At the Keeyask site, the bedrock is predominantly greywacke gneiss, with amphibolite predominant in the Powerhouse area. The greywacke gneiss and amphibolites commonly contain bands or sills of granitic material. Diabase dykes crosscut all rock types in the areas, and vary from short narrow features to continuous wider dykes. The contacts between the diabase dykes and the host rocks are typically open near surface, but generally tighten with depth.

All rock types within the Gull Rapids area are typically fresh, strong to very strong, with medium to widely spaced joints. Most of the joints are tight with little or no alteration. Bedrock drilling at the locations of the North, Central and South Dams, Powerhouse, Spillway and Transition Structures has shown no evidence of significant geological weaknesses. The bedrock appears to be competent and suitable for the proposed Principal Structures at this site.

4 Foundation Conditions along the Transmission Tower Spur

4.1 Site Investigations

The investigations of the Keeyask GS site commenced in the early 1960s. The Stage II and Stage III Explorations were conducted in 1988 and 1990. Since then, additional investigations have been carried out in the Winter 1990/1991, Summer 1991, Winter 1991/1992, Winter 1999/2000 and Summer 2003.

During the 2003 Summer Subsurface Investigation three drill holes were advanced from a platform over water, along the alignment of the Powerhouse Stage I Cofferdam (BH 03-039 to BH 03-041 inclusive). All drill holes were advanced a minimum of 3 m into bedrock. Boreholes 03-039 and 03-040 are located in the vicinity of the TTS.

4.2 Overburden

The site investigations performed in the riverbed area in the vicinity of the Principal Structures generally indicate the existence of a thin layer of silty sand to sandy silt alluvium overburden overlying the bedrock. The thickness of the overburden in the riverbed generally varies between zero and 0.7 m. Two boreholes (03-039 and 03-040) located near the proposed TTS indicated the depth of alluvium varies between 0.2 m to 0.3 m.

4.3 Bedrock

The bedrock encountered near the vicinity of the proposed TTS (G-0064, G-0015, G-0016, 03-039, and 03-040) is generally very strong to strong greywacke gneiss or granite, of good to excellent rock quality. Local zones of broken and/or lost core are present and are attributed

to open joints and zones of closely spaced jointing. The bedrock has undergone ancient deformation and therefore contains healed shear zones and well defined foliation.

5 Design of the Transmission Tower Spur

5.1 Transmission Towers

The towers required on the TTS will be self-supporting, up to 47 m high with a square base of 15 m x 15 m (49 ft x 49 ft). Each transmission tower has been assumed to be anchored to four 4.5 m x 4.5 m concrete footings based on sizing of tower footings used at Limestone GS. The footings will extend approximately 5.0 m in depth into the TTS. Each of the four single circuit structures will have a centre to centre spacing of 45.72 m.

The locations of the transmission towers were provided to KGS ACRES by Manitoba Hydro in an email dated October 20, 2009. A minimum distance of 10 m has been provided around the tower footprint in all directions to provide access for maintenance vehicles and the ability to increase the size of the tower footprint during final design, if required. In order to accommodate for the clearance around the towers and still maintain the required minimum distance of 3 m between the toe of the TTS and the edge of the Tailrace Channel we have incorporated the placement of transition fill and road topping materials over the top of the riprap along the north edge of the TTS (see Section A-A in Figure 4). We have considered the alternative of shifting the towers to the south to provide the additional clearances, however this would require realignment and lengthening of the Powerhouse Cofferdam in order to allow for the construction of the tower footings in-the-dry.

The construction methodology will have the tower foundations constructed in-the-dry concurrently with the placement of the rock and granular fill. At this time the design of the foundation footings for the towers is currently under review by Manitoba Hydro.

5.2 Design Water Levels

The Tailrace water levels applicable for the design of the proposed Keeyask GS are presented in Memorandum GN-4.1.2, “Hydraulic Design Criteria” [Ref 1], Memorandum GN-2.2, “Updated Backwater Study Winter Conditions” [Ref 2] and GN-2.8, “River Management During Construction” [Ref 3].

The following maximum water levels were considered for the preliminary design of TTS crest elevation:

- Extreme Maximum Tailrace Water Elevation el. 142.2 m
- Winter Water Level During Construction el. 143.7 m

5.3 Crest Elevation

The selection of the crest elevation for the TTS considered the following:

(a) Design Water Level

Winter Water Level During Construction el. 143.7 m.

(b) Freeboard Requirements

Under winter (ice cover) conditions a minimum freeboard allowance of 1.5 m shall be provided for cofferdams, with crest elevations to be governed by winter conditions. The additional freeboard allowance under winter conditions is provided due to the uncertainty in the prediction of staging due to ice [Ref 1].

(c) Settlement Requirements

Settlement of the TTS is assumed as being 1% of the height of the embankment, as described in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref 4]. For an approximately 10 m high embankment, the estimated settlement is about 0.10 m. Settlement of the foundation will be based on the foundation materials. The foundation on the riverbed is assumed to have negligible settlement.

The elevation of the crest for the TTS, as determined from the above criteria, is shown in Table 1.

As a minimum, the crest of the TTS was rounded up to the nearest 0.5 m and set at a level equal to that of the Stage I Powerhouse Cofferdam at el. 145.5 m. As discussed in memorandum GN-4.2.2.11 Section 6, the crest elevation of the Powerhouse Stage I Cofferdam is governed by winter ice conditions and is based on the same criteria as the TTS.

The selected crest el of 145.5 m was based on the "outside" cofferdam location (i.e. the outside protective rock groin of the Powerhouse Cofferdam was incorporated into the TTS). At this time, the crest requirement for the TTS for the "inside" cofferdam location as illustrated in Figures 3 and 4 has not been reviewed.

It is believed that there is an opportunity to lower the crest elevation of the TTS for the current location which would decrease the fill quantities for the TTS and possibly decrease the Powerhouse and Tailrace summer level cofferdams quantities. The total saving in fill material quantities for the cofferdams would likely be in the range of 3% to 8% and they would be derived from the opportunity to reset the alignment of the cofferdams to shorten their length. The savings in the fill quantities for the TTS would likely range from 5% to 15% depending on the reduction in the crest elevation. In addition, the lowering of the crest of the TTS would have a positive impact on the construction schedule resulting in possible cost savings for the project.

In an effort to close Stage IV Studies, it is recommended that the assessment discussed above be deferred into Final Design.

The freeboard requirements for the TTS for Final Design will be based on the Geotechnical Design Criteria for Final Design. As a minimum, the crest of the TTS will be set at a level which satisfies the more critical of the following two conditions:

1) Full Supply Level for Stephens Lake plus wind effects for the 1:1000 year wind event.

2) Probable Maximum Flood level plus wind effects for the 1:2 year wind event.

In addition, the freeboard requirements are long-term conditions, and therefore must take into account any potential settlement of the embankment and foundations of the structure.

5.4 Access Ramp

The access ramp will consist of a Class 6 rockfill embankment, with a granular transition zone and a road topping surface. The granular transition fill on top of the rockfill will be 0.30 m thick. The road topping located on top of the granular transition material will be 0.20 m thick. The access ramp will have a maximum grade of 6%, a minimum crest width of 8 m, and side slopes set at 2H:1V.

The access ramp will be tied into the downstream berm of the Central Dam.

6 Transmission Tower Spur Construction Materials

The materials to be used in the TTS will consist of Class 6 rockfill, Class 4 filter, Class 2a bedding and Class RT.

The Class 6 rockfill material will come from the excavations on the north side of the river. The granular fill materials of Class 2a, 3 and RT will be sourced from granular Deposit G-3 on the north side of the river. The location of the potential borrow sources are shown in Figure 3.

The gradation limits for the materials to be utilized in the construction of the TTS are given in Table 2. The properties of the fills to be used for the design and construction of the embankment are provided in Memorandum GN-4.1.3, “Geotechnical Design Criteria [Ref 4], and summarized in Tables 2 and 3. A written description of the characteristics of the various classes of fills follows.

Pervious Granular Fill (Class 2a)
Class 2a fill shall be a uniform to well graded pit run sandy gravel with less than 7% (by weight) passing 0.075 mm. It shall be used to form a base on top of the coarse filter (Class 4).

Coarse Filter (Class 4)
Class 4 shall be a clean gravel of gravel sized crushed rock, processed as required so that the gradation limits are satisfied. Class 4 fill will be used as a filter layer adjacent to Classes 2a, 3 and 5, and as a bedding layer overlying the Class 6 rockfill.

Rockfill (Class 6)
Rockfill Class 6 shall be well grade rock obtained form the required rock excavations and/or quarries, with a maximum particle size of 500 mm. Processing may not be required, other than selective removal of oversized pieces.

Riprap (Class 7 and Class 8)
Riprap Class 7 and Class 8 will be selected, narrowly graded, broken or blasted, sound, durable rock to be used for riprap protection of the slopes of the TTS. The riprap sizing will be based on Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref 1] requirements.

Road Topping (Class RT)
Road Topping will be processed sands and gravels with a trace to some fines, or gravel sized crushed rock with some sand and a trace to some fines. Road topping fill will be used fro the toping of the TTS crest and its access ramp.

7 Stability Analysis

Stability analyses were not carried out for the TTS. It has been deemed that the effective angle of shearing resistance (ϕ') of the rockfill matrix will provide acceptable factors of safety, greater than the minimum for the various conditions as presented in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref 5].

8 Construction of Transmission Tower Spur

8.1 Powerhouse Stage I Cofferdam

Stage I Diversion will close off the north channel of the river and direct the river's flow into the south portion of the channel. As part of the Stage I Diversion, the Powerhouse Cofferdam with a crest set at el. 145.5 m will be extended from the north bank to the east bank of the North Island, closing the North Channel and encompassing the Powerhouse area. An initial protective rockfill groin, parallel to the southern leg of the Powerhouse Stage I Cofferdam will extend east into the river (see Figure 3).

The Powerhouse Stage I Cofferdam will be a zoned earth and rockfill embankment. The cofferdam will consist of two, upstream and downstream, rockfill (Class C) groins and an impervious silty sand till (Class A) filled in between these two rockfill groins. On the "dry

side" rockfill groin, a zone of granular transition (Class B) will be placed between the impervious zone Class A and the rockfill Class C. The "dry side" rockfill groin will be constructed out of finer rockfill than the "wet side" rockfill groin, or at least a zone of finer rockfill material will be placed along the interface area with the Class B transition zone. A layer of road topping material (Class RT) will be placed on the access road of the cofferdam. The cross section of the Powerhouse Stage I Cofferdam is shown in Figure 4. Further details are discussed in Memorandum GN-4.2.2.11 "Earthworks Structures – Cofferdams Arrangements and Design" [Ref 6].

8.2 Transmission Tower Spur

The TTS will be a zoned granular and rockfill embankment, constructed within the Powerhouse Stage I and Tailrace Summer Level Cofferdams. The TTS will be approximately 200 m long and the major portion of the rockfill used in the construction will be rockfill Class 6. The side slopes of the TTS will rest at 1.5H:1V for material placed in-the-wet and 1.8H:1V for material place in-the-dry. Riprap erosion protection consisting of Class 7 or 8 shall be placed on the slopes and shall have a minimum thickness of 2 x D_{50}. The rockfill groin (dry side) of the Powerhouse Stage I Cofferdam will be incorporated into the TTS.

The TTS will be constructed inside the Powerhouse Stage I and Tailrace Summer Level Cofferdams. Constructing the TTS in-the-dry will allow sufficient time to construct the concrete footings for the towers and most of the spur prior to the removal of the cofferdams.

The construction of the spur will be carried out in two phases. The first phase will be performed in Year Y-1 and will include the completion of the spur and transmission towers for Powerhouse Units 1, 2, 5, 6 and 7 (the two western most towers and the eastern most tower). The remainder of the TTS will be completed in Year Y following the excavation of the remaining impervious fill from the Powerhouse Cofferdam located within the footprint of the spur.

The construction will be sequenced to allow construction of the towers footings to be performed in-the-dry. Class 6 rockfill will be placed inside the Powerhouse Stage I Cofferdam and Tailrace Summer Level Cofferdam up to el 140.0 m, directly against the internal rockfill slope of the cofferdams. Each concrete footing will then be cast in place on a 0.5 m thick layer of Class 4 material wrapped with a non-woven geotextile fabric. Backfilling will continue using Class 6 rockfill materials around each footing up to el 144.6 m as shown in Figure 4. Above el 144.6, the backfill will consist of up to 0.4 m of coarse filter (Class 4) up to el 145.0 m, overlain with 0.3 m of Class 2a and a 0.2 m thick layer of Road Topping (Class RT) up to el 145.5 m as shown in Figure 4.

As discussed in Section 5.3 it is envisioned that there is an opportunity to lower the crest elevation of the TTS which will result in some cost savings. If there is a reduction in crest then the top elevation of the Class 6 material will be lowered from el. 144.6 m. All material zoning

layers above the Class 6, i.e. the use of Class 4, 2a and RT are not expected to change. The cost savings associated with the decrease in the crest elevation will generally be associated with the decrease in quantities for the Class 6, Class 7 and Class 8 rockfill products.

8.3 Riprap Armouring

Although the TTS will be located away from the path of direct flow from the Powerhouse Tailrace Channel, the spur will encroach into the flow path of discharge from the Spillway. As such, the spur will experience significant flow velocities (on the south side) during the operation of the Spillway under high flood events. High flow velocities will also be present on the north side of the spur during normal Powerhouse Tailrace Channel operations. Therefore it is deemed that riprap erosion protection is required.

The design floods and methodology used to size the riprap are provided in Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref 1].

For assessing the minimum level of riprap erosion protection for the TTS, the following were considered:

- Wind event having a return period of 1:1,000 years.
- A range of design floods, 1:1,000, 1:10,000 and the Probable Maximum Flood (PMF).
- Maximum discharge through the Powerhouse Tailrace Channel.

The hydraulic design of the TTS is discussed in detail in Memorandum GN-4.3.21, "Hydraulic Design of TTS" [Ref 7].

The median riprap size (D_{50}) to prevent erosion due to wave action was computed along with the median size of riprap to prevent erosion due to high flow velocities under the discharge conditions considered. The summary of riprap sizing for the TTS is presented in Table 4. In addition, the riprap sizes selected will provide protection against wave action from a 1:1 000 year wind event, and all flood events, ranging in magnitude up to and including the PMF, discharged through the Principal Structures.

The riprap sizing determined above was based on an earlier geometry and location of the TTS, which was subsequently extended to the east by 5.4 m and to the south by 6.0 m to provide adequate space for maintenance vehicles. The effect of these changes on the erosion protection requirements are anticipated to be small and will be reviewed in further detail during final design.

8.4 Excavation and Foundation Preparations

Virtually the entire length of the TTS will be founded on alluvium or bedrock. Foundation preparation is not required.

8.5 Quantities

A summary of the estimated quantities of the material required for the construction of the TTS is presented in Table 5.

Prepared By

D.M. Fuchs

P.R. Pantel

Attach
References
Tables 1 to 5
Figures 1 to 5

DMF:PRP:spa

References

1. KGS Acres Ltd., Memorandum GN-4.1.2, “Keeyask Generating Station – Stage IV Studies GR-4 - Hydraulic Design Criteria”, Manitoba Hydro File 00195-06330-0004, Rev 1, June 2011.

2. KGS Acres Ltd., Memorandum GN-2.2, “Keeyask Generating Station – Stage IV Studies GR-4 – Updated Backwater Study Winter Conditions”, Manitoba Hydro File 00195-11410-0028, Rev. 1, July 2009.

3. KGS Acres Ltd., Memorandum GN-2.8, “Keeyask Generating Station – Stage IV Studies GR-4 - River Management During Construction”, Manitoba Hydro File 00195-10420-0002, Rev 0, June 2011.

4. Canadian Dam Association, “Dam Safety Guidelines”, Canadian Dam Association, Edmonton, January 1999.

5. KGS Acres Ltd., Memorandum GN-4.1.3, “Keeyask Generating Station – Stage IV Studies GR-4 - Geotechnical Design Criteria”, Manitoba Hydro File 0195-06310-0002, Rev 0, August 2010.

6. KGS Acres Ltd., Memorandum GN-4.2.2.11, “Keeyask Generating Station - Stage IV Studies GR-4 - Earthworks Structures - Cofferdams Arrangements and Design”, Manitoba Hydro File 00195-21600-0004, Rev 0, June 2011.

7. KGS Acres Ltd., Memorandum GN-4.3.21, “Keeyask Generating Station – Stage IV Studies - Axis GR-4 - Hydraulic Design of Transmission Tower Spur”, Manitoba Hydro File 00195-11400-0035, Rev. 0, March 2009.

8. Bohm, C.0., Bowerman, M.S. and Downey, M.W. 2006: Bedrock Geology of the Gull Rapids Area, Manitoba (part of NTS 54D6): Manitoba Science, Technology, Energy and Mines, Manitoba Geological Survey, Open File Report OF2006-32.

Tables

Tables 1 to 5

Table 1: Crest Requirements

Item Description	Transmission Tower Spur	Selected for Design
Winter Water Levels During Construction [(1)]	143.7	-
Freeboard [(2)]	1.50	-
Settlement Allowance (m) [(3)]	0.10	-
Foundation Settlement (m) [(4)]	0.0	-
Crest Elevation (m)	145.3	145.5[(5)]

Notes:

(1) From Memorandum GN-4.2.2.11 "Cofferdam Arrangement and Design" and GN-2.2 "Updated Backwater Study Winter Conditions.

(2) Freeboard of 1.5 m shall be provided during construction for winter conditions.

(3) Based on 1% of an average TTS height of 10 m.

(4) Dependent upon foundation material, assumed to be negligible for riverbed as the majority of the settlement (if any) will likely occur immediately during the placement of the rockfill for the base of the TTS.

(5) Crest elevation is rounded up the nearest 0.5 m.

Table 2: Construction Materials - Gradation Limits

US Standard Sieve Size	Limits - Percent Passing (by weight)									
	Class 1	**Class 2a**	**Class 2b**	**Class 3**	**Class 4**	**Class 5**	**Class 6**	**Class 7**	**Class 8**	**Class RT**
(1100 mm)									100	
(1000 mm)									50-100	
(900 mm)								100	30-50	
(800 mm)								75-100	0-20	
(760 mm)									0	
(650 mm)								35-50		
(600 mm)								20-40		
(550 mm)								0-15		
(500 mm)						100	100	0		
(400 mm)										
(300 mm)						70-100	50-100			
(200 mm)						60-80	30-100			
(150 mm)	100									
(100 mm)							0-80			
75 mm	98-100	100	100		100					
37.5 mm	95-100									
25 mm										100
19 mm	90-100	90-100	90-100	100	90-100	10-35	0-35			85-100
12.5 mm						0-27				70-100

US Standard Sieve Size	Limits - Percent Passing (by weight)									
	Class 1	**Class 2a**	**Class 2b**	**Class 3**	**Class 4**	**Class 5**	**Class 6**	**Class 7**	**Class 8**	**Class RT**
9.5 mm				90-100	60-100					
4.75 mm	80-100	70-100	70-100	80-100	40-85	0-10	0-10			35-60
2 mm	70-100	55-100	50-100	60-100	15-60					25-45
0.85 mm					0-35					
.425 mm	50-92	10-55	0-80	15-50	0-15					10-25
.15 mm	38-80			0-10	0-7					
.075 mm	30-70	0-5	0-10	0-5						4-10
0.01 mm	10-40									
0.002 mm	0-20									

Table 3: Construction Materials – Properties

Fill	Moist Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Effective Angle of Shearing Resistance ϕ' (deg)	Effective Cohesion c' (kPa)	Hydraulic Conductivity k (cm/s)	Pore Pressure Parameter (r_u)
Transmission Tower Spur						
Impervious Fill (Class 1)	19.9	20.3	33	0	10^{-5}	0.3
Granular Fill (Class 2a)	21.2	22.1	35	0	10^{-3}	0
Fine Filter (Class 3)	21.2	22.1	35	0	10^{-2}	0
Coarse Filter (Class 4)	22.2	22.7	35	0	10^{-1}	0
Riprap Bedding (Class 5)	18.7	21.1	45	0	Free Draining	0
Rockfill (Class 6)	20.1	21.4	45	0	Free Draining	0
Riprap (Class 7 and 8)	18.7	21.1	45	0	Free Draining	0
Road topping (Class RT)	22.2	22.7	35	0	Free Draining	0
Powerhouse Cofferdam material incorporated into the Transmission Tower Spur						
Rockfill (Class C)	19.5	21.3	45	0	Free Draining	0

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of Riprap Sizing for the TTS

Location	Required Size of Riprap (D_{50}) (m)	Rockfill Class[(2)]
Powerhouse Side	0.41[(1)]	Class 7
Spillway Side	0.41[(1)]	Class 8
At east end of TTS	0.75	Class 8

Notes:

(1) Based on minimum stone mass criterion imposed in Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref 1].

(2) All rockfill used shall consist of hard, dense and resistant blocks of rock. Low specific gravity rock or altered rock and shale are not acceptable.

(3) The limits of the riprap armouring are shown in Figure 5.

Table 5
Keeyask GS – Stage IV Studies, Axis GR-4
Quantities for Construction

Item	Description	Unit	Quantity
Fill	Class 2 - Transition Fill	m^3	4,900
	Class 4 – Coarse Filter	m^3	6,700
	Class 6 - Rockfill	m^3	114,400
	Class 7 – Riprap	m^3	6,100
	Class 8 - Riprap	m^3	12,900
	Class RT - Road Topping	m^3	3,000

Figures

Figures 1 to 5

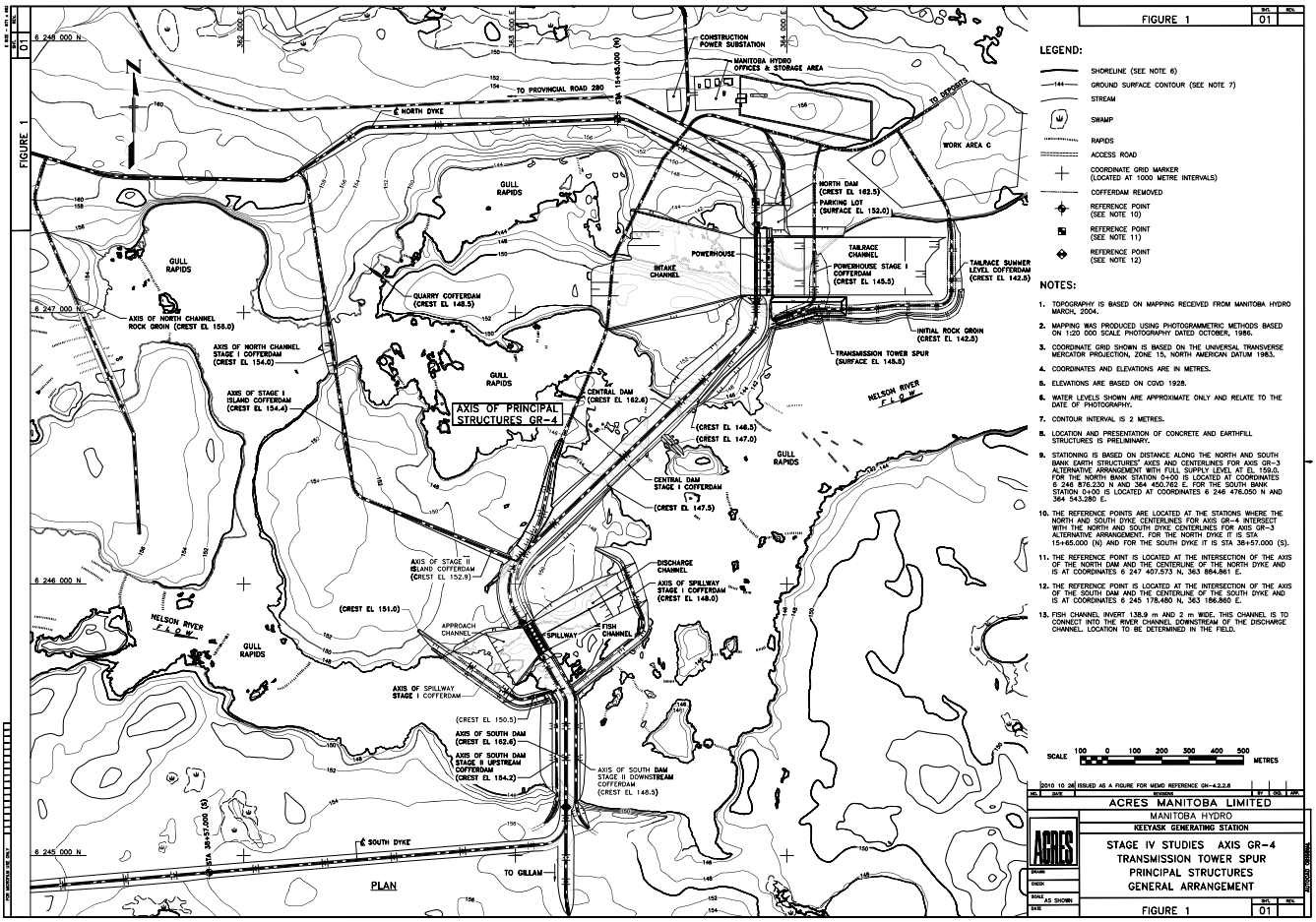


FIGURE 1
01
LEGEND:
SHORELINE (SEE NOTE 6)
144 GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
REFERENCE POINT (SEE NOTE 12)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURES' AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0. FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 248 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.
SCALE 100 0 100 200 300 400 500 METRES
2010 10 26 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.6
ACRES MANITOBA LIMITED
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
TRANSMISSION TOWER SPUR
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT
FIGURE 1
01
PLAN
N
6 248 000 N
6 247 000 N
6 246 000 N
6 245 000 N
392 000 E
393 000 E
394 000 E
STA 15+65.000 (N)
STA 38+57.000 (S)
℄ NORTH DYKE
℄ SOUTH DYKE
TO PROVINCIAL ROAD 280
TO GILLAM
TO THOMPSON
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO OFFICES & STORAGE AREA
WORK AREA C
NORTH DAM (CREST EL 162.5)
PARKING LOT (SURFACE EL 152.0)
POWERHOUSE
TAILRACE CHANNEL
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.5)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 142.5)
INTAKE CHANNEL
INITIAL ROCK GROIN (CREST EL 142.5)
TRANSMISSION TOWER SPUR (SURFACE EL 145.5)
QUARRY COFFERDAM (CREST EL 148.5)
AXIS OF NORTH CHANNEL ROCK GROIN (CREST EL 156.0)
AXIS OF NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
AXIS OF STAGE I ISLAND COFFERDAM (CREST EL 154.4)
AXIS OF PRINCIPAL STRUCTURES GR-4
CENTRAL DAM (CREST EL 162.6)
(CREST EL 146.5)
(CREST EL 147.0)
CENTRAL DAM STAGE I COFFERDAM
(CREST EL 147.5)
AXIS OF STAGE II ISLAND COFFERDAM (CREST EL 152.0)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM (CREST EL 148.0)
(CREST EL 151.0)
APPROACH CHANNEL
SPILLWAY
FISH CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
AXIS OF SOUTH DAM (CREST EL 162.6)
AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 154.2)
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
NELSON RIVER FLOW
GULL RAPIDS

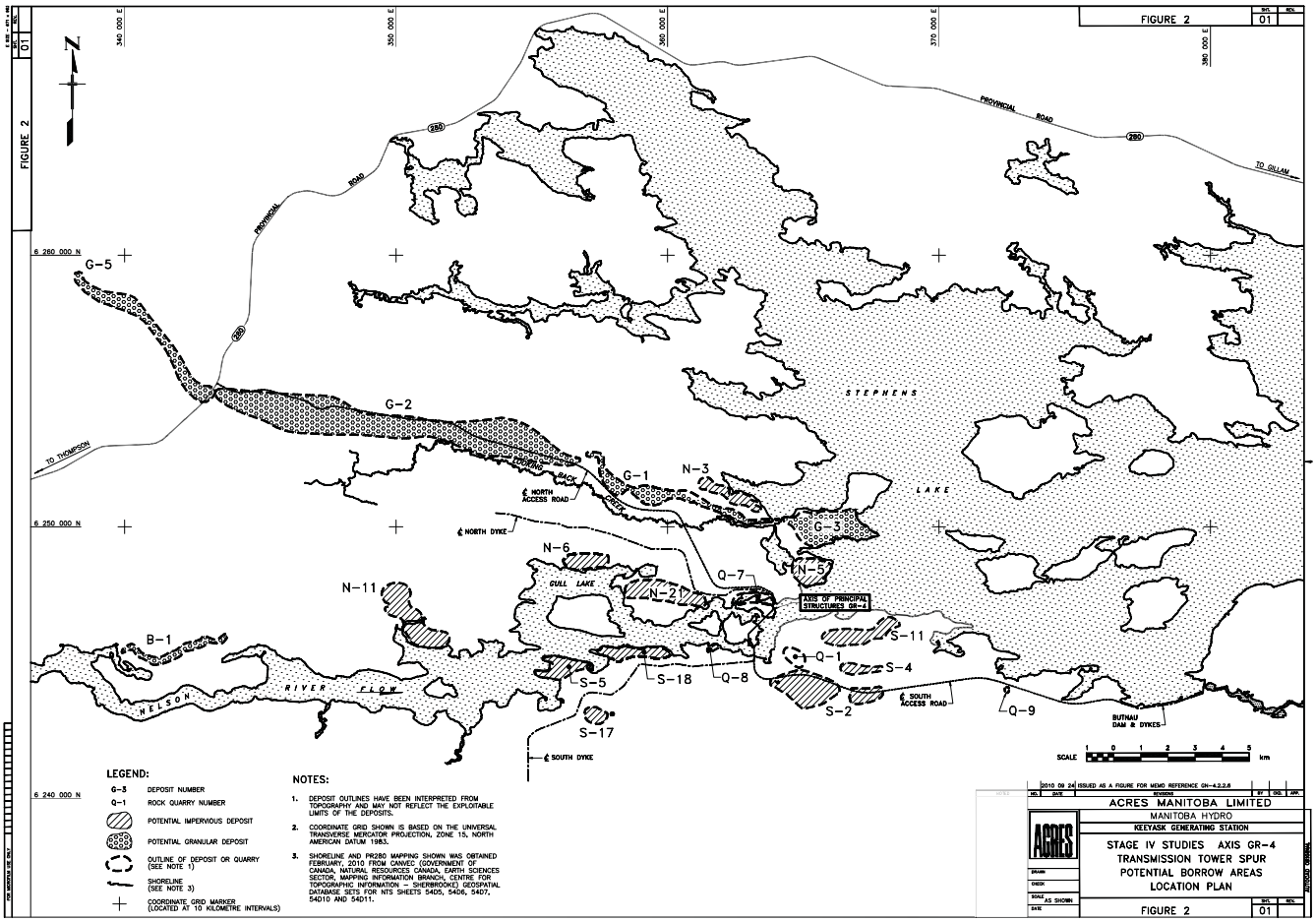
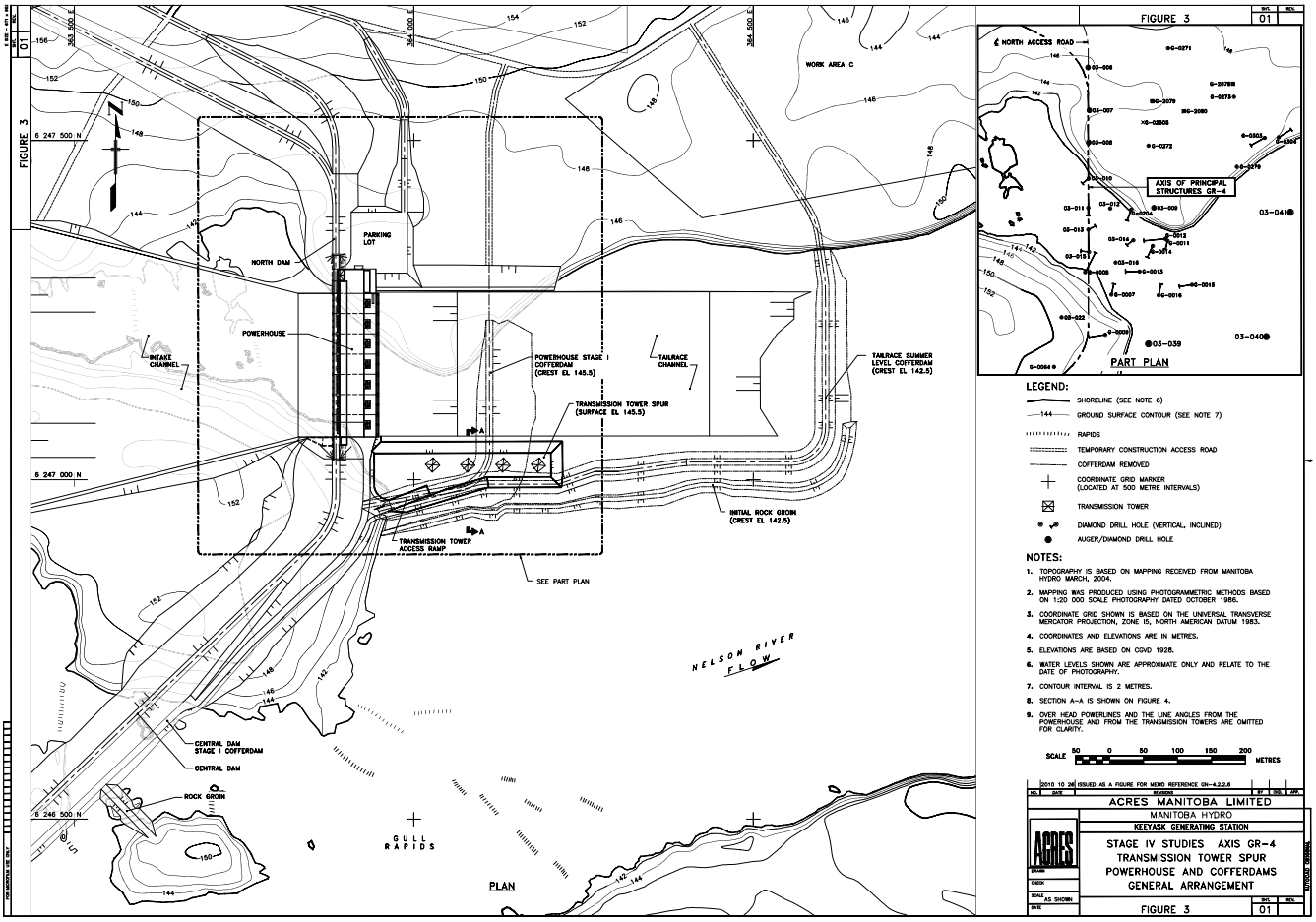


FIGURE 2
340 000 E
350 000 E
360 000 E
370 000 E
380 000 E
PROVINCIAL ROAD
280
TO GILLAM
TO THOMPSON
6 260 000 N
6 250 000 N
6 240 000 N
G-5
G-2
G-1
G-3
N-1
N-2
N-3
N-4
N-5
N-6
N-11
Q-1
Q-7
Q-8
Q-9
S-2
S-4
S-5
S-11
S-17
S-18
B-1
STEPHENS
LAKE
GULL LAKE
NELSON
RIVER
FLOW
NORTH ACCESS ROAD
NORTH DYKE
SOUTH ACCESS ROAD
SOUTH DYKE
AXIS OF PRINCIPAL STRUCTURES GR-4
BUTNAU DAM & DYKES
SCALE
0 1 2 3 4 5 km
LEGEND:
G-5 DEPOSIT NUMBER
Q-1 ROCK QUARRY NUMBER
POTENTIAL IMPERVIOUS DEPOSIT
POTENTIAL GRANULAR DEPOSIT
OUTLINE OF DEPOSIT OR QUARRY (SEE NOTE 1)
SHORELINE (SEE NOTE 3)
COORDINATE GRID MARKER (LOCATED AT 10 KILOMETRE INTERVALS)
NOTES:
1. DEPOSIT OUTLINES HAVE BEEN INTERPRETED FROM TOPOGRAPHY AND MAY NOT REFLECT THE EXPLOITABLE LIMITS OF THE DEPOSITS.
2. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
3. SHORELINE AND PR280 MAPPING SHOWN WAS OBTAINED FEBRUARY, 2010 FROM CANVEC (GOVERNMENT OF CANADA, NATURAL RESOURCES CANADA, EARTH SCIENCES SECTOR, MAPPING INFORMATION BRANCH, CENTRE FOR TOPOGRAPHIC INFORMATION — SHERBROOKE) GEOSPATIAL DATABASE SETS FOR NTS SHEETS 54D0, 54D6, 54D7, 54D10 AND 54D11.
2010 09 24 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.6
ACRES MANITOBA LIMITED
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
TRANSMISSION TOWER SPUR
POTENTIAL BORROW AREAS
LOCATION PLAN
FIGURE 2
01



LEGEND:

- SHORELINE (SEE NOTE 6)
- 144 GROUND SURFACE CONTOUR (SEE NOTE 7)
- RAPIDS
- TEMPORARY CONSTRUCTION ACCESS ROAD
- COFFERDAM REMOVED
- COORDINATE GRID MARKER (LOCATED AT 500 METRE INTERVALS)
- TRANSMISSION TOWER
- DIAMOND DRILL HOLE (VERTICAL, INCLINED)
- AUGER/DIAMOND DRILL HOLE

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. SECTION A–A IS SHOWN ON FIGURE 4.
9. OVER HEAD POWERLINES AND THE LINE ANGLES FROM THE POWERHOUSE AND FROM THE TRANSMISSION TOWERS ARE OMITTED FOR CLARITY.

SCALE 50 0 50 100 150 200 METRES

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MANITOBA HYDRO

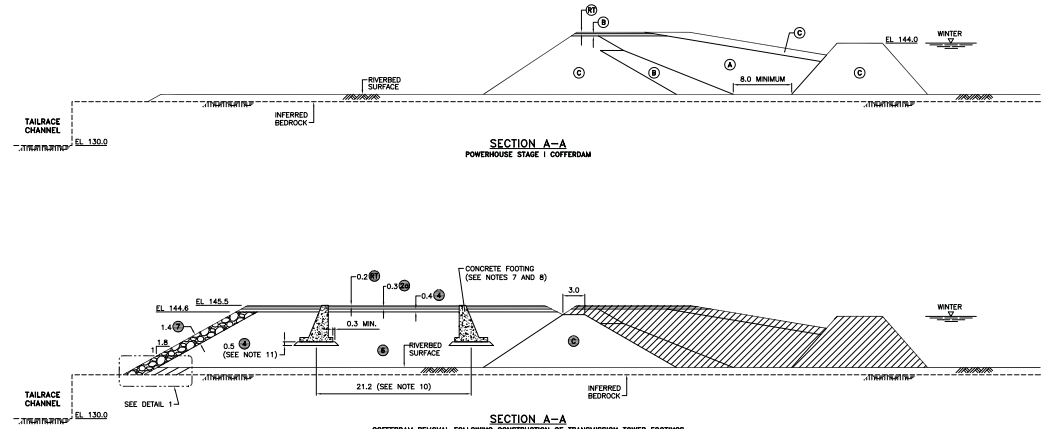
KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
TRANSMISSION TOWER SPUR
POWERHOUSE AND COFFERDAMS
GENERAL ARRANGEMENT

DRAWN
CHECK
SCALE AS SHOWN
DATE

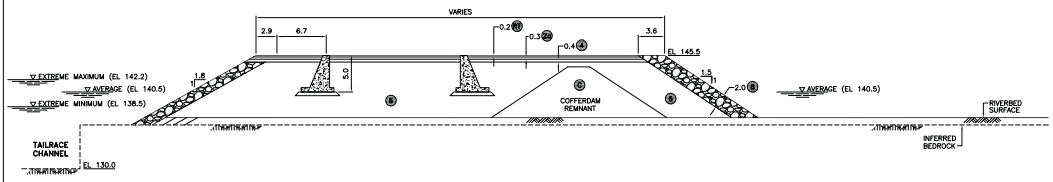
FIGURE 3 01

FIGURE 4 | SHT. 01 REV.

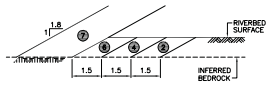


SECTION A–A
POWERHOUSE STAGE I COFFERDAM

SECTION A–A
COFFERDAM REMOVAL FOLLOWING CONSTRUCTION OF TRANSMISSION TOWER FOOTINGS



SECTION A–A
FINAL TRANSMISSION TOWER SPUR



DETAIL 1
N.T.S.

LEGEND:

- A — IMPERVIOUS FILL
- B — GRANULAR (TRANSITION) FILL
- C — ROCKFILL
- RT — ROAD TOPPING
- 2A — PERVIOUS GRANULAR FILL
- 4 — TRANSITION – CRUSHED ROCK
- 5 — ROCKFILL
- 7 — RIPRAP
- 8 — RIPRAP
- WATER LEVEL
- COFFERDAM REMOVAL
- RIPRAP (SEE NOTE 5)

NOTES:

1. ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. ELEVATIONS ARE BASED ON CGVD 1928.
3. FREEBOARD ALLOWANCE APPROXIMATELY 3.3 METRES ABOVE MAXIMUM TAILWATER ELEVATION.
4. ADDITIONAL ROCK ARMOUR SHALL BE PLACED AS REQUIRED, WHERE HIGHER VELOCITIES ARE ANTICIPATED.
5. CLASS 7 OR 8 (2 X D_{50}) ROCKFILL ARMOUR PROTECTION.
6. PRIOR TO THE REMOVAL OF THE POWERHOUSE COFFERDAM, ALL EXPOSED RIVERBED SLOPES WITHIN THE TAILRACE CHANNEL SHALL HAVE FILTERS AND RIPRAP INSTALLED TO MINIMIZE THE HYDRAULIC FORCES ACTING ON THE ALLUVIUM. THIS IS TO PREVENT THE POWERHOUSE DISCHARGE FLOWS FROM UNDERMINING THE TRANSMISSION TOWER SPUR.
7. FOOTINGS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY. FOOTINGS WILL BE REVIEWED DURING THE FINAL DESIGN.
8. CONCRETE FOOTINGS WILL EXTEND INTO THE TRANSMISSION TOWER SPUR.
9. THE LOCATION OF SECTION A–A IS SHOWN ON FIGURE 3.
10. THE CENTER–TO–CENTER SPACING OF THE TOWER FOOTINGS WILL BE 15.0 M, HOWEVER THE ORIENTATION OF THE TOWERS RELATIVE TO SECTION A–A RESULTS IN THE INCREASED DISTANCE INDICATED.
11. CLASS 4 TO BE WRAPPED WITH NON–WOVEN GEOTEXTILE FILTER FABRIC.

SCALE 5 0 5 10 15 20 METRES

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MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR–4
TRANSMISSION TOWER SPUR
AND RELATED COFFERDAM SECTION

DRAWN
CHECK
SCALE AS SHOWN
DATE

FIGURE 4 | SHT. 01 REV.

AUTOCAD ORIGINAL

FOR HORIZONTAL USE ONLY

E SIZE — ANSI A SIZE
REV. 01

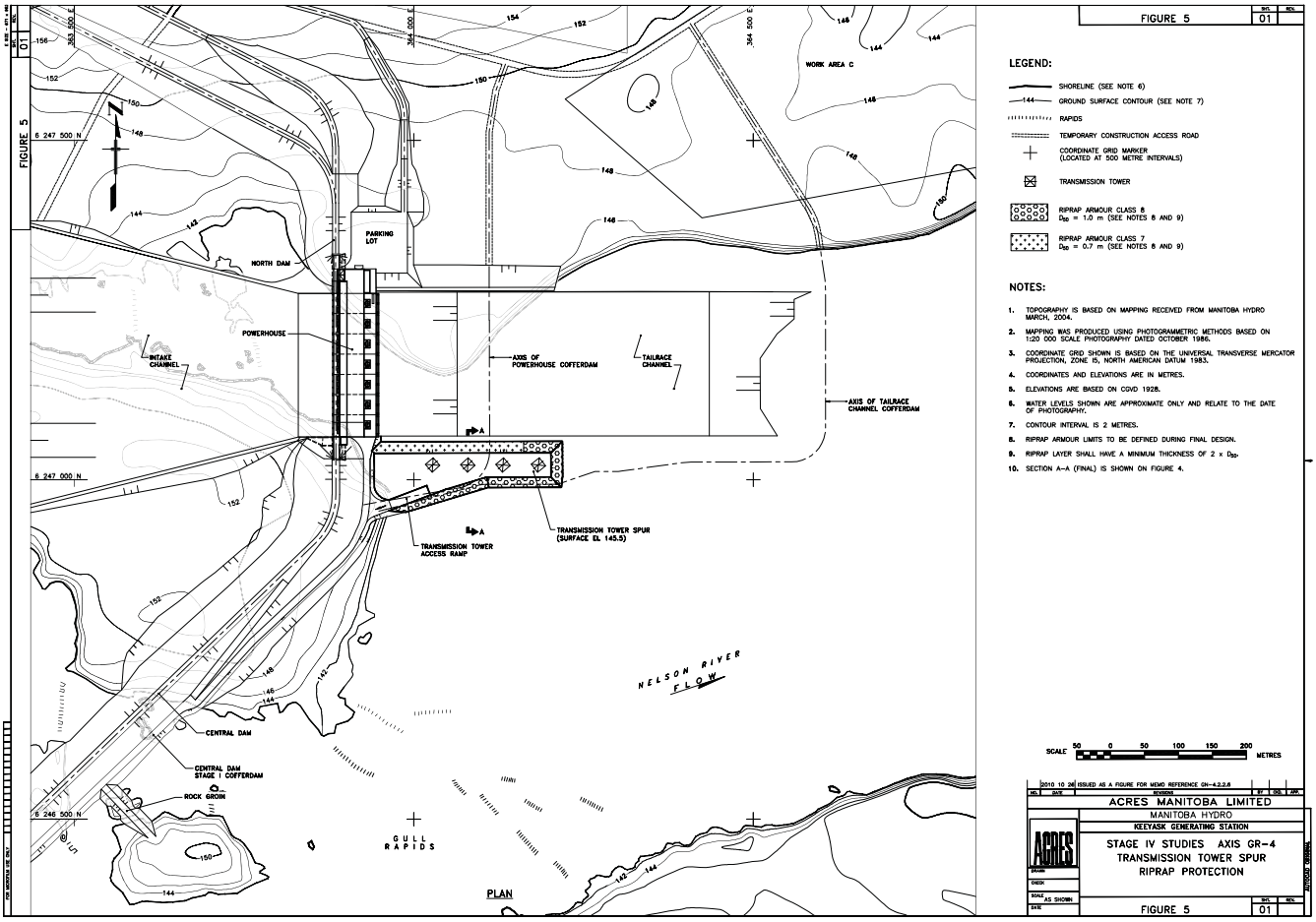

FIGURE 5 | REV. 01

LEGEND:

- SHORELINE (SEE NOTE 6)
- 144 GROUND SURFACE CONTOUR (SEE NOTE 7)
- RAPIDS
- TEMPORARY CONSTRUCTION ACCESS ROAD
- COORDINATE GRID MARKER (LOCATED AT 500 METRE INTERVALS)
- TRANSMISSION TOWER
- RIPRAP ARMOUR CLASS 8 D_{50} = 1.0 m (SEE NOTES 8 AND 9)
- RIPRAP ARMOUR CLASS 7 D_{50} = 0.7 m (SEE NOTES 8 AND 9)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. RIPRAP ARMOUR LIMITS TO BE DEFINED DURING FINAL DESIGN.
9. RIPRAP LAYER SHALL HAVE A MINIMUM THICKNESS OF 2 x D_{50}.
10. SECTION A–A (FINAL) IS SHOWN ON FIGURE 4.

SCALE 50 0 50 100 150 200 METRES

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ACRES MANITOBA LIMITED

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
TRANSMISSION TOWER SPUR
RIPRAP PROTECTION

DESIGN	
CHECK	
SCALE	AS SHOWN
DATE	

FIGURE 5 | REV. 01

AUTOCAD DRAWING

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.7
Rev. 0, March 17, 2010

Grouting and Pressure Relief

Manitoba Hydro File 00195-20100-0002_02

Prepared by Phil Pantel (P. PANTEL) March 17, 2010
KGS Acres Ltd.

Checked by (R. HALIM) March 17, 2010
KGS Acres Ltd.

Reviewed by (for R. Dewar) Mar 17, 2010
KGS Acres Ltd.

Approved by (N. J. Smith) March 17, 2010
KGS Acres Ltd.

Accepted by DS MAGNUSSON MARCH 30, 2010
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I. R. Dewar	**Date**	March 24, 2010
		File No.	P10008(16).27.07
From	P. R. Pantel	**cc**	
Subject	**Keeyask Generating Station Stage IV Studies – Axis GR-4 Grouting and Pressure Relief Memorandum GN-4.2.2.7, Rev 0 Manitoba Hydro File 00195-20100-0002_02**		

1 Introduction

The site of the proposed Keeyask Generating Station (Keeyask GS) is located on the Nelson River approximately 730 km north of Winnipeg, at the head of Stephens Lake. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is the Gull Rapids site. The proposed axis for the structure crosses the Nelson River at the foot of the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam.

The general arrangement of the proposed Keeyask GS is shown in Figure 1. The layout corresponds to that proposed for Axis GR-4 development, with an FSL at el 159.0 m. The selected arrangement consists of two long riverbank dykes, on the north and on the south, linked to the Principal Structures, which span the Nelson River at Gull Rapids. The Principal Structures consist of a Powerhouse Complex constructed on the north side of the river, which will be connected to the North Dyke, through a 99 m long North Dam and a short concrete Transition Structure. The Spillway would be located on an island, approximately 1.5 km south of the Powerhouse. A Central Dam will connect the Spillway and the Powerhouse. The Spillway will be connected to the South Dam by means of a short concrete Transition Structure. The South Dam, approximately 565 m long, will connect to the South Dyke on the south riverbank.

This memorandum reviews the bedrock conditions and permeability, and summarizes the requirements for curtain grouting and rock surface treatments below the main structures. It also reviews the requirements for a Drainage Curtain (pressure relief drains) within the foundation of the Powerhouse, below the Draft Tube. The estimated length of drilling and grouting materials required for installation of the drainage curtain and curtain grouting is also provided.

Seepage control measures will be installed below the following main structures:

- North Dam
- Powerhouse North Transition
- Service Bay
- Powerhouse
- Powerhouse South Transition
- Central Dam
- Spillway North Transition
- Spillway
- Spillway South Transition
- South Dam.

2 Site Conditions

2.1 General

The bedrock within the Gull Rapids area shows a varied degree of metamorphic and cataclastic texture. The site is located within the Canadian Shield physiographic region, near the boundary between the Churchill and Superior provinces. The Churchill-Superior boundary zone is poorly defined and contains cataclastic rocks, which resulted from collision of the two plates during the late Hudsonian orogeny. Downstream or to the east, the bedrock consists of metasedimentary rocks and igneous intrusive rocks, the same type as in the foundations for the Kettle GS. West and upstream of the site, and occasionally within the site area, the bedrock consists of a different group of metasedimentary and igneous intrusive rocks. At the site itself, the bedrock is primarily greywacke gneiss. Diabase dykes crosscut all rock types in the area, varying from short narrow features to continuous wider lineations.

Topographically, relief in the area is low and the morphology is generally planar, except where glacial deposits, abandoned or present day river channels or bedrock highs are encountered.

2.2 Bedrock Conditions

The bedrock conditions at the Keeyask GS site were defined during field and laboratory investigations conducted in 1988, 1991, 1994 and during the 2003 exploration programs. A detailed description of the bedrock conditions is summarized in the 1991 Summer Subsurface Investigation Report [Ref 1]. The Precambrian bedrock within the area consists of granite, greywacke gneiss and diabase. The greywacke gneiss contains bands or sills of granitic material throughout. Diabase occurs as continuous and discontinuous dykes that have intruded the greywacke gneiss and granite. The contacts between the diabase dykes and the host rock vary between tight to open. Typically, they are open at ground surface, and tighten with depth.

All rock types within the Gull Rapids area are typically fresh, strong to very strong, and have widely to medium spaced jointing. Most of the joints appear tight with little or no alteration. Open joints are typically widely to very widely spaced, slightly to faintly altered and may be infilled with clay. Carbonate and chlorite coatings were frequently observed on joint surfaces [Ref 1, 2, 3 and 4].

2.2.1 Powerhouse and North Dam Area Jointing

In the area of the Powerhouse and North Dam, four major and two minor joint sets were identified from a total of 708 orientated core measurements, not including discontinuities described as healed [Ref 2]. The following list summarizes the joint set orientations for the bedrock in this area.

Powerhouse Joint Set	Orientation			Description
	Strike	Dip	Dip Direction	
$J1_{PH}$	198º	1º	288º	Major, sub horizontal
$J2_{PH}$	320º	30º	50º	Major
$J3_{PH}$	60º	23º	150º	Major
$J4_{PH}$	237º	25º	327º	Major
$J5_{PH}$	335º	80º	65º	Minor, sub vertical joint set
$J6_{PH}$	65º	53º	155º	Minor

Notes:
(1) Dip from horizontal. Dip direction is 90° right of strike.
(2) PH - Powerhouse

2.2.2 Central Dam Area Jointing

In the area of the Central Dam, four major joint sets were identified from a total of 25 orientated core measurements, not including discontinuities described as healed [Ref 3]. The following list summarizes the joint set orientations for the bedrock in this area.

Central Dam Joint Set	Orientation			Description
	Strike	Dip	Dip Direction	
$J1_{CD}$	80º	17º	170º	Major
$J2_{CD}$	108º	88º	198º	Major, sub vertical
$J3_{CD}$	273º	70º	3º	Major
$J4_{CD}$	261º	25º	351º	Major

Notes:
(1) Dip from horizontal. Dip direction is 90º right of strike.
(2) CD – Central Dam.

2.2.3 Spillway and South Dam Jointing

In the area of the Spillway and South Dam, two major and two minor joint sets were identified from a total of 364 orientated core measurements, not including discontinuities described as healed [Ref 4]. The following list summarizes the joint set orientations for the bedrock in this area.

Spillway Joint Set	Orientation			Description
	Strike	Dip	Dip Direction	
$J1_{SP}$	25º	32º	115º	Major
$J2_{SP}$	126º	30º	216º	Major
$J3_{SP}$	210º	25º	300º	Minor
$J4_{SP}$	103º	72º	193º	Minor

Notes:
(1) Dip from horizontal. Dip direction is 90º right of strike.
(2) SP - Spillway

3 Water Pressure Tests

Water Pressure Tests (WPT) were performed in selected portions of the drill holes in order to evaluate the permeability of the bedrock at the site. The WPT data was analyzed to provide a related unit of permeability, Lugeon (Lu), that is commonly used in grouting and dam design [Ref 5]. An accurate assessment of the bedrock permeability is required for design of curtain grouting, drainage curtain (pressure relief drains) and construction of dewatering systems.

A total of 571 WPTs were completed and interpreted at the Keeyask GS for the Axis GR-4 development (i.e., 420 in 1991, 53 in 1999 and 98 in 2003). The results were reduced, to consider only the WPT results from drill holes located adjacent to the Principal Axis. Tests completed within the quarries, along the dyke alignments and along Axes GR-3 and GR-5 were ignored, yielding 251 relevant WPT results along Axis GR-4. A summary of the drill holes along Axis GR-4 is shown in Appendix A, Table A1. The location of the drill holes is shown in Figure 3. Stage lengths used during the WPTs range between 2.0 m and 20.41 m, averaging 3.6 m in length. Approximately 95% of all test stage lengths were less than 6 m in length.

The analysis of the WPT results assumed that Lugeons less than 0.1 indicate No Take (Lu = 0) and results between 0.1 and 1.49 are reported as 1 Lugeon. All other Lugeon values are rounded to the nearest number (i.e., 1.5 to 2.4 = 2 Lu). WPT results greater than 100 Lu were assigned a value of 101 Lu, as values in this upper range represent relatively high bedrock permeability. The permeability of the bedrock is based on the interpreted Lugeon value and described as follows [Ref 1].

Bedrock Permeability	**Lugeon Value**
High	> 100
Medium	5 – 100
Low	1 – 5
Practically Impermeable	0

The WPT results along Axis GR-4 are shown in Figure 4. The results include WPTs in which the pump capacity was exceeded during the test.

As shown in Figure 5, approximately 57% of the results indicate values of zero Lu, thus indicating that the foundations are generally tight or practically impermeable. However, approximately 28.2% of the results are between 1 to 5 Lu, 12.4% range from 10 to 100 Lu and 2.4% exceeded 100 Lu.

As shown in Figure 6, the Lugeon values (reported based on the geodetic elevation of the middle of their test zones) for the entire range of WPT along Axis GR-4 are sorted into four elevation ranges. Figure 6 indicates that the trendlines for Lu values between 0 to 5 Lu, and 20 to 101 Lu are diverging. This indicates that the bedrock permeability at the Keeyask GS along Axis GR-4 decreases and the rock becomes tighter with depth.

4 Grout Curtain Design

4.1 General

Seepage control measures, which will be designed to limit uplift/pore pressures, piping potential and leakage quantities below the main structures at the Keeyask GS, may be attained by means of curtain grouting. Curtain grouting shall be installed prior to the installation of the drainage curtain (pressure relief holes) below the concrete structures, and prior to commencement of the placement of impervious materials within the earthen structures. Grouting principles and the preliminary design of curtain grouting for the Keeyask GS is discussed below.

4.2 Single or Multiple Rows of Grouting Curtain

The use of a single or multiple rows of grouting curtain was evaluated during design in terms of the quality of the grouting. The use of single line grout curtains are typically not recommended due to this procedure assuming that geologic conditions will allow for hydraulic connectivity between grout holes [Ref. 6]. However, the results of WPT along the Keeyask GS Axis GR-4 (as discussed in Section 3) indicate that the bedrock permeability is generally low to practically impermeable, with relatively few zones of medium permeability and only rare areas of high permeability. Based on the results of these tests, the use of a single row of grouting curtain which is constructed in accordance with the split spacing principal (i.e., grouting of primary, secondary, and tertiary holes) is expected to be suitable. The grouting hole pattern will need to be closely spaced to provide an overlap of grout penetration as it travels through the joint apertures to prevent the occurrence of ungrouted joints between drill holes.

4.3 Grout Curtain Location

Based on the current understanding of the Keeyask GS site, curtain grouting will consist of a single row curtain located below the concrete structures, and a single row curtain below the North, Central and South Dams.

The grout holes will be located about 3 m inside the upstream face of the concrete structures (see Figure 2), while under the earthfill structures (i.e., North, Central and South Dams); the grout holes will be located beneath the impervious core along the centre line axis of the Principal Structures (see Figure 2).

The curtain grouting alignment, shown in Figure 3, is approximately 2,700 m in length, extending from the North Abutment to the South Abutment of the Nelson River.

4.4 Depth of Grout Curtain

Guidelines provided in the Memorandum GN-4.1.3, "Geotechnical Design Criteria [Ref 7] indicate that the depth of the grout hole should be at least 70% of the design head, i.e. the difference between the headwater (FSL el 159.0 m) and the average tailwater level (el 140.5 m), or at least 6 m deep (vertically). The design head is determined to be approximately 18.5 m. Therefore based on the hydraulic head, the vertical depth of the grout hole should be at least 13 m (70%×18.5 m). The depth of the grout curtain will also be influenced by the geologic conditions. Based on our understanding of the bedrock conditions in the Project Area, it is anticipated that all grout holes will bottom in sound, relatively impervious rock.

The Powerhouse Intake and the Spillway structures will be founded at elevations 128.0 m and 134.9 m respectively, where the bedrock foundations are anticipated to be tight with little grout take [Ref 2 and 3]. The impervious core of the North, Central and South dams will be founded at bedrock surface and some grout takes are anticipated in these areas.

For Axis GR-4, the majority of the investigation holes were concentrated in the proposed Powerhouse and Spillway areas. The WPT results in these areas are reviewed separately because of the complex geology at the site.

Powerhouse Area

As shown in Figure 7 (Plot 1), higher Lugeon values were interpreted during the 1991 investigation and they indicate that stage lengths of higher grout takes, would most likely be encountered. However, the majority of the boreholes advanced in 1991 are generally located downstream of the Axis of Principal Structures for GR-4.

A closer observation of the Lugeon values along the current alignment of Axis GR-4 indicates that the bedrock is generally tighter and less permeable than the previous Powerhouse location investigated in 1991. However, Figure 7 (Plot 2) also shows that stage lengths of higher grout takes (ranging between 10 to 101 Lu), are anticipated in the Powerhouse South Transition area (Boreholes 03-015 and G-0008). These stages may require thicker grout mixes and/or a sanded grout mixture to limit migration of the grout while still maintaining interconnectivity with adjacent grouting holes.

As shown in Figure 7 (Plot 2), the Lugeon values converge to 0 and 1 Lu in the Powerhouse area below el 113.0 m, representing a practical depth at which grouting will probably not provide a benefit. Thus a grout hole length of 16.0 m (15 m vertically) for grouting in the Powerhouse area is recommended.

For further discussion on the bedrock conditions in the Powerhouse area, reference Memorandum GN-1.5.4, "Review of Bedrock Conditions in the Powerhouse Area" [Ref 2].

Spillway Area

As shown in Figure 8, the Lugeon values, interpreted within the grouting zone, indicate that the bedrock is practically impermeable. The four highest Lugeon values shown in Figure 8 (Plot 1) were interpreted in the 1991 investigation holes (G-0045, G-0047 and G-0049) located approximately 150 to 200 m downstream of the Axis of Principal Structures.

A closer observation of the Lugeon values along the current alignment of Axis GR-4 indicates that the bedrock is generally tighter and less permeable than the previous Spillway location investigated in 1991 (Figure 8, Plot 2). The Lugeon values observed below the base elevation of the grout line generally range between 0 to 1 Lu. For most rock grouting applications, a target permeability range of 1 to 2 Lugeons is achievable. The grouting of bedrock with a permeability value lower than 1 or 2 Lugeons is generally not practical. However, due to the limited number of boreholes advanced along the Axis of Principal Structures in the Spillway area, it is deemed prudent to at least execute pilot/exploratory holes – 1st Stage primary grout holes at 12 – 24 m spacings beneath the Spillway Structure.

A Lugeon value of 6 was recorded in Borehole 03-027 (in the Spillway South Transition Area) in a test zone between el 125.50 m to el 120.54 m. Therefore it is prudent to extend the depth of the exploratory grout holes to el 120.5 m to intercept this zone. As shown in Figure 8 (Plot 2), the Lugeon values converge to 1 and 2 Lu in the Spillway area below el 122.0 m, representing a practical depth at which grouting will probably not provide a benefit. Thus an exploratory grout hole length of 16.6m (15.6 m vertically) in the Spillway area is recommended. If needed, curtain grouting would then consist of a single row curtain 14 m in length (13 m vertically) beneath the Spillway North Transition and the Spillway Structures and then consist of a single row curtain 16.6 m in depth (15.6 m vertically), located beneath the Spillway South Transition.

For further discussion on the bedrock conditions in the Spillway area, reference Memorandum GN-1.5.5, “Review of Bedrock Conditions in the Spillway Area” [Ref 4].

North, Central and South Dam Areas

Curtain grouting beneath the earthfill structures would consist of a single row curtain 13.0 m deep (i.e., based on an 18.5 m design head).

4.5 Grout Curtain Hole Orientation

The orientation of the proposed single row grout curtain is designed to maximize the number of joints intersected by the grouting holes. The orientation of the grout holes varies along the length of the Principal Axis. Grout holes with a 270º/70º (trend/plunge) is proposed below the Powerhouse Structure and the North Dam, holes with a 315º/70º (trend/plunge) for the Central Dam, and 240º/70º (trend/plunge) for the Spillway and the north section of the South Dam. The orientation of the grout holes for the river portion of the South Dam is set at 270º/60º (trend/plunge) for South Dam (Figure 2).

Due to the limited jointing information along the earthfill structures, it is deemed prudent to advance additional investigation holes along the curtain axis – 1st Stage primary grout holes at 36 m spacings, once the cofferdams are in place and the bedrock surface is exposed. The vertical depth of the exploratory holes can be limited to a depth of 15 m (i.e., proposed depth of curtain grouting plus 2 m).

4.6 Foundation Preparation

Prior to the commencement of the grouting operation, any loose materials will be removed, and the rock surface will be cleaned so that surface leaks can be identified. Dental concrete and/or reinforced and anchored concrete grout cap will be placed, as required.

4.7 Grouting Materials and Grout Mixes

The cement grout mixture will be designed to suit the particular conditions encountered in the rock for each drill hole. The cement grouting mixtures may include, but are not limited to combinations of some the following materials:

- **Water -** Mixing water for grout will be fresh and clean, free from oil, acid, alkali, salts, organic matter or other deleterious substances.
- **Cement** - such as Low alkali Type 10 Portland cement, Type 20, or Micro Cement (for joint aperture less than 50 microns).
- **Supplementary Cementitious Materials (SCM)** - such as Fly Ash and Silica Fume.
- **Admixtures** - such as pumping aids, superplasticizers, water reducer and or retarder agents (each one adjusts grout properties for specific applications).
- **Additives** - such as Bentonite (of the type that disperses easily in water). Bentonite improves the stability under pressure and reduces shrinkage and bleeding of the grout.
- **Sand** - (if required) will consist of clean, dry, durable stone particles, free from clay lumps and deleterious materials.

Whenever sand is added to the grout mixture, it will conform to the following grading requirements.

Standard Sieve Size		Percentage Passing (by weight)
No. 8	2.36 mm	100
No. 16	1.18 mm	95 – 100
No. 30	0.60 mm	60 – 85
No. 50	0.30 mm	30 – 50
No. 100	0.15 mm	10 – 30
No. 200	0.075 mm	0 - 5

Fly ash is used as a pozzolan to replace up to 25% of the cement where cheaper fly ash is available. Fly ash can also be used with Type 20 cement to produce a grout which is better resistant to aggressive groundwater.

Silica Fume can be used up to 10% by weight of cement to produce a stronger, less permeable grout with enhanced stability and resistance to pressure filtration.

Selection of Cement Type

The flow of the grout is influenced by the gradation of all its components. The maximum particle size of cement is relevant when considering the flow of the grout through the fissure width (i.e. joint aperture). Cements are available in a wide range of particle sizes ranging from fine to microfine cements. The D_{95} particle size and practical grouting range for various cement grouts are provided below.

Cement Product	D_{95} of Grains (microns)	Practical Joint Aperture Range (microns) (1)
Portland Cement (Type 10, 20)	80 to 100	>400
High Early Strength Cement	40 to 60	>200
Microfine Cement	10 to 12	>50

Note:
(1) General rule of thumb is that the aperture be greater than 4 x D_{95} of the selected cement product.

For reasons of low cost, ordinary Portland cement is always preferred on cement grouting projects, provided that the joint aperture is greater than 400 microns. For a nominal extra cost, high early strength Portland cement will provide enhanced penetration for joint apertures down to 200 microns, due to its lower D_{95} size. For cement grouting of rock joint apertures down to 50 microns, microfine cement is recommended [Ref 5]. Microfine cement is normally a proprietary blend of finely ground blast furnace slag and Portland cement.

The average joint aperture must be determined in order to properly select the cement product for the Keeyask site. The average joint aperture along Axis GR-4 can be estimated using the following equation by Snow (1968) that relates hydraulic conductivity (estimated from the Lugeon values) to fracture spacing and aperture [Ref 8]:

$$K = \frac{\gamma_w N b^3}{6\nu}$$

Where,

K = Hydraulic conductivity of the rock (m/s)
γ_w = Unit weight of water (kN/m^3)
b = Joint aperture (m)
ν = Dynamic viscosity of water
N = Joints per meter

As discussed in Section 3, the WPT results along Axis GR-4 ranged between 0 to 101 Lu (pump capacity). Based on the Rock Quality Designation (RQD), the average joint spacing is approximately 285 mm. The estimated average joint aperture (b), for Axis GR-4, is shown below for various Lugeon values.

	Lugeons					
	1 ($1.3x10^{-7}$ m/s)	2 ($2.6x10^{-7}$ m/s)	4 ($5.2x10^{-7}$ m/s)	8 ($1.0x10^{-6}$ m/s)	15 ($2.0x10^{-6}$ m/s)	30 ($3.9x10^{-6}$ m/s)
Joint Aperture in microns	270 to 300	340 to 380	430 to 470	535 to 595	675 to 750	840 to 945

Note:
(1) The ranges shown are based on the dynamic viscosity of water, for water temperature varying between 16° and 4° Celsius (i.e., 270 to 300).

Based on the two tables above, the use of Normal Portland Cement for curtain grouting at the Keeyask GS is considered to be suitable for grouting the WPT zones with Lugeon values greater than 2 Lu. Although not currently recommended, should it be required to grout the zones with Lu values equal to or less than 2 Lu, consideration should be given to the use of high early strength Portland cement. As mentioned previously, high early strength Portland cement will provide enhanced penetration for joint apertures down to 200 microns, due to its smaller D_{95} size.

Although it is possible that Normal Portland Cement can be passed into joints with apertures less than 400 microns by using a thinner watery grout mix, the additional water content in the grout mix will likely reduce the durability of the grout mix. In addition, joint entry by the grout particles may be blocked if there is sufficient number of D_{max} particles entering the joint simultaneously.

The use of Microfine cement was considered as an alternative to normal Portland cement or high early strength Portland cement due to its effectiveness in penetrating into joints that are impenetrable by the other cements. Microfine cement suspension grouts provide a viable alternative to closely spaced grouting holes since they:

(a) possess sufficiently small particle sizes that enable penetration into reasonably small voids

(b) exhibit relatively low viscosity and yield strength properties to enable injectability

(c) are compatible with a variety of additives used to manipulate set time and solids concentration of the mix

(d) manifest sufficient compressive strength and permeability reduction in most grouted formations [Ref. 9].

However, Microfine cements are more expensive than ordinary Portland cements due to their high manufacturing cost. Based on the estimation for Wuskwatim Generating Station, the price of the standard cement delivered to site is in the order of [redacted] However, the

price for the Microfine cement is in the order of [redacted] (based on $[redacted] assuming the Microfine cement is 1.4 tonne per cubic meter). This is 6 times of the price of the ordinary Portland cement. Assuming the Microfine cement takes are the same as that of Portland cement (408 tonnes), the cost induced by replacing normal Portland cement by Microfine cement will increase grouting cost by approximately [redacted] This is not considered to be a cost-effective option due to the increased cost and there is no assurance that the quality of the grout curtain will be significantly increased.

Grout Starter Mix

For each application of cement grout, it is necessary to select a starter mix, and then as grouting of the stage length proceeds, decide whether to thicken the mix during grouting. Choosing a good starter mix will come from experience after several primary holes have been grouted. Case studies have shown that grout mixtures thinner than 3:1 water to cement by volume have not proved to be durable or are suspect. Therefore, the thinnest grout mix for Keeyask is set at 3:1, which is a reasonable compromise between penetration and durability [Ref 5]. Grouting for each stage length should always begin using the thinnest mix and be gradually thickened as required. Jumping a mix (i.e. going from 3:1 to 1:1) should be avoided, as this may block the stage length prematurely. A guideline for starting grout mix is shown below.

Conditions	Water to Cement Ratio (w/c by volume[(2)])
WPT Results: 0 to 5 Lu [(1)]	3:1
WPT Results: 5 to 30 Lu	2:1
WPT Results: > 30 Lu	1:1
Joints greater than 2.5 mm wide	0.8:1
	0.6:1

Notes:

(1) For enhanced penetration of joint apertures down to 200 microns, the starter mix should begin with high early strength cement at 3:1, followed by normal cement at 3:1, and then 2:1 and so on.

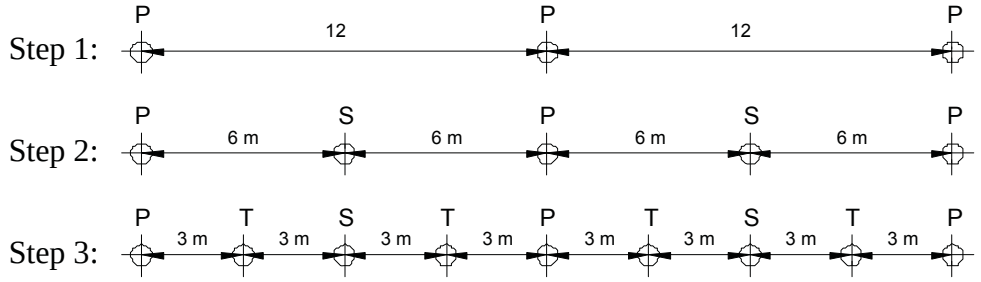
(2) Bentonite and other admixtures should be added to ensure <4% bleed and 34 – 38 seconds Marsh viscosity for w/c = 3/1 & 2/1, and <2% bleed and 38 – 45 seconds Marsh viscosity of w/c=1/1; 0.81 & 0.6/1.

4.8 Grouting Principles

In general, grouting is the process of injecting cementitious liquid (grout) into a controlled or limited space (stage length) along a drill hole. The grout mix is gradually thickened until the zone receiving the grout meets or exceeds the refusal criteria (acceptance level). This process is repeated in a neighbouring drill hole until the material between the drill holes is, in essence, relatively impermeable (i.e., lugeon value equal to or less than 1). Additionally, pressure is typically applied at the collar of the drill hole to promote grout migration, and to combat excess hydraulic heads (head differential between river water level and base of foundation), which may exist within the drill hole.

Typically, the grouting procedure used in the curtain grouting is the split spacing principle. The split spacing principle is the principle or process of progressively closing a grout curtain by means of drilling and grouting additional holes (if required) midway between holes which have previously been drilled and grouted, i.e., secondary holes are drilled and grouted on either side of any primary holes which exceed the refusal criteria. If the grout takes continue to exceed the refusal criteria in the secondary holes, the process is repeated using tertiary holes on either side of the secondary holes, as necessary until closure is achieved. In this way, final spacing of grouted holes after primaries are executed will be 12 m, after secondaries 6 m, after tertiaries 3 m, and so on as shown in the following diagram.

Split Spacing Principle



Notes:

P: Primary Holes spaced at 12 m on center;
S: Secondary Holes spaced at 6 m on center;
T: Tertiary Holes spaced at 3 m on center;
Grouting

The ground conditions at the Keeyask GS are suitable for using the Stop Grouting Method. The Stop Grouting Method is defined as the drilling of a hole to its full depth in one continuous operation and the subsequent washing and grouting of the hole in sections (stage lengths) from the bottom upward by means of a packer that is set at progressively shallower depths.

Grout migration is controlled, if necessary, by the gradual thickening of the grout mixtures during injection. In general, the grout mixtures at the Keeyask GS will range from water to cement ratios (w:c) between 3:1 (by volume) and 0.6:1, using normal Portland cement. Admixtures such as Bentonite (2 to 3% by weight on cement) and sand (5 – 10% by weight on cement) if at all needed, and super-plasticizers may be required to provide a full range of pumpable and thick grout mixtures, with limited bleed water and improved void or back filling properties. The sand size typically ranges between 1 mm and 0.075 mm, with less than 5% by mass smaller than 0.075 mm.

Grouting of each stage length is continued until the refusal criterion is attained. In general, the grouting of any stage length commences with the thinnest grout mixture (3:1 w:c by volume) and is continued until the stage length practically/virtually refuses to take grout at the maximum grouting pressure specified, for a period of 10 minutes (i.e., refusal criteria).

Grout takes exceeding the refusal criteria are gradually thickened and the procedure is repeated until either the refusal criteria is attained, or the stage length continues to accept the thickest practical grout mixture at the maximum grouting pressure specified, for a period of 10 minutes. The maximum thickness of the grout mixture will be determined in the field during a trial mix program. Stage lengths that do not attain the refusal criteria are abandoned and the process recommences after the injected grout sets or hardens, or a sanded grout mixture is injected into the stage length to limit the migration of the grout. Typically, an additional hole is required on either side of the grouted hole, which does not attain refusal, (i.e., the split spacing principle).

4.9 Curtain Grouting Installation Quantities

Depending on laminar or turbulent flow, hydraulic theory suggests that pressure losses in the order of 40% to 80% occur in the first 1 m radius of the grouting hole (while the grout is still flowing). This, combined with the already low permeability of the rock mass at Keeyask suggests that the communication of the grout between Basic Pattern (i.e. P, S, T holes)spaced at 6 m on centre may not be achievable. Therefore, in order to achieve closure of the grout curtain, it is prudent to set the final spacing of Basic Pattern (Primary, Secondary and Tertiary) grout holes at 3 m on centre.

The number of grout holes, depth of hole, and quantity of grout mixture required for curtain grouting are based on the WPT results, as shown in Figure 5. The assumptions used for estimating grouting quantities are as follows:

- 72% of the WPT results are between 0 Lu and 1 Lu
- 18% are between 2 and 10 Lu
- 10% are greater than 10 Lu
- WPT results indicate that the rock becomes tighter with depth.

Drilling and cement requirements for curtain grouting at the Keeyask GS have been estimated using the results from the WPTs, and assuming two packer settings per hole during grouting. A total of 1,327 grout holes are estimated consisting of 928 grout holes required for the Basic Pattern (Primary, Secondary, and Tertiary), 367 Quaternary and 32 Quinary grout holes. The total length of drilling is estimated to be approximately 17,100 m, with approximately 408 tonnes of Portland cement. In addition, approximately 10.1 tonnes of Bentonite and 2.0 tonnes of sand, and approximately 664 hours of packer setting time for 1327 WPTs, and about 4,740 packer settings are required for curtain grouting.

The amount of packer setting time for testing, and the number of packer settings for grouting, were estimated using the WPT results. The results shown in Figure 5 indicate that, over 72% of the zones tested, the rock conditions were tight (i.e., 0 to 1 Lu). Therefore, the total hours for WPTs, and the total number of packer settings required for grouting, are based on the assumption that the grout take will be negligible in 72% of the packer settings.

Exploratory investigative work to test the effectiveness of the grouting has been estimated at 180 m of NX coring.

Refer to Appendix A, Table A2 for a complete summary of the grouting quantities estimate.

5 Rock Surface Treatment

The basic objectives of rock surface treatment are:

- to remove overhangs and steep areas to avoid low stress zones within the fill and hydraulic fracture potential,
- to obtain a good bond between the impervious core materials and the foundation,
- to defend against erosion of embankment materials into the foundations by filling and covering surface cracks in the foundation,
- to remove erodible, weak, unstable, compressible, loose or pervious materials to ensure a foundation of adequate strength and appropriate permeability.

Although the bedrock along Axis GR-4 is relatively sound, discontinuities such as potholes, low lying irregularities, joints, faults, minor weathered zones and shear zones can create irregularities requiring treatment. The reader will note that potholes in igneous rock is interpreted in terms of the process of abrasion of the bedrock by rotary currents inside the potholes, and vibration of pebbles and stones by the water, that gradually wore the holes deeper and deeper into the bedrock.

All unsuitable materials will be removed and the foundation surface will be shaped to provide a sufficiently regular surface such that earthfill, concrete, or grout can be placed. This may require the removal of unsuitable and loose materials by handwork, picking, brooming, water jetting, or compressed air. Ponding water from the cleaning operations must also be removed.

The rock foundation will be shaped by filling in local low lying irregularities or depressions with concrete or grout, and by removing points, promontories, overhangs and steep surfaces. Shaping the foundation rock will provide a sufficiently regular surface such that earthfill can be placed. If shaping requires blasting, proper blasting procedures will be used to ensure that the permeability, strength and integrity of the rock is not adversely affected.

Rock surfaces that flake or disintegrate rapidly on exposure will be protected or covered immediately with embankment material or concrete.

Treatment of the exposed rock surface after the removal of unsuitable overlying materials depends on the type of rock and the irregularities present. Typical foundation treatment methods for Keeyask are shown in Figure 9, and may consist of treatment programs as described below.

Dental Concrete

Dental concrete will be used to fill or shape holes, grooves, extensive areas of vertical surfaces, and sawteeth created by bedding planes, joints, and other irregularities such as previously cleaned depressions, shear zones, large joints or buried channels.

Formed dental concrete will be used to fillet steep slopes and fill overhangs.

Shaping the foundation with dental concrete may be appropriate when the smoothing of irregularities requires excessively large quantities of excavation, or when it requires blasting that may damage the foundation.

Blanket Grouting

Blanket Grouting will be used to treat selected geologic defects such as fractures and shear zones.

Blanket Grouting is a procedure in which relatively closely spaced and usually shallow holes are drilled and grouted for the purpose of reducing the permeability of the upper portions of the bedrock under the site of an embankment dam.

Slush Grouting

Slush Grouting will be used to treat narrow surface cracks and surface irregularities.

Slush Grouting is the application of cement slurry to surface rock as a means of filling narrow surface cracks and surface irregularities or by “brooming” to prevent slaking. The cement slurry will consist of a neat cement grout (cement and water) or a sand-cement grout (sand, cement and water).

6 Drainage Curtain Design and Quantity

Dams are subject to seepage through their foundation and abutments. Seepage may result in excess fluid pressures or uplift pressures beneath elements of the dam, thereby becoming a safety concern with respect to the stability of the dam. Pressure-relief wells are an integral part of a dam’s seepage control system. These wells are generally designed to be free flowing and placed to intercept zones in dam foundation materials that are sensitive to elevated fluid pressures. Pressure-relief wells reduce excess fluid pressures by allowing water to be safely

discharged from the zone affected by the excess fluid pressure (U.S. Army Corps of Engineers [USACE], 1992).

The foundation drainage system for Keeyask GS consists of holes drilled downstream of the grout curtain designed to intercept and control seepage through the bedrock foundation beneath the Powerhouse structure so as to reduce uplift pressures under the structure. The drainage curtain (pressure relief), in the form of drain holes drilled into the foundation of the draft tube, will be installed to lower excess hydraulic pressures within the rock mass, and along the concrete/bedrock contact during maintenance of an unwatered unit. Foundation drains will be 76 mm in diameter and spaced at 3 m centers. They will be drilled 14 m into rock along the wall of the draft tube elbow, and 3 m into rock below the invert of the draft tube. Sleeves or collars, installed prior to placement of the concrete, will expedite drilling of the drainage holes. Over 294 permanent pressure relief holes totalling over 1,344 m of rock drilling is estimated.

The water pressure tests carried out in the Spillway area have shown that the bedrock underneath the spillway concrete structure is impervious or tight, as shown in Figure 8. Consequently, it is expected that seepage through the bedrock will be minor. Therefore, no drainage gallery or pressure relief drains located along the spillway concrete structure have been considered necessary, and the structures were designed to accommodate full uplift pressures.

In addition, for the stability of rock excavations, 50 mm diameter temporary drains will be installed in the Powerhouse and Spillway areas. It is estimated that approximately 201 temporary drain holes, totalling 1,206 m of rock drilling will be required.

A complete summary of the estimated pressure relief quantities is shown in Appendix A, Table A3.

7 Environmental and Special Considerations

The use of grease, rod dope, drill mud or other lubricants on the drill rods, as an aid to drilling will not be permitted during the grouting program.

Any weathered and/or weak rock along the grout curtain alignment will be excavated and a grout cap will be installed.

The faults lines and shear zones identified in the Gull Rapids area are ancient, inactive and healed. They are not anticipated to be problematic. However, additional grout holes will be advanced should such features be identified during the grouting program. This will ensure that the feature is suitably handled.

Very irregular foundation profiles are anticipated along the North, Central and South Dams. Therefore, it is advisable to flatten any small knobs along the line of the grout curtain to give a fairly uniform surface for grouting.

As with concrete, the leachates (from the grout mix) will be trapped by the hydration of the Portland cement and the pozzolanic interaction. However, during the construction process any bleed water from the grout may have temporary environmental consequences. Therefore, any bleed water identified at the rock surface will be prevented from running off into adjacent water courses.

It is prudent to delay the placement of any dental concrete required for foundation shaping to facilitate the placement of fill, until after the grouting program is finished. Otherwise the dental concrete may become dislodged during grouting.

Earthfill placement may not be permitted over dental concrete for a minimum of 72 hours after concrete placement to allow the concrete time to develop sufficient strength to withstand the stress caused by placing earthfill.

Prepared By

P.R. Pantel

Attach
References
Appendix A
Figures 1 – 9

PRP:spa

References

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3. Acres Manitoba Limited, Memorandum G-1.4.3, “Gull Generating Station, Stage 4 Studies, “Investigation Program, Bedrock Geology, Review of Bedrock Conditions, GR-3 Axis – North Abutment”, Manitoba Hydro File 00195-11610-0012.

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5. Houlsby, A.C., “Construction and Design of Cement Grouting”, John Wiley and Sons, Inc., 1990.

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9. Raymond J. Krizek, Stanley F. Pepper, Slurries in Geotechnical Engineering, The Twelfth Spencer J. Buchanan Lecture, 2004. https://ceprofs.civil.tamu.edu/briaud/buchanan%20web/Lectures/Twelfth%20Buchanan%20Lecture.pdf

Figures

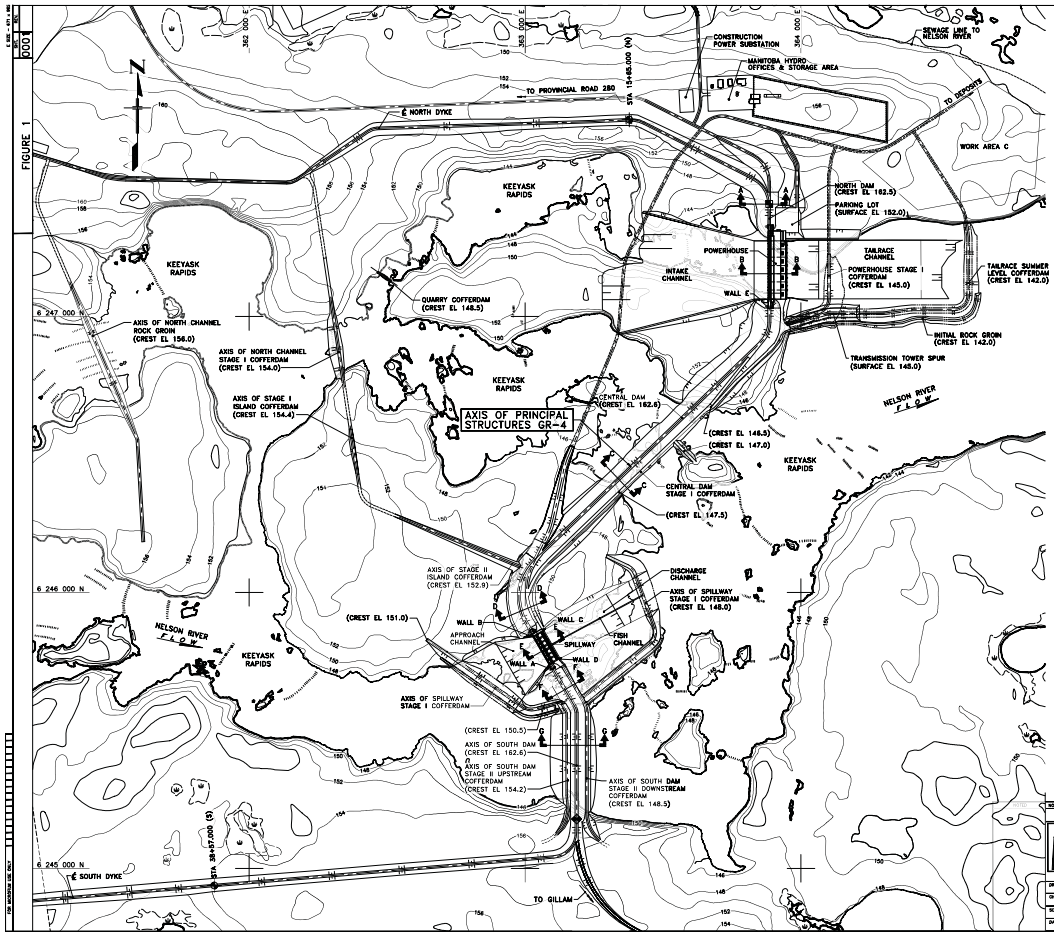


FIGURE 1 — 001

LEGEND:

- SHORELINE (SEE NOTE 6)
- GROUND SURFACE CONTOUR (SEE NOTE 7)
- STREAM
- SWAMP
- RAPIDS
- ACCESS ROAD
- COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
- COFFERDAM REMOVED
- REFERENCE POINT (SEE NOTE 10)
- REFERENCE POINT (SEE NOTE 11)
- REFERENCE POINT (SEE NOTE 12)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0 FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 478.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT; FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. SEE FIGURE 2 FOR SECTIONS.

SCALE 1:5000 — 100 0 100 200 300 400 500 METRES

NO.	DATE	REVISION	BY	CHK	APP.
0	2009 07 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GH-4.2.2.7, REV 0			

ACRES MANITOBA LIMITED

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4 GROUTING AND PRESSURE RELIEF PRINCIPAL STRUCTURES GENERAL ARRANGEMENT AND LOCATION OF SECTIONS

DRAWN
SHEIN
SCALE AS SHOWN
DATE

FIGURE 1 — DWG. 001 — REV.

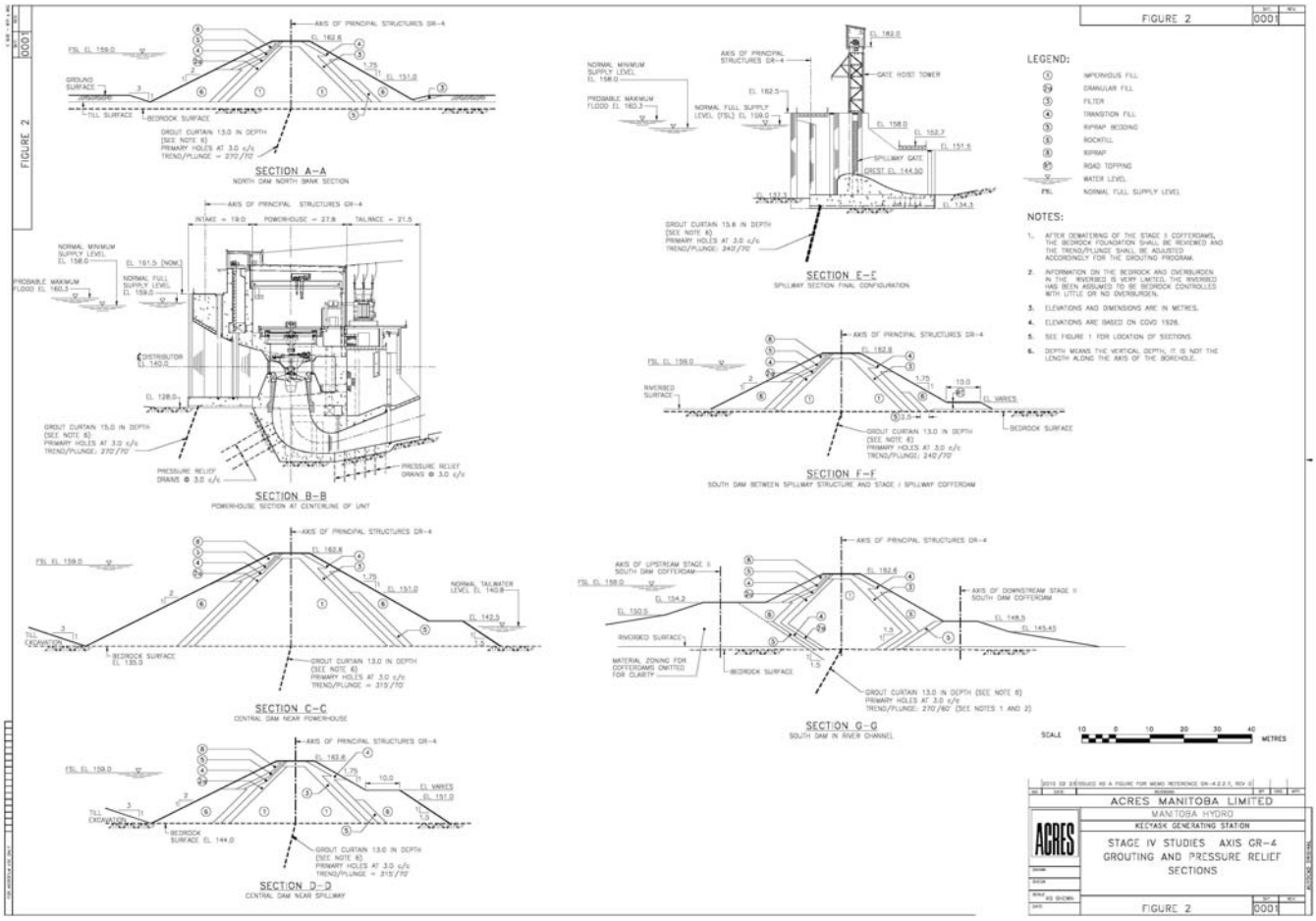


FIGURE 2
0001
SECTION A–A
NORTH DAM NORTH BANK SECTION
SECTION B–B
POWERHOUSE SECTION AT CENTERLINE OF UNIT
SECTION C–C
CENTRAL DAM NEAR POWERHOUSE
SECTION D–D
CENTRAL DAM NEAR SPILLWAY
SECTION E–E
SPILLWAY SECTION FINAL CONFIGURATION
SECTION F–F
SOUTH DAM BETWEEN SPILLWAY STRUCTURE AND STAGE I SPILLWAY COFFERDAM
SECTION G–G
SOUTH DAM IN RIVER CHANNEL
LEGEND:
IMPERVIOUS FILL
GRANULAR FILL
FILTER
TRANSITION FILL
RIPRAP BEDDING
ROCKFILL
RIPRAP
ROAD TOPPING
WATER LEVEL
FSL NORMAL FULL SUPPLY LEVEL
NOTES:
1. AFTER DEWATERING OF THE STAGE I COFFERDAMS, THE BEDROCK FOUNDATION SHALL BE REVIEWED AND THE TREND/PLUNGE SHALL BE ADJUSTED ACCORDINGLY FOR THE GROUTING PROGRAM.
2. INFORMATION ON THE BEDROCK AND OVERBURDEN IN THE RIVERBED IS VERY LIMITED. THE RIVERBED HAS BEEN ASSUMED TO BE BEDROCK CONTROLLED WITH LITTLE OR NO OVERBURDEN.
3. ELEVATIONS AND DIMENSIONS ARE IN METRES.
4. ELEVATIONS ARE BASED ON CGVD 1928.
5. SEE FIGURE 1 FOR LOCATION OF SECTIONS.
6. DEPTH MEANS THE VERTICAL DEPTH. IT IS NOT THE LENGTH ALONG THE AXIS OF THE BOREHOLE.
SCALE 10 0 10 20 30 40 METRES
ACRES MANITOBA LIMITED
MANITOBA HYDRO
KEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
GROUTING AND PRESSURE RELIEF
SECTIONS
FIGURE 2
0001

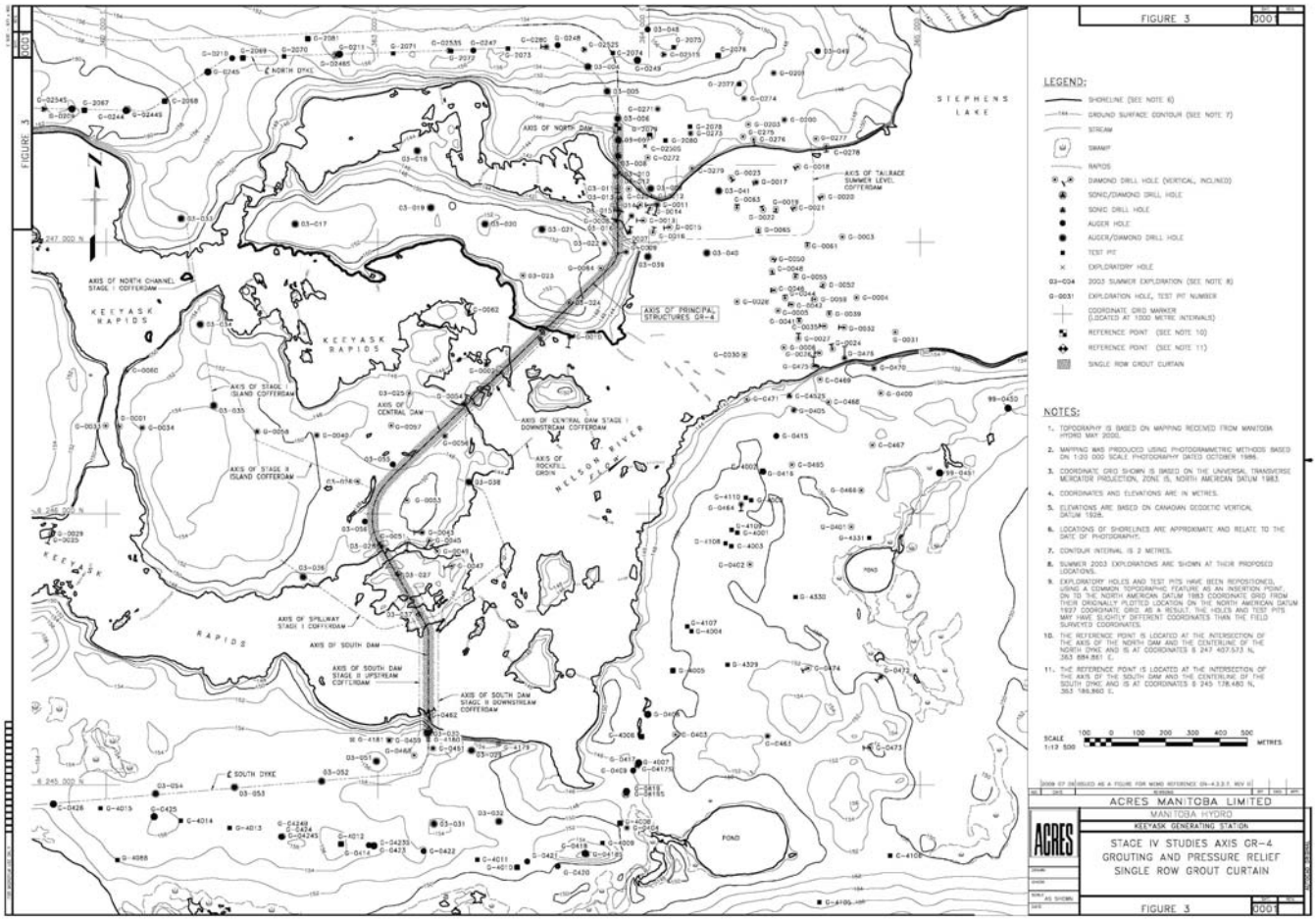
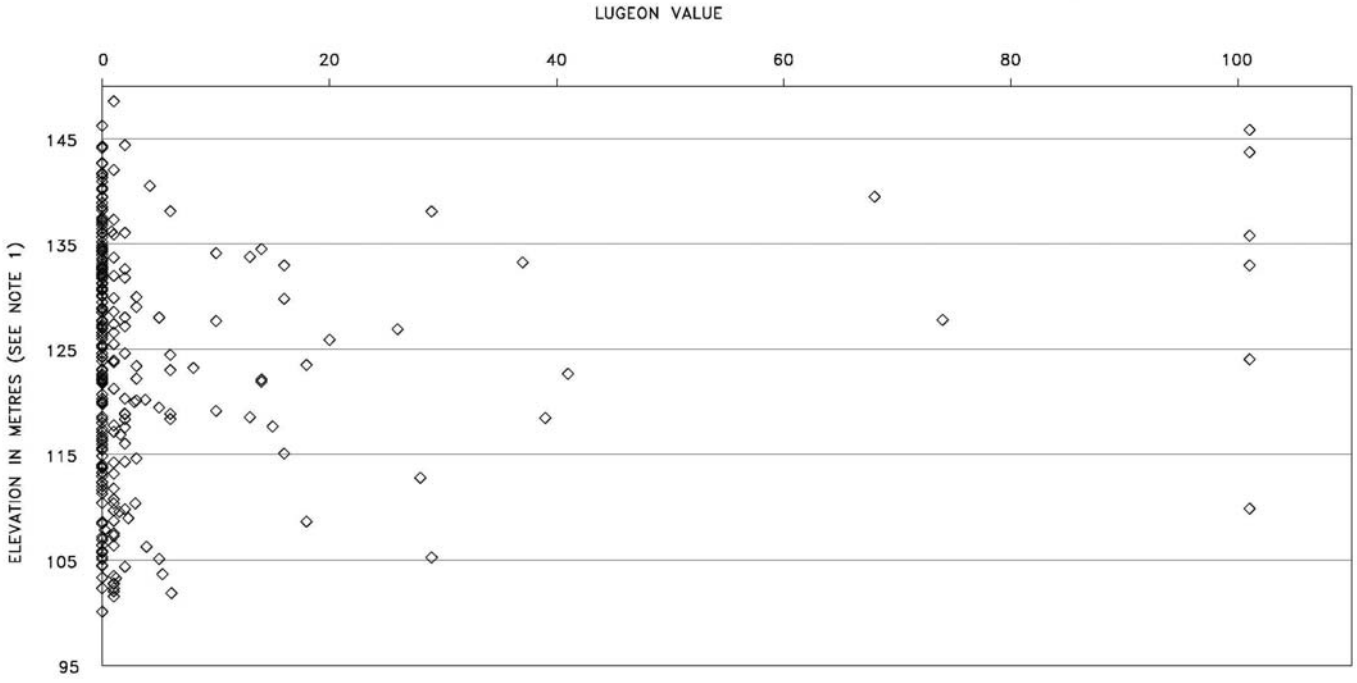


FIGURE 3
2001
LEGEND:
SHORELINE (SEE NOTE 6)
GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
DIAMOND DRILL HOLE (VERTICAL, INCLINED)
SONIC/DIAMOND DRILL HOLE
SONIC DRILL HOLE
AUGER HOLE
AUGER/DIAMOND DRILL HOLE
TEST PIT
EXPLORATORY HOLE
03-024 2003 SUMMER EXPLORATION (SEE NOTE 8)
G-0031 EXPLORATION HOLE, TEST PIT NUMBER
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
SINGLE ROW GROUT CURTAIN
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. SUMMER 2003 EXPLORATIONS ARE SHOWN AT THEIR PROPOSED LOCATIONS.
9. EXPLORATORY HOLES AND TEST PITS HAVE BEEN REPOSITIONED, USING A COMMON TOPOGRAPHIC FEATURE AS AN INSERTION POINT, ON TO THE NORTH AMERICAN DATUM 1983 COORDINATE GRID FROM THEIR ORIGINALLY PLOTTED LOCATION ON THE NORTH AMERICAN DATUM 1927 COORDINATE GRID. AS A RESULT, THE HOLES AND TEST PITS MAY HAVE SLIGHTLY DIFFERENT COORDINATES THAN THE FIELD SURVEYED COORDINATES.
10. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES E 247 407.573 N, 363 884.861 E.
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES E 243 178.480 N, 363 186.860 E.
SCALE 1:10 000
100 0 100 200 300 400 500 METRES
ACRES MANITOBA LIMITED
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS CR-4 GROUTING AND PRESSURE RELIEF SINGLE ROW GROUT CURTAIN
FIGURE 3
2001
KEEYASK RAPIDS
STEPHENS LAKE
NELSON RIVER
POND
AXIS OF NORTH DAM
AXIS OF NORTH CHANNEL STAGE I COFFERDAM
AXIS OF STAGE I ISLAND COFFERDAM
AXIS OF STAGE II ISLAND COFFERDAM
AXIS OF CENTRAL DAM
AXIS OF CENTRAL DAM STAGE I DOWNSTREAM COFFERDAM
AXIS OF ROCKFILL DRAIN
AXIS OF PRINCIPAL STRUCTURES CR-4
AXIS OF TAILRACE SUMMER LEVEL COFFERDAM
AXIS OF SPILLWAY STAGE I COFFERDAM
AXIS OF SOUTH DAM
AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM
℄ NORTH DYKE
℄ SOUTH DYKE
E 247 000 N
E 245 000 N
FIGURE 3

FIGURE 4 | SHT. 0001 | REV.



NOTES:

1) TEST ZONES ARE PLOTTED AT THE MIDPOINT OF THE TEST ZONE.
2) ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
3) TEST ZONE LENGTHS ALONG CORE AXIS VARIES BETWEEN 1.34 AND 20.41 METRES, AVERAGING 3.6 METRES.
4) A TOTAL OF 251 WPT WERE UNDERTAKEN IN THE VICINITY OF AXIS GR-4.
5) WPT EXCEEDING PUMP CAPACITY, ARE ASSIGNED A VALUE OF 101.
6) FOR A LIST OF DRILL HOLES USED IN THIS ANALYSIS REFERENCE APPENDIX A, TABLE A1 .

NO.	DATE	REVISIONS	BY	CKD.	APP.
	2009 07 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.7, REV. 0			

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
GROUTING AND PRESSURE RELIEF
LUGEON VALUE ELEVATION

DRAWN | CHECK | SCALE AS SHOWN | DATE

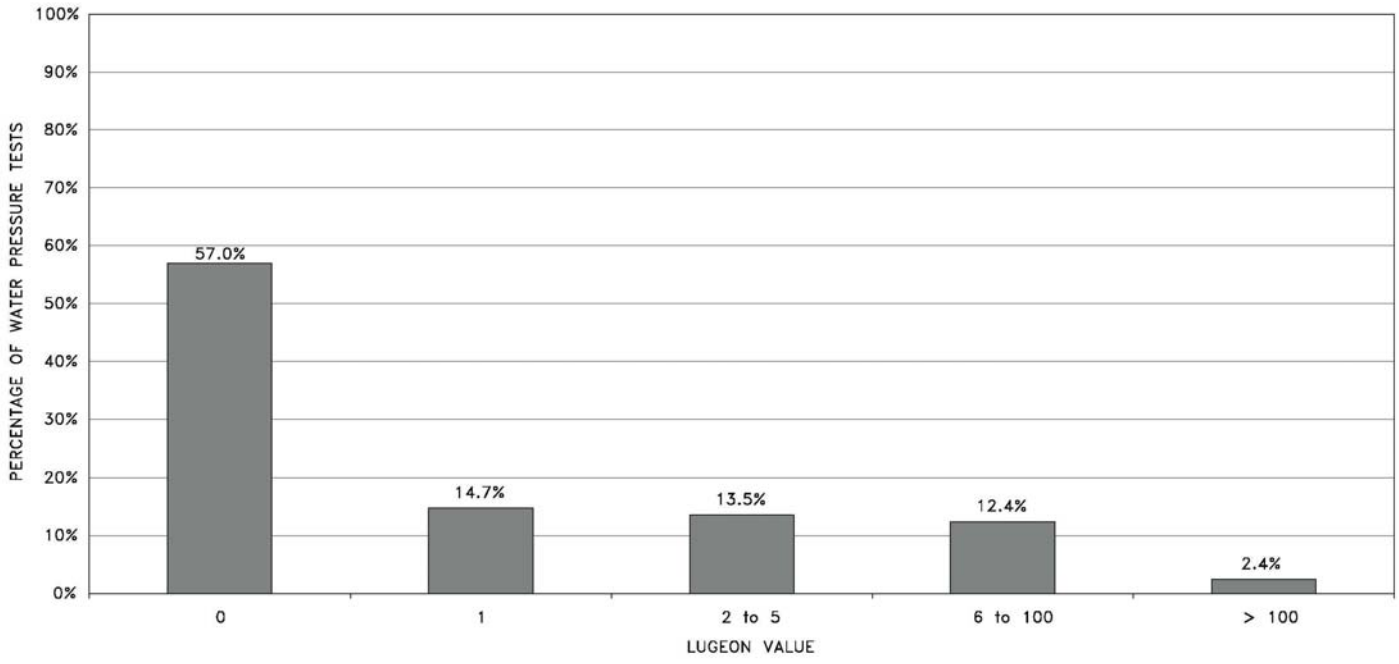
FIGURE 4 | SHT. 0001 | REV.

AUTOCAD ORIGINAL

FOR WORKFLOW USE ONLY

FIGURE 4 | SHT. 0001 | REV.

FIGURE 5 | SHT. 0001 | REV.



NOTES:

1) FOR A LIST OF DRILL HOLES USED IN THIS ANALYSIS SEE APPENDIX A, TABLE A1 .
2) A TOTAL OF 251 WPT WERE UNDERTAKEN IN THE VICINITY OF AXIS GR-4.
3) A TOTAL OF 909.48 M OF ROCK WAS TESTED, INCLUDING THE OVERLAP OF THE TEST ZONES.
4) WPT EXCEEDING PUMP CAPACITY OR GREATER THAN 100 LUGEONS ARE ASSIGNED A VALUE OF 101.
5) THE ADDITIVE SUM OF THE PERCENTAGES MAY NOT ADD UP TO 100% DUE TO ROUNDING.
6)

0	PRACTICALLY IMPERMEABLE
1 TO 5	LOW PERMEABILITY
6 TO 100	MEDIUM PERMEABILITY
> 100	HIGH PERMEABILITY

NO.	DATE	REVISIONS	BY	CKD.	APP.
	2009 07 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.7, REV. 0			

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
GROUTING AND PRESSURE RELIEF
RANGE OF WATER PRESSURE
TESTS RESULTS ALONG THE
PRINCIPAL AXIS

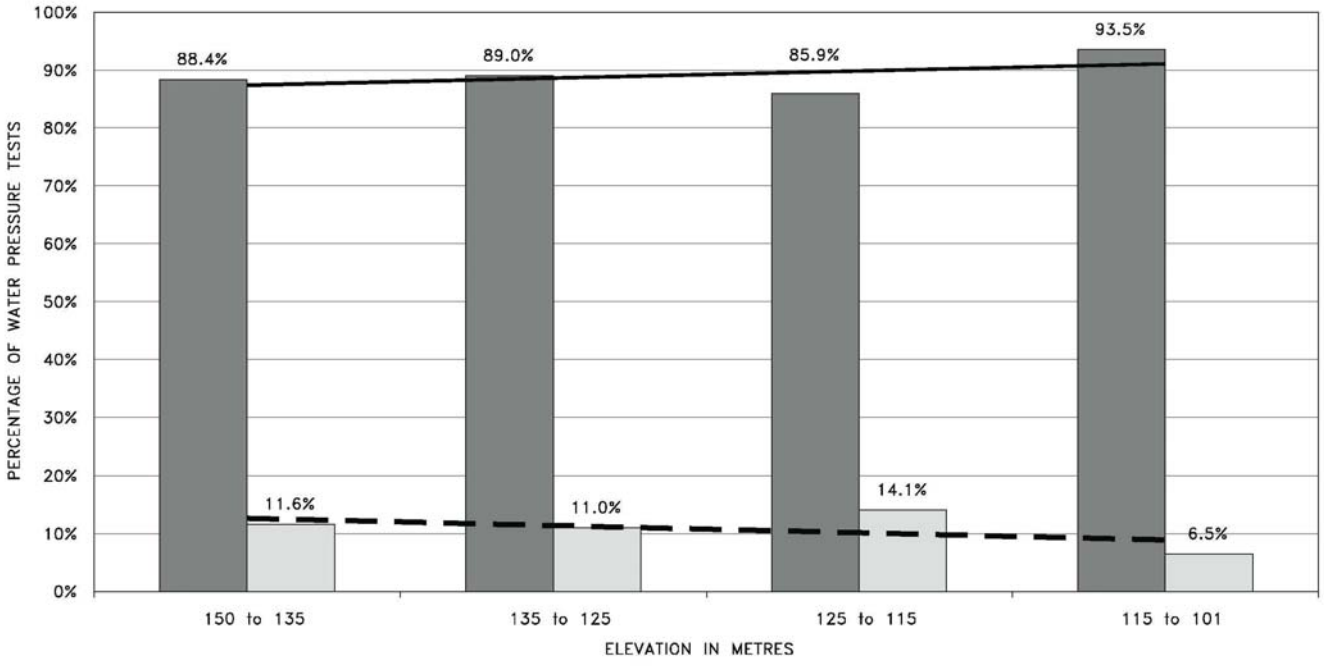
DRAWN
CHECK
SCALE AS SHOWN
DATE

FIGURE 5 | SHT. 0001 | REV.

FOR MICROFILM USE ONLY

AUTOCAD ORIGINAL

FIGURE 6 0001



LEGEND

0 TO 10 Lu

> 10 Lu

TRENDLINE (TIGHT TO 10 Lu)

TRENDLINE (>10 TO 100 Lu)

NO.	DATE	REVISIONS	BY	CKD.	APP.
	2009 07 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.7, REV. 0			

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MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
GROUTING AND PRESSURE RELIEF
PERCENTAGE OF WATER PRESSURE
TESTS FOR SPECIFIED RANGE
IN ELEVATION

DRAWN

CHECK

SCALE AS SHOWN

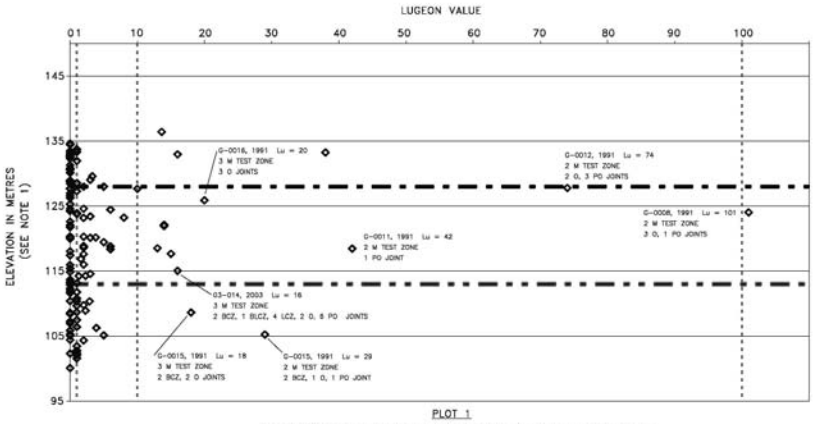
DATE

FIGURE 6 0001

FOR MICROFILM USE ONLY

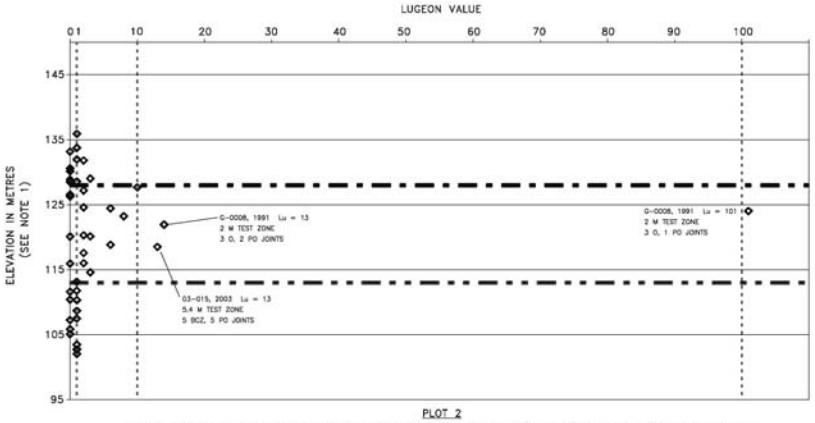
AUTOCAD ORIGINAL

FIGURE 7 0001



PLOT 1

WATER PRESSURE TEST RESULTS IN POWERHOUSE AREA – SEE NOTE 5



PLOT 2

WATER PRESSURE TEST RESULTS ALONG AXIS OF PRINCIPAL STRUCTURES IN POWERHOUSE AREA – SEE NOTE 6

LEGEND:

– · –	BASE ELEVATION OF CONCRETE IN POWERHOUSE AT GROUT CURTAIN LOCATION
– · –	PROPOSED DEPTH OF GROUTING HOLES (EL. 113.0 METRES)
O	OPEN JOINT
PO	PARTLY OPEN JOINT
BCZ	BROKEN CORE ZONE
BLCZ	BROKEN LOST CORE ZONE
LCZ	LOST CORE ZONE
3 PO	FEATURE
	NUMBER OF FEATURES IDENTIFIED IN TEST ZONE

NOTES:

1. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
2. TEST ZONE LENGTHS ALONG CORE AXIS VARIES BETWEEN 2.0 AND 8.9 METRES, AVERAGING 3.7 METRES.
3. LUGEON VALUES ARE PLOTTED BASED ON THE MIDPOINT ELEVATION OF THEIR TEST ZONES.
4. WATER PRESSURE TESTS EXCEEDING PUMP CAPACITY, ARE ASSIGNED A VALUE OF 101.
5. A TOTAL OF 126 WATER PRESSURE TESTS WERE UNDERTAKEN IN THE POWERHOUSE AREA.
6. A TOTAL OF 43 WATER PRESSURE TESTS WERE UNDERTAKEN ALONG THE AXIS OF PRINCIPAL STRUCTURES IN THE POWERHOUSE AREA. (ie. BOREHOLES 03-008, 03-010, 03-011, 03-013, 03-015, 03-022, AND G-0008)

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
GROUTING AND PRESSURE RELIEF
REVIEW OF POWERHOUSE AREA

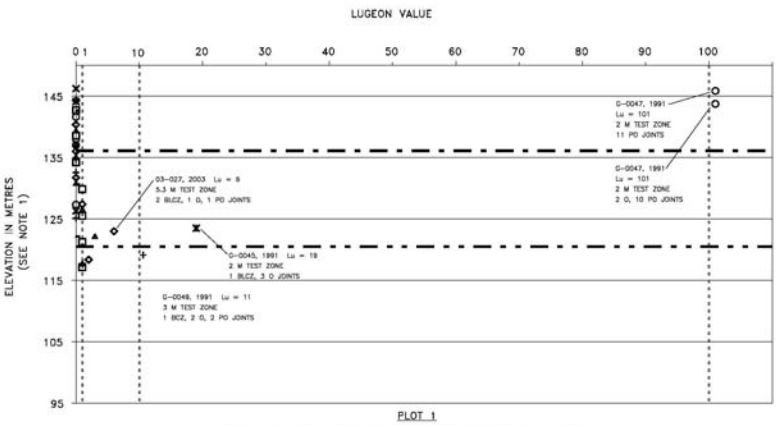
DRAWN

DESIGN

SCALE AS SHOWN

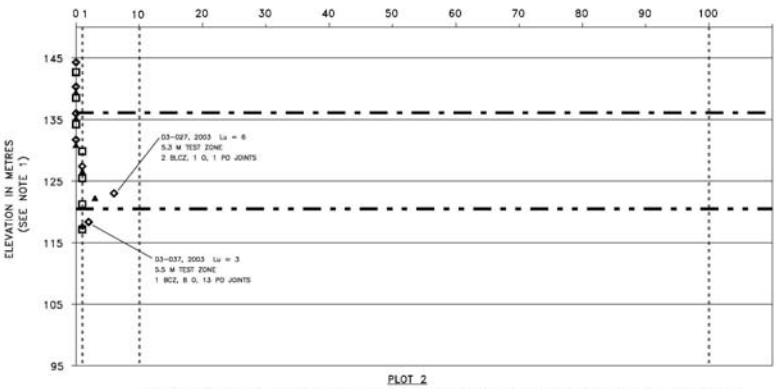
DATE

FIGURE 7 0001



PLOT 1

WATER PRESSURE TEST RESULTS IN SPILLWAY AREA – SEE NOTE 5



PLOT 2

WATER PRESSURE TEST RESULTS ALONG AXIS OF PRINCIPAL STRUCTURES IN SPILLWAY AREA – SEE NOTE 6

LEGEND:

	BASE ELEVATION OF CONCRETE IN POWERHOUSE AT GROUT CURTAIN LOCATION
	PROPOSED DEPTH OF GROUTING HOLES (EL. 120.5 METRES)
O	OPEN JOINT
PO	PARTLY OPEN JOINT
BCZ	BROKEN CORE ZONE
BLCZ	BROKEN LOST CORE ZONE
LCZ	LOST CORE ZONE
3 PO	FEATURE
	NUMBER OF FEATURES IDENTIFIED IN TEST ZONE

NOTES:

1. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
2. TEST ZONE LENGTHS ALONG CORE AXIS VARIES BETWEEN 2.0 AND 20.41 METRES, AVERAGING 4.5 METRES
3. LUGEON VALUES ARE PLOTTED BASED ON THE MID POINT ELEVATION OF THEIR TEST ZONES.
4. WATER PRESSURE TESTS EXCEEDING PUMP CAPACITY, ARE ASSIGNED A VALUE OF 101.
5. A TOTAL OF 39 WATER PRESSURE TESTS WERE UNDERTAKEN IN THE SPILLWAY AREA.
6. A TOTAL OF 20 WATER PRESSURE TESTS WERE UNDERTAKEN ALONG THE AXIS OF PRINCIPAL STRUCTURES IN THE SPILLWAY AREA (ie. BOREHOLES 03-027, 03-028 AND 03-037).

2012 03 12 ISSUED AS A FIGURE FOR MDMO REFERENCE GN-4.2.2.7, REV 0

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4 GROUTING AND PRESSURE RELIEF REVIEW OF SPILLWAY AREA

DRAWN

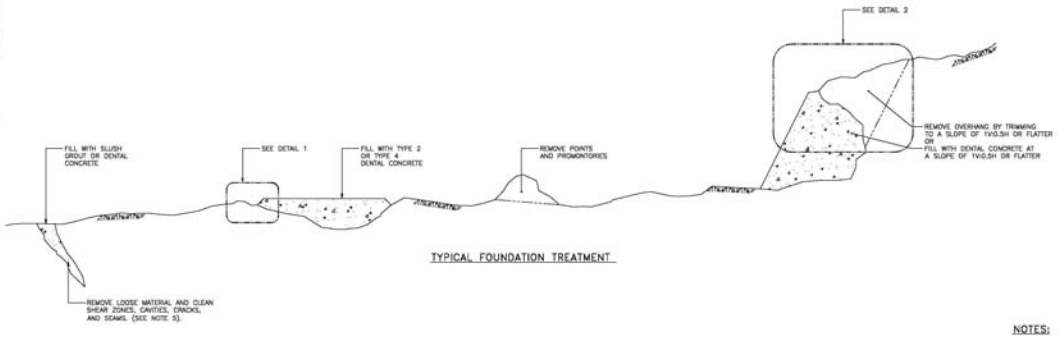
DESIGN

SCALE AS SHOWN

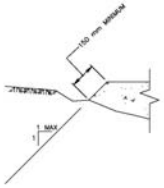
DATE

FIGURE 8 0001

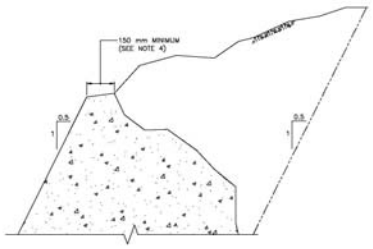
FIGURE 9 0001



TYPICAL FOUNDATION TREATMENT



SEE DETAIL 1



SEE DETAIL 2

NOTES:

1. ALL DIMENSION ARE IN METRES UNLESS NOTED OTHERWISE.
2. BENEATH THE IMPERVIOUS CORE, ALL OVERHANG MUST BE REMOVED. STEPPED SURFACES STEEPER THAN 1V:0.5H AND HIGHER THAN 150 mm SHOULD BE EXCAVATED AND TREATED WITH DENTAL CONCRETE TO FORM A SLOPE OF 1V:05H OR FLATTER.
3. OUTSIDE THE IMPERVIOUS CORE, ALL OVERHANGS SHOULD BE REMOVED AND STEPPED SURFACES STEEPER THAN 1V:05H AND HIGHER THAN 1.5 M SHOULD BE EXCAVATED AND TREATED WITH DENTAL CONCRETE.
4. SLOPE CONCRETE AWAY FROM ROCK CONTACT TO PROPAGATE DRAINAGE.
5. LOOSE OR UNSUITABLE MATERIALS WITHIN THE DISCONTINUITIES WILL BE TREATED AS FOLLOWS:

   OPENINGS NARROWER THAN 50 mm WILL BE CLEANED TO A DEPTH OF THREE TIMES THE WIDTH OF THE OPENING.

   OPENINGS WIDER THAN 50 mm AND NARROWER THAN 1.5 M WILL BE CLEANED TO A DEPTH OF THREE TIMES THE WIDTH OF THE OPENING UP TO A SET MAXIMUM OF 1.5 M.

SCALE 1:1000 — 20 0 20 40 60 80 100 METRES

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MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4 GROUTING AND PRESSURE RELIEF TYPICAL FOUNDATION TREATMENT AND DETAILS

DRAWN — DESIGN — SCALE AS SHOWN — DATE

FIGURE 9 0001

Appendix A

Table A1
Keeyask Generating Station - Axis GR-4
Drill Holes along Axis GR-4

Exploratory Hole No.	Datum(1) (m)	B/R Depth (m)	E.O.H. Depth (m)	Orientation (deg.)	
				Dip from Horizontal	Azimuth
03-006	146.21	7.72	18.00	90	-
03-007	146.16	8.13	18.27	90	-
03-008	146.68	9.53	20.29	90	-
03-009	147.08	8.97	18.99	90	-
03-010	140.88	0.89	40.62	70	231
03-011	140.99	3.71	38.72	70	182
03-012	140.98	0.00	37.83	90	-
03-013	141.34	0.53	39.73	70	63
03-014	140.42	0.81	39.93	70	234
03-015	141.18	2.69	38.85	70	277
03-016	140.95	0.71	37.36	90	-
03-022	149.54	9.86	25.05	90	-
03-023	151.35	5.11	20.14	90	-
03-024	153.10	0.45	15.46	90	-
03-025	146.97	2.46	16.85	90	-
03-027	148.16	0.00	34.79	70	315
03-028	149.29	3.02	36.53	70	315
03-029	152.46	10.96	21.15	90	-
03-030	155.64	15.67	26.56	90	-
03-037	146.01	2.80	33.08	70	315
03-039	140.68	0.28	3.38	90	-
03-055	149.20	N/E	3.81	90	-
03-056	148.76	N/E	3.81	90	-
G-0002	144.57	0.00	17.44	45	278
G-0008	141.50	13.11	58.01	45	28
G-0009	148.11	14.83	60.80	60	262
G-0010	142.18	2.46	55.81	45	188
G-0011	140.05	0.94	55.44	45	195
G-0012	139.95	1.61	55.80	45	264
G-0013	136.91	1.73	51.05	60	270
G-0014	140.11	4.03	46.10	60	205
G-0015	139.81	4.32	46.66	60	262
G-0016	139.76	4.73	42.36	60	12
G-0043	150.69	2.25	42.06	45	270
G-0045	149.65	2.13	45.39	45	207
G-0047	149.04	1.46	40.91	45	257
G-0049	144.93	4.04	42.49	45	320
G-0051	149.00	7.96	41.08	45	185
G-0053	151.13	5.05	8.35	90	-
G-0054	143.53	0.00	50.30	45	143
G-0055	140.55	9.43	30.54	60	20
G-0056	147.88	4.10	6.93	90	-
G-0057	147.42	6.47	10.08	90	-
G-0062	144.52	0.00	49.95	45	20
G-0064	149.12	5.39	8.61	90	-
G-0272	148.47	9.75	22.29	90	-
G-0279	147.25	12.44	18.26	90	-
G-0459	154.94	12.52	17.93	90	-
G-0460	155.40	13.28	21.15	90	-
G-0461	155.20	14.35	19.53	90	-
G-0462	145.50	0.88	49.77	45	62

Notes:
(1) Datum varies: ground surface, barge platform or other.
(2) The elevation recorded is either the platform elevation or the top of the collar elevation.
(3) Elevations reported are based on the Canadian Government Vertical Datum 1928.
(4) Depths are measured along axis of core.
(5) N/E Not Encountered.

Table A2
Keeyask Generating Station
Stage IV Studies - Axis GR-4
Grouting Quantity Estimate

Location	WPT Results - Assumed Representative																
						<1 Lu	1-10 Lu	> 10 Lu				>1 Lu				>1 Lu	
						72%	18%	10%				40%				10%	
					kg/m	15	30	60			kg/m	30			kg/m	20	
	Primary, Secondary and Tertiary									Quarternary				Quinary			
				Length		Assumed Cement Take					Length		Cement		Length		Cement
	Distance (m)	Spacing (m)	No. of Holes	Hole (m)	Drilled (m)	<1 Lu (kg)	1-10 Lu (kg)	> 10 Lu (kg)	Sub Total (kg)	No. of Holes	Hole (m)	Drilled (m)	Take (kg)	No. of Holes	Hole (m)	Drilled (m)	Take (kg)
North Dam	99	3	34	14.0	476	5,141	2,570	2,856	10,567	13	8.4	109	3,276	1	8.4	8	160
P/H N. Transition	20	3	8	16.0	128	1,382	691	768	2,842	3	16.0	48	1,440	0	16.0	-	-
Service Bay	36	3	13	16.0	208	2,246	1,123	1,248	4,618	5	16.0	80	2,400	0	16.0	-	-
Powerhouse	212	3	72	16.0	1,152	12,442	6,221	6,912	25,574	28	16.0	448	13,440	2	16.0	32	640
P/H Corners (1)	-	-	6	16.0	96	1,037	518	576	2,131	2	16.0	32	960	0	16.0	-	-
P/H S. Transition	32	3	12	16.0	192	2,074	1,037	1,152	4,262	4	16.0	64	1,920	0	16.0	-	-
Central Dam	1,600	3	535	14.0	7,490	80,892	40,446	44,940	166,278	214	8.4	1,798	53,928	21	8.4	176	3,520
S/P N. Transition	24	3	9	14.0	126	1,361	680	756	2,797	3	14.0	42	1,260	0	14.0	-	-
Spillway	119	3	41	14.0	574	6,199	3,100	3,444	12,743	16	14.0	224	6,720	1	14.0	14	280
S/P S. Transition	20	3	8	16.6	133	1,434	717	797	2,948	3	16.6	50	1,494	0	16.6	-	-
South Dam	565	3	190	14.0	2,660	28,728	14,364	15,960	59,052	76	8.4	638	19,152	7	8.4	58	1,160
Totals	**2,727**		**928**		**13,235**				**293,813**	**367**		**3,533**	**105,990**	**32**		**288**	**5,760**

	No. of Holes	**1,327**		
(i)	Drilled (m)	**17,056**		
(ii)	NX core drilling (m)	**180**		Based on additional drilling beneath dams
(iii)	Packer Settings (each)	**4,738**	5	m. Stage Length 2 settings per hole
(iv)	Washing and pressure testing (hr)	**664**		2 tests per every second hole (approx. 30 minutes per test)
(v)	Cement in grout mix (kg)	**408,263**		
(vi)	Bentonite in grout mix (kg)	**10,139**	2.5%	Cement Mass
(vii)	Sand in grout mix (kg)	**2,041**	0.5%	Cement Mass

Notes:
(1) Represents fanning of grout holes at the corner of the excavation.
(2) Below the Powerhouse and Spillway structures, grouting will be a single row grout curtain, 16.0 m and 16.6 m in length, respectively.
(3) Below dams, grouting will be a single row grout curtain, 14 m in length.
(4) Cement and sand in grout mix quantities include amounts required to fill NX core drilling holes, assuming 15 kg/m of cement (No bentonite necessary).
(5) Secondary and Tertiary holes are advanced to 60% of the primary hole depth (9 m in length).

Table A3
Keeyask Generating Station
Stage IV Studies - Axis GR-4
Temporary and Permanent Drainage Estimate

Location/Structure	Width (m)	Area of Face (m2)	Drain Hole Details								
			Type		Number				Total Lengths		
			Permanent	Temporary	Spacing (centers) (m)	Permanent	No. of Collars	Temporary	Hole Length (m)	Permanent (76 mm dia.) (m)	Temporary (50 mm dia.) (m)
North Dam	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Powerhouse North Transition	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Service Bay - Excavation - right face	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
- Excavation - u/s face	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
- Excavation - d/s face	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Powerhouse - Excavation - left face	-	1,230	-	Yes	6	-	-	35	6	-	210
- Excavation - right face	-	1,010	-	Yes	6	-	-	29	6	-	174
- Excavation - u/s face	-	3,180	-	Yes	6	-	-	89	6	-	534
- Draft Tube - floor	-	n/a	Yes	-	-	252	252	-	3	756	-
- Draft Tube - wall	-	n/a	Yes	-	-	42	42	-	14	588	-
Powerhouse South Transition	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Central Dam	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Spillway North Transition	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Spillway - Excavation - left face	-	430	-	Yes	6	-	-	12	6	-	72
- Excavation - right face	-	400	-	Yes	6	-	-	12	6	-	72
- Excavation - u/s face	-	360	-	Yes	6	-	-	10	6	-	60
- Excavation - d/s face	-	480	-	Yes	6	-	-	14	6	-	84
Spillway South Transition	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
South Dam	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
		Totals:				**294**		**201**		**1,344**	**1,206**

Notes:
(1) No temporary drain holes required for service bay due to assumption of excavation of less than 1 m.
(2) Powerhouse left face area based on bedrock el 137.3 m, right face based on bedrock el 133.0 m.
(3) Draft tube drain holes based on civil design descriptions.
(4) Spillway design calculations assumed to be the same as for the Powerhouse.
(5) n/a - not applicable

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Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.8

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Rock Excavation and Support

Manitoba Hydro File 00195-13200-0009_03

Prepared by _____ (P. PAWTEL) June 24, 2011
KGS Acres Ltd.

Checked by _____ (R.A. HALIM) June 24, 2011
KGS Acres Ltd.

Reviewed by _____ (I.R. Dewar) June 24, 2011
KGS Acres Ltd.

Approved by _____ (N.J. Smith) June 24, 2011
KGS Acres Ltd.

Accepted by _____ (M. ST. LAURENT) MARCH 22, 2013
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	June 24, 2011
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From	H. Liu, R.A. Halim and P.R. Pantel	**cc**	
Subject	**Keeyask Generating Station Stage IV Studies - Axis GR-4 Rock Excavation and Support Memorandum GN-4.2.2.8, Rev 1 Manitoba Hydro File 00195-13200-0009_03**		

1 Introduction

The proposed site of the Keeyask Generating Station (Keeyask GS) is located on the lower Nelson River, approximately 730 km north of Winnipeg at the head of Stephens Lake. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is the Keeyask site. The proposed axis for the structures crosses the Nelson River at Gull Rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam.

The general arrangement of the Principal Structures is shown in Figure 1. The layout corresponds to that proposed for Axis GR-4, with an FSL at el 159.0 m. The selected arrangement consists of two long river bank dykes, one on the north and the other on the south, linked to the Principal Structures which span the Nelson River at Keeyask Rapids. The Principal Structures consist of a Powerhouse Complex, Spillway and Central Dam. The Powerhouse will be constructed on the north side of the river, and will be connected to the North Dyke through a 100 m long North Dam and a short Concrete Transition Structure. The Spillway will be located on an existing island, approximately 1.5 km south of the Powerhouse, with a Central Dam connecting the two structures. On the south riverbank, the South Dyke will be connected to the Spillway via a South Dam approximately 550 m long.

The Principal Concrete Structures at Keeyask will be founded on bedrock with significant rock excavation required for the Powerhouse and Spillway Structures. Specifically, bedrock excavations are required as follows:

Powerhouse Complex and associated Intake and Tailrace Channels

Spillway Structure and associated Approach and Discharge Channels.

This memorandum reviews the conditions and properties of the bedrock and summarizes the requirements for rock support and excavation at the Keeyask site. The analyses performed for this report were based on the bedrock conditions defined during field and laboratory investigations conducted in 1988, 1991, 1999, 2003 and 2004. Detailed descriptions of the bedrock conditions are provided in the 1991 Summer Subsurface Investigation Report [Ref 1], and the Review of Bedrock Conditions in the Powerhouse and Spillway memoranda [Ref 2 and 3]. A quantity estimate for the required support and associated drilling, pumping requirements for care and control of water within the excavations, and the amount of blasting required is also provided herein. The estimates include potential overbreak quantities, and the total length of drilling required for preshearing and line drilling. Quantities used in the cost estimate assume that all excavated rock faces will be treated. However, actual treatment levels will vary according to the foundation conditions encountered during construction.

2 Physiography and Site Geology

In general, the Keeyask GS site is located within the Canadian Shield physiographic region, near the boundary between the Churchill and Superior geological provinces. The Precambrian bedrock within the area consists of granite, tonalite, pegmatite, amphibolite, greywacke gneiss, granodiorite gneiss, granite rocks and diabase. The greywacke gneiss contains bands or sills of granitic material throughout. Diabase occurs as continuous and discontinuous dykes that have intruded the amphibol greywacke gneiss, iron formation, granodiorite gneiss, and granite rocks. The contacts between the diabase dykes and the host rock are typically open at ground surface, but appear to tighten with depth.

All rock types within the Keeyask GS area are typically fresh, strong to very strong, with widely to moderately spaced jointing. The Principal Concrete Structures are located in an area where the bedrock geology is comprised predominantly of greywacke gneiss and amphibolite.

3 Bedrock Conditions

3.1 Joint Sets

At Keeyask GS the orientation of the joint sets vary significantly across the site, with different dominant sets occurring within the Powerhouse and Spillway areas. A summary of the joint sets, for each area, is provided in Table 1. The orientations were based on a total of 708 and 364 oriented core measurements (not including discontinuities described as healed) for the Powerhouse and Spillway areas, respectively. Polar density plots derived from the measurements are shown in Figures 2 and 3, and stereonet projections of the dominant joint sets are presented in Figures 4 and 5.

The joint sets were ordered by frequency of occurrence in each area, and the denotation PH and SP identifies the areas where the joint set occurs. The designation of major or minor was based on the concentration of the poles on the stereoplots, with the latter having lower concentrations. Major and minor joint sets have limited continuity, and generally are not mapable features; however, they act together in statistically defined sets to influence the geotechnical behavior of the rock mass.

Figure 6 indicates that, to some degree, the frequency of all the joint sets vary with depth. Specifically, the frequency of occurrence for $J6_{PH}$ appears to increase with depth, and $J5_{PH}$ appears to decrease with depth. However, the frequency of the remainder of the joints sets encountered in the Powerhouse area is relatively constant with depth. There appears to be no discernable trend in the frequency of occurrence for any of the joint sets encountered in the Spillway area. With the exception of $J1_{PH}$, the joint sets encountered at Keeyask GS are generally acute to sub-vertical. Consequently, the depth of occurrence is simply the depth at which the borehole intersects the joint; which is dependent on the orientation of the borehole with the respect to the orientation of the joint, as well as the offset of the borehole from the surface expression of the joint.

Sub-horizontal Joint Set $J1_{PH}$ is a sheeting joint, likely the product of stress relief or unloading. Joint Sets $J2_{PH}$ and $J5_{PH}$ are concordant with the foliation identified during previous exploration programs conducted in the area of the Powerhouse Structure. Consequently, the angle of friction for these joint sets may be less than that of the others. Joint Sets $J3_{PH}$ and $J4_{PH}$ strike roughly parallel, while dipping in opposite directions. Joint Sets $J5_{PH}$ and $J6_{PH}$ have significantly lower frequencies of occurrence than the other joint sets in the Powerhouse Area; however, the analysis revealed that their orientations resulted in critical cases developing on the various excavation faces. Likewise, Joint Sets $J3_{SP}$ and $J4_{SP}$ were recorded less frequently than the other joint sets in the Spillway Area; however, they represent distinctly separate joint sets, with well defined boundaries. Therefore, it was deemed prudent to evaluate the effects they would have on the stability of the various excavation faces as well.

Several relationships can be drawn between the Joint Sets in the Powerhouse Area and those in the Spillway Area. Joint Sets $J1_{SP}$ and $J3_{SP}$ have similar strikes (approximately 180^{o} from each other) and dip angles, and are sub-parallel to Joint Sets $J3_{PH}$ and $J4_{PH}$, respectively. The Spillway and Powerhouse Joint Sets $J2_{SP}$ and $J2_{PH}$ also have strikes approximately 180^{o} from each other, and equal dip angles.

The investigation programs indicate that the majority of the joints appear tight with little or no alteration. Open joints are typically widely to very widely spaced, slightly to faintly altered and may be infilled with clay. Carbonate and chlorite coatings were frequently observed on joint surfaces [Ref 1, 2].

3.2 Rock Mass Properties

In the Principal Concrete Structures area, 31 drill holes were reviewed to assess the quality and permeability of the bedrock. A list of the drill holes in the Powerhouse and Spillway areas are provided in Appendix A, Tables A1 and A2, respectively. The information contains both NQ and HQ size drill holes. The size of the hole, and the use of either a single or triple barrel rod, has an affect on the quality of the sample obtained; HQ and triple barrels typically producing samples with fewer fractures.

The rock quality was reviewed in terms of the rock quality designation (RQD) and drill core recovery (percent length of core recovered from each length of drilled depth).

3.2.1 Rock Quality Designation

The RQD is defined as the ratio of the summation of the lengths of pieces of sound rock core (NQ size), individually greater than 100 mm in length (based on natural breaks), to the total length drilled. It is generally reported on a run by run basis.

Table 2 provides a typical relationship between RQD and the anticipated engineering quality of the rock [Ref 4].

The RQD values for each of the drill holes, on a run by run basis, are presented on the drill hole summaries listed in Tables A1 and A2. As listed in these tables, the RQD for the upper 40 m of the rock mass in both the Powerhouse and Spillway areas typically varies between 0% and 100%, averaging 89%.

The RQD values measured in all drill holes located near the Powerhouse and Spillway structures are shown in Figures 7 and 8, respectively. In the Powerhouse area, most of the coring runs with RQD values of less than 75% occurred in the upper 30 m of the bedrock. Whereas in the Spillway area, the majority of the runs with RQD values of less than 75% occurred in the upper 15 m of bedrock. In general, zones in which lower RQD values have been reported, occur where the boreholes have intersected one of the joint sets. As most of the joint sets are inclined to the horizontal, the depth of intersection will vary.

It is presently envisioned that bedrock excavation for the Principal Structures will be carried out in 6 m lifts, thus the drilling results were divided into 6 m intervals. In the Powerhouse area the average RQD (per interval) ranges from 83% to 93% (Figure 9). As indicated in Figure 10, the average RQD (per interval) in the Spillway area ranges from 77% to 94%. In both areas the average RQD for the all of the intervals is 75% or higher, resulting in classifications of good to excellent quality rock.

3.2.2 Drill Core Recovery

The percent recovery is defined as the ratio of the total length of the pieces of rock core retrieved from the drill run to the total length of the drill run. A percentage core recovery less than 100% indicates a core loss, reflecting core which has been ground by the drilling process or contains open features (e.g., joints, fractures, etc). The percent recovery for each of the drill holes analyzed on a run by run basis, are presented on the drill hole summaries listed in Tables A1 and A2.

The percent recovery measured in all of the drilled holes located near the Powerhouse and Spillway structures are shown in Figures 11 and 12, respectively. Low recovery zones typically occur where the boreholes have intersected one of the joint sets. However, the percent recovery will typically increase with depth, as the aperture width and infilling of the joints decrease with depth.

As listed in Table A1, the percent recovery for the upper 40 m of the rock mass in the Powerhouse area varies between 73% and 100%, averaging 99%. Figure 9 indicates that when the results are evaluated based on 6 m depth intervals, the average percent recovery in the area of the Powerhouse is not less than 99% for any of the intervals.

As listed in Table A2, in the Spillway area the percent recovery for the upper 40 m of the rock mass varies between 68 % and 100 %, averaging 99 %. Figure 10 indicates that in the Spillway area the average percent recovery per 6 m depth interval varies with depth, with values typically greater than 94%.

3.2.3 Rock Mass Permeability

The permeability of the rock mass was estimated using the water pressure test results completed during the 1991 and 1999 field investigation programs. In summary, the rock mass permeability ranges between tight (0 Lugeons) to over 100 Lugeons, averaging approximately 20 Lugeons (Lu) in the upper 30 m of the bedrock [Ref 5].

An approximate permeability was estimated by assuming that 1 Lu is equal to a hydraulic conductivity of approximately 10^{-5} cm/s [Ref 6]. A range of permeabilities between 1 Lu and 100 Lu (or equivalent 10^{-5} and 10^{-3} cm/s, respectively) was used in Section 4.1 to estimate the amount of seepage inflow through the bedrock that is expected during excavation of the Principal Concrete Structures.

4 Rock Excavation

Construction of the Principal Structures and exploitation of the rock quarries at Keeyask GS will require excavation of bedrock material that will have to be blasted to facilitate removal.

The early stages of quarry blasting will yield useful information for blast designs associated with the Principal Structures; such as, specific charge quantity, delay and charge weighting relationships, and vibration correlations.

The excavated material from the Principal Structures excavation will be used for the production of the concrete coarse aggregate, Class 4, Class 5, Class 6, Class 7, Class 8 and Road Topping (RT).

4.1 Control of Water

The Powerhouse and Spillway Structure excavations, including excavation for their associated channels, will be conducted within the confines of cofferdams. An estimate of the seepage rate that may be expected within the Powerhouse and Spillway excavations was carried out to determine what pumping capacity would likely be required.

The estimated seepage quantities range between 0.02 and 15 L/s, which are essentially negligible for the large Principal Structure's excavations. The seepage quantities estimated for the cofferdam and its alluvial foundation will likely be significantly greater than that for the bedrock, and thus no additional pumping capacity will be required to handle the seepage originating through the bedrock.

4.2 Excavation Design

The nature of the discontinuities encountered at the site will influence the stability of the rock excavations. As was stated in Section 3 of this memorandum the joints tend to be discontinuous and widely spaced, and the bedrock is generally classified as good quality rock. Consequently, the selected excavation slopes were driven primarily by constructability considerations and hydraulic design. The excavation faces for the structures and associated channels will be constructed at a 1H:12V slope; with the exception of the West (Upstream) face of the Powerhouse rock noses, which will be constructed at a 1H:2V slope.

4.3 Controlled Blasting Techniques

Controlled blasting will be employed during all excavations for the Principal Concrete Structures and associated channels. Blasting for the final rock faces of these structures will be undertaken using pre-shear (pre-splitting) and line drilling techniques, or some variation thereof. Pre-shear blasting techniques consist of drilling a single row of closely spaced perimeter holes, loading all or select holes lightly, and then firing them before any adjoining main excavation area is blasted. This technique produces a crack along the line of perimeter holes to which a subsequent primary blast can break. Smooth wall blasting is similar to pre-splitting, except the drill holes are spaced farther apart, and they are fired either at the same time as the main production blast or after. Line drilling consists of drilling a single row of very closely spaced perimeter holes and not loading them, except for the possible use of a toe load to aid in splitting the rock webs between adjacent holes. With closely spaced holes, it is anticipated that with subsequent primary blasts in the area, a crack along the line of holes will be established to form a neat excavation face.

It has been assumed that pre-shear blasting techniques will be used around the perimeter of all rock excavations, and that line drilling will be used around the rock noses situated between the units of the Powerhouse.

4.4 Blasting Design Requirements

The blast design should consider the experience gained during work done for other generating stations in the area, that exhibit similar subsurface conditions.

4.4.1 Control of Over-break and Under-break

During blasting operations, every effort should be made to minimize over-break quantities. Several techniques that proved successful during excavations performed for other generating stations in the area have been incorporated into the recommendation for Keeyask GS, as discussed below.

To produce a smooth final face and minimize damage beyond the neat lines the holes for controlled perimeter blasting will not be less than 37 mm in diameter or spaced less than 0.5 m centers. In addition, care must be taken to ensure the production blasts do not cause joints to open beyond the neat lines of the excavation. Consequently, all blast holes within 6 m (vertically and horizontally) of a final face will not be less than 75 mm in diameter, and will be loaded and detonated in a sequence to produce minimum damage to the final face when the main charge is fired.

Care should be taken during drilling to minimize drill hole wander, as this can adversely affect the results of blasting; producing an irregular face. To minimize irregularities in the base of the excavation, sub-drilling should be restricted to within 0.3 m of the neat line.

To minimize over-break at excavation corners, an additional two or three unloaded holes should be split-spaced between the loaded holes on each side of the corner, and delays should not be permitted.

In addition to pre-splitting; a buffer row, at a reduced burden and spacing, should be drilled adjacent to the pre-split wall in order to cushion the final wall from the effects of the main blast and still provide enough energy to ensure that the blast breaks back to the pre-split wall. Buffer hole column loading should be between 75% and 100% of the production hole load; therefore, increased stemming length and reduced hole to hole spacing will be required to ensure enough energy is produced. Recommended spacing for the buffer holes is 1.5 m, and these holes should be vertical. The maximum relief of burden should be provided on the row of holes in front of the buffer line by omitting on consecutive delay periods. This will allow the row time to move out and minimize back pressure.

4.4.2 Blast Pattern

The burden and spacing used will vary in order to achieve the required powder factor, without overloading the individual holes. The rock type, presence of open joints, degree of fragmentation required, degree of confinement of the blast, and the expected efficiency of the loading process all affect the choice of burden and spacing.

Fragmentation of rock is a function of burden, hole spacing, loading and delay sequence. Care should be taken to coordinate these variables in such a way as to produce ideal fragmentation, keeping in mind that the blasted rock will be utilized as coarse aggregate and rock fill.

The length of the blast should not be greater than two times the width of the blast. To reduce drill hole wander, bench heights are limited to 6 m.

4.4.3 Blast Guidelines

Vibration control during blasting will be required where blasting activities are undertaken in the vicinity of grouting operations or concrete structures, to ensure the integrity of the operations are not undermined by blast induced vibrations. The peak particle velocity (PPV), as determined by a seismograph measurement, at the grout curtains or concrete structures must conform to the guidelines given in Table 3. Peak particle velocity will also have to be maintained below 75 mm/s near completed sections of the earthfill structures.

Monitoring of the PPV will be required to ensure the limits are not exceeded, and to calibrate the site vibration levels. The blast design will also have to incorporate other requirements, regarding setback distances and charge weights, to satisfy environmental guidelines. These are addressed separately, in Section 7 of this memorandum.

5 Rock Support Design

5.1 Joint Analysis

The joint configuration, with respect to the various excavation faces, has been examined using stereonet projections and wedge analysis. Computer programs from Rockscience Inc., DIPS version 5.0 Graphical and Statistical Analysis of Orientation Data and SWEDGE version 4.0-3D Surface Wedge Analysis for Slopes, were utilized to aid in the rock support design and analysis. The DIPS program was used to identify the dominant joint sets, and analyze the potential for failure of the rock slope in three different modes: Toppling, Planar Sliding, and Wedge Sliding. Once this analysis was complete, the computer program SWEDGE was used to obtain a Factor of Safety regarding translational slip along the wedge boundaries.

The following excavation face configurations have been examined with respect to the dominant joint sets, using stereonet projections and wedge analysis:

North and South Walls of the Powerhouse, and associated Intake and Tailrace Channels

- North Wall of Intake: 104^o (Azimuth)
- South Wall of Intake: 256^o (Azimuth)
- North Wall of the Powerhouse and Tailrace: 90^o (Azimuth)
- South Wall of the Powerhouse and Tailrace: 270^o (Azimuth)

West (Upstream) Wall of Powerhouse: 0^o (Azimuth)

North and South Walls of the Spillway Approach and Discharge Channels

- North Wall of Approach: 82^o (Azimuth)
- South Wall of Approach: 218^o (Azimuth)
- North Wall of Discharge: 60^o (Azimuth)
- South Wall of Discharge: 243^o (Azimuth)

Note that no significant rock excavation is exposed on an east face of either of the concrete structures, or the west face of the Spillway, and thus no analysis has been performed for these faces. The upstream wall of the Powerhouse Structure is irregular, with rock noses jutting out of the face. The north and south walls of the rock noses will have orientations very similar to those analyzed for the channels. Consequently, it was determined that only the West Walls required independent analysis.

The rock faces along the structure and Channel walls are anticipated to be sloped at 1H:12V, resulting in a dip angle of 85^{o} (from horizontal). The West (Upstream) face of the Powerhouse rock noses will have a shallower dip angle of 63° (from horizontal).

The effect the dominant joints, as well as combinations of intersecting joints, will have on the stability of the various excavation faces are outlined below. Stereonets showing the resulting combinations are provided in Appendix B. Recommended treatment options are discussed in Section 6 of this memorandum.

Several factors contribute to instabilities developing along joint contacts within a rock mass, the following are three critical factors:

the exposure of the joint on the excavation face,

the dip angle of the joint with respect to:

- the friction angle along the joint contact,
- the slope of the excavation face.

Planar sliding may occur along a joint surface if the joint dips out of the face of the wall. However, the probability of sliding is based on whether the dip angle of the joint is shallower than the slope of the face, as well as steeper than the friction angle along the joint surface. Toppling instabilities are usually the result of the presence of near vertical joints with similar strikes to that of the excavation face. These joints may dip into the face of the excavation; however, due to their sub-vertical nature, toppling instabilities may still form within the rock mass.

The probability of failure was assessed for each of the excavation faces based on the percentage of poles lying within the region of either planar sliding or toppling, as defined by the DIPS program, for a given joint set. These are visual estimates designed only to identify high risk areas. A more quantitative analysis was performed to assess wedge instabilities, with the SWEDGE program employed to arrive at a Factor of Safety against sliding along the joint intersections.

5.2 Joint Instability

5.2.1 Powerhouse Structure

In the Powerhouse area four major and two minor joint sets were identified, and are listed in Table 1. The geological profile and bedrock excavation limits below the Powerhouse are shown in Figure 13.

The maximum rock cut height in the Powerhouse area will be approximately 26 m. The exposure of the critical joint sets, with respect to the various excavation faces within the Powerhouse structure are outlined in Table 4.

Although Joint Set $J1_{PH}$ dips out of the south walls of both channels, this joint set has an extremely shallow dip, and therefore will not pose a threat to the stability of the rock mass for any of the excavation faces.

As was stated previously, the probability of failure cannot be assessed solely on the basis of exposure of the joints along the face of the walls. The other contributing factors were incorporated into the evaluation and the resulting possible rock mass instabilities are presented in Tables 5 and 6, for toppling and planar sliding respectively. The ranking, from high to none, was utilized as a description of potential for toppling or sliding in order to discern between areas where failures were anticipated to occur, and would require subsequent rock support, and those of a less critical nature.

Table 6 indicates that there is a possibility of planar sliding along the various joint surfaces. Joint set $J2_{PH}$ may introduce instabilities in the rock mass along the South Walls of the Intake and Tailrace Channels, as well as the West Wall of the Powerhouse Structure. Joint Set $J5_{PH}$ is sub-vertical; therefore, its dip direction can vary by 180^{o}. Consequently, there is a potential for planar sliding along Joint Set $J5_{PH}$ out of the North, South and West Walls of the Powerhouse Structure and its associated Channels. The table also indicates that Joint Set $J6_{PH}$ may introduce instabilities into the rock mass along the North Walls of the Channels, and the West Wall of the Powerhouse Structure. The joint influences and the stability of the Powerhouse are discussed elsewhere.

5.2.2 Spillway Structure

Two major and two minor joint sets were identified in the Spillway area. They are listed in Table 1. The geological profile and bedrock excavation limits below the Spillway are shown in Figure 14.

The maximum rock cut height in the Spillway area is approximately 12.1 m. The exposure of the critical joint sets, with respect to the various excavation faces within the Spillway, are outlined in Table 7.

As was stated previously, the potential for failure cannot be assessed solely on the basis of exposure of the joints along the face of the walls. The other contributing factors were incorporated into the evaluation, and the possibility of instabilities being introduced into the rock mass due to a potential for toppling and planar sliding is presented in Tables 8 and 9, respectively. The ranking, from high to none, was utilized to discern between areas where failures were anticipated to occur, and would require subsequent rock support, and those of a less critical nature.

The analysis indicates that along the south wall of the Approach and Discharge Channels instabilities may be introduced into the rock mass due to a potential for toppling along Joint Sets $J1_{SP}$ and $J4_{SP}$, respectively.

As shown in Table 9, the presence of Joint Sets $J1_{SP}$, $J2_{SP}$ and $J4_{SP}$ result in a relatively moderate potential for planar sliding along the North Walls of both the Approach and Discharge Channels. The analysis revealed that Joint Set $J2_{SP}$ may also compromise the stability of the South Wall of the Approach Channel. However, the probability of a block sliding along this plane, out of the wall, is relatively low because the strike of Joint Set $J2_{SP}$ is nearly perpendicular to that of this wall. The shallow nature of Joint Set $J3_{SP}$ precludes sliding along this plane even where the joints dip out of the face of the wall, as with the South Walls of the Channels.

5.3 Wedge Stability

Determination of whether a wedge, formed by the intersection of two joints, may slide out of the face of the wall is primarily influenced by two criteria:

The line of intersection formed by the two joints must have a trend that plunges out of the face of the wall.

The slope of the wall must be steeper than the plunge of the line of intersection.

If both criteria are met, the wedge formed by the intersection of the two joint sets may compromise the stability of the wall. The probability of failure is influenced by the steepness of the plunge of the line of intersection. With a very shallow plunge and high friction angles along the surface of the joints, the driving forces along the wedge boundary may not exceed the resisting forces.

5.3.1 Powerhouse Area

The exposure of the lines of intersection between the joint sets, with respect to the excavation faces within the Powerhouse structure, is summarized in Table 10. The safety factors, calculated with SWEDGE version 4.0-3D for Surface Wedge Analysis for Slope, are summarized in Table 11.

As can be seen in Table 10 and Table 11, there are several cases where the line of intersection formed between two joint sets dips out of the face of the excavation. Over 82% (29 out of 35) of the wedges formed by the joints intersection have a Factor of Safety (50%) above 1.5. All of the cases can be categorized into five different scenarios. In the first two scenarios the factor of safety against sliding is greater than 1.5, and consequently it can be reasonably assessed that the stability of the excavation face will not be compromised. For the third scenario, the factor of safety is typically very low; however, the weight and size of the wedges formed will be sufficiently low to suggest that failure will not be critical. The final two scenarios occur where the factor of safety is less than 1.5, and the size and weight of the blocks is great enough to indicate that a rock support system will have to be utilized to stabilize the wedges.

Scenario 1

The following lines of intersection plunge at such shallow angles that even where the line plunges out of the face of the wall, sliding will not occur: $J1_{PH}$&$J2_{PH}$; $J1_{PH}$&$J3_{PH}$; $J1_{PH}$ &$J4_{PH}$; $J1_{PH}$&$J5_{PH}$; $J1_{PH}$&$J6_{PH}$; $J2_{PH}$&$J5_{PH}$; $J3_{PH}$&$J4_{PH}$; $J3_{PH}$&$J6_{PH}$; and $J4_{PH}$&$J6_{PH}$.

Scenario 2

The line of intersection between the following Joint Sets, dip out of the face of the specified excavations:

- $J2_{PH}$&$J3_{PH}$ and $J2_{PH}$ &$J6_{PH}$ - West Wall of the Powerhouse,
- $J2_{PH}$&$J4_{PH}$ - South Wall of both Channels,
- $J4_{PH}$&$J5_{PH}$ - South Wall of the Intake Channel.

However, the Factor of Safety against sliding is sufficiently high to suggest that the aforementioned cases will not pose a threat to the stability of these walls.

Scenario 3

There are three critical cases where the line of intersection trends nearly parallel to, and dips out of, the wall:

- $J2_{PH}$&$J3_{PH}$ - North Wall of the Intake Channel,
- $J2_{PH}$&$J4_{PH}$ - West Wall of the Powerhouse,
- $J2_{PH}$ $J6_{PH}$ - South Wall of the Tailrace Channel.

Where this occurs, small slivers/wedges of rock may form and slide out of the face of the excavation. The relatively small size and weight of these blocks typically results in factors of safety significantly less than 1.0.

Scenario 4

For the following cases the lines of intersection dip out of the face of the excavation, and could potentially compromise the stability of the rock mass:

- $J3_{PH}$&$J5_{PH}$ and $J2_{PH}$ &$J3_{PH}$ - North Wall of the Tailrace,
- $J3_{PH}$&$J5_{PH}$ - North Wall of the Intake Channel and West Wall of the Powerhouse,
- $J4_{PH}$&$J5_{PH}$ - South Wall of the Tailrace Channel.

However, in the aforementioned cases the Factor of Safety is only less than 1.5 when the rock mass is completely saturated. Therefore, the installation of drain holes along the face of the excavations will increase the stability of the rock mass.

Scenario 5

The most critical cases occur with the wedges formed between Joint Sets $J5_{PH}$ &$J6_{PH}$, with respect to the North Walls of both Channels and the West Wall of the Powerhouse. These wedges will pose a threat to the stability of the rock mass along the aforementioned walls and will require a rock support system to stabilize the wedges, details of which are discussed in Section 6. Of note is that Joint Sets $J5_{PH}$ and $J6_{PH}$ have a low frequency of occurrence. Consequently, even though the aforementioned scenarios have low Factors of Safety, the probability of occurrence is also low.

5.3.2 Spillway Area

The exposure of the lines of intersection between the joint sets, with respect to the excavation faces within the Spillway structure, is summarized in Table 12. The safety factors, calculated with SWEDGE version 4.0 – 3D for Surface Wedge Analysis for Slope, are summarized in Table 13.

As can be seen in Table 12 and Table 13, over 82% (19 out of 23) of the wedges formed by the joints intersection have a Factor of Safety (50%) above 1.5, although the line of intersection between Joint Sets $J1_{SP}$&$J2_{SP}$ dips out of the face of both of the North Walls, the Factor of Safety against sliding is sufficiently high to indicate that the stability of this wall will not be compromised by wedges formed between these joint sets. Conversely, a rock support system will be required to ensure wedges formed between Joint Sets $J1_{SP}$&$J4_{SP}$ do not slide out of the North Walls of the Channels.

Although the lines of intersection between Joint Sets $J1_{SP}$&$J3_{SP}$, and $J2_{SP}$&$J4_{SP}$ dip out of the face of the South Walls of both Channels, the plunge of these lines is not sufficiently steep to allow mobilization of a wedge. The lines of intersections formed between Joint Sets $J2_{SP}$&$J3_{SP}$ and $J3_{SP}$&$J4_{SP}$ do dip out of the face of the South Walls at steep enough angles to facilitate sliding of the wedges. Consequently, instabilities may be introduced into the rock mass during and following excavation. The critical case only occurs when the rock mass is completely saturated; therefore, installation of drain holes along the face of the wall is required to facilitate stabilization of the rock mass. However, the weight and size of the wedges formed between Joint Sets $J2_{SP}$&$J3_{SP}$ and $J3_{SP}$&$J4_{SP}$ will be small enough to suggest that the removal of these wedges can be undertaken easily with the excavation equipment. The removal of these small wedges eliminates the hazard of these wedges popping out into the excavation and hence eliminates the need for detailed rock support for this particular joint set.

In summary, several of the joint sets and wedges developed are oriented in such a way that, for the aforementioned slope faces, they introduce instability in the rock mass. Therefore, permanent rock treatment consisting of drainage holes and patterned rock support will be required for stability and safety.

The rock stability analysis summarized in this memorandum assessed sliding failure along single discontinuities and discontinuity intersections that may develop between two joint sets; however, failure involving more than two joint sets or internal deformation and fractures was not assessed. Measures should be employed in the field to identify and secure any instabilities, even those not identified in this analysis.

The severity of a failure, and the subsequent prevention techniques employed, will depend primarily on the weight of the block mobilized. Toppling or scaling of the rock can usually be controlled by removal of the loose blocks during excavation, and the use of wire mesh to contain future instabilities. However, if the toppling blocks are sufficiently large, more rigorous support may be required. Planar and wedge sliding typically involves larger blocks, and as such usually requires the use of rock bolts or anchors.

6 Treatment Considerations

6.1 Minimum Treatment

Planning for rock support is necessary; however, the design can only be considered preliminary until rock excavations expose the actual conditions within the bedrock. In general, all excavated rock faces will be assumed to receive the following minimum rock treatment:

Overall rock face will be sloped at maximum 1H:12V, assuming approximately 6 m bench heights and a 0.5 m toe-offset per bench. Along the west face of the Powerhouse rock noses the specified 1H:2V slope can be achieved by sloping the bench faces from the vertical. Note that the maximum rock cut height at Keeyask will be approximately 26 m, located within the Powerhouse excavation.

Rock faces will be inspected and scaled during construction.

A 6 x 6 m pattern of 6 m long, 50 mm diameter drain holes will be required. These holes will be inclined at 6° from the horizontal, dipping out of the rock face.

Temporary protection from falling rock will be required and provided by chain link mesh, which will cover all rock slopes within the construction area where falling rock will pose a threat to human safety. The chain link mesh will be fastened as required to the rock slope using 0.6 m long mechanical mesh pins. It has been assumed that mesh pins will be installed at a rate of 1 pin per 20 m^2 of facial area.

Rock dowels will be utilized along the edge of the excavations for additional support and for use in connection with the chain link mesh. These dowels will be spaced at 6 m centers, 1.5 m back from a stable final cut line, and prior to the commencement of excavation. It is presently envisioned that 20 mm diameter dowels, 1.5 m in length, installed vertically, will be utilized. Cement grout will be used to anchor the dowels.

6.2 Recommended Rock Support

In addition to the minimum treatment noted above, it is recommended that permanent rock support be provided for the South and North Walls of the channel rock cuts, outside the confines of the Powerhouse and Spillway structures. Temporary rock support will also be required on all excavation faces within the concrete structures. After placement of structural concrete, the rock treatment in these areas becomes less significant. The height and/or slope of the excavations along the East Wall of the Powerhouse, and the East and West Walls of the Spillway, are sufficiently low such that unstable rock blocks or fragments exposed during excavation may be treated using spot bolts, or the complete removal of the infringing rock block or wedge type specified will depend on the type of support required, excavation height, dominant joint set and drainage conditions along the excavation face. The rock bolt pattern for the temporary support of the rock faces will differ from that of the permanent design because the project specifications, as stated in the Geotechnical Design Criteria [Ref 3], require a Factor of Safety for temporary and permanent slopes of 1.3 and 1.5, respectively.

Table 14 summarizes the anticipated rock treatment recommended for each of the excavation faces at Keeyask GS. The bolts for the Channels Walls are considered permanent, and thus require cement grout encapsulation for corrosion protection.

Actual bolt configurations will be confirmed during final design and modified to suit the actual conditions encountered during excavation. To simplify installation and reduce the potential intersection or interference of drilling drain holes across previously installed rock bolts, bolting will also be inclined at 6° from the horizontal (dipping out of the face).

Spot bolting of individual prominent features which cannot be treated by pattern bolting will be treated by adjusting or relocating the existing pattern bolts.

7 Environmental Issues and Mitigation Alternatives

The bedrock excavated will be competent enough to be reused as concrete aggregate and rock product. Consequently, it is currently envisaged that all of the excavated bedrock material will be utilized for construction materials, and will not require allocation for permanent storage.

All rock excavations will proceed within the confines of a cofferdam; therefore, any fine grained material that is disturbed during excavation will not be released into the river. As such, the fish habitat in the vicinity of the Generating Station should not be adversely affected.

A system of pumps will be employed within the cofferdams to remove infiltrated water during excavation for some of the structures. It is presently envisioned that the pumps will release the water back into the river. Water pumped into the river shall meet with the Canadian Environmental Quality Guidelines [Ref 7] for freshwater containing aquatic life and Manitoba water quality standards [Ref 8]; details of the dewatering schemes will be provided in the Stage IV design memoranda for the cofferdams.

To ensure the impact on the native fish species and other aquatic life is limited, blasting designs will employ techniques that conform to the Fisheries Act. Care will also be taken regarding the blast induced vibrations and the chemicals used in the explosives.

Blast induced noise may also interfere with the nesting activities of wildlife in the area, and cause individual animals to avoid the work area or alter their pattern of movement. Production of these noises cannot be avoided and their affects are anticipated to be short term, only for the duration of the construction. The Quarry Minerals Regulation prohibits blasting between 4 pm and 9 am, at any time on a Saturday, Sunday or Statutory holiday, unless otherwise approved by the director under the Mines and Minerals Act [Ref 10]. To limit the disturbance to wildlife, excavation activities will not be continuous within a 24-hour period.

Recommended setback distances (m) from the center of detonation of a confined explosive, to the fish habitat, to achieve the 100 kPa guideline criteria for various substrates are provided in Table 15.

8 Quantity Estimates

8.1 Rock Excavation

Quantity estimates for the volume of rock that will be removed during excavation of the Principal Structures were undertaken, along with an estimate of over-break and the length of rock drilling required for pre-shear and line drilling.

Rock Excavation and over-break quantities are presented in Appendix C, Table C1 and C2. In total it is estimated over 1,997,000 m^3 of rock will be excavated for construction of the Principal Structures at Keeyask GS. The volumes measured assume neat excavation lines as the pay-lines. Over-break volumes were estimated, assuming the walls are excavated to within 100 mm of the pay-line, and the inverts are excavated to within 200 mm of the pay-line. In total, approximately 5,700 m^3 of additional rock may be excavated outside the boundaries shown on the construction drawings for the concrete structures. As well, an additional 77,000 m^3 of rock may be excavated outside the boundaries of the channel excavations depicted on the construction drawings.

As part of the controlled blasting techniques employed for the excavation of rock, approximately 62,100 m of pre-shear drilling and approximately 7,300 m of line drilling was estimated. Details of the estimate performed for the drilling that will be required as part of the blasting operations is presented in the attached Table C3.

8.2 Rock Support

Applying the recommendations provided herein would require an estimated 12,200 m of rock bolting consisting of about 1,280 bolts, ranging in length from 3 m to 12 m. However, this is a conservative estimate; as it is anticipated that after the actual rock conditions are exposed during construction, the majority of the provisional rock support will not be required. Therefore, an allowance of approximately two thirds of this number is recommended for cost estimating purposes (8,100 m of rock bolting, and 850 bolts). For simplicity, it is assumed that all bolts will be grouted. Additionally, approximately 11,400 m^2 of safety mesh and 160 m of 1.5 m rock dowels, installed vertically along the edge of the channels have also been estimated. Refer to Appendix D, Table D1 for a detailed summary of the estimated rock support.

For temporary drainage of the rock mass during construction, it is estimated that about 1,200 m of drilling will be required. Additionally, over 5,500 m of drilling was estimated for installation of permanent drain holes along the Channel excavations. All of the drainage holes will consist of 50 mm diameter holes, on 6 m centers. Estimate details are provided in Appendix D, Table D2.

Prepared By

H. Liu

R.A. Halim

P.R. Pantel

Attach
References
Tables 1 to 15
Appendices A to D
Figures 1 – 14

PRP/RAH:spa

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Tables

Tables 1 to 15

Table 1
Keeyask GS – Stage IV Studies, Axis GR-4
Joint Set Orientation

Joint Set		Orientation			Description	Designation
		Strike	Dip[(1)]	Dip Direction[(1)]		
Powerhouse	$J1_{PH}$	198°	1°	288°	Sub-horizontal	Major
	$J2_{PH}$	320°	30°	50°	-	Major
	$J3_{PH}$	60°	23°	150°	-	Major
	$J4_{PH}$	237°	25°	327°	-	Major
	$J5_{PH}$	335°	80°	65°	Sub-vertical	Minor
	$J6_{PH}$	65°	53°	155°	-	Minor
Spillway	$J1_{SP}$	25°	32°	115°	-	Major
	$J2_{SP}$	126°	30°	216°	-	Major
	$J3_{SP}$	210°	25°	300°	-	Minor
	$J4_{SP}$	103°	72°	193°	Sub-vertical	Minor

Notes:
[(1)]Dip measured from horizontal. Dip direction is 90° right of strike.

Table 2
Keeyask GS – Stage IV Studies, Axis GR-4
RQD Classification System

RQD (%)	Rock Quality
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
<25	Very Poor

Table 3
Keeyask GS – Stage IV Studies, Axis GR-4
Peak Particle Velocity near Concrete and Grout

Time in Place	Peak Particle Velocity (mm/s)
Less than 6 hours	10
Less than 28 days	50
After 28 days	75

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Powerhouse Area - Joint Set Exposure at Rock Faces

Joint Set	Intake Channel		Tailrace Channel		West Wall of Powerhouse
	North Wall	South Wall	North Wall	South Wall	
$J1_{PH}$	Oblique to face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J2_{PH}$	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips out of face
$J3_{PH}$	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips out of face
$J4_{PH}$	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J5_{PH}$	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips out of face
$J6_{PH}$	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips out of face

Table 5
Keeyask GS – Stage IV Studies, Axis GR-4
Powerhouse Area - Potential for Toppling

Joint Set	Intake Channel		Tailrace Channel		West Wall of Powerhouse
	North Wall	South Wall	North Wall	South Wall	
$J1_{PH}$	None	None	None	None	None
$J2_{PH}$	V. Low	None	None	None	None
$J3_{PH}$	None	None	None	None	None
$J4_{PH}$	None	None	None	None	None
$J5_{PH}$	None	None	None	None	V. Low
$J6_{PH}$	None	High	None	High	None

Notes:
High = 50%-100% V. Low = < 10%
Moderate = 50%- 25% None = 0%
Low = 25%-10%

Table 6
Keeyask GS – Stage IV Studies, Axis GR-4
Powerhouse Area - Potential for Planar Sliding

Joint Set	Intake Channel		Tailrace Channel		West Wall of Powerhouse
	North Wall	South Wall	North Wall	South Wall	
$J1_{PH}$	None	None	None	None	None
$J2_{PH}$	None	Moderate	None	Moderate	Moderate
$J3_{PH}$	None	None	None	None	None
$J4_{PH}$	None	None	None	None	None
$J5_{PH}$	V. Low	Moderate	V. Low	Moderate	Low
$J6_{PH}$	High	None	High	None	Low

Notes:
High = 50%-100% V. Low =<10%
Moderate = 50%- 25% None = 0%
Low = 25%-10%

Table 7
Keeyask GS – Stage IV Studies, Axis GR-4
Spillway Area - Joint Set Exposure at Rock Faces

Joint Set	Approach Channel		Discharge Channel	
	North Wall	South Wall	North Wall	South Wall
$J1_{SP}$	Dips out of face	Dips into face	Dips out of face	Dips into face
$J2_{SP}$	Dips out of face	Oblique to face	Dips out of face	Dips into face
$J3_{SP}$	Dips into face	Dips out of face	Dips into face	Dips out of face
$J4_{SP}$	Dips out of face	Dips into face	Dips out of face	Dips into face

Table 8
Keeyask GS – Stage IV Studies – Axis GR-4
Spillway Area - Potential for Toppling

Joint Set	Approach Channel		Discharge Channel	
	North Wall	South Wall	North Wall	South Wall
$J1_{SP}$	None	Low	None	None
$J2_{SP}$	None	None	None	None
$J3_{SP}$	None	None	None	None
$J4_{SP}$	None	None	None	Low

Notes:
High = 50%-100% V. Low =<10%
Moderate = 50%- 25% None = 0%
Low = 25%-10%

Table 9
Keeyask GS – Stage IV Studies, Axis GR-4
Spillway Area - Potential for Planar Sliding

Joint Set	Approach Channel		Discharge Channel	
	North Wall	South Wall	North Wall	South Wall
$J1_{SP}$	Moderate	None	Moderate	None
$J2_{SP}$	Moderate	Low	Moderate	None
$J3_{SP}$	None	V. Low	None	V. Low
$J4_{SP}$	High	none	High	None

Notes:
High = 50%-100% V. Low =<10%
Moderate = 50%- 25% None = 0%
Low = 25%-10%

Table 10
Keeyask GS – Stage IV Studies, Axis GR-4
Powerhouse Area - Wedge Intersection Exposure at Rock Faces

Wedges	Trend/ Plunge	Intake Channel		Tailrace Channel		West Wall of Powerhouse
		North Wall	South Wall	North Wall	South Wall	
$J1_{PH}$&$J2_{PH}$	321°/01°	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J1_{PH}$&$J3_{PH}$	238°/01°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips into face
$J1_{PH}$&$J4_{PH}$	238°/01	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips into face
$J1_{PH}$&$J5_{PH}$	335°/01°	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J1_{PH}$&$J6_{PH}$	244°/01°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips into face
$J2_{PH}$&$J3_{PH}$	107°/17°	Parallel to face	Dips into face	Dips out of face	Dips into face	Dips out of face
$J2_{PH}$&$J4_{PH}$	2°/21°	Dips into face	Dips out of face	Dips into face	Dips out of face	Parallel to face
$J2_{PH}$&$J5_{PH}$	336°/09°	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J2_{PH}$&$J6_{PH}$	86°/25°	Dips into face	Dips into face	Parallel to face	Parallel to face	Dips out of face
$J3_{PH}$&$J4_{PH}$	238°/01°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips into face
$J3_{PH}$&$J5_{PH}$	150°/23°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips out of face
$J3_{PH}$&$J6_{PH}$	67°/03°	Dips into face	Parallel to face	Dips into face	Dips out of face	Dips out of face
$J4_{PH}$&$J5_{PH}$	339°/24°	Dips into face	Dips out of face	Dips into face	Dips out of face	Dips into face
$J4_{PH}$&$J6_{PH}$	243°/03°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips into face
$J5_{PH}$&$J6_{PH}$	142°/52°	Dips out of face	Dips into face	Dips out of face	Dips into face	Dips out of face

Table 11
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of the Wedge Stability Analysis for Powerhouse

Structure	Face	Joint Intersection		Factor of Safety		Comments
		Joints	Trend/Plunge	FS(50%)	FS(100%)	
Intake Channel	North Wall	$J1_{PH}$&$J3_{PH}$	238°/01°	72.47	59.94	Bench W = 1,416 m , wt = 1.89×10^{7} t, H=22.5 m
		$J1_{PH}$ &$J4_{PH}$	238°/01°	76.79	62.88	Bench W = 1,420 m, wt = 1.70×107 t, H= 22.5 m
		$J1_{PH}$ &$J6_{PH}$	244°/01°	65.1	53.78	Bench W = 1,131 m, wt = 1.45×107 t, H= 22.5 m
		$J2_{PH}$ &$J3_{PH}$	107°/17°	0.58	0	Bench W= 3.0 m, wt = 207 t, H= 22.5 m
		$J3_{PH}$ &$J4_{PH}$	238°/01°	77.04	63.75	Bench W= 1,380 m, wt = 1.91×107 t, H= 22.5 m
		$J3_{PH}$ &$J5_{PH}$	150°/23°	1.91	1.42	Bench W= 37.4 m, wt = 2.52×104 t, H= 22.5 m
		$J4_{PH}$ &$J6_{PH}$	243°/03°	21.21	17.48	Bench W= 306 m, wt = 2.76×105 t, H= 22.5 m
		$J5_{PH}$ &$J6_{PH}$	142°/52°	0.57	0.12	Bench W= 9.5 m, wt = 1,799 t, H= 22.5 m
	South Wall	$J1_{PH}$ &$J2_{PH}$	321°/01°	56.37	46.62	Bench W= 1,404 m, wt = 2.14×107 t, H = 22.5 m
		$J1_{PH}$ &$J5_{PH}$	335°/01°	69.69	57.62	Bench W= 1,859 m, wt = 2.76×107 t, H = 22.5 m
		$J2_{PH}$ &$J4_{PH}$	2°/21°	2.23	1.78	Bench W = 55 m, wt = 1.01×105 t, H= 22.5 m
		$J2_{PH}$ &$J5_{PH}$	336°/09°	9.27	7.65	Bench W = 133 m, wt = 5.05×104 t, H= 22.5 m
		$J3_{PH}$ &$J6_{PH}$	67°/03°	35.21	26.06	Bench W = 61 m, wt = 6.06×104 t, H= 22.5 m
		$J4_{PH}$ &$J5_{PH}$	339°/24°	1.93	1.5	Bench W = 48 m, wt = 6.95×104 t, H= 22.5 m
Tailrace Channel	North Wall	$J1_{PH}$ &$J3_{PH}$	238°/01°	72.47	59.91	Bench W = 1,037 m, wt = 1.48×107 t, H = 22.5 m
		$J1_{PH}$ &$J4_{PH}$	238°/01°	76.73	62.39	Bench W = 1,041 m, wt = 1.28×107 t, H = 22.5 m
		$J1_{PH}$ &$J6_{PH}$	244°/01°	65.09	53.7	Bench W = 766 m, wt = 1.04×107 t, H = 22.5 m
		$J2_{PH}$ &$J3_{PH}$	107°/17°	1.76	0.47	Bench W = 20 m, wt = 1.03×104 t, H = 22.5 m
		$J3_{PH}$ &$J4_{PH}$	238°/01°	77.04	63.74	Bench W = 1,011 m, wt = 1.92×106 t, H = 22.5 m
		$J3_{PH}$ &$J5_{PH}$	150°/23°	1.914	1.47	Bench W = 45 m, wt = 4.35×104 t, H = 22.5 m
		$J4_{PH}$ &$J6_{PH}$	243°/03°	21.2	17.43	Bench W = 212 m, wt = 2.64×105 t, H = 22.5 m
		$J5_{PH}$ &$J6_{PH}$	142°/52°	0.58	0.20	Bench W = 12.5 m, wt = 3,984 t, H = 22.5 m
	South Wall	$J1_{PH}$ &$J2_{PH}$	321°/01°	56.37	46.61	Bench W = 1206 m, wt = 1.65×107 t, H = 22.5 m
		$J1_{PH}$ &$J5_{PH}$	335°/01°	69.68	57.6	Bench W = 1717 m, wt = 2.28×107 t, H = 22.5 m
		$J2_{PH}$ &$J4_{PH}$	2°/21°	2.23	1.78	Bench W = 57 m, wt = 7.65×104 t, H = 22.5 m
		$J2_{PH}$ &$J5_{PH}$	336°/09°	9.28	7.71	Bench W = 124 m, wt = 5.55×104 t, H = 22.5 m
		$J2_{PH}$ &$J6_{PH}$	86°/25°	0.74	0	Bench W = 2.43 m, wt = 171 t, H = 22.5 m
		$J3_{PH}$ &$J6_{PH}$	67°/03°	35.49	28.24	Bench W = 159 m, wt = 1.01×105 t, H = 22.5 m
		$J4_{PH}$ &$J5_{PH}$	339°/24°	1.92	1.46	Bench W = 45 m, wt = 3.99×104 t, H = 22.5 m
West Wall	West Wall	$J2_{PH}$ &$J3_{PH}$	107°/17°	2.78	2.24	Bench W = 68 m, wt = 7.90×104 t, H = 22.5 m
		$J2_{PH}$ &$J4_{PH}$	2°/21°	0	0	Bench W = 0.50 m, wt = 4.5 t, H = 22.5 m
		$J2_{PH}$ &$J6_{PH}$	86°/25°	1.98	1.55	Bench W = 47 m, wt = 3.52×104 t, H = 22.5 m
		$J3_{PH}$ &$J5_{PH}$	150°/23°	1.92	1.40	Bench W = 25 m, wt = 1.63×104 t, H = 22.5 m
		$J3_{PH}$ &$J6_{PH}$	67°/03°	35.66	29.66	Bench W = 382 m, wt = 1.58×105 t, H = 22.5 m
		$J5_{PH}$ &$J6_{PH}$	142°/52°	0.66	0.32	Bench W = 9.6 m, wt = 2,337 t, H = 22.5 m

Notes:
(1) Bench W = wedge width on upper face
(2) wt = wedge weight
(3) H = wedge height
(4) FS (50%) and FS (100%) refer to Factor of Safety with the water infill 50% and 100% height of the joint respectively

Table 12
Keeyask GS – Stage IV Studies, Axis GR-4
Spillway Area - Wedge Intersection Exposure at Rock Faces

Wedges	Trend/ Plunge	Approach Channel		Discharge Channel	
		North Wall	South Wall	North Wall	South Wall
$J1_{SP}$&$J2_{SP}$	167°/21°	Dips out of face	Dips into face	Dips out of face	Dips into face
$J1_{SP}$&$J3_{SP}$	27°/1.5°	Dips into face	Dips out of face	Dips into face	Dips out of face
$J1_{SP}$&$J4_{SP}$	115°/32°	Dips out of face	Dips into face	Dips out of face	Dips into face
$J2_{SP}$&$J3_{SP}$	265°/21°	Parallel to face	Dips out of face	Dips into face	Dips out of face
$J2_{SP}$&$J4_{SP}$	278°/15°	Dips into face	Dips out of face	Dips into face	Dips out of face
$J3_{SP}$&$J4_{SP}$	275°/23°	Parallel to face	Dips out of face	Dips into face	Dips out of face

Table 13
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of the Wedge Stability Analysis for Spillway

Structure	Face	Joint Selection		Factory of Safety		Comments
		Joints	Trend/Plunge	FS(50%)	FS(100%)	
Approach Channel	North Wall	$J1_{SP}$&$J2_{SP}$	167˚/21˚	2.33	1.86	Bench W = 64 m , wt = 7.48×10^4 t, t, H=25 m
		$J1_{SP}$ &$J2_{SP}$	167˚/21˚	2.33	1.86	Bench W = 20.5 m, wt = 2,450 t, H=8 m
		$J1_{SP}$ &$J4_{SP}$	115˚/32˚	1.29	0.90	Bench W= 6.5 m, wt = 476 t, H= 8 m
	South Wall	$J1_{SP}$ & $J3_{SP}$	27˚/1.5˚	39.92	33.04	Bench W= 201 m, wt = 1.22×10^6 t, , H = 25 m
		$J1_{SP}$ & $J3_{SP}$	27˚/1.5˚	39.92	33.04	Bench W= 64 m, wt = 3.99×10^4 t, , H = 8 m
		$J2_{SP}$ & $J3_{SP}$	265˚/21˚	1.73	1.23	Bench W = 15 m, wt = 5,490 t, H= 8 m
		$J2_{SP}$ & $J4_{SP}$	278˚/15˚	5.87	4.61	Bench W = 25 m, wt = 940 t, H= 8 m
		$J3_{SP}$ & $J4_{SP}$	275˚/23˚	1.74	1.33	Bench W = 15.5 m, wt = 6,283 t, H= 8 m
		$J3_{SP}$ & $J4_{SP}$	275˚/23˚	1.69	1.01	Bench W = 7.3 m, wt = 509 t, H = 8 m
Discharge Channel	North Wall	$J1_{SP}$ &$J2_{SP}$	167˚/21˚	2.33	1.86	Bench W = 61.2 m, wt = 7.61×10^4 t, H = 25 m
		$J1_{SP}$ &$J2_{SP}$	167˚/21˚	2.33	1.86	Bench W = 19.6 m, wt = 2,493 t, H = 8 m
		$J1_{SP}$ & $J4_{SP}$	115˚/32˚	1.30	0.96	Bench W = 10.0 m, wt = 872 t, H = 8 m
	South Wall	$J1_{SP}$ & $J3_{SP}$	27˚/1.5˚	39.92	33.03	Bench W = 627 m, wt = 1.11×10^6 t, H = 25 m
		$J1_{SP}$ & $J3_{SP}$	27˚/1.5˚	39.92	33.03	Bench W = 201 m, wt = 3.63×10^4 t, H = 8 m
		$J2_{SP}$ & $J3_{SP}$	265˚/21˚	1.60	0.37	Bench W = 7.4 m, wt = 385 t, H = 8 m
		$J2_{SP}$ & $J4_{SP}$	278˚/15˚	5.84	4.35	Bench W = 16.4 m, wt = 640 t, H = 8 m
		$J3_{SP}$ & $J4_{SP}$	275˚/23˚	1.71	1.14	Bench W = 9.6 m, wt = 879 t, H = 8 m

Notes:

(1) Bench W = wedge width on upper face
(2) wt = wedge weight
(3) H = wedge height
(4) FS (50%) and FS (100%) refer to Factor of Safety with the water infill 50% and 100% height of the joint respectively

Table 14
Keeyask GS – Stage IV Studies, Axis GR-4
Powerhouse and Spillway Excavations - Rock Treatment

Excavation Face		Bolt Spacing[(1)] (m x m)	Bolt Diameter (mm)	Bolt Length (m)	Type of Support
Powerhouse Complex	North Wall	4 x 4	25	6	Temporary
	South Wall	Spot	25	6	Temporary
	West Wall (U/S)	3 x 3	25	12	Temporary
	East Wall (D/S)	Spot	25	3	Temporary
Powerhouse Channels	North Walls	4 x 4	25	6	Permanent
	South Walls	8 x 8 and 8 x 5	25	6	Permanent
Spillway Structure	North Wall	3 x 3	25	12	Temporary
	South Wall	Spot	25	6	Temporary
	West Wall (U/S)	Spot	25	3	Temporary
	East Wall (D/S)	Spot	25	3	Temporary
Spillway Channels	North Walls	3 x 3	25	12	Permanent
	South Walls	6 x 6	25	6	Permanent

Note:
(1) Along the channel excavations the specified bolt spacing will only be applied within 10 m of the edge of concrete, outside these limits the bolt pattern will be reduced to spot bolting.

Table 15
Keeyask GS – Stage IV Studies, Axis GR-4
Setback Distances from Center of Detonation - Confined Explosives

Setback Distances (m)	Weight of Explosive Charge (kg)							
	0.5	1	2	5	10	25	50	100
Distance to fish habitat [(1)]	3.5	5.0	7.1	11.0	15.9	25.0	35.6	50.3
Distance to spawning habitat [(2)]	10.7	15.1	21.4	33.7	47.8	75.5	106.7	150.9

Notes:
(1) The guidelines requires that the explosive charge does not induce an overpressure in the fish bladder which exceed 100 kPa.
(2) The guidelines require that the explosive charge cannot produce a PPV which exceed 13 mm/s in spawning beds.

Appendix A

Bedrock Summary Logs
Table A1 and A2

Table A1

GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-008	Geological	Drill Run			Recovery		RQD		Jts	Average[3]	Recovery (%)	RQD (%)	Joints per run
Date: Jul. 24/03	Description[3]	Depth (m - along core axis)		Length	Length		Length		per	Spacing	0 25 50 75 100	0 25 50 75 100	0 5 10 15 20 25
		From	To	(m)	(m)	(%)	(m)	(%)	run	per Jt. (mm)			
Dip: 90 deg.	Greywacke Gneiss: grey to dark grey, fine grained to very fine grained, strong, indistinct foliation at 0 to 30 degrees to the core axis, moderately spaced, good to excellent quality, low permeability.	9.61	11.13	1.52	1.51	99	1.38	91	8 +	168			
Azimuth: - deg.		11.13	12.66	1.53	1.47	96	1.31	86	10 +	134			
Grd. Elev.: 146.68 m		12.66	14.18	1.52	1.52	100	1.39	91	7	190			
Depth: 20.29 m		14.18	15.71	1.53	1.53	100	1.29	84	7	191			
		15.71	17.23	1.52	1.52	100	1.35	89	8	169			
		17.23	18.74	1.51	1.51	100	1.51	100	3	378			
		18.74	19.41	0.67	0.67	100	0.67	100	1	335			
		19.41	20.29	0.88	0.71	81	0.65	74	5 +	118			
			Total =	10.68	10.44		9.55		49				
			Average =	1.34		97.8		89	6	214			

Drill Hole: 03-009	Geological	Drill Run			Recovery		RQD		Jts	Average[3]	Recovery (%)	RQD (%)	Joints per run
Date: Jul. 18/03	Description[3]	Depth (m - along core axis)		Length	Length		Length		per	Spacing	0 25 50 75 100	0 25 50 75 100	0 5 10 15 20 25
		From	To	(m)	(m)	(%)	(m)	(%)	run	per Jt. (mm)			
Dip: 90 deg.	Greywacke Gneiss: grey to dark grey, very fine grained, strong, indistinct foliation at 10 to 15 degrees to the core axis, closely spaced, good quality, low permeability.	8.97	9.74	0.77	0.77	100	0.66	86	7	96			
Azimuth: - deg.		9.74	9.91	0.17	0.17	100	0.13	76	2	57			
Grd. Elev.: 147.08 m		9.91	10.31	0.40	0.40	100	0.32	80	2	133			
Depth: 18.99 m		10.31	10.91	0.60	0.60	100	0.47	78	9	60			
		10.91	11.29	0.38	0.38	100	0.30	79	2	127			
		11.29	12.87	1.58	1.58	100	1.10	70	21 +	72			
		12.87	14.40	1.53	1.53	100	1.26	82	12	118			
		14.40	15.94	1.54	1.54	100	1.39	90	8	171			
		15.94	17.46	1.52	1.52	100	1.25	82	13 +	109			
		17.46	18.99	1.53	1.50	98	1.43	93	6	214			
			Total =	10.02	9.99		8.31		82				
			Average =	1.00		99.7		83	8	121			

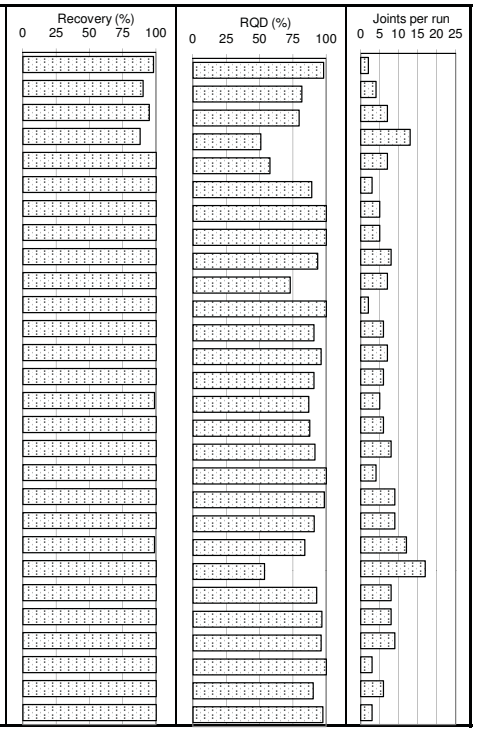
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-010
Date: Jul. 27/03

Dip: 70 deg.
Azimuth: 231 deg.
Grd. Elev.: 138.16 m
Depth: 43.52 m

Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery Length (%)	RQD Length (m)	RQD Length (%)	Jts per run	Average[3] Spacing per Jt. (mm)
3.79-32.44 Greywacke gneiss: light to medium greenish grey, coarse to very coarse grained, strong, distinct to indistinct planar to irregular foliation, closely to moderately spaced, good quality, low permeability, occasional granitic bands.	3.79	5.34	1.55	1.52	98	1.52	98	2 +	507
	5.34	6.16	0.82	0.74	90	0.67	82	4 +	148
	6.16	7.68	1.52	1.44	95	1.21	80	7 +	180
	7.68	9.27	1.59	1.40	88	0.81	51	13 +	100
32.44-39.50 Granite: light to medium greenish grey, coarse to very coarse grained, strong, no distinct foliation, very closely spaced, good quality, low permeability.	9.27	9.72	0.45	0.45	100	0.26	58	7	56
	9.72	10.82	1.10	1.10	100	0.98	89	3 +	275
39.50-43.52 Greywacke gneiss: light to medium greenish grey, coarse to very coarse grained, moderately strong, distinct to indistinct planar to irregular foliation, moderately spaced, excellent quality, low permeability, occasional granite intrusions.	10.82	12.35	1.53	1.53	100	1.53	100	5	255
	12.35	13.87	1.52	1.52	100	1.52	100	5	253
	13.87	15.40	1.53	1.53	100	1.43	93	8	170
	15.40	16.91	1.51	1.51	100	1.10	73	7	189
	16.91	18.44	1.53	1.53	100	1.53	100	2	510
	18.44	19.96	1.52	1.52	100	1.38	91	6	217
	19.96	21.48	1.52	1.52	100	1.46	96	7	190
	21.48	23.00	1.52	1.52	100	1.38	91	6	217
	23.00	24.52	1.52	1.50	99	1.32	87	5 +	250
	24.52	26.05	1.53	1.53	100	1.34	88	6	219
	26.05	27.57	1.52	1.52	100	1.39	91	8	169
	27.57	29.09	1.52	1.52	100	1.52	100	4	304
	29.09	30.49	1.40	1.40	100	1.38	99	9	140
	30.49	32.03	1.54	1.54	100	1.40	91	9	154
	32.03	33.58	1.55	1.53	99	1.30	84	12 +	118
	33.58	35.09	1.51	1.51	100	0.81	54	17	84
	35.09	36.62	1.53	1.53	100	1.42	93	8	170
	36.62	38.10	1.48	1.48	100	1.43	97	8	164
	38.10	39.66	1.56	1.56	100	1.50	96	9	156
	39.66	41.21	1.55	1.55	100	1.55	100	3	388
	41.21	42.74	1.53	1.53	100	1.38	90	6	219
	42.74	43.52	0.78	0.78	100	0.76	97	3	195
		Total =	39.73	39.31		35.28		189	
		Average =	1.42		98.9		89	7	209



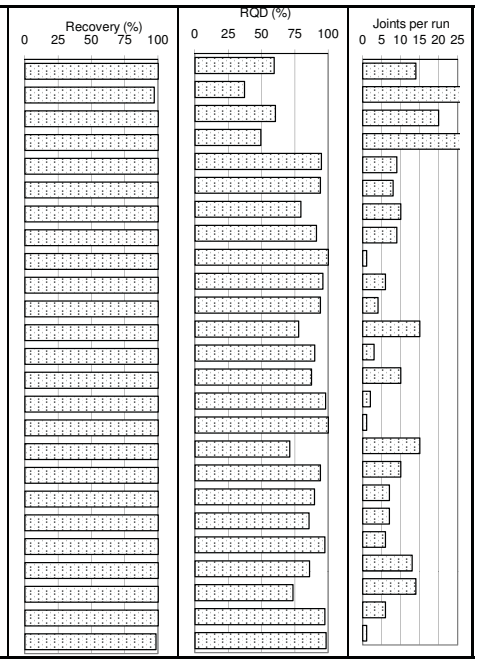
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-011
Date: Jul. 31/03

Dip: 70 deg.
Azimuth: 230 deg.
Grd. Elev.: 136.31 m
Depth: 43.70 m

Barge Hole

Geological Description**	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
8.69-19.58 Granite, light grey, medium to coarse grained, strong, closely spaced, good quality, low permeability, hard drilling, occasional amphibolite xenoliths.	8.69	9.85	1.16	1.16	100	0.69	59	14	77
	9.85	11.17	1.32	1.28	97	0.49	37	26 +	47
	11.17	12.66	1.49	1.49	100	0.90	60	20 +	71
19.58-24.62 Amphibolite: dark grey to black, very fine grained, strong, distinct foliation varies from 90 to 45 degrees to the core axis, moderately spaced, good to excellent quality, low impermeable.	12.66	14.20	1.54	1.54	100	0.76	49	32 +	47
	14.20	15.76	1.56	1.56	100	1.48	95	9 +	156
	15.76	17.27	1.51	1.51	100	1.42	94	8	168
24.62-41.16 Granite: white to light pink, fine to medium grained, strong, no foliation evident, closely spaced, good quality, practically impermeable.	17.27	18.83	1.56	1.56	100	1.24	79	10	142
	18.83	20.07	1.24	1.24	100	1.13	91	9	124
	20.07	20.28	0.21	0.21	100	0.21	100	1	105
41.16-43.70 Amphibolite: same as above, closely spaced, excellent quality, low permeability.	20.28	21.76	1.48	1.48	100	1.42	96	6	211
	21.76	23.29	1.53	1.53	100	1.44	94	4	306
	23.29	24.82	1.53	1.53	100	1.19	78	15	96
	24.82	26.30	1.48	1.48	100	1.33	90	3	370
	26.30	27.80	1.50	1.50	100	1.31	87	10 +	136
	27.80	29.32	1.52	1.52	100	1.49	98	2	507
	29.32	30.84	1.52	1.52	100	1.52	100	1	760
	30.84	32.36	1.52	1.52	100	1.08	71	15 +	95
	32.36	33.87	1.51	1.51	100	1.42	94	10 +	137
	33.87	35.40	1.53	1.53	100	1.37	90	7	191
	35.40	36.93	1.53	1.53	100	1.31	86	7	191
	36.93	38.49	1.56	1.56	100	1.52	97	6	223
	38.49	39.99	1.50	1.50	100	1.29	86	13 +	107
	39.99	41.57	1.58	1.58	100	1.16	73	14 +	105
	41.57	43.10	1.53	1.53	100	1.49	97	6 +	219
	43.10	43.70	0.60	0.59	98	0.59	98	1	295
		Total =	35.01	34.96		29.25		249	
		Average =	1.40		99.9		84	10	140



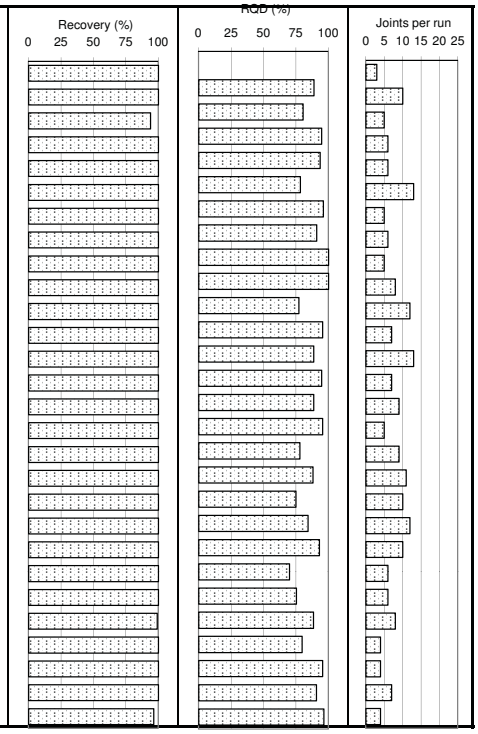
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-012
Date: Jul. 31/03

Dip: 90 deg.
Azimuth: - deg.
Grd. Elev.: 137.78 m
Depth: 41.05 m

Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
3.22-14.85 Amphibolite: dark grey to black, fine grained, moderately strong to strong, no distinct foliation, closely spaced, good quality, low permeability, contains fine to coarse-grained white to light grey granitic bands. Foliation and granitic bands varies from 30 to 45 degrees to the core axis. Granitic bands locally cross cut foliation.	3.22	3.45	0.23	0.23	100	0.00	0	3	58
	3.45	4.97	1.52	1.52	100	1.35	89	10	138
	4.97	6.45	1.48	1.39	94	1.19	80	5 +	232
	6.45	7.97	1.52	1.52	100	1.44	95	6	217
	7.97	9.50	1.53	1.53	100	1.43	93	6	219
14.85-21.05 Granite: light grey/green to light pink/green, medium to coarse grained, strong, no distinct foliation, closely spaced, good quality, low impermeability.	9.50	11.02	1.52	1.52	100	1.19	78	13	109
	11.02	12.55	1.53	1.53	100	1.47	96	5	255
	12.55	14.07	1.52	1.52	100	1.38	91	6	217
21.05-34.10 Amphibolite: same as above, strong, with granite intrusions.	14.07	15.61	1.54	1.54	100	1.54	100	5	257
	15.61	17.12	1.51	1.51	100	1.51	100	8	168
	17.12	18.61	1.49	1.49	100	1.15	77	12	115
	18.61	20.14	1.53	1.53	100	1.46	95	7	191
	20.14	21.71	1.57	1.57	100	1.39	89	13	112
34.10-41.05 Granite: same as above, greyish green, closely spaced, good quality, low impermeability.	21.71	23.22	1.51	1.51	100	1.43	95	7	189
	23.22	24.72	1.50	1.50	100	1.33	89	9	150
	24.72	26.26	1.54	1.54	100	1.47	95	5	257
	26.26	27.80	1.54	1.54	100	1.20	78	9	154
	27.80	29.31	1.51	1.51	100	1.33	88	11	126
	29.31	30.66	1.35	1.35	100	1.01	75	10	123
	30.66	32.18	1.52	1.52	100	1.28	84	12	117
	32.18	33.76	1.58	1.58	100	1.47	93	10	144
	33.76	34.56	0.80	0.80	100	0.56	70	6	114
	34.56	35.41	0.85	0.85	100	0.64	75	6	121
	35.41	36.45	1.04	1.03	99	0.92	88	8 +	114
	36.45	36.94	0.49	0.49	100	0.39	80	4	98
	36.94	38.46	1.52	1.52	100	1.45	95	4	304
	38.46	39.96	1.50	1.50	100	1.36	91	7	188
	39.96	41.05	1.09	1.05	96	1.05	96	4	210
		Total =	37.83	37.69		33.39		211	
		Average =	1.35		99.6		88	8	178



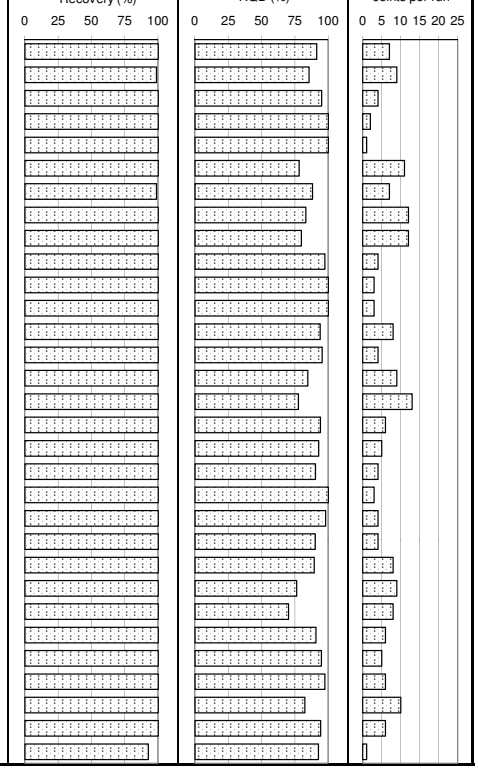
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-013
Date: Aug. 5/03

Dip: 69 deg.
Azimuth: 63 deg.
Grd. Elev.: 137.32 m
Depth: 44.01 m

Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery Length (%)	RQD Length (m)	RQD Length (%)	Jts per run	Average[3] Spacing per Jt. (mm)
4.75-14.45 Granite: white to light grey with some pink/green, medium to coarse grained, strong, good quality, medium permeability.	4.81	6.08	1.27	1.27	100	1.16	91	7	159
	6.08	7.67	1.59	1.57	99	1.36	86	9 +	157
14.45-29.08 Amphibolite: black/dark green, fine grained, strong, good quality, medium permeability, occasional granite intrusions.	7.67	8.46	0.79	0.79	100	0.75	95	4	158
	8.46	9.11	0.65	0.65	100	0.65	100	2	217
	9.11	9.52	0.41	0.41	100	0.41	100	1	205
29.08-44.01 Granite: same as above, low permeability.	9.52	11.08	1.56	1.56	100	1.22	78	11	130
	11.08	12.61	1.53	1.51	99	1.35	88	7 +	189
	12.61	14.16	1.55	1.55	100	1.29	83	12	119
	14.16	15.69	1.53	1.53	100	1.22	80	12	118
	15.69	17.26	1.57	1.57	100	1.53	97	4	314
	17.26	18.86	1.60	1.60	100	1.60	100	3	400
	18.86	20.36	1.50	1.50	100	1.50	100	3	375
	20.36	21.85	1.49	1.49	100	1.40	94	8	166
	21.85	23.39	1.54	1.54	100	1.47	95	4	308
	23.39	24.96	1.57	1.57	100	1.33	85	9	157
	24.96	26.48	1.52	1.52	100	1.18	78	13	109
	26.48	28.00	1.52	1.52	100	1.43	94	6	217
	28.00	28.84	0.84	0.84	100	0.78	93	5	140
	28.84	29.87	1.03	1.03	100	0.93	90	4	206
	29.87	30.98	1.11	1.11	100	1.11	100	3	278
	30.98	32.47	1.49	1.49	100	1.46	98	4	298
	32.47	33.38	0.91	0.91	100	0.82	90	4	182
	33.38	34.43	1.05	1.05	100	0.94	90	8	117
	34.43	35.53	1.10	1.10	100	0.84	76	9	110
	35.53	36.50	0.97	0.97	100	0.68	70	8	108
	36.50	37.48	0.98	0.98	100	0.89	91	6	140
	37.48	39.00	1.52	1.52	100	1.44	95	5	253
	39.00	40.57	1.57	1.57	100	1.53	97	6	224
	40.57	42.05	1.48	1.48	100	1.22	82	10	135
	42.05	43.60	1.55	1.55	100	1.46	94	6	221
	43.60	44.01	0.41	0.38	93	0.38	93	1 +	190
		Total =	39.20	39.13		35.33		194	
		Average =	1.26		99.8		90	6	201



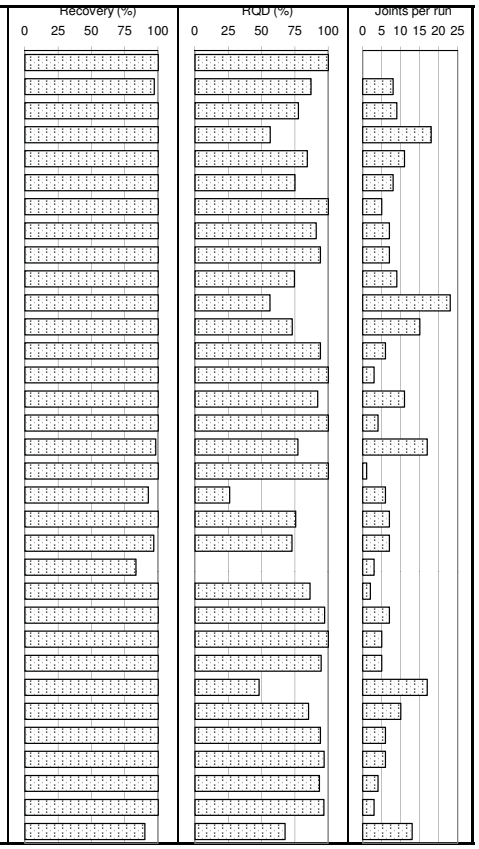
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-014
Date: Aug. 19/03

Dip: 68 deg.
Azimuth: 231 deg.
Grd. Elev.: 137.52 m
Depth: 43.02 m

Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery Length (%)	RQD Length (m)	RQD Length (%)	Jts per run	Average[3] Spacing per Jt. (mm)
3.71-30.25 Greywacke gneiss: dark grey, medium to fine grained, strong, distinct foliation at 30 to 60 degrees to the core axis, closely spaced, low permeability, occasional granitic intrusions.	3.71	4.09	0.38	0.38	100	0.38	100	0	380
	4.09	5.48	1.39	1.35	97	1.21	87	8	154
	5.48	6.73	1.25	1.25	100	0.97	78	9	125
	6.73	8.27	1.54	1.54	100	0.87	56	18 +	81
	8.27	9.80	1.53	1.53	100	1.29	84	11 +	128
30.25-37.20 Granite: light pink to orange, coarse grained, strong, closely spaced, practically impermeable, dark inclusions.	9.80	10.36	0.56	0.56	100	0.42	75	8	62
	10.36	11.32	0.96	0.96	100	0.96	100	5	160
	11.32	12.85	1.53	1.53	100	1.39	91	7	191
	12.85	14.37	1.52	1.52	100	1.43	94	7	190
	14.37	15.91	1.54	1.54	100	1.15	75	9	154
	15.91	17.42	1.51	1.51	100	0.85	56	23	63
	17.42	18.97	1.55	1.55	100	1.13	73	15	97
	18.97	20.16	1.19	1.19	100	1.12	94	6	170
	20.16	20.66	0.50	0.50	100	0.50	100	3	125
	20.66	22.17	1.51	1.51	100	1.39	92	11	126
	22.17	23.75	1.58	1.58	100	1.58	100	4	316
	23.75	24.85	1.10	1.08	98	0.85	77	17 +	61
	24.85	25.33	0.48	0.48	100	0.48	100	1	240
	25.33	25.87	0.54	0.50	93	0.14	26	6 +	77
	25.87	26.73	0.86	0.86	100	0.65	76	7	108
	26.73	27.61	0.88	0.85	97	0.64	73	7 +	110
	27.61	27.91	0.30	0.25	83	0.00	0	3 +	75
	27.91	28.35	0.44	0.44	100	0.38	86	2	147
	28.35	29.81	1.46	1.46	100	1.42	97	7	183
	29.81	31.25	1.44	1.44	100	1.44	100	5	240
	31.25	32.76	1.51	1.51	100	1.43	95	5	252
	32.76	34.30	1.54	1.54	100	0.74	48	17	86
	34.30	35.85	1.55	1.55	100	1.32	85	10	141
	35.85	37.37	1.52	1.52	100	1.43	94	6	217
	37.37	38.95	1.58	1.58	100	1.53	97	6 +	226
	38.95	40.30	1.35	1.35	100	1.26	93	4	270
	40.30	41.82	1.52	1.52	100	1.47	97	3	380
	41.82	43.02	1.20	1.08	90	0.81	67	13 +	86
		Total =	39.31	39.01		32.63		263	
		Average =	1.19		99.2		83	8	149



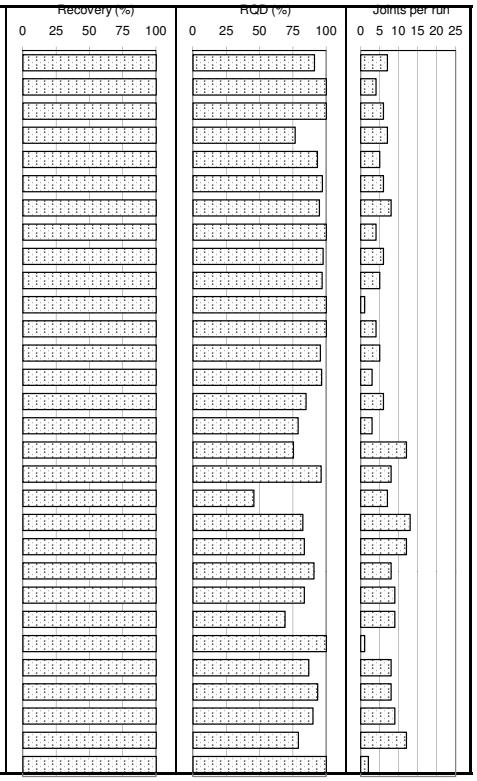
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-015
Date: Aug. 11/03

Dip: 70 deg.
Azimuth: 277 deg.
Grd. Elev.: 136.51 m
Depth: 43.82 m

Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
7.66-25.79 Amphibolite: black to dark green, fine to very fine grained, strong, indistinct to distinct foliation varying between 0 and 35 degrees to the core axis, moderately spaced, low permeability.	7.66	8.57	0.91	0.91	100	0.83	91	7	114
	8.57	10.05	1.48	1.48	100	1.48	100	4	296
	10.05	11.63	1.58	1.58	100	1.58	100	6	226
25.79-43.82 Granite: pink/grey to white, medium grained, moderately strong to strong, closely spaced, good quality, practically impermeable.	11.63	12.70	1.07	1.07	100	0.82	77	7	134
	12.70	14.22	1.52	1.52	100	1.42	93	5	253
	14.22	15.59	1.37	1.37	100	1.33	97	6	196
	15.59	17.11	1.52	1.52	100	1.44	95	8	169
	17.11	18.60	1.49	1.49	100	1.49	100	4	298
	18.60	19.90	1.30	1.30	100	1.27	98	6	186
	19.90	21.45	1.55	1.55	100	1.50	97	5 +	258
	21.45	21.72	0.27	0.27	100	0.27	100	1	135
	21.72	23.20	1.48	1.48	100	1.48	100	4	296
	23.20	23.87	0.67	0.67	100	0.64	96	5	112
	23.87	24.73	0.86	0.86	100	0.83	97	3	215
	24.73	26.25	1.52	1.52	100	1.29	85	6	217
	26.25	26.44	0.19	0.19	100	0.15	79	3 +	48
	26.44	27.78	1.34	1.34	100	1.01	75	12	103
	27.78	29.05	1.27	1.27	100	1.22	96	8	141
	29.05	29.62	0.57	0.57	100	0.26	46	7 +	71
	29.62	30.98	1.36	1.36	100	1.12	82	13 +	97
	30.98	32.50	1.52	1.52	100	1.27	84	12	117
	32.50	34.01	1.51	1.51	100	1.37	91	8	168
	34.01	35.46	1.45	1.45	100	1.21	83	9	145
	35.46	36.27	0.81	0.81	100	0.56	69	9	81
	36.27	37.25	0.98	0.98	100	0.98	100	1	490
	37.25	38.77	1.52	1.52	100	1.32	87	8	169
	38.77	40.16	1.39	1.39	100	1.30	94	8	154
	40.16	41.57	1.41	1.41	100	1.27	90	9 +	141
	41.57	43.14	1.57	1.57	100	1.24	79	12	121
	43.14	43.82	0.68	0.68	100	0.68	100	2	227
		Total =	36.16	36.16		32.63		198	
		Average =	1.21		100.0		90	7	182



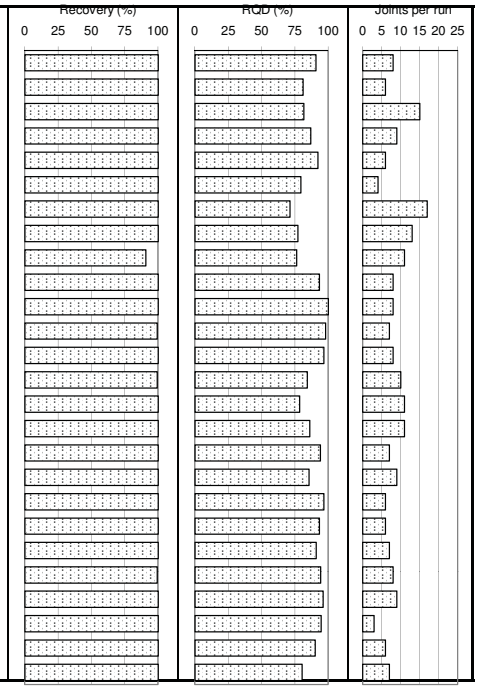
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: 03-016
Date: Jul. 22/03

Dip: 90 deg.
Azimuth: - deg.
Grd. Elev.: 137.30 m
Depth: 41.01 m

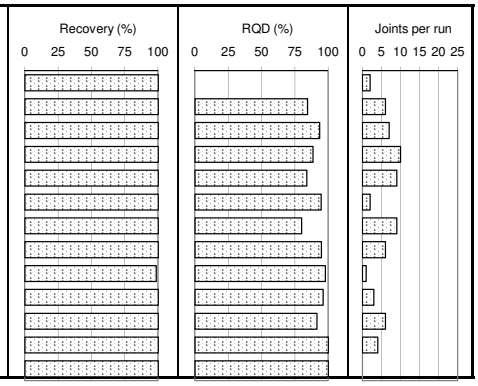
Barge Hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average** Spacing per Jt. (mm)
4.36-8.15 Greywacke gneiss: medium to dark grey, fine grained, moderately strong to strong, distinct to indistinct irregular and planar foliation, closely spaced, good quality.	4.36	5.87	1.51	1.51	100	1.37	91	8	168
	5.87	6.45	0.58	0.58	100	0.47	81	6	83
	6.45	7.97	1.52	1.52	100	1.24	82	15	95
8.15-19.45 Granite: pink to light grey, fine to coarse grained, strong, no foliation evident, closely spaced, good quality.	7.97	9.50	1.53	1.53	100	1.33	87	9	153
	9.50	10.28	0.78	0.78	100	0.72	92	6	111
	10.28	11.01	0.73	0.73	100	0.58	79	4	146
	11.01	12.54	1.53	1.53	100	1.09	71	17	85
	12.54	14.07	1.53	1.53	100	1.18	77	13	109
	14.07	15.59	1.52	1.38	91	1.16	76	11 +	127
	15.59	17.11	1.52	1.52	100	1.42	93	8	169
	17.11	18.64	1.53	1.53	100	1.53	100	8	170
	18.64	20.17	1.53	1.52	99	1.50	98	7 +	191
	20.17	21.69	1.52	1.52	100	1.47	97	8	169
	21.69	23.21	1.52	1.51	99	1.28	84	10 +	138
	23.21	24.75	1.54	1.54	100	1.21	79	11	128
	24.75	26.26	1.51	1.51	100	1.30	86	11	126
	26.26	27.80	1.54	1.54	100	1.45	94	7	193
	27.80	29.32	1.52	1.52	100	1.30	86	9	152
	29.32	30.84	1.52	1.52	100	1.47	97	6	217
	30.84	32.34	1.50	1.50	100	1.40	93	6	214
	32.34	33.87	1.53	1.53	100	1.39	91	7	191
	33.87	35.42	1.55	1.54	99	1.46	94	8 +	172
	35.42	36.95	1.53	1.53	100	1.47	96	9	153
	36.95	38.46	1.51	1.51	100	1.43	95	3	377
	38.46	39.99	1.53	1.53	100	1.38	90	6	219
	39.99	41.01	1.02	1.02	100	0.82	80	7	128
		Total =	36.65	36.48		32.42		220	
		Average =	1.41		99.5		88	8	166



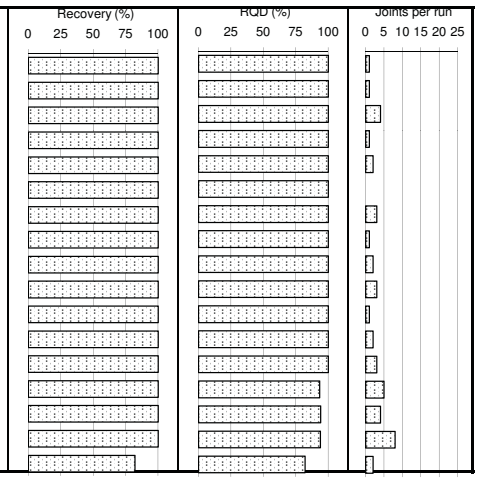
Drill Hole: 03-022
Date: Aug. 31/03
Dip: 90 deg.
Azimuth: - deg.
Grd. Elev.: 149.54 m
Depth: 25.05 m

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average** Spacing per Jt. (mm)
9.86-15.40 Amphibolite: black to dark green, medium to fine grained, distinct irregular foliation at various angles to the core axis, moderately strong to strong, closely spaced, good quality.	9.86	9.99	0.13	0.13	100	0.00	0	2	43
	9.99	11.28	1.29	1.29	100	1.09	84	6	184
15.40-20.05 Granite: light grey to white, and pink, medium to very coarse grained (pegmatitic), strong, indistinct to locally weak foliation at various angles to core axis, closely spaced, good quality.	11.28	12.82	1.54	1.54	100	1.44	94	7	193
	12.82	14.38	1.56	1.56	100	1.38	88	10	142
	14.38	15.50	1.12	1.12	100	0.94	84	9	112
	15.50	15.87	0.37	0.37	100	0.35	95	2	123
	15.87	17.42	1.55	1.55	100	1.24	80	9	155
	17.42	18.95	1.53	1.53	100	1.45	95	6	219
	18.95	20.35	1.40	1.38	99	1.37	98	1 +	700
	20.35	21.93	1.58	1.58	100	1.52	96	3	395
	21.93	23.46	1.53	1.53	100	1.40	92	6	219
	23.46	24.83	1.37	1.37	100	1.37	100	4	274
	24.83	25.05	0.22	0.22	100	0.22	100	0	220
		Total =	15.19	15.17		13.77		65	
		Average =	1.17		99.9		91	5	230



Drill Hole: G-0007
Date: Oct. 2/88
Dip: 45 deg.
Bearing: 19 deg.
Datum 140.19 m
Depth: 28.94 m

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
3.86-22.88 Granite: white to whitish grey, medium grained to coarse grained, strong, moderately spaced jointing, excellent rock quality.	3.86	4.56	0.70	0.70	100	0.70	100	1	350
	4.56	6.08	1.52	1.52	100	1.52	100	1	760
22.88-23.39 Diabase: dark grey to green, very fine grained, slight foliation evident at 30 to 40 degrees to the core axis, strong, moderately spaced jointing, excellent rock quality.	6.08	7.66	1.58	1.58	100	1.58	100	4	316
	7.66	9.15	1.49	1.49	100	1.49	100	1	745
23.39-28.94 Granite: as described above, good to excellent rock quality.	9.15	10.72	1.57	1.57	100	1.57	100	2	523
	10.72	12.23	1.51	1.51	100	1.51	100	0	1510
	12.23	13.75	1.52	1.52	100	1.52	100	3	380
	13.75	15.25	1.50	1.50	100	1.50	100	1	750
	15.25	16.80	1.55	1.55	100	1.55	100	2	517
	16.80	18.34	1.54	1.54	100	1.54	100	3	385
	18.34	19.85	1.51	1.51	100	1.51	100	1	755
	19.85	21.34	1.49	1.49	100	1.49	100	2	497
	21.34	22.88	1.54	1.54	100	1.54	100	3	385
	22.88	24.38	1.50	1.50	100	1.40	93	5	250
	24.38	25.91	1.53	1.53	100	1.44	94	4 +	306
	25.91	27.38	1.47	1.47	100	1.38	94	8	163
	27.38	28.94	1.56	1.28	82	1.28	82	2 +	427
		Total =	25.08	24.80		24.52		43	
		Average =	1.48		98.9		98	3	570

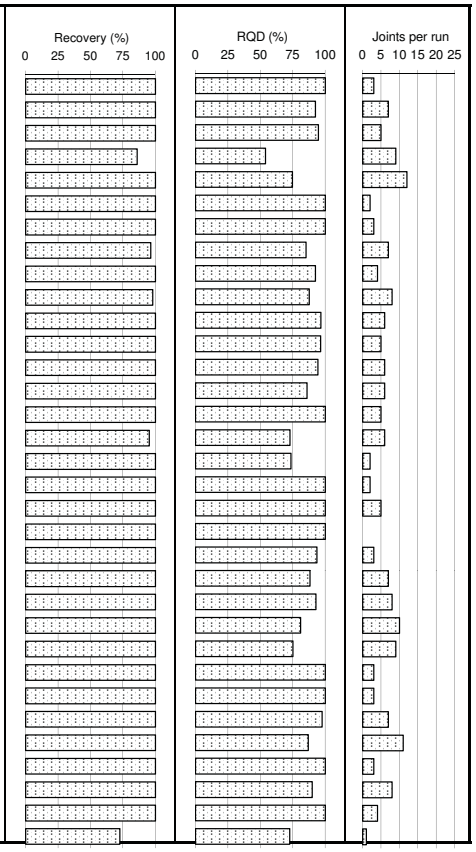


GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0008
Date: Jun, 6/91

Dip: 45 deg.
Bearing: N28E
Datum 141.50 m
Depth: 58.01 m

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
13.06-19.75 Greywacke gneiss: dark grey to greenish grey with pink to light grey granite bands throughout, fine grained, strong, distinct foliation at 35 to 45 degrees to the core axis. Closely to moderately spaced jointing, excellent rock quality, low permeability.	13.11	14.40	1.29	1.29	100	1.29	100	3	323
	14.40	15.73	1.33	1.33	100	1.23	92	7	166
	15.73	17.25	1.52	1.52	100	1.44	95	5	253
19.75-25.93 Granite: light pink, medium to coarse grained, strong, no foliation evident, hematitic stained, upper contact is intact, irregular and sharp, closely to moderately spaced jointing, excellent rock quality, low permeability.	17.25	18.53	1.28	1.10	86	0.69	54	9 +	110
	18.53	19.83	1.30	1.30	100	0.97	75	12	100
	19.83	20.64	0.81	0.81	100	0.81	100	2	270
25.93-35.08 Greywacke gneiss: as descripbed above.	20.64	21.87	1.23	1.23	100	1.23	100	3	308
	21.87	23.49	1.62	1.56	96	1.38	85	7 +	195
35.08-40.66 Granite: as described above.	23.49	24.93	1.44	1.44	100	1.33	92	4	288
40.66-44.97 Greywacke gneiss: as described above.	24.93	26.39	1.46	1.43	98	1.28	88	8 +	159
44.97-49.35 Granite: light grey, coarse grained, strong, no foliation evident, hematite staining throughout, upper contact is irregular, intact and sharp, closely to moderately spaced jointing, good to excellent rock quality, low permeability.	26.39	27.88	1.49	1.49	100	1.44	97	6	213
	27.88	29.54	1.66	1.66	100	1.60	96	5	277
	29.54	30.93	1.39	1.39	100	1.31	94	6	199
49.35-55.13 Greywacke gneiss: as described above.	30.93	32.50	1.57	1.57	100	1.35	86	6	224
55.13-58.01 Granite: light grey, medium to coarse grained, strong, no foliation evident, hematite staining, possible containing garnet, upper contact is a joint, planar, rough, tight, black coating, minor carbonate, trace iron sulphides, slickensides and at 40 degrees to the core axis, moderately spaced jointing, excellent rock quality, low permeability.	32.50	34.08	1.58	1.58	100	1.58	100	5	263
	34.08	35.55	1.47	1.40	95	1.07	73	6 +	200
	35.55	35.89	0.34	0.34	100	0.25	74	2	113
	35.89	37.32	1.43	1.43	100	1.43	100	2	477
	37.32	38.84	1.52	1.52	100	1.52	100	5	253
	38.84	39.14	0.30	0.30	100	0.30	100	0	300
	39.14	40.52	1.38	1.38	100	1.29	93	3	345
	40.52	41.96	1.44	1.44	100	1.27	88	7	180
	41.96	43.47	1.51	1.51	100	1.40	93	8	168
	43.47	45.00	1.53	1.53	100	1.24	81	10 +	139
	45.00	46.48	1.48	1.48	100	1.11	75	9	148
	46.48	48.06	1.58	1.58	100	1.58	100	3	395
	48.06	49.23	1.17	1.17	100	1.17	100	3	293
	49.23	50.89	1.66	1.66	100	1.62	98	7	208
	50.89	52.48	1.59	1.59	100	1.38	87	11	133
	52.48	54.14	1.66	1.66	100	1.66	100	3	415
	54.14	55.63	1.49	1.49	100	1.34	90	8	166
	55.63	57.17	1.54	1.54	100	1.54	100	4	308
	57.17	58.01	0.84	0.61	73	0.61	73	1 +	305
		Total =	44.90	44.33		40.71		180	
		Average =	1.36		98.7		91	5	248



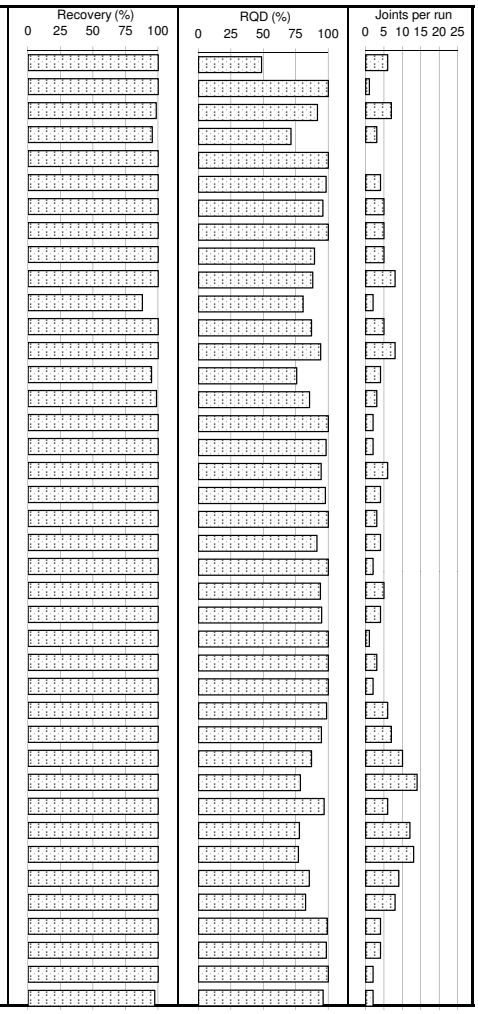
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0011
Date: Jun. 15, 91

Dip: 45 deg.
Bearing: S16W
Datum 140.05 m
Depth: 55.44 m

Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
0.94-43.87 Greywacke gneiss: medium to dark grey, fine grained, indistinct to strongly foliated at 30 to 45 degrees to the core axis, strong, moderately spaced jointing, excellent rock quality, with granitic stringers and intrusions.	0.94	2.01	1.07	1.07	100	0.52	49	6 +	153
	2.01	2.44	0.43	0.43	100	0.43	100	1	215
	2.44	3.76	1.32	1.30	98	1.21	92	7 +	163
43.87-55.44 Granite: light grey, coarse to medium grained, very strong, upper contact at 90 degrees, close to moderately close jointing, good to excellent rock quality.	3.76	4.63	0.87	0.83	95	0.62	71	3 +	208
	4.63	5.14	0.51	0.51	100	0.51	100	0	510
	5.14	6.84	1.70	1.70	100	1.67	98	4	340
	6.84	8.06	1.22	1.22	100	1.17	96	5	203
	8.06	9.82	1.76	1.76	100	1.76	100	5	293
	9.82	11.14	1.32	1.32	100	1.18	89	5	220
	11.14	12.80	1.66	1.66	100	1.46	88	8	184
	12.80	14.04	1.24	1.09	88	1.00	81	2 +	363
	14.04	15.35	1.31	1.31	100	1.14	87	5	218
	15.35	16.86	1.51	1.51	100	1.42	94	8	168
	16.86	17.64	0.78	0.74	95	0.59	76	4 +	148
	17.64	19.09	1.45	1.43	99	1.24	86	3 +	358
	19.09	20.54	1.45	1.45	100	1.45	100	2	483
	20.54	21.16	0.62	0.62	100	0.61	98	2	207
	21.16	22.81	1.65	1.65	100	1.56	95	6	236
	22.81	23.69	0.88	0.88	100	0.86	98	4	176
	23.69	24.82	1.13	1.13	100	1.13	100	3	283
	24.82	26.43	1.61	1.61	100	1.47	91	4	322
	26.43	28.10	1.67	1.67	100	1.67	100	2	557
	28.10	29.57	1.47	1.47	100	1.38	94	5	245
	29.57	31.16	1.59	1.59	100	1.51	95	4	318
	31.16	32.67	1.51	1.51	100	1.51	100	1	755
	32.67	34.28	1.61	1.61	100	1.61	100	3	403
	34.28	35.75	1.47	1.47	100	1.47	100	2	490
	35.75	37.32	1.57	1.57	100	1.55	99	6	224
	37.32	38.83	1.51	1.51	100	1.43	95	7	189
	38.83	40.38	1.55	1.55	100	1.35	87	10	141
	40.38	41.87	1.49	1.49	100	1.17	79	14 +	99
	41.87	43.45	1.58	1.58	100	1.53	97	6	226
	43.45	44.98	1.53	1.53	100	1.19	78	12	118
	44.98	46.46	1.48	1.48	100	1.14	77	13	106
	46.46	48.04	1.58	1.58	100	1.35	85	9	158
	48.04	49.53	1.49	1.49	100	1.23	83	8 +	166
	49.53	50.91	1.38	1.38	100	1.37	99	4	276
	50.91	52.31	1.40	1.40	100	1.38	99	4	280
	52.31	53.96	1.65	1.65	100	1.65	100	2	550
	53.96	55.44	1.48	1.44	97	1.42	96	2 +	480
		Total =	54.50	54.19		49.91		201	
		Average =	1.36		99.4		92	5	270



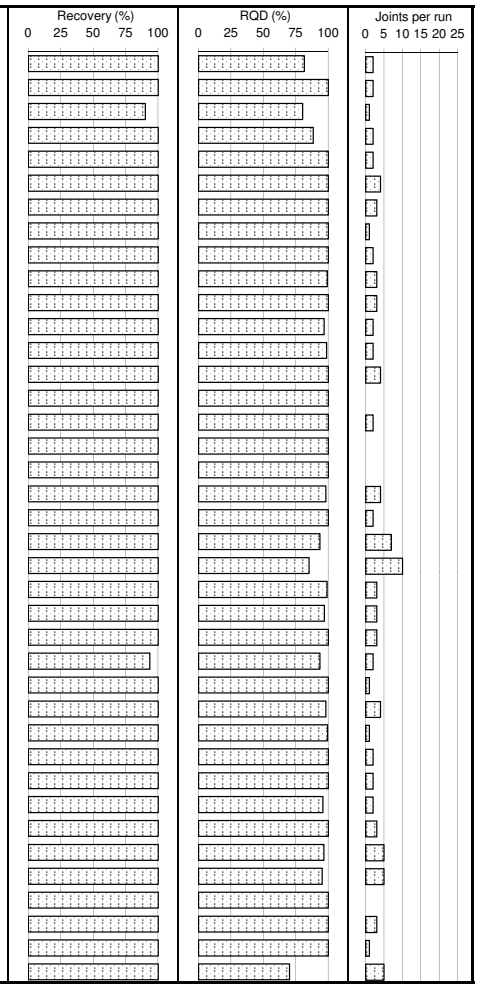
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0012
Date: Jun. 18, 91

Dip: 45 deg.
Bearing: S85W
Datum 139.95 m
Depth: 55.80 m

Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
1.61-14.89 Greywacke gneiss: dark grey, with light grey granite, fine grained, strong, distinct foliation at 45 to 60 degrees to the core axis, moderately spaced jointing, excellent rock quality, medium permeability.	1.61	2.37	0.76	0.76	100	0.62	82	2 +	253
	2.37	4.00	1.63	1.63	100	1.63	100	2	543
14.89-44.06 Granite: medium grey, medium to coarse grained, strong, no foliation evident, moderately to widely spaced jointing, excellent rock quality, low permeability, upper contact is intact, planar, sharp and at 45 degrees to the core axis.	4.00	5.42	1.42	1.28	90	1.14	80	1 +	640
	5.42	6.96	1.54	1.54	100	1.36	88	2 +	513
	6.96	8.45	1.49	1.49	100	1.49	100	2	497
	8.45	9.99	1.54	1.54	100	1.54	100	4	308
	9.99	11.20	1.21	1.21	100	1.21	100	3	303
	11.20	12.61	1.41	1.41	100	1.41	100	1	705
	12.61	14.09	1.48	1.48	100	1.48	100	2	493
	14.09	15.62	1.53	1.53	100	1.52	99	3	383
	15.62	17.18	1.56	1.56	100	1.56	100	3	390
	17.18	18.69	1.51	1.51	100	1.46	97	2	503
	18.69	20.28	1.59	1.59	100	1.57	99	2	530
	20.28	21.78	1.50	1.50	100	1.50	100	4	300
	21.78	23.33	1.55	1.55	100	1.55	100	0	1550
	23.33	24.91	1.58	1.58	100	1.58	100	2	527
	24.91	25.63	0.72	0.72	100	0.72	100	0	720
	25.63	26.58	0.95	0.95	100	0.95	100	0	950
	26.58	28.11	1.53	1.53	100	1.50	98	4 +	306
	28.11	29.65	1.54	1.54	100	1.54	100	2	513
	29.65	31.03	1.38	1.38	100	1.29	93	7	173
	31.03	32.58	1.55	1.55	100	1.32	85	10	141
	32.58	33.66	1.08	1.08	100	1.07	99	3	270
	33.66	34.66	1.00	1.00	100	0.97	97	3	250
	34.66	35.87	1.21	1.21	100	1.21	100	3	303
	35.87	37.26	1.39	1.30	94	1.30	94	2 +	433
	37.26	38.55	1.29	1.29	100	1.29	100	1	645
	38.55	40.10	1.55	1.55	100	1.52	98	4	310
	40.10	41.67	1.57	1.57	100	1.56	99	1	785
	41.67	43.19	1.52	1.52	100	1.52	100	2	507
	43.19	44.61	1.42	1.42	100	1.42	100	2	473
	44.61	46.28	1.67	1.67	100	1.60	96	2	557
	46.28	47.87	1.59	1.59	100	1.59	100	3	398
	47.87	49.37	1.50	1.50	100	1.45	97	5	250
	49.37	51.03	1.66	1.66	100	1.58	95	5	277
	51.03	51.31	0.28	0.28	100	0.28	100	0	280
	51.31	52.70	1.39	1.39	100	1.39	100	3 +	348
	52.70	54.23	1.53	1.53	100	1.53	100	1	765
	54.23	55.80	1.57	1.57	100	1.10	70	5 ++	262
		Total =	54.19	53.96		52.32		103	
		Average =	1.39		99.6		97	3	521



GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0013
Date: Jun. 21, 91

Dip: 60 deg.
Bearing: 270 deg.
Datum 140.00 m
Depth: 51.05 m

Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
1.73-19.44 Granite: medium grey, medium to coarse grained, strong, no foliation evident, widely spaced jointing, excellent rock quality, low permeability, hematite staining throughout, minor biotite to chlorite alteration throughout.	1.73	2.38	0.65	0.65	100	0.43	66	0 +	650
	2.38	3.88	1.50	1.50	100	1.12	75	4 +	300
	3.88	5.33	1.45	1.45	100	1.45	100	0	1450
19.44-28.09 Greywacke gneiss: dark grey, medium to coarse grained, strong, widely spaced jointing, excellent rock quality, low permeability, upper contact is intact, irregular and gradational.	5.33	6.69	1.36	1.36	100	1.36	100	0	1360
	6.69	8.35	1.66	1.66	100	1.66	100	0	1660
28.09-40.86 Granite: light pink, medium to coarse grained, strong, no foliation evident, widely spaced jointing, excellent rock quality, low permeability, upper contact is gradational.	8.35	9.86	1.51	1.51	100	1.51	100	0	1510
	9.86	11.46	1.60	1.60	100	1.60	100	2	533
	11.46	12.93	1.47	1.47	100	1.47	100	1	735
40.86-44.87 Greywacke gneiss: dark grey with light grey granite, fine to medium grained, strong, no foliation evident, widely spaced jointing, excellent rock quality, low permeability, gneissic texture is not apparent from core surface but inspection of machine break surfaces shows a well developed gneissic texture, upper contact is intact, planar, sharp and at 75 degrees to the core axis.	12.93	14.49	1.56	1.56	100	1.56	100	1	780
	14.49	16.05	1.56	1.56	100	1.56	100	2	520
	16.05	17.49	1.44	1.44	100	1.44	100	0	1440
	17.49	18.97	1.48	1.48	100	1.42	96	0 +	1480
44.87-51.73 Granite: light grey, coarse grained, strong, no foliation evident, widely spaced jointing, excellent rock quality, upper contact is intact, planar, sharp and at 55 degrees to the core axis.	18.97	20.54	1.57	1.57	100	1.51	96	4	314
	20.54	21.98	1.44	1.44	100	1.44	100	3	360
	21.98	23.60	1.62	1.62	100	1.62	100	1	810
	23.60	25.02	1.42	1.42	100	1.42	100	2	473
	25.02	26.65	1.63	1.63	100	1.54	94	4	326
	26.65	28.07	1.42	1.42	100	1.42	100	4	284
	28.07	28.70	0.63	0.63	100	0.63	100	4	126
	28.70	29.83	1.13	1.13	100	1.02	90	2 +	377
	29.83	31.23	1.40	1.40	100	1.40	100	0	1400
	31.23	32.78	1.55	1.55	100	1.55	100	0	1550
	32.78	34.36	1.58	1.58	100	1.56	99	2	527
	34.36	35.91	1.55	1.55	100	1.49	96	3	388
	35.91	37.48	1.57	1.57	100	1.57	100	1	785
	37.48	38.17	0.69	0.69	100	0.62	90	2	230
	38.17	38.89	0.72	0.72	100	0.72	100	4	144
	38.89	40.51	1.62	1.62	100	1.62	100	1	810
	40.51	42.01	1.50	1.50	100	1.50	100	1	750
	42.01	43.58	1.57	1.57	100	1.57	100	1	785
	43.58	45.05	1.47	1.47	100	1.47	100	2	490
	45.05	45.47	0.42	0.42	100	0.36	86	2	140
	45.47	46.79	1.32	1.32	100	1.32	100	1	660
	46.79	48.13	1.34	1.34	100	1.34	100	1	670
	48.13	49.59	1.46	1.46	100	1.46	100	0	1460
	49.59	51.05	1.46	1.31	90	1.31	90	0	1310
		Total =	49.32	49.17		48.04		55	
		Average =	1.37		99.7		97	2	881

Recovery (%) 0 25 50 75 100
RQD (%) 0 25 50 75 100
Joints per run 0 5 10 15 20 25

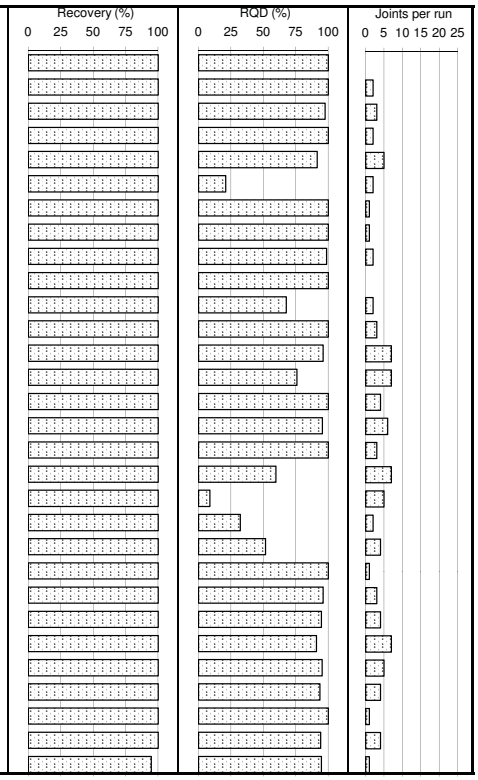
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0014
Date: Jun. 25, 91

Dip: 60 deg.
Bearing: S26W
Datum 140.11 m
Depth: 46.10 m

Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
4.03-12.82 Granite: light to medium grey, medium to coarse grained, strong, no foliation evident, widely spaced jointing, good to excellent rock quality, low permeability.	4.03	4.73	0.70	0.70	100	0.70	100	0	700
	4.73	6.31	1.58	1.58	100	1.58	100	2	527
12.82-33.31 Greywacke gneiss: dark grey with light grey granite bands, fine grained, strong, distinct foliation at 25 to 35 degrees to the core axis, widely to moderately spaced jointing, good to excellent rock quality, low permeability, upper contact is intact, sharp and irregular.	6.31	7.89	1.58	1.58	100	1.54	97	3 +	395
	7.89	8.91	1.02	1.02	100	1.02	100	2	340
	8.91	10.44	1.53	1.53	100	1.40	92	5 +	255
	10.44	11.20	0.76	0.76	100	0.16	21	2 ++	253
33.31-37.41Granite: light grey, medium to coarse grained, strong, no foliation evident, widely spaced jointing, excellent rock quality, upper contact is intact, planar, sharp and at 50 degrees to the core axis.	11.20	12.68	1.48	1.48	100	1.48	100	1	740
	12.68	14.00	1.32	1.32	100	1.32	100	1	660
	14.00	15.50	1.50	1.50	100	1.48	99	2 +	500
37.41-46.10 Greywacke gneiss: dark grey, fine grained, strong, distinct foliation at 20 to 45 degrees to the core axis, moderately to widely spaced jointing, excellent rock quality, low permeability, upper contact is a joint, planar, rough, tight, carbonate coated and at 30 degrees to the core axis.	15.50	17.15	1.65	1.65	100	1.65	100	0	1650
	17.15	18.63	1.48	1.48	100	1.00	68	2 +	493
	18.63	20.14	1.51	1.51	100	1.51	100	3	378
	20.14	21.66	1.52	1.52	100	1.46	96	7 +	190
	21.66	23.15	1.49	1.49	100	1.13	76	7 +	186
	23.15	24.69	1.54	1.54	100	1.54	100	4	308
	24.69	26.25	1.56	1.56	100	1.49	96	6	223
	26.25	27.76	1.51	1.51	100	1.51	100	3	378
	27.76	29.22	1.46	1.46	100	0.87	60	7 +	183
	29.22	30.71	1.49	1.49	100	0.13	9	5 ++	248
	30.71	31.92	1.21	1.21	100	0.39	32	2 ++	403
	31.92	33.47	1.55	1.55	100	0.80	52	4 ++	310
	33.47	35.07	1.60	1.60	100	1.60	100	1	800
	35.07	35.58	0.51	0.51	100	0.49	96	3	128
	35.58	37.08	1.50	1.50	100	1.42	95	4	300
	37.08	38.50	1.42	1.42	100	1.29	91	7	178
	38.50	40.00	1.50	1.50	100	1.43	95	5	250
	40.00	41.55	1.55	1.55	100	1.45	94	4	310
	41.55	43.05	1.50	1.50	100	1.50	100	1	750
	43.05	44.60	1.55	1.55	100	1.46	94	4	310
	44.60	46.10	1.50	1.42	95	1.42	95	1 +	710
		Total =	42.07	41.99		36.22		98	
		Average =	1.40		99.8		86	3	425



GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0015
Date: Jun. 27, 91

Dip: 60 deg.
Bearing: S82W
Datum 139.81 m
Depth: 46.66 m

Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
4..32-46.66 Greywacke gneiss: dark grey to black with light to medium grey granite, fine grained, strong, indistinct to distinct foliation at 30 to 40 degrees to the core axis, closely to moderately spaced jointing, good to excellent rock quality, low permeability.	4.32	6.15	1.83	1.59	87	1.26	69	7 +	199
	6.15	7.81	1.66	1.55	93	1.51	91	4 +	310
	7.81	8.08	0.27	0.27	100	0.27	100	0	270
	8.08	9.50	1.42	1.42	100	1.39	98	4	284
	9.50	10.85	1.35	1.35	100	1.29	96	6	193
	10.85	12.31	1.46	1.46	100	1.46	100	5	243
	12.31	13.76	1.45	1.38	95	1.19	82	7 +	173
	13.76	15.21	1.45	1.45	100	1.38	95	5	242
	15.21	16.75	1.54	1.54	100	1.32	86	8	171
	16.75	18.15	1.40	1.40	100	1.40	100	2	467
	18.15	19.61	1.46	1.46	100	1.42	97	2	487
	19.61	21.30	1.69	1.69	100	1.12	66	7 +	211
	21.30	22.50	1.20	1.20	100	1.16	97	6	171
	22.50	24.01	1.51	1.51	100	1.51	100	5	252
	24.01	25.57	1.56	1.56	100	1.48	95	8	173
	25.57	26.55	0.98	0.98	100	0.85	87	8	109
	26.55	28.04	1.49	1.49	100	0.90	60	12	115
	28.04	28.64	0.60	0.60	100	0.49	82	7 +	75
	28.64	30.15	1.51	1.51	100	1.46	97	4	302
	30.15	31.60	1.45	1.45	100	1.29	89	8	161
	31.60	33.15	1.55	1.55	100	1.55	100	5	258
	33.15	34.60	1.45	1.45	100	1.45	100	0	1450
	34.60	36.00	1.40	1.40	100	1.35	96	5 +	233
	36.00	37.54	1.54	1.54	100	1.32	86	4 +	308
	37.54	39.04	1.50	1.50	100	1.39	93	4 +	300
	39.04	40.57	1.53	1.53	100	1.30	85	10 +	139
	40.57	42.19	1.62	1.62	100	1.62	100	5	270
	42.19	42.33	0.14	0.14	100	0.14	100	0	140
	42.33	43.85	1.52	1.52	100	1.46	96	7	190
	43.85	45.09	1.24	1.24	100	1.24	100	3	310
	45.09	46.66	1.57	1.50	96	1.48	94	4 +	300
		Total =	42.34	41.85		38.45		162	
		Average =	1.37		98.8		91	5	260

Recovery (%) 0 25 50 75 100

RQD (%) 0 25 50 75 100

Joints per run 0 5 10 15 20 25

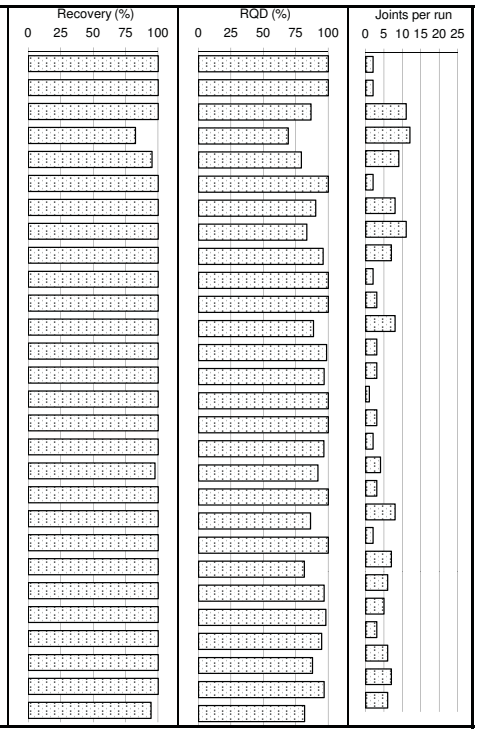
GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0016
Date: Jun. 28, 91

Dip: 60 deg.
Bearing: N13W
Datum 139.76 m
Depth: 42.36 m

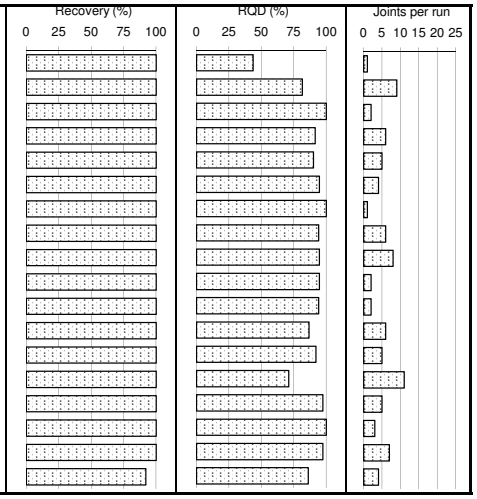
Barge hole

Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery Length (%)	RQD Length (m)	RQD Length (%)	Jts per run	Average[3] Spacing per Jt. (mm)
4.73-39.65 Granite: light grey to light brownish grey, medium to coarse grained, strong, no foliation evident, moderately spaced jointing, good to excellent rock quality, medium permeability, occasional mafic clot.	4.73	5.35	0.62	0.62	100	0.62	100	2	207
	5.35	5.93	0.58	0.58	100	0.58	100	2	193
	5.93	7.50	1.57	1.57	100	1.36	87	11	131
	7.50	8.99	1.49	1.23	83	1.03	69	12 +	95
	8.99	10.44	1.45	1.38	95	1.15	79	9 +	138
	10.44	10.83	0.39	0.39	100	0.39	100	2	130
	10.83	12.37	1.54	1.54	100	1.39	90	8	171
	12.37	13.88	1.51	1.51	100	1.26	83	11	126
	13.88	15.40	1.52	1.52	100	1.46	96	7	190
	15.40	16.84	1.44	1.44	100	1.44	100	2	480
	16.84	18.13	1.29	1.29	100	1.29	100	3	323
	18.13	19.70	1.57	1.57	100	1.39	89	8	174
	19.70	21.16	1.46	1.46	100	1.44	99	3	365
	21.16	22.74	1.58	1.58	100	1.53	97	3	395
	22.74	24.13	1.39	1.39	100	1.39	100	1	695
	24.13	25.62	1.49	1.49	100	1.49	100	3	373
	25.62	27.07	1.45	1.45	100	1.40	97	2	483
	27.07	28.69	1.62	1.58	98	1.49	92	4 +	316
	28.69	30.03	1.34	1.34	100	1.34	100	3	335
	30.03	31.63	1.60	1.60	100	1.38	86	8	178
	31.63	33.08	1.45	1.45	100	1.45	100	2	483
	33.08	34.11	1.03	1.03	100	0.84	82	7	129
	34.11	35.63	1.52	1.52	100	1.47	97	6	217
	35.63	37.15	1.52	1.52	100	1.49	98	5	253
	37.15	37.93	0.78	0.78	100	0.74	95	3	195
	37.93	39.40	1.47	1.47	100	1.29	88	6	210
	39.40	40.93	1.53	1.53	100	1.48	97	7	191
	40.93	42.36	1.43	1.35	94	1.17	82	6 +	193
		Total =	37.63	37.18		34.75		146	
		Average =	1.34		98.8		92	5	256



GN-4.2.2.8, Rev 1 -Table A1

Drill Hole: G-0204 Date: Sept. 30/88 Dip: 45 deg. Bearing: 230 deg. Datum 141.96 m Depth: 30.79 m	Geological Description[3]	Drill Run Depth (m - along core axis) From	To	Length (m)	Recovery Length (m)	Recovery (%)	RQD Length (m)	RQD (%)	Jts per run	Average[3] Spacing per Jt. (mm)
	4.73-39.65 Granite: light grey to light brownish grey, medium to coarse grained, strong, no foliation evident, moderately spaced jointing, good to excellent rock quality, medium permeability, occasional mafic clot.	4.72	4.88	0.16	0.16	100	0.07	44	1	80
		4.88	6.40	1.52	1.52	100	1.24	82	9 +	152
		6.40	7.92	1.52	1.52	100	1.52	100	2	507
	39.65-42.36 Greywacke gneiss: dark grey to medium grey, fine to medium grained, occasionally coarse grained, strong, distinct foliation at 20 degrees to the core axis, moderately jointed, good rock quality, low permeability, contains some coarse quartz/feldspar grains, rounded and rotated, some angular fragments, upper contact is a joint, tight, sharp and at 20 degrees to the core axis.	7.92	9.44	1.52	1.52	100	1.39	91	6	217
		9.44	10.97	1.53	1.53	100	1.38	90	5 +	255
		10.97	12.49	1.52	1.52	100	1.44	95	4 +	304
		12.49	14.00	1.51	1.51	100	1.51	100	1	755
		14.00	15.52	1.52	1.52	100	1.43	94	6	217
		15.52	17.05	1.53	1.53	100	1.45	95	8	170
		17.05	18.58	1.53	1.53	100	1.45	95	2	510
		18.58	20.11	1.53	1.53	100	1.44	94	2	510
		20.11	21.62	1.51	1.51	100	1.31	87	6 +	216
		21.62	23.14	1.52	1.52	100	1.40	92	5	253
		23.14	24.66	1.52	1.52	100	1.08	71	11 +	127
		24.66	26.20	1.54	1.54	100	1.50	97	5	257
		26.20	27.71	1.51	1.51	100	1.51	100	3	378
		27.71	29.27	1.56	1.56	100	1.52	97	7	195
		29.27	30.79	1.52	1.40	92	1.31	86	4 +	280
			Total =	26.07	25.95		23.95		87	
			Average =	1.45		99.5		92	5	296



Notes:

(1) Chart Depths are approximate.

(2) Elevation is approximate.

(3) Refer to geological logs for a complete description

(4) For barge holes, elevation noted is the riverbed elevation.

\+ Defines the minimum discontinuities counted in a core run, where missing or broken core is present.

++ defines a core run with discontinuities too numerous to count, or the entire core run is broken or lost.

Table A2

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: 03-027 Date: Sept. 9/03 Dip: 70 deg. Azimuth: 315 deg. Grd. Elev.: 147.17 m Depth: 34.79 m	Geological Desciption [3]	Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)	Recovery (%) 0 25 50 75 100	RQD (%) 0 25 50 75 100	Joints per run 0 5 10 15 20 25
	0.00-1.08 Greywacke gneiss: dark grey fine to medium grained, distinct foliation at 60 to 80 degrees to the core axis, strong, moderately spaced jointing, good to excellent rock quality, low permeability.	0.00	1.62	1.62	1.59	98	1.24	77	13	+	114			
		1.62	3.13	1.51	1.51	100	1.44	95	5		252			
		3.13	4.68	1.55	1.55	100	1.46	94	5		258			
	1.08-28.45 Diabase: dark green, very fine grained, no foliation evident, moderately spaced jointing, good to excellent rock quality, low permeability.	4.68	6.28	1.60	1.60	100	1.43	89	4		320			
		6.28	7.78	1.50	1.50	100	1.43	95	6		214			
		7.78	8.23	0.45	0.45	100	0.45	100	1		225			
	28.45-30.95 Greywacke gneiss: same as above, only weakly distinct foliation.	8.23	9.36	1.13	1.13	100	1.13	100	0		1130			
		9.36	10.85	1.49	1.49	100	1.39	93	7		186			
		10.85	12.41	1.56	1.56	100	1.50	96	6		223			
	30.95-34.79 Granite: pink to light grey, medium to coarse grained, locally pegmatitic, strong to very strong, closely spaced jointing, good rock quality low permeability.	12.41	13.96	1.55	1.55	100	1.55	100	1		775			
		13.96	15.48	1.52	1.52	100	1.50	99	4		304			
		15.48	17.03	1.55	1.55	100	1.55	100	2		517			
		17.03	18.52	1.49	1.49	100	1.49	100	2		497			
		18.52	20.03	1.51	1.51	100	1.44	95	6		216			
		20.03	21.43	1.40	1.40	100	1.40	100	2		467			
		21.43	23.01	1.58	1.58	100	1.50	95	5		263			
		23.01	24.61	1.60	1.60	100	1.60	100	5		267			
		24.61	26.17	1.56	1.56	100	1.51	97	5		260			
		26.17	27.77	1.60	1.60	100	1.57	98	3		400			
		27.77	29.39	1.62	1.60	99	1.33	82	14	+	107			
		29.39	30.69	1.30	1.30	100	0.96	74	13		93			
		30.69	31.76	1.07	1.05	98	0.48	45	13	+	75			
		31.76	33.28	1.52	1.52	100	1.43	94	6		217			
		33.28	33.57	0.29	0.23	79	0.01	3	2	++	77			
		33.57	34.79	1.22	1.19	98	0.92	75	11		99			
		Totals =		34.79	34.63		31.71		141					
		Average =		1.39		99.5		91		6	245			

(for notes see page 8)

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: 03-028 Date: Sept.5/03 Dip: 70 deg. Azimuth: 315 deg. Grd. Elev.: 148.79 m Depth: 36.53 m	Geological Desciption [3]	Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)	Recovery (%) 0 25 50 75 100	RQD (%) 0 25 50 75 100	Joints per run 0 5 10 15 20 25
	3.02-11.00 Greywacke gneiss: as described above	3.02	3.83	0.81	0.81	100	0.73	90	8		90			
	11.00-13.90 Granite: light to medium grey, medium to coarse grained, strong, closely spaced jointing, excellent rock quality, low permeability.	3.83	5.19	1.36	1.11	82	0.62	46	11		93			
		5.19	6.53	1.34	1.34	100	1.34	100	9	+	134			
		6.53	8.03	1.50	1.50	100	1.12	75	12		115			
	13.90-36.47 Greywacke gneiss: as described above.	8.03	9.60	1.57	1.57	100	1.30	83	14		105			
		9.60	10.82	1.22	1.22	100	0.62	51	17		68			
		10.82	12.39	1.57	1.57	100	1.57	100	4		314			
		12.39	13.91	1.52	1.52	100	1.36	89	7		190			
		13.91	15.46	1.55	1.55	100	1.30	84	10	+	141			
		15.46	16.95	1.49	1.49	100	1.49	100	5		248			
		16.95	18.49	1.54	1.54	100	1.45	94	7		193			
		18.49	19.09	0.60	0.60	100	0.60	100	2		200			
		19.09	20.60	1.51	1.51	100	1.36	90	11		126			
		20.60	21.81	1.21	1.21	100	0.90	74	11		101			
		21.81	23.31	1.50	1.50	100	1.31	87	13		107			
		23.31	24.88	1.57	1.57	100	1.42	90	10		143			
		24.88	26.39	1.51	1.51	100	1.35	89	12		116			
		26.39	27.96	1.57	1.57	100	1.35	86	12		121			
		27.96	29.40	1.44	1.44	100	1.43	99	2		480			
		29.40	30.98	1.58	1.58	100	1.36	86	9		158			
		30.98	32.46	1.48	1.48	100	1.27	86	7		185			
		32.46	34.01	1.55	1.47	95	1.30	84	4		294			
		34.01	35.58	1.57	1.57	100	1.48	94	9		157			
		35.58	36.53	0.95	0.89	94	0.86	91	6	+	127			
		Totals =		33.51	33.12		28.89		212					
		Average =		1.40		98.8		86		9	157			

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: 03-037	Geological	Drill Run			Recovery		RQD		Jts		Average [3]	Recovery (%)	RQD (%)	Joints per run
Date: Sept.5/03	Desciption [3]	Depth (m - along core axis)			Length		Length		per		Spacing	0 25 50 75 100	0 25 50 75 100	0 5 10 15 20 25
		From	To	(m)	(m)	(%)	(m)	(%)	run		per Jt. (mm)			
Dip: 70 deg.	2.80-33.08 Greywacke gneiss: dark grey to black, fine to medium grained, distinct to weakly distinct foliation at various angles to the core axis, generally between 65 to 85 degrees, strong, moderately spaced jointing, excellent rock quality, low permeability.	2.80	3.62	0.82	0.82	100	0.79	96	4		164			
Azimuth: 315 deg.		3.62	5.19	1.57	1.54	98	0.84	54	13	+	110			
Grd. Elev.: 146.01 m		5.19	6.59	1.40	1.40	100	1.27	91	11		117			
Depth: 33.08 m		6.59	6.66	0.07	0.07	100	0.07	100	0		70			
		6.66	6.70	0.04	0.04	100	0.04	100	0		40			
		6.70	8.25	1.55	1.55	100	1.47	95	7		194			
		8.25	9.73	1.48	1.48	100	1.27	86	9		148			
		9.73	11.29	1.56	1.56	100	1.56	100	7		195			
		11.29	12.82	1.53	1.53	100	1.37	90	11		128			
		12.82	13.54	0.72	0.72	100	0.42	58	10		65			
		13.54	14.31	0.77	0.77	100	0.64	83	7		96			
		14.31	15.86	1.55	1.55	100	1.38	89	10		141			
		15.86	17.41	1.55	1.55	100	1.31	85	11		129			
		17.41	17.93	0.52	0.52	100	0.43	83	2		173			
		17.93	18.94	1.01	1.01	100	0.93	92	5		168			
		18.94	20.46	1.52	1.52	100	1.33	88	10		138			
		20.46	21.93	1.47	1.47	100	1.35	92	9		147			
		21.93	23.50	1.57	1.57	100	1.24	79	15		98			
		23.50	25.03	1.53	1.53	100	1.27	83	14		102			
		25.03	26.55	1.52	1.52	100	0.82	54	24		61			
		26.55	28.07	1.52	1.52	100	1.21	80	12	+	117			
		28.07	29.61	1.54	1.54	100	1.54	100	8		171			
		29.61	31.14	1.53	1.53	100	1.53	100	7		191			
		31.14	32.64	1.50	1.50	100	1.47	98	6		214			
		32.64	33.08	0.44	0.42	95	0.42	95	0		420			
			Totals =	30.28	30.23		25.97		212					
			Average =	1.21		99.8		86		8	142			

(for notes see page 8)

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: G-0043 Date: Jul.26/91 Dip: 45 deg. Azimuth: 270 deg. Grd. Elev.: 150.69 m Depth: 42.06 m	Geological Desciption [3]	Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)
	2.25-42.25 Greywacke gneiss: dark grey, fine grained, strong, indistinct foliation, moderately spaced jointing, excellent rock quality, practically impermeable, granitic bands throughout. Granite bands are medium to light pink, medium to coarse grained, strong, no foliation evident.	2.25	3.62	1.37	1.37	100	1.26	92	5	+	228
		3.62	4.80	1.18	1.18	100	1.18	100	5		197
		4.80	4.91	0.11	0.11	100	0.07	64	3		28
		4.91	6.54	1.63	1.63	100	1.40	86	6	+	233
		6.54	7.89	1.35	1.35	100	0.73	54	3	+	338
		7.89	9.45	1.56	1.56	100	1.52	97	3		390
		9.45	10.88	1.43	1.43	100	1.43	100	2		477
		10.88	12.37	1.49	1.49	100	1.33	89	4		298
		12.37	13.75	1.38	1.38	100	1.33	96	3		345
		13.75	15.33	1.58	1.58	100	1.53	97	4		316
		15.33	16.86	1.53	1.53	100	1.43	93	1	+	765
		16.86	18.41	1.55	1.55	100	1.46	94	5	+	258
		18.41	19.69	1.28	1.28	100	0.96	75	6	+	183
		19.69	20.40	0.71	0.71	100	0.71	100	1		355
		20.40	21.79	1.39	1.39	100	1.39	100	1		695
		21.79	22.86	1.07	1.07	100	1.07	100	1		535
		22.86	24.46	1.60	1.60	100	1.44	90	9		160
		24.46	25.20	0.74	0.69	93	0.58	78	2	+	230
		25.20	26.62	1.42	1.42	100	1.33	94	10		129
		26.62	28.24	1.62	1.62	100	1.62	100	5		270
		28.24	28.88	0.64	0.64	100	0.61	95	2		213
		28.88	30.44	1.56	1.56	100	1.56	100	2		520
		30.44	30.81	0.37	0.37	100	0.31	84	3		93
		30.81	31.88	1.07	1.07	100	1.07	100	1		535
		31.88	33.30	1.42	1.42	100	1.10	77	4	+	284
		33.30	34.80	1.50	1.50	100	1.35	90	5		250
		34.80	35.88	1.08	1.08	100	0.45	42	4	+	216
		35.88	37.48	1.60	1.60	100	1.55	97	5	+	267
		37.48	38.98	1.50	1.50	100	1.50	100	1		750
		38.98	40.52	1.54	1.54	100	1.41	92	10	+	140
		40.52	42.06	1.54	1.54	100	1.21	79	9		154
			Totals =	39.81	39.76		35.89		125		
			Average =	1.28		99.9		90		4	316

Recovery (%) 0 25 50 75 100 | RQD (%) 0 25 50 75 100 | Joints per run 0 5 10 15 20 25

(for notes see page 8)

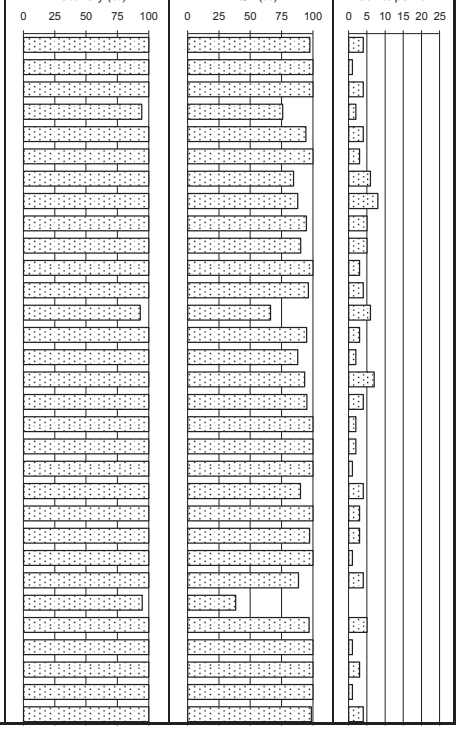
Drill Hole: G-0045
Date: Jul.28/91
Dip: 45 deg.
Azimuth: 207 deg.
Grd. Elev.: 149.65 m
Depth: 45.39 m

Geological Desciption [3]

2.13-33.24 Greywacke gneiss: dark grey to black with light grey granitic material, fine grained, strong, distinct foliation at 35 degrees to the core axis, moderately to widely spaced jointing, good to excellent rock quality, low permeability. Contains a mixture of granite and greywacke gneiss.

33.24-45.38 Granite: light grey, medium to coarse grained, no foliation evident, moderately to widely spaced jointing, good to excellent rock quality, medium permeability.

Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)
2.13	3.35	1.22	1.22	100	1.19	98	4		244
3.35	4.86	1.51	1.51	100	1.51	100	1		755
4.86	6.37	1.51	1.51	100	1.51	100	4		302
6.37	7.99	1.62	1.53	94	1.23	76	2	+	510
7.99	9.41	1.42	1.42	100	1.34	94	4	+	284
9.41	10.83	1.42	1.42	100	1.42	100	3		355
10.83	12.26	1.43	1.43	100	1.21	85	6		204
12.26	13.83	1.57	1.57	100	1.38	88	8		174
13.83	15.38	1.55	1.55	100	1.47	95	5		258
15.38	16.81	1.43	1.43	100	1.29	90	5		238
16.81	18.39	1.58	1.58	100	1.58	100	3		395
18.39	19.80	1.41	1.41	100	1.36	96	4		282
19.80	21.25	1.45	1.35	93	0.96	66	6	+	193
21.25	22.82	1.57	1.57	100	1.49	95	3	+	393
22.82	23.48	0.66	0.66	100	0.58	88	2		220
23.48	24.98	1.50	1.50	100	1.40	93	7		188
24.98	25.81	0.83	0.83	100	0.79	95	4		166
25.81	27.34	1.53	1.53	100	1.53	100	2		510
27.34	28.84	1.50	1.50	100	1.50	100	2		500
28.84	30.37	1.53	1.53	100	1.53	100	1		765
30.37	31.78	1.41	1.41	100	1.27	90	4		282
31.78	33.28	1.50	1.50	100	1.50	100	3		375
33.28	34.42	1.14	1.14	100	1.11	97	3		285
34.42	35.16	0.74	0.74	100	0.74	100	1		370
35.16	36.12	0.96	0.96	100	0.85	89	4		192
36.12	38.07	1.95	1.85	95	0.75	38	-	++	-
38.07	39.38	1.31	1.31	100	1.27	97	5		218
39.38	40.85	1.47	1.47	100	1.47	100	1		735
40.85	42.39	1.54	1.54	100	1.54	100	3		385
42.39	43.85	1.46	1.46	100	1.46	100	1		730
43.85	45.39	1.54	1.54	100	1.52	99	4		308
Totals =		43.26	42.97		39.75		105		
Average =		1.40		99.3		92		4	408



(for notes see page 8)

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: G-0047 Date: Jul.30/91 Dip: 45 deg. Azimuth: 257 deg. Grd. Elev.: 149.04 m Depth: 40.91 m	Geological Desciption [3]	Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)
	1.46-40.91 Greywacke gneiss: dark grey, fine grained, strong, indistinct foliation at 40 to 50 degrees to the core axis, moderately spaced jointing, good to excellent rock quality, low permeability, granitic bands throughout. Granite bands are medium to light grey, medium to coarse grained, strong, no foliation evident.	1.46	1.80	0.34	0.34	100	0.00	0	6	+	49
		1.80	3.47	1.67	1.67	100	1.37	82	14		111
		3.47	4.86	1.39	1.39	100	1.25	90	8		154
		4.86	6.34	1.48	1.48	100	1.42	96	6		211
		6.34	7.85	1.51	1.51	100	1.33	88	8		168
		7.85	9.30	1.45	1.45	100	1.40	97	8		161
		9.30	9.68	0.38	0.38	100	0.21	55	4		76
		9.68	10.89	1.21	1.21	100	1.00	83	9		121
		10.89	12.30	1.41	1.41	100	1.10	78	12	+	108
		12.30	13.84	1.54	1.54	100	1.36	88	5		257
		13.84	15.35	1.51	1.51	100	1.49	99	4		302
		15.35	16.91	1.56	1.35	87	1.09	70	3	+	338
		16.91	18.56	1.62	1.28	79	1.56	96	4	+	256
		18.56	19.91	1.35	1.35	100	0.65	48	11	+	113
		19.91	20.98	1.07	1.07	100	0.92	86	8		119
		20.98	22.56	1.58	1.58	100	1.49	94	8		176
		22.56	24.12	1.56	1.56	100	1.53	98	5		260
		24.12	25.71	1.59	1.59	100	1.30	82	5		265
		25.71	27.09	1.38	1.38	100	1.30	94	4		276
		27.09	28.55	1.46	1.46	100	1.43	98	5		243
		28.55	29.14	0.59	0.56	95	0.56	95	1	+	280
		29.14	30.76	1.62	1.57	97	1.52	94	4	+	314
		30.76	31.86	1.10	1.10	100	1.10	100	1		550
		31.86	33.45	1.59	1.59	100	1.54	97	4		318
		33.45	34.35	0.90	0.90	100	0.90	100	0		900
		34.35	34.91	0.56	0.56	100	0.56	100	1		280
		34.91	36.17	1.26	1.26	100	1.21	96	2		420
		36.17	37.56	1.39	1.34	96	1.34	96	4	+	268
		37.56	38.99	1.43	1.43	100	1.43	100	3		358
		38.99	40.30	1.31	1.31	100	1.31	100	2		437
		40.30	40.91	0.61	0.61	100	0.61	100	1		305
			Totals =	39.42	38.74		35.28		160		
			Average =	1.27		98.3		89		5	245

Recovery (%) 0 25 50 75 100

RQD (%) 0 25 50 75 100

Joints per run 0 5 10 15 20 25

(for notes see page 8)

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: G-0049
Date: Aug.1/91
Dip: 45 deg.
Azimuth: 320 deg.
Grd. Elev.: 144.93 m
Depth: 42.49 m

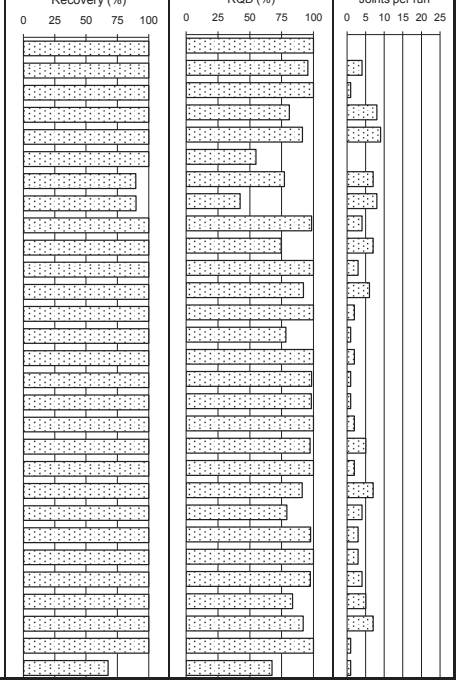
Geological Desciption [3]

4.04-7.87 Greywacke gneiss: dark grey fine grained, strong, indistinct foliation, moderately to widely spaced jointing, good to excellent rock quality, low permeability, granitic bands throughout.

7.87-28.93 Granite: light grey, medium to coarse grained, strong, no foliation evident, moderately spaced jointing, good rock quality, low permeability, numerous healed joints throughout.

28.93-42.49 Greywacke gneiss: same as above.

Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)
4.04	4.11	0.07	0.07	100	0.07	100	0		70
4.11	5.01	0.90	0.90	100	0.86	96	4		180
5.01	6.63	1.62	1.62	100	1.62	100	1		810
6.63	8.00	1.37	1.37	100	1.11	81	8		152
8.00	9.63	1.63	1.63	100	1.49	91	9		163
9.63	10.05	0.42	0.42	100	0.23	55	-	++	-
10.05	11.01	0.96	0.86	90	0.74	77	7	+	108
11.01	12.38	1.37	1.23	90	0.58	42	8	+	137
12.38	13.83	1.45	1.45	100	1.43	99	4		290
13.83	15.45	1.62	1.62	100	1.21	75	7	+	203
15.45	17.01	1.56	1.56	100	1.56	100	3		390
17.01	18.55	1.54	1.54	100	1.42	92	6		220
18.55	20.01	1.46	1.46	100	1.46	100	2		487
20.01	21.58	1.57	1.57	100	1.23	78	1	+	785
21.58	22.99	1.41	1.41	100	1.41	100	2		470
22.99	23.64	0.65	0.65	100	0.64	98	1		325
23.64	24.86	1.22	1.22	100	1.20	98	1		610
24.86	26.02	1.16	1.16	100	1.16	100	2		387
26.02	27.54	1.52	1.52	100	1.48	97	5		253
27.54	29.02	1.48	1.48	100	1.48	100	2		493
29.02	30.61	1.59	1.59	100	1.45	91	7		199
30.61	32.05	1.44	1.44	100	1.14	79	4		288
32.05	33.44	1.39	1.39	100	1.36	98	3		348
33.44	34.98	1.54	1.54	100	1.54	100	3		385
34.98	36.57	1.59	1.59	100	1.55	97	4		318
36.57	37.97	1.40	1.40	100	1.17	84	5	+	233
37.97	39.59	1.62	1.62	100	1.49	92	7		203
39.59	40.98	1.39	1.39	100	1.39	100	1		695
40.98	42.49	1.51	1.02	68	1.02	68	1		510
Totals =		38.45	37.72		34.49		108		
Average =		1.33		98.1		90		4	353



(for notes see page 8)

GN-4.2.2.8 Rev 0, Table A2

Drill Hole: G-0051
Date: Aug.2/91
Dip: 45 deg.
Azimuth: 185 deg.
Grd. Elev.: 149 m
Depth: 41.08 m

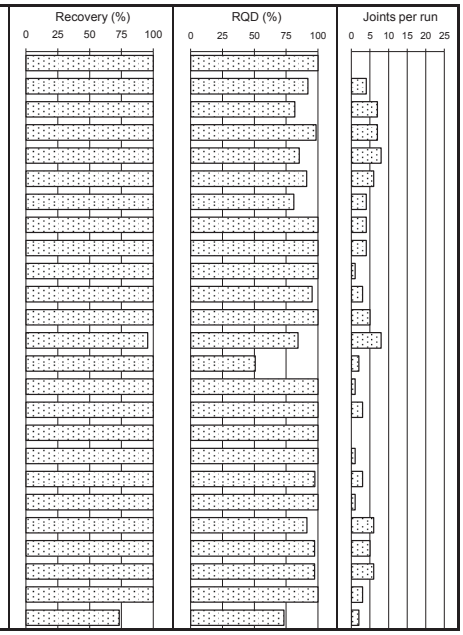
Geological Desciption [3]

7.96-26.53 Greywacke gneiss: dark grey to black, fine grained, strong, indistinct to distinct foliation at 20 to 30 degrees to the core axis, moderately spaced jointing, good to excellent rock quality, low permeability.

26.53-36.30 Granite: medium grey, medium grained, locally coarse grained, strong, no foliation evident, fresh, widely spaced jointing, excellent rock quality, practically impermeable.

36.30-41.08 Greywacke gneiss: dark grey to black, fine grained, strong, indistinct foliation at 30 degrees to the core axis, moderately spaced jointing, excellent rock quality, low permeability.

Drill Run Depth (m - along core axis) From	To	(m)	Recovery Length (m)	(%)	RQD Length (m)	(%)	Jts per run		Average [3] Spacing per Jt. (mm)
7.96	8.29	0.33	0.33	100	0.33	100	0		330
8.29	9.43	1.14	1.14	100	1.05	92	4		228
9.43	10.91	1.48	1.48	100	1.21	82	7	+	185
10.91	12.86	1.95	1.95	100	1.92	98	7		244
12.86	14.35	1.49	1.49	100	1.27	85	8	+	166
14.35	15.81	1.46	1.46	100	1.33	91	6		209
15.81	16.44	0.63	0.63	100	0.51	81	4		126
16.44	17.42	0.98	0.98	100	0.98	100	4		196
17.42	18.95	1.53	1.53	100	1.53	100	4		306
18.95	20.42	1.47	1.47	100	1.47	100	1		735
20.42	21.96	1.54	1.54	100	1.47	95	3		385
21.96	23.48	1.52	1.52	100	1.52	100	5		253
23.48	25.07	1.59	1.52	96	1.34	84	8	+	169
25.07	26.53	1.46	1.46	100	0.74	51	2	++	487
26.53	26.91	0.38	0.38	100	0.38	100	1		190
26.91	28.39	1.48	1.48	100	1.48	100	3		370
28.39	29.58	1.19	1.19	100	1.19	100	0		1190
29.58	31.05	1.47	1.47	100	1.47	100	1		735
31.05	32.67	1.62	1.62	100	1.58	98	3		405
32.67	34.13	1.46	1.46	100	1.46	100	1		730
34.13	35.73	1.60	1.60	100	1.46	91	6		229
35.73	37.23	1.50	1.50	100	1.46	97	5		250
37.23	38.78	1.55	1.55	100	1.51	97	6		221
38.78	40.22	1.44	1.44	100	1.44	100	3		360
40.22	41.08	0.86	0.63	73	0.63	73	2		210
Totals =		33.12	32.82		30.73		94		
Average =		1.32		99.1		93		4	349



Technical Notes:

A) For drill run no. 2, the detailed fracture logs do not indicate any LCZ, BLCZ or BCZ, therefore we assume that 1.11 m recovery recorded in gINT is a transcribing error. The recovery has therefore been set at 100% for drill run no. 2

B) The calculated run length for drill run no. 3 is 1.34 m and not the 1.52 m shown in the gINT database. Assumed it is a transcribing error. The run length is interpreted as 1.34 m

C) For drill run no. 22(32.45 m to 34.01 m) the Geological Logs shows recovery as 1.47/1.47 for 100%. The calculated run length is 1.55 not 1.47 m. The Detailed Fracture Logs do not indicate any LCZ, BLCZ or BCZ, therefore we assume the recovery is 1.55/1.55 for 100%

D) The maximum core that can be recovered with the core tube is 165 m. Therefore a run length greater than 165 m only occurs when there is a core loss. Review of the original field notes to hole G-0051 shows that one 1.2 runs were combined (i.e. one ended run is absent or missing) between 10.91 m and 12.86 m for a drill run of 1.95 m

(for notes see page 8)

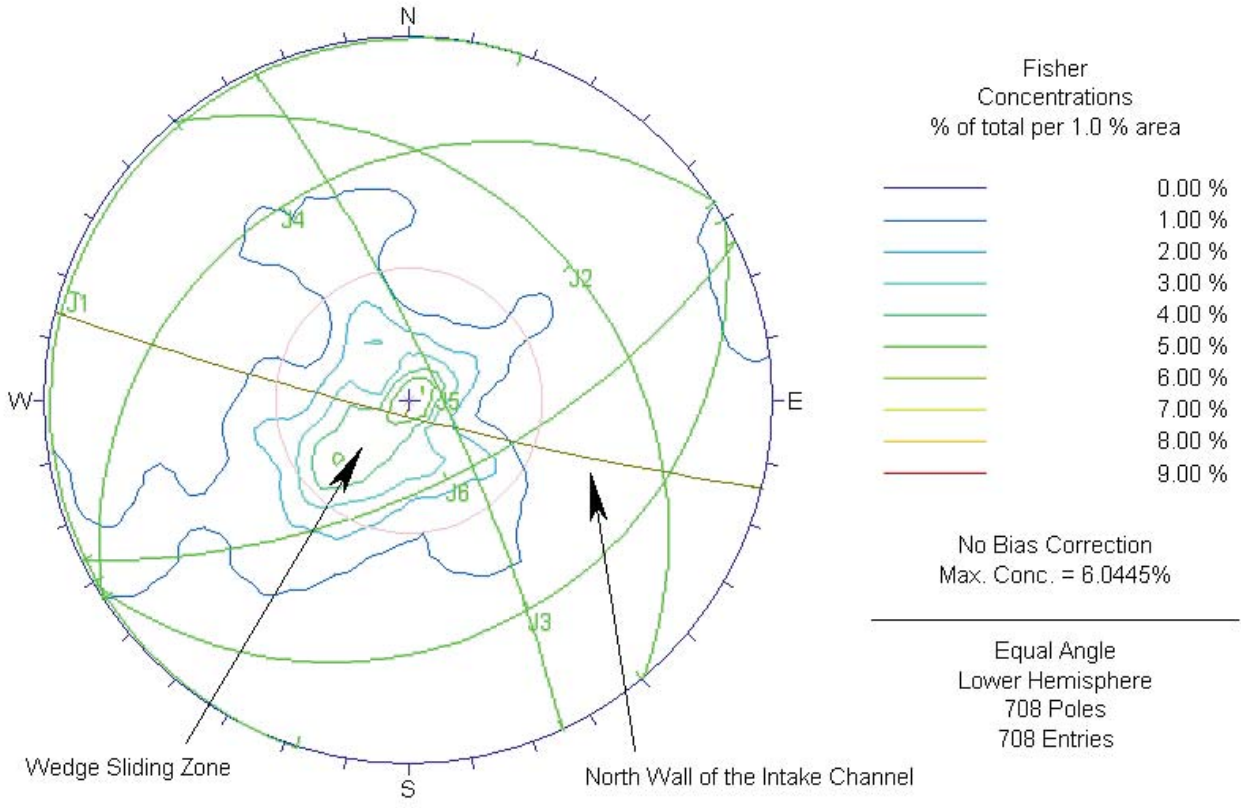
Appendix B

Stereonet Projections

Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Intake Channel (Az. 104)

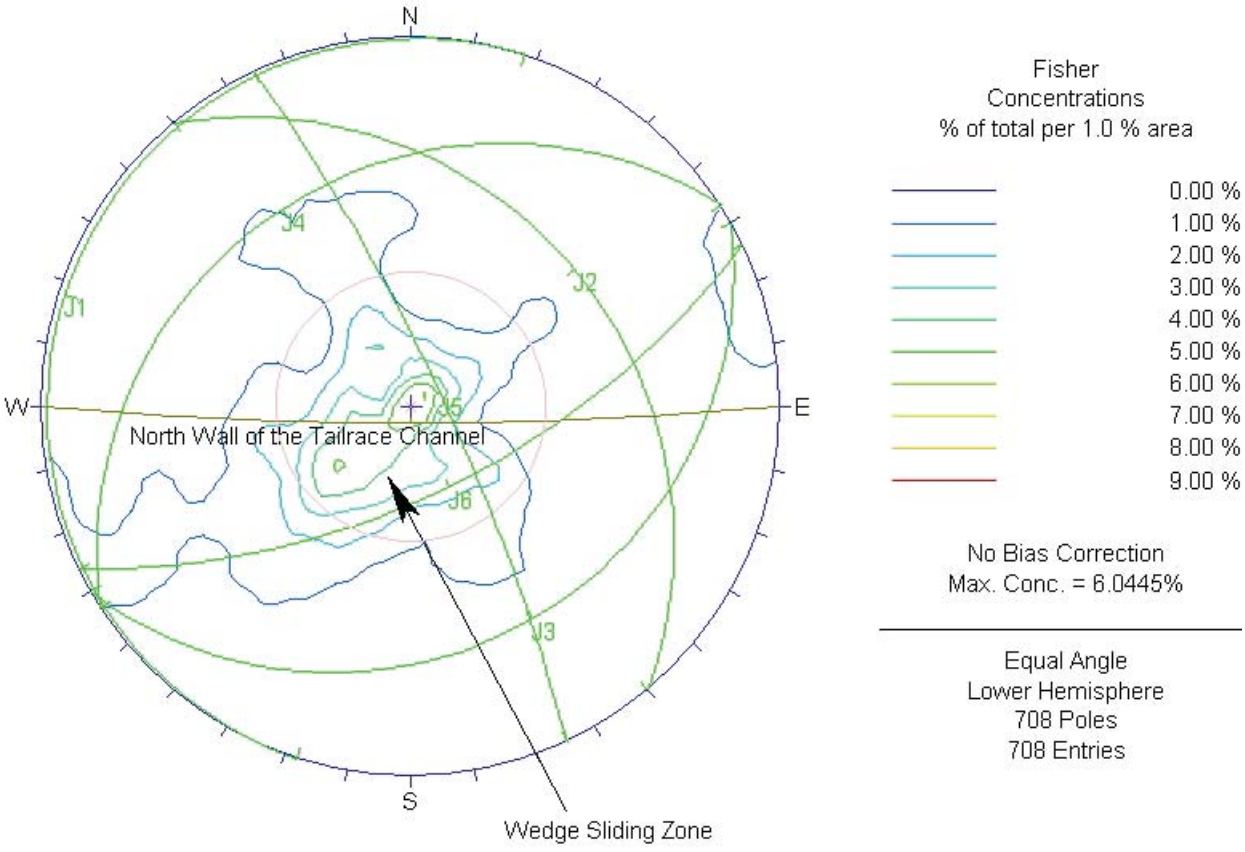
Wedges Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Tailrace Channel (Az. 90)

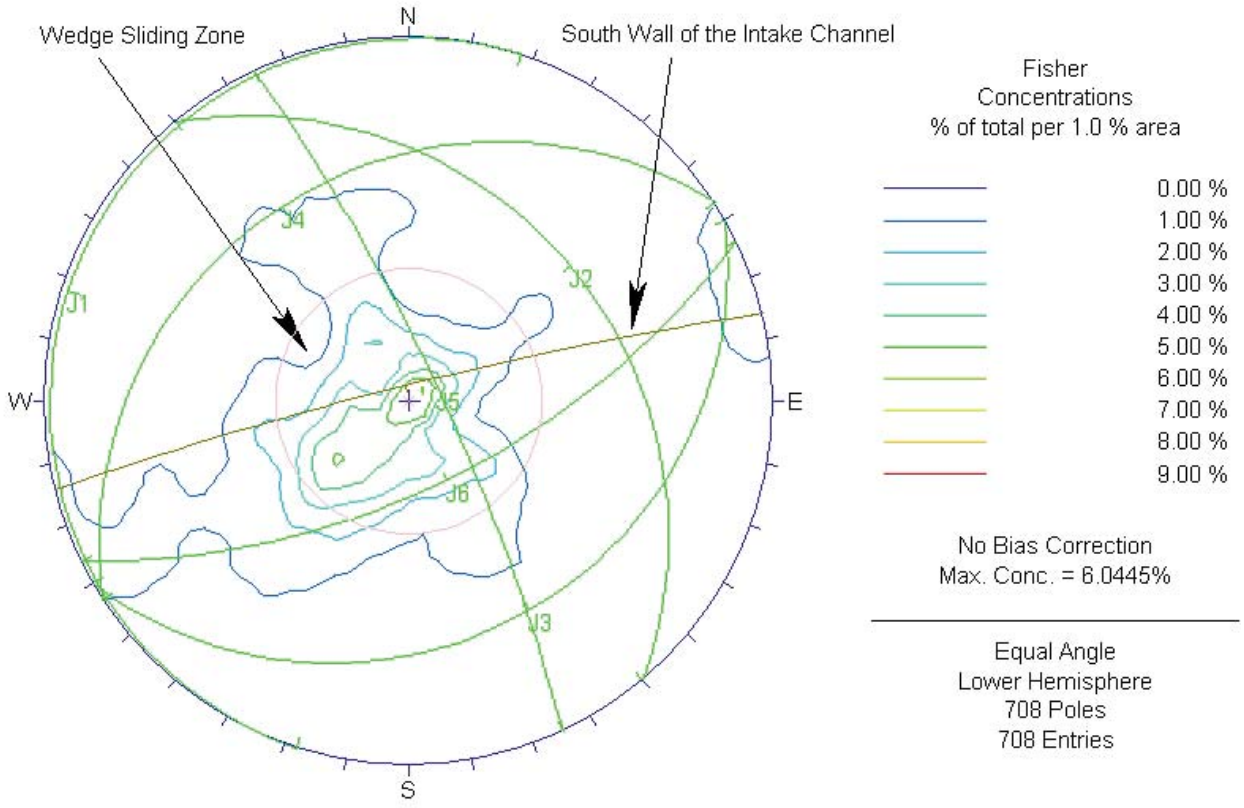
Wedges Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Intake Channel (Az. 256)

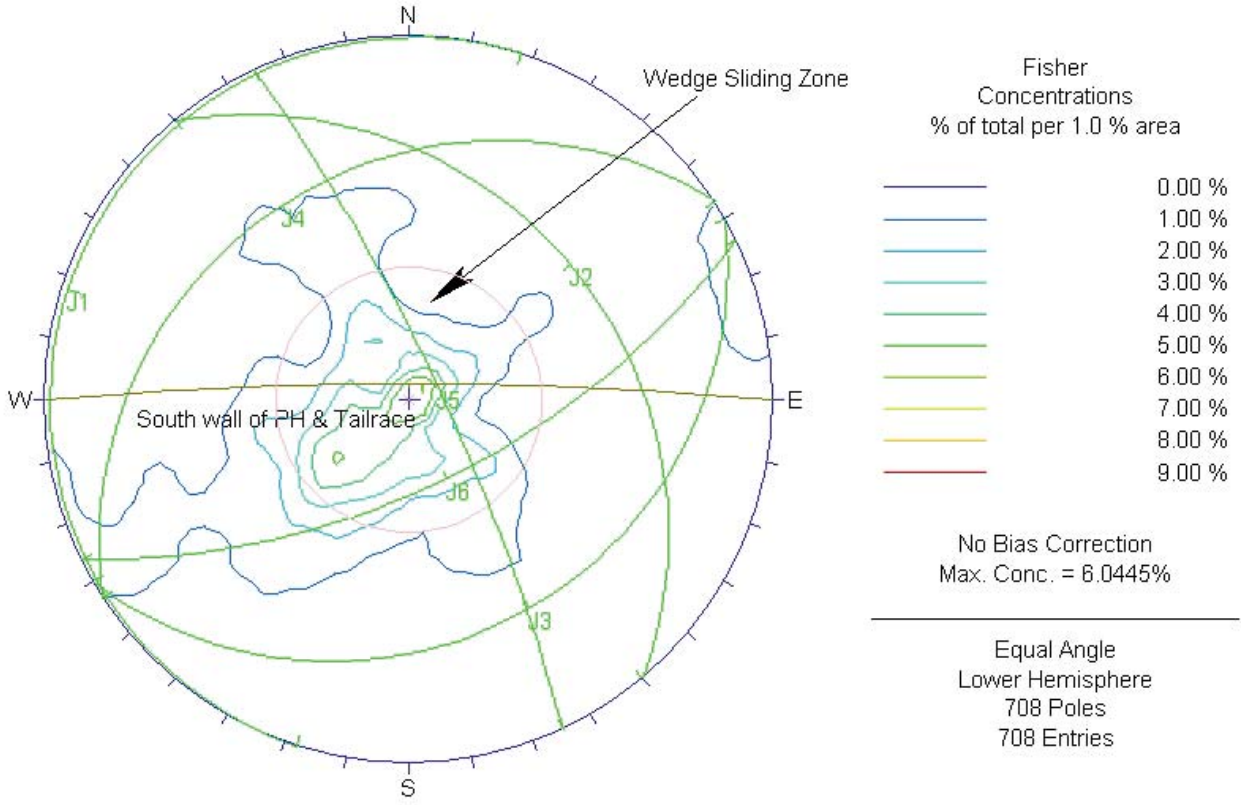
Wedges Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Tailrace Channel (Az. 270)

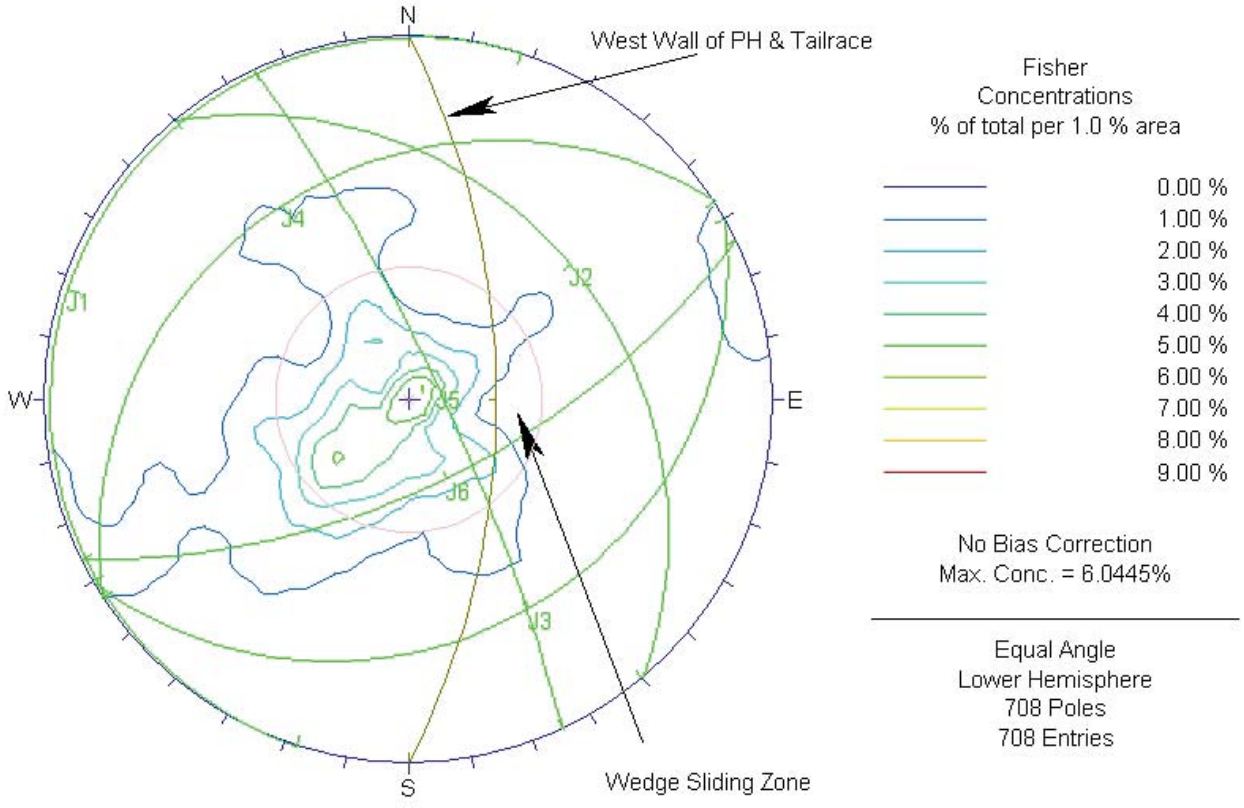
Wedges Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: West Wall of the Powerhouse and Tailrace (Az. 0)

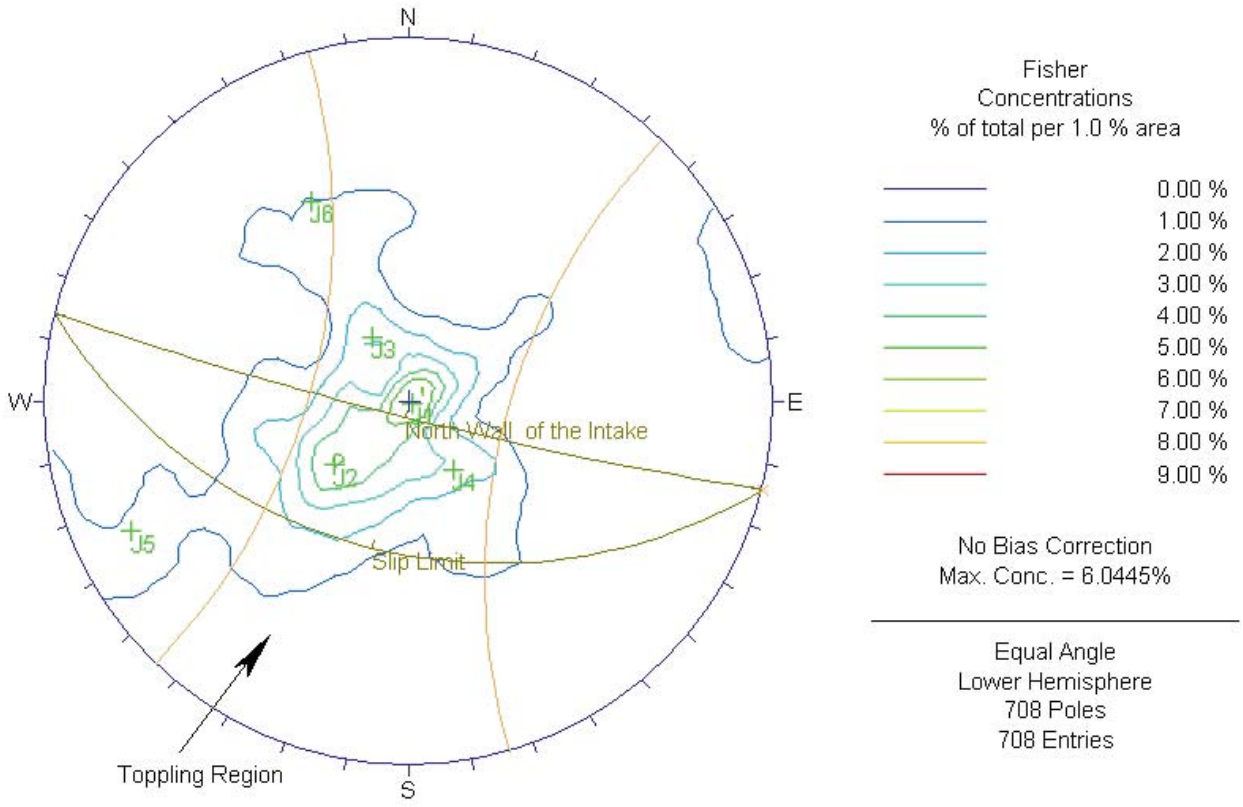
Wedges Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Intake Channel (Az 104)

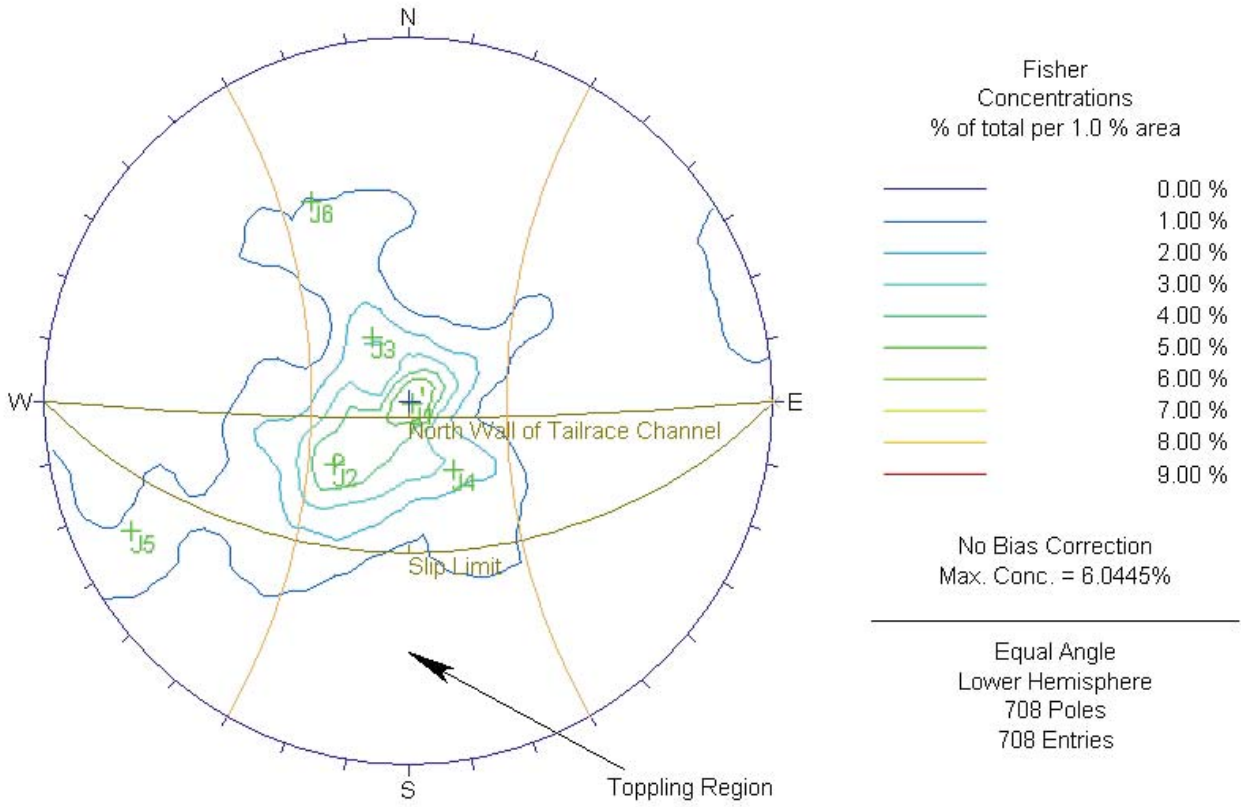
Toppling Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Tailrace Channel (Az 90)

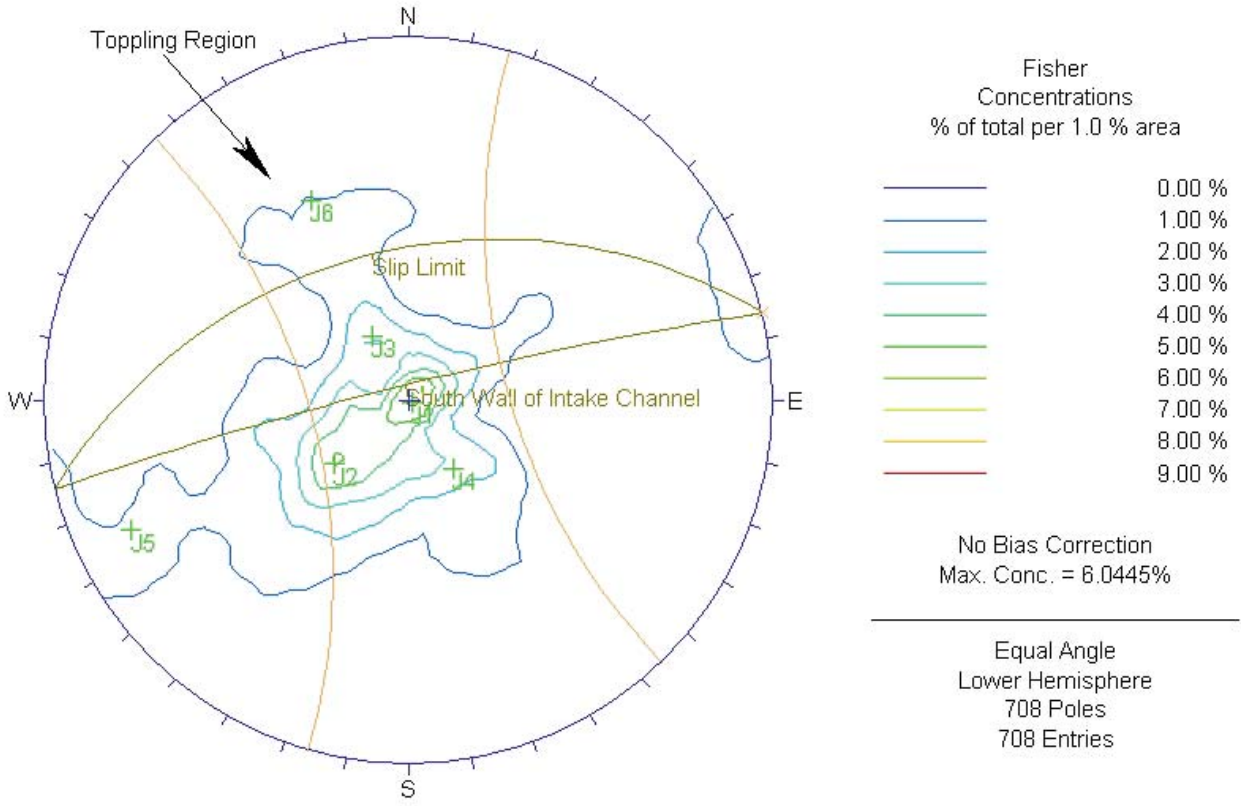
Toppling Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Intake Channel (Az 256)

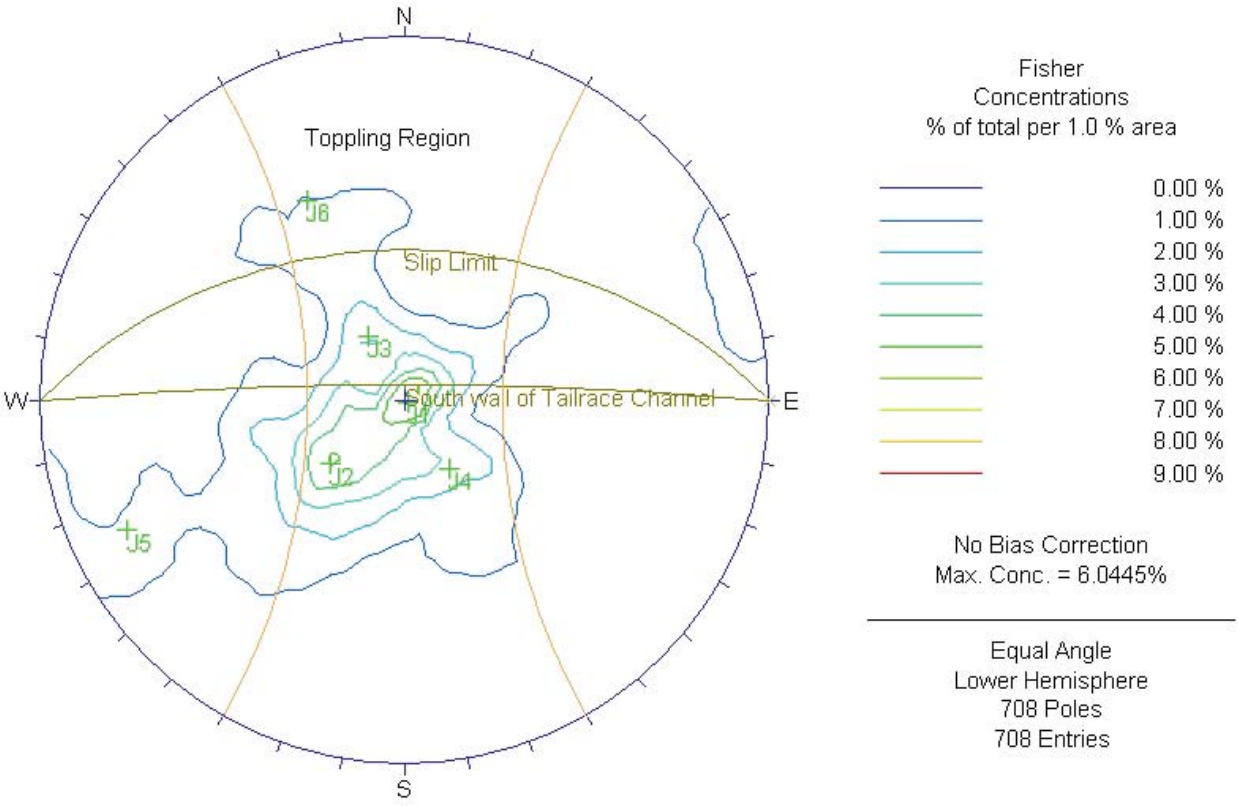
Toppling Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Tailrace Channel (Az 270)

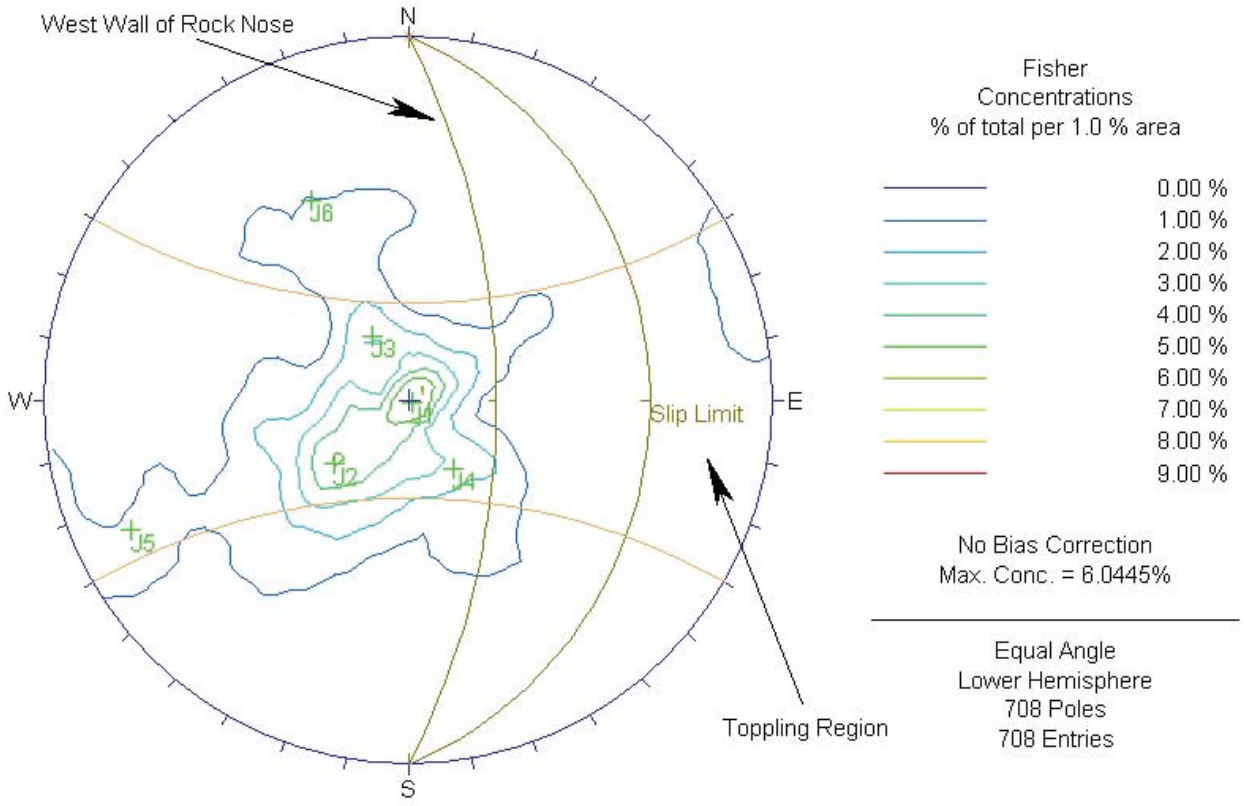
Toppling Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: West Wall of the Rock Nose (Az 0)

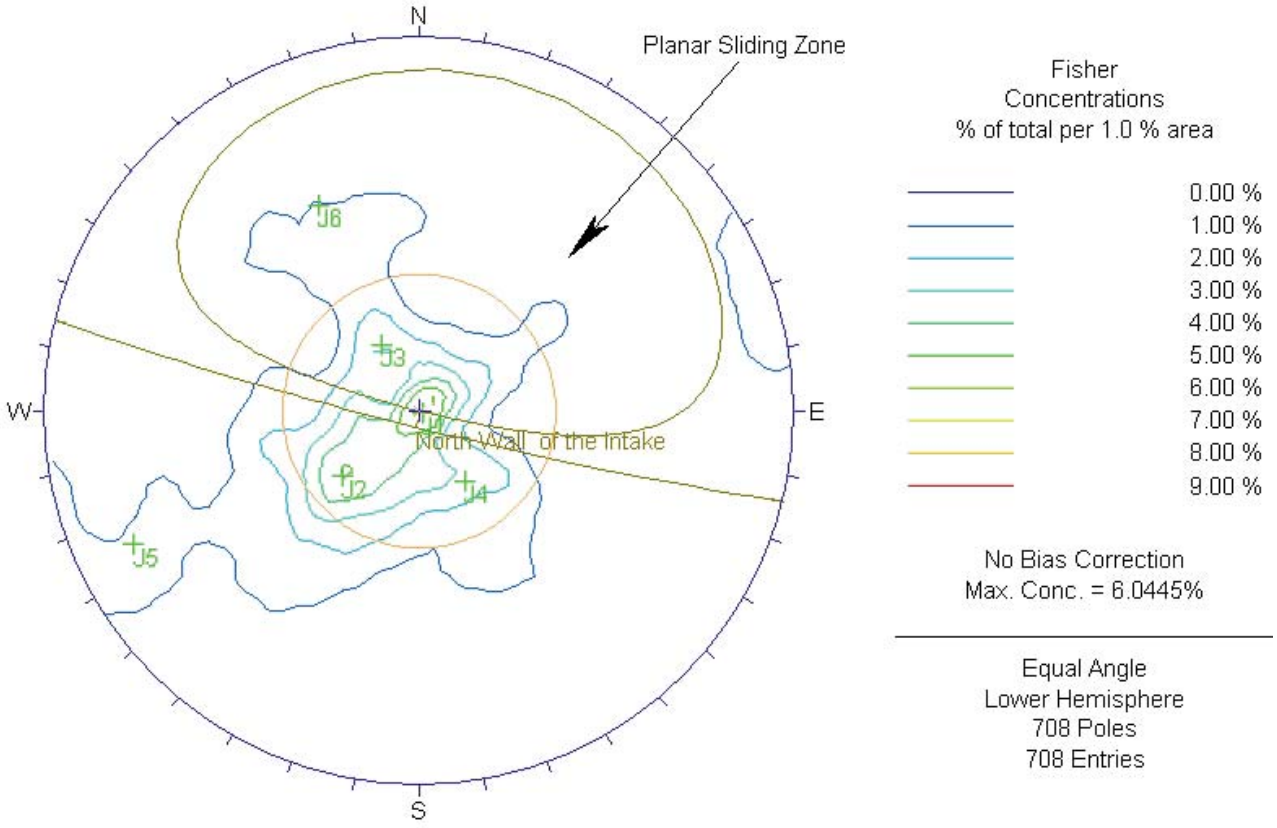
Toppling Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Intake Channel (Az 104)

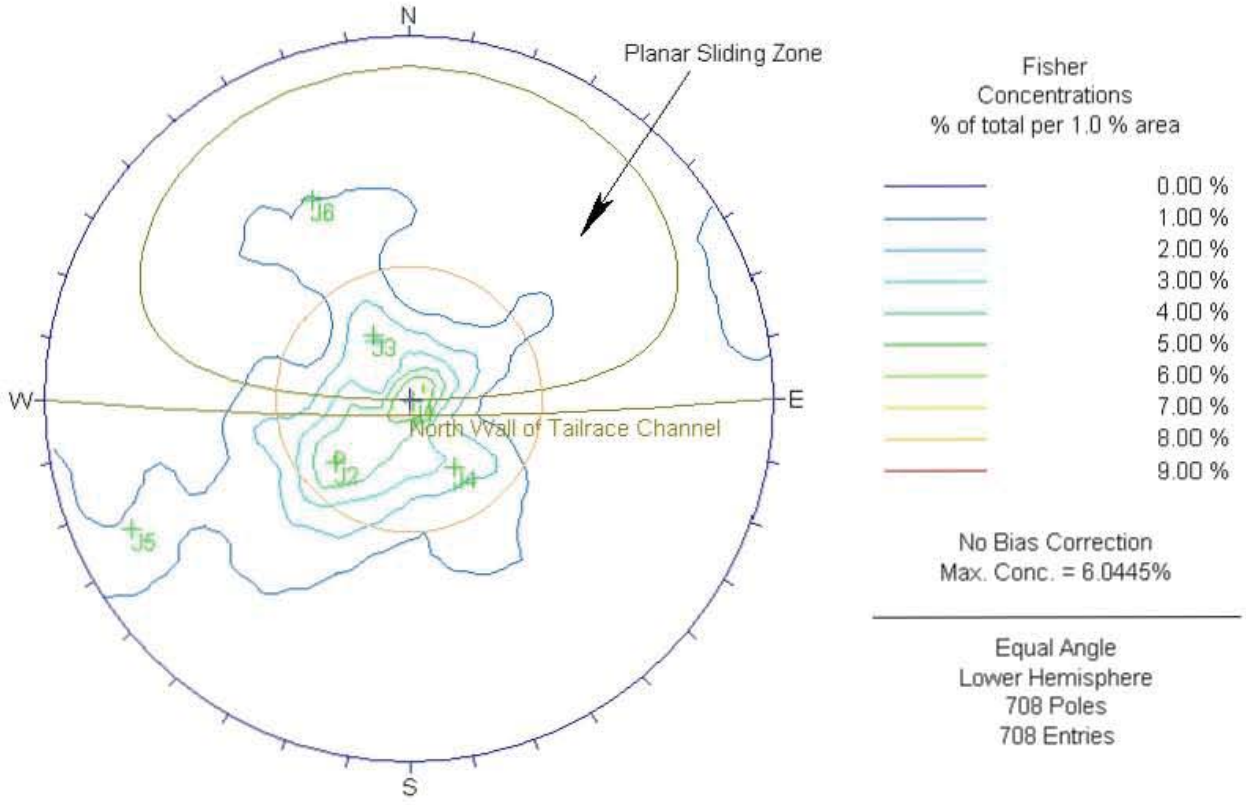
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: North Wall of the Tailrace Channel (Az 90)

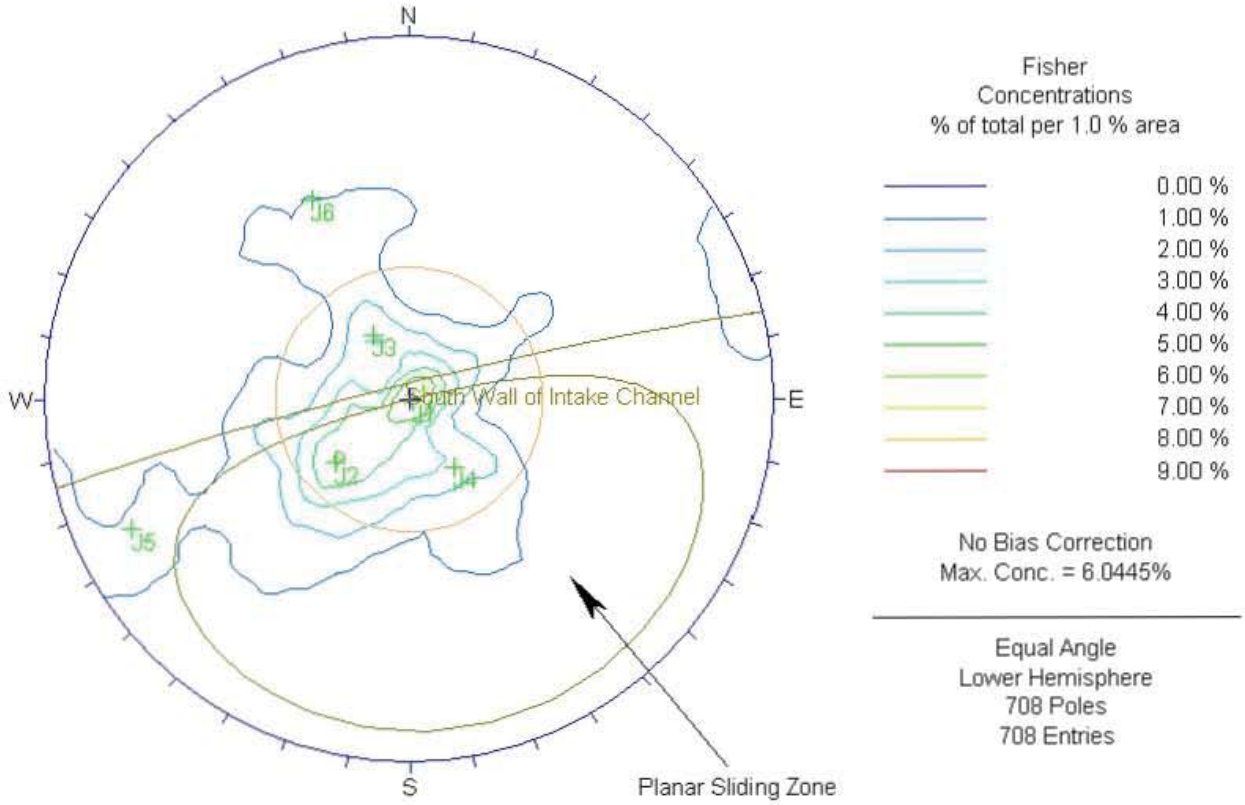
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Intake Channel (Az 256)

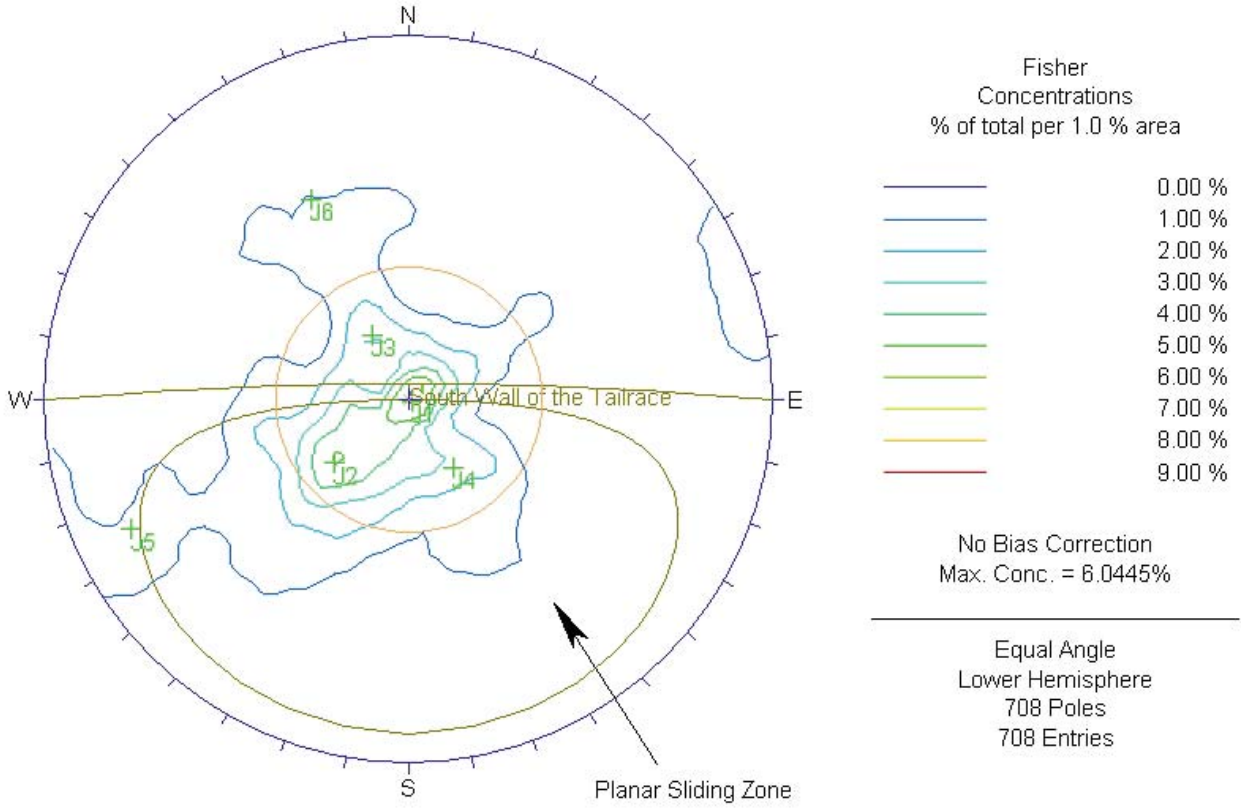
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: South Wall of the Tailrace Channel (Az 270)

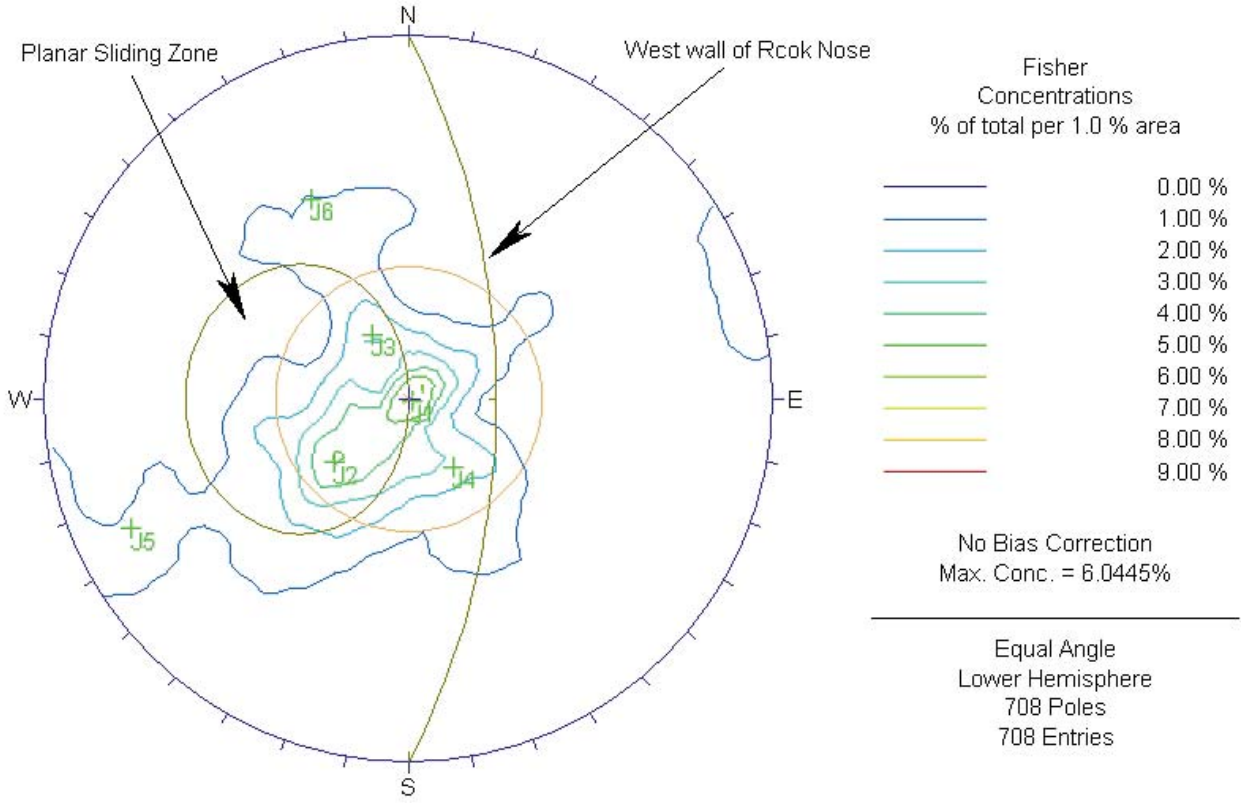
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Powerhouse Area: West Wall of Rock Nose (Az 0)

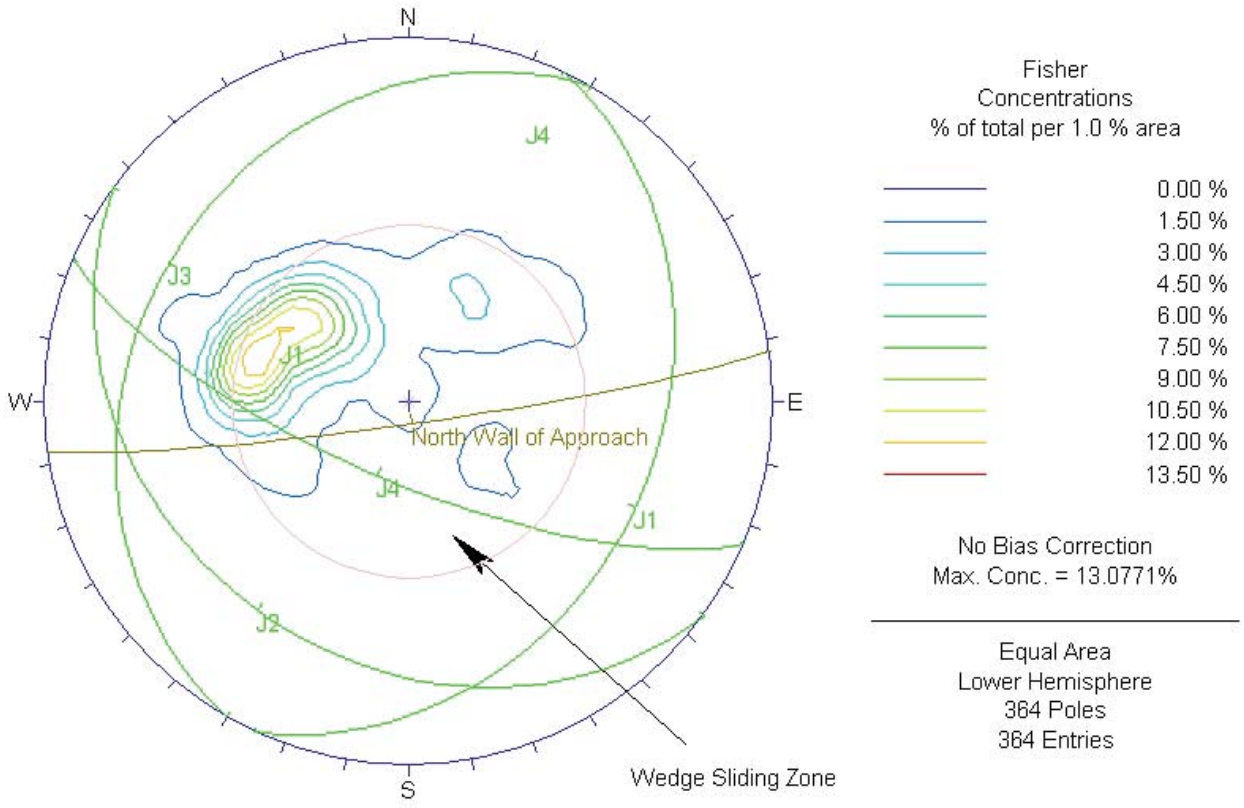
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Approach Channel (Az 82)

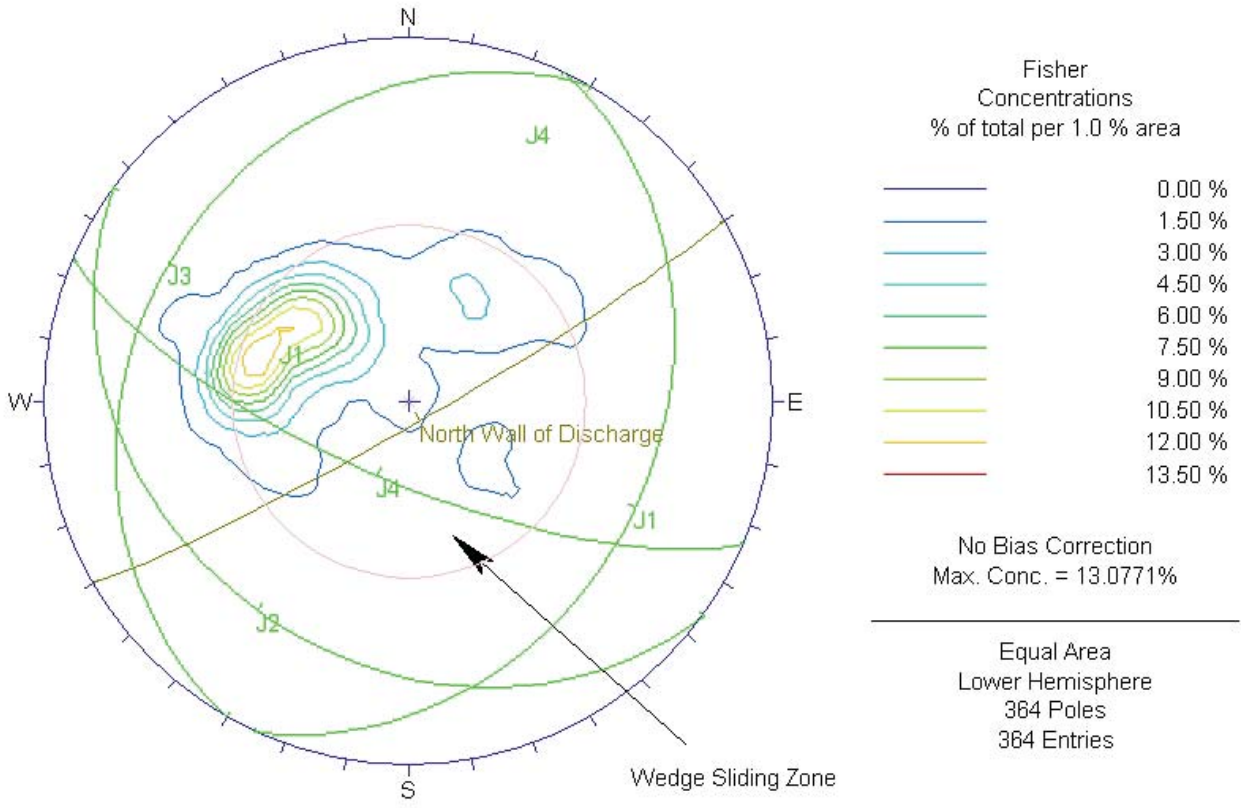
Wedge Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Discharge Channel (Az 60)

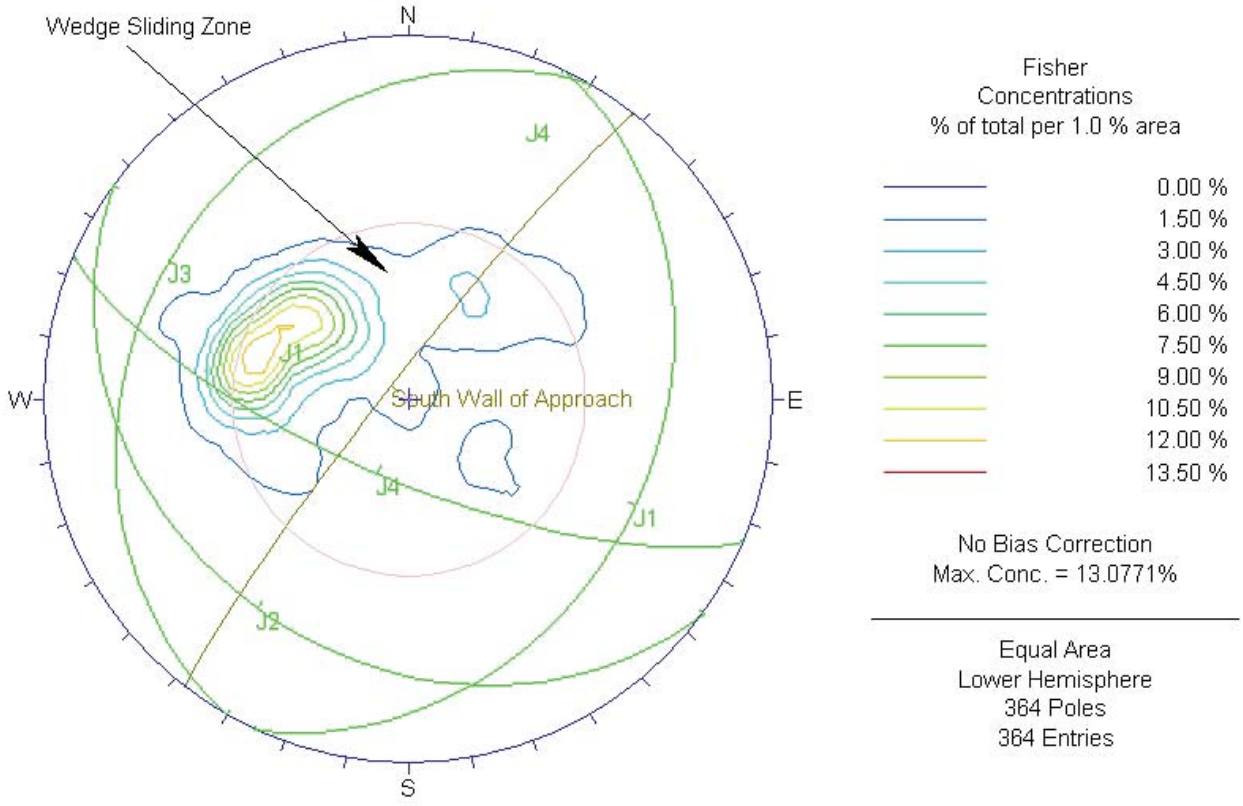
Wedge Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Approach Channel (Az 218)

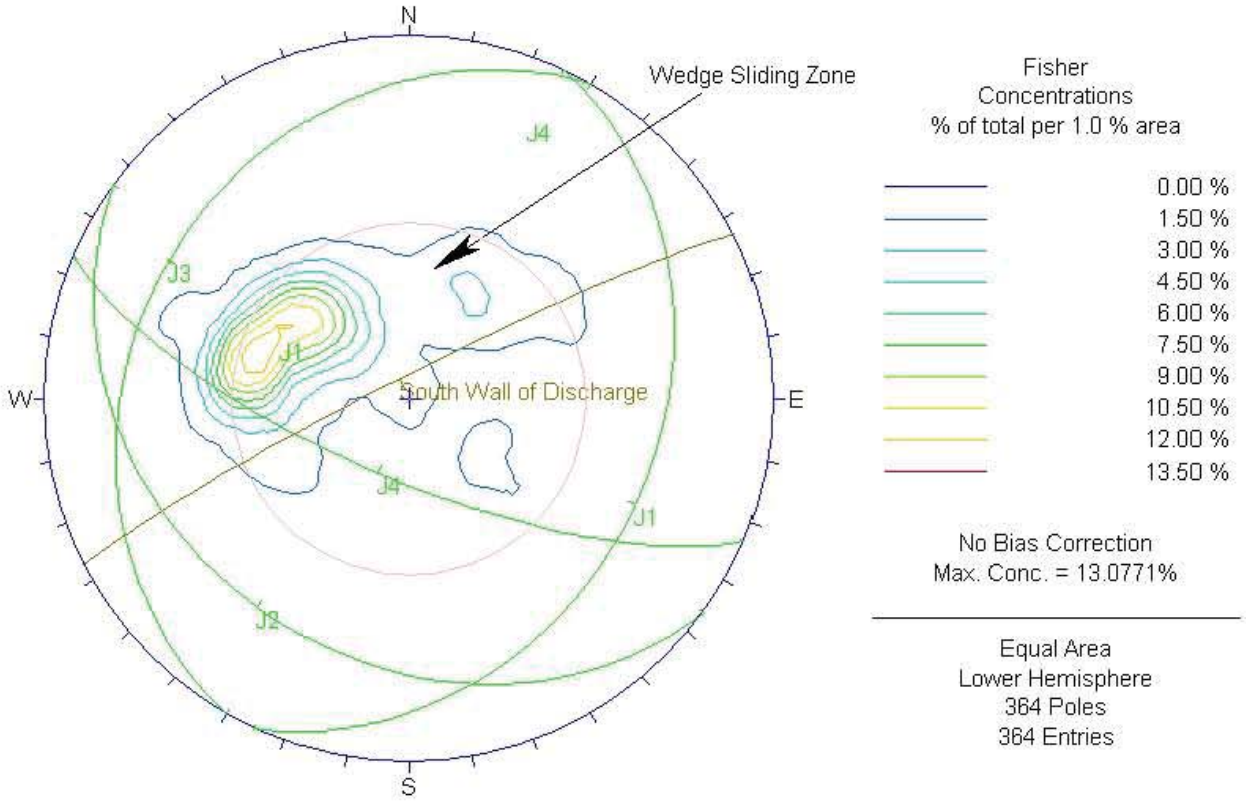
Wedge Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Discharge Channel (Az 243)

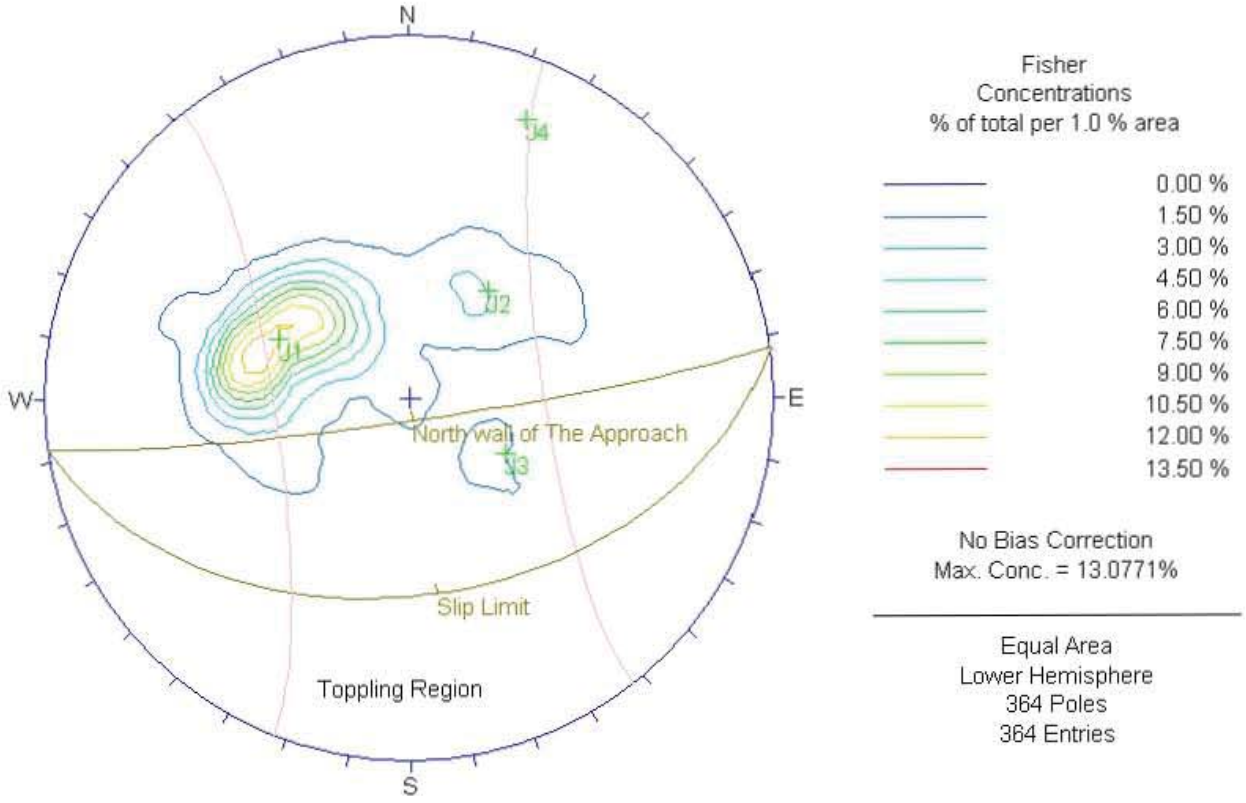
Wedge Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Approach Channel (Az 82)

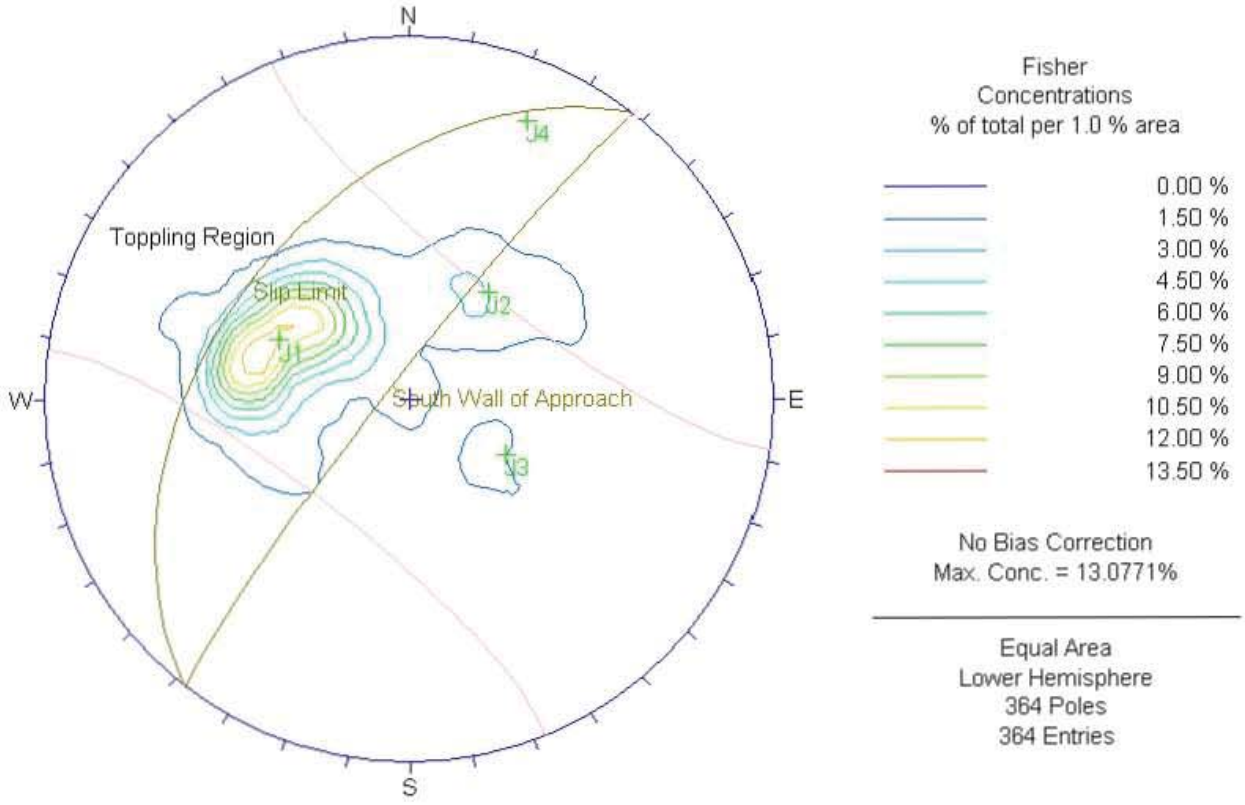
Toppling Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Approach Channel (Az 218)

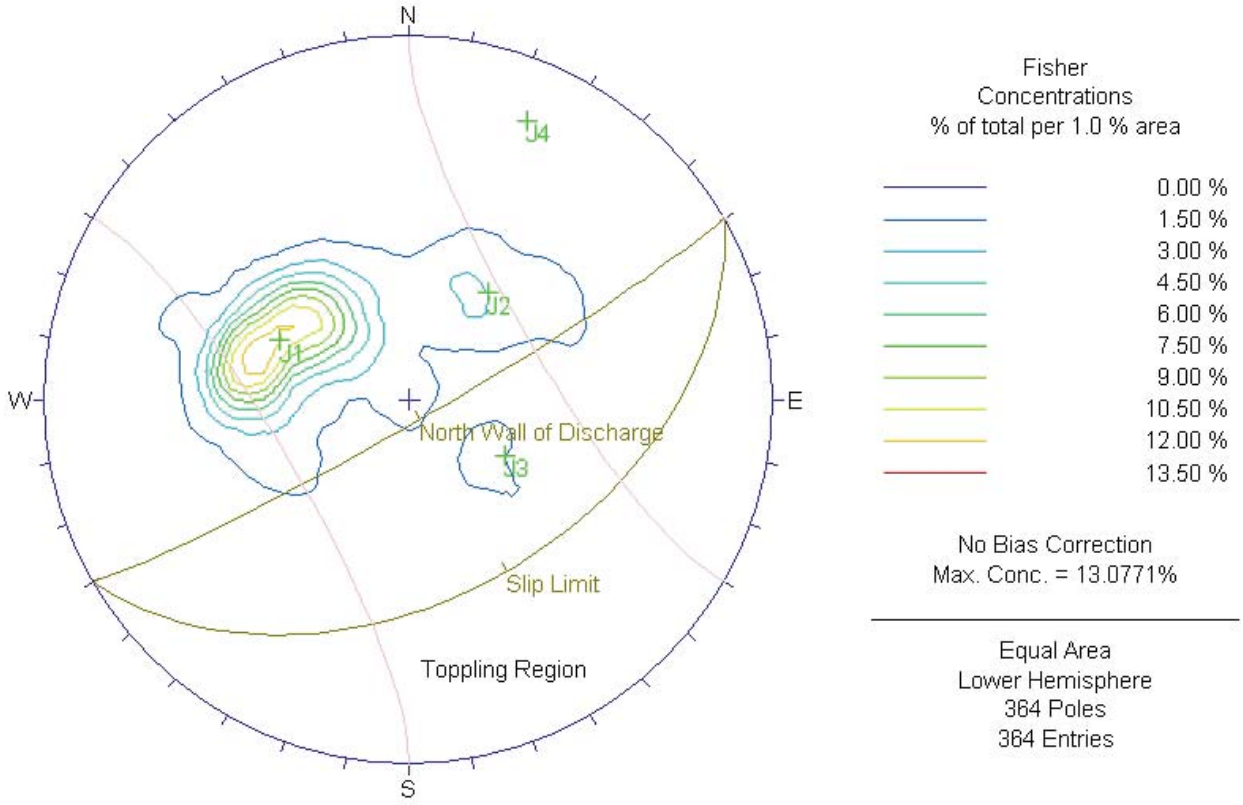
Toppling Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Discharge Channel (Az 60)

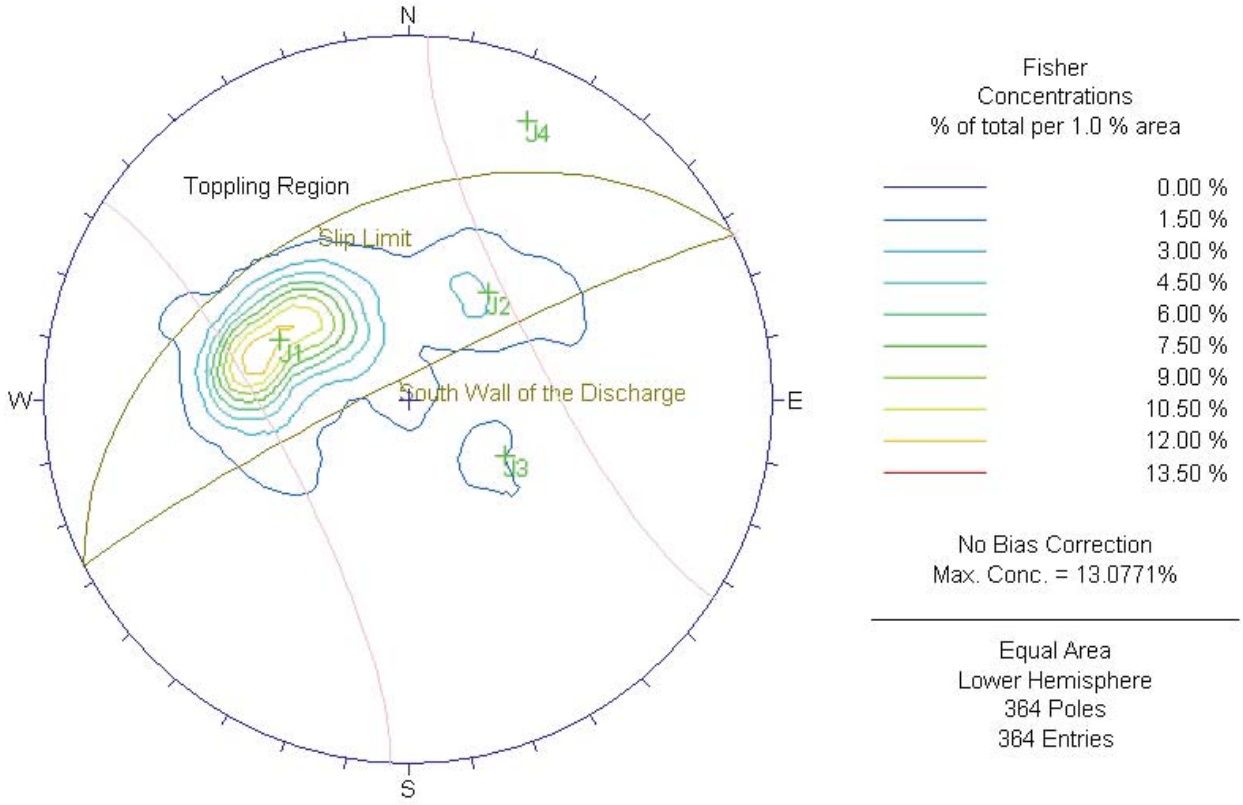
Toppling Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Discharge Channel (Az 243)

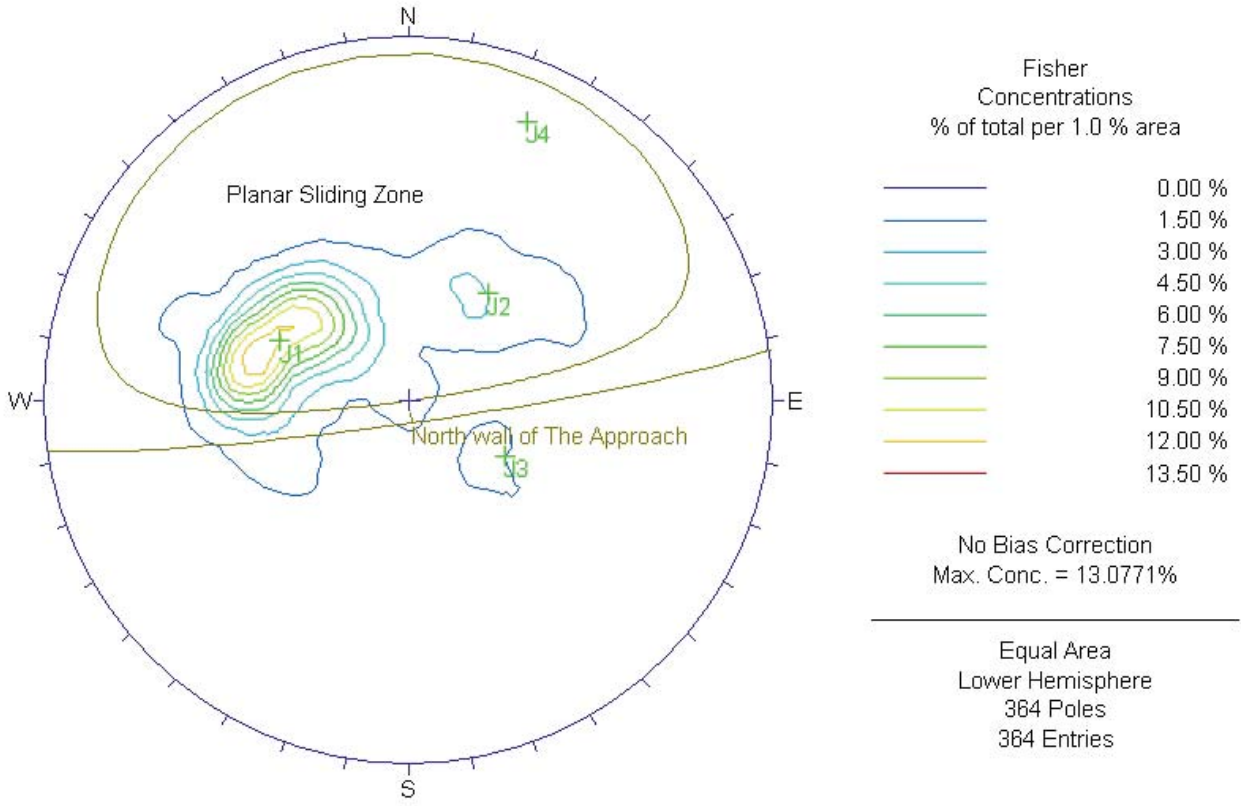
Toppling Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Approach Channel (Az 82)

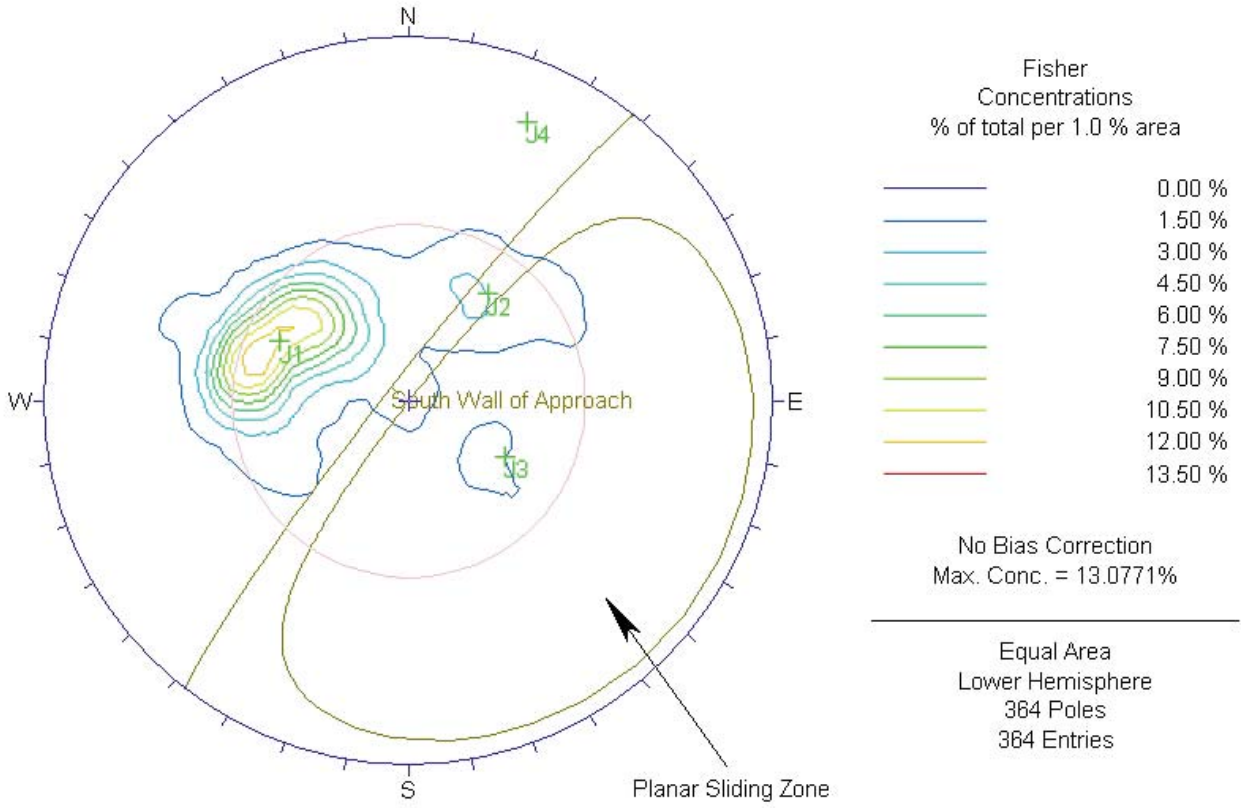
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Approach Channel (Az 218)

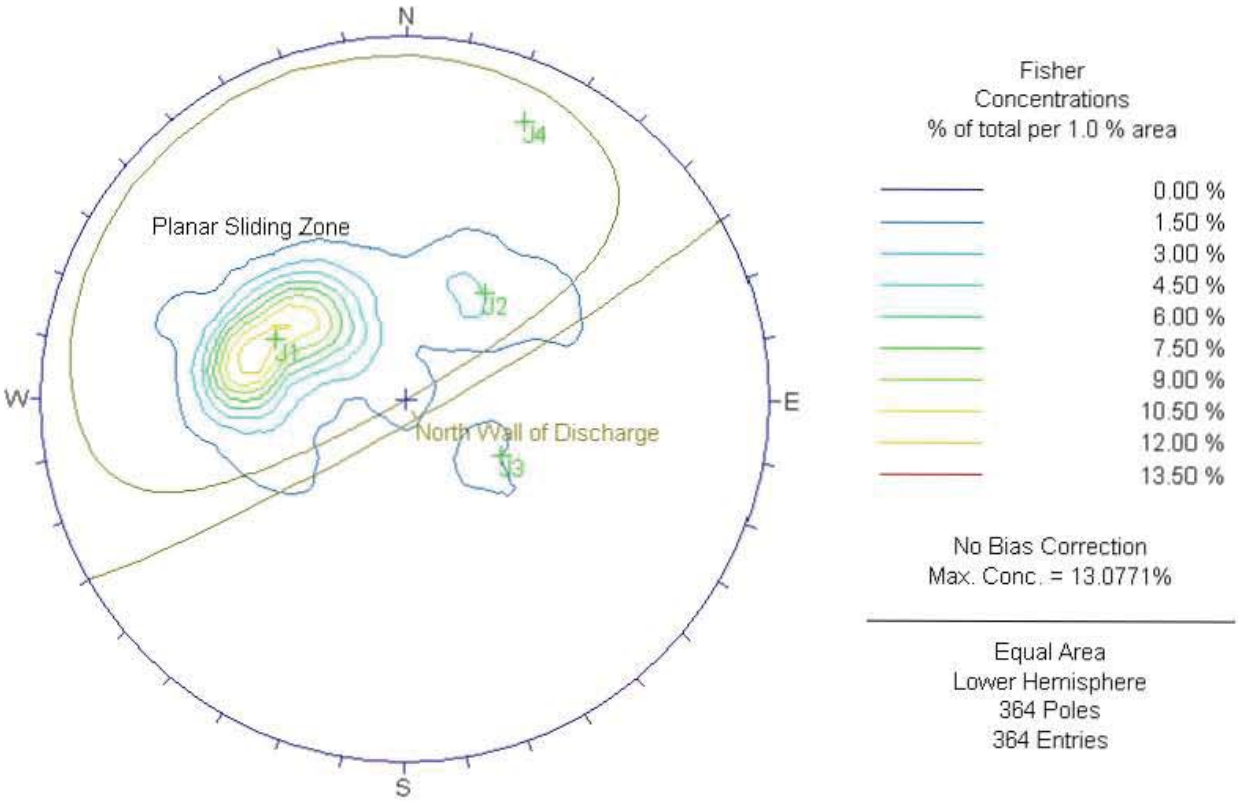
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: North Wall of the Discharge Channel (Az 60)

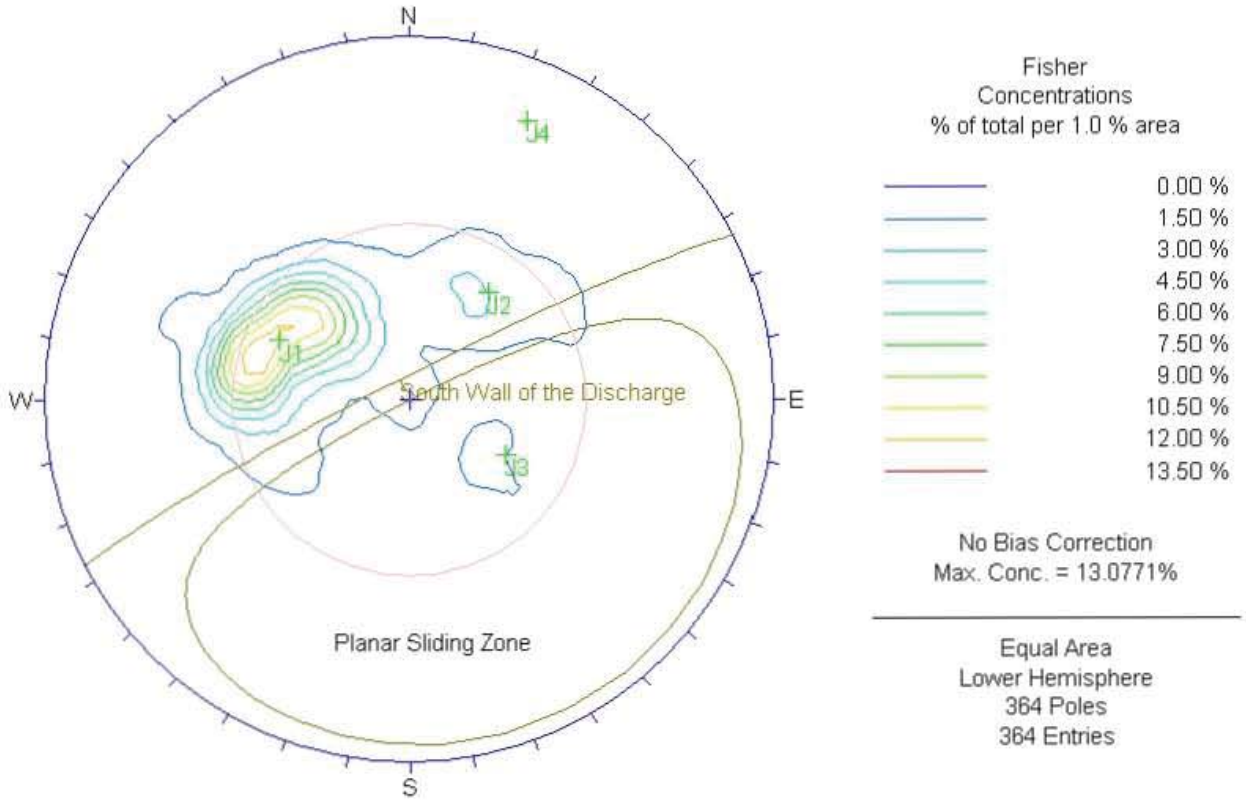
Planar Sliding Analysis



Keeyask Generation Station – Axis GR4

Spillway Area: South Wall of the Discharge Channel (Az 243)

Planar Sliding Analysis



Appendix C

Quantity Estimate – Rock Excavation

Table C1
Keeyask GS - Stage IV Studies, Axis GR-4
Bedrock Excavation - Quantity Estimate

Structure	Length (m)	Volume (m^3)
Powerhouse		
Concrete Structures	212.0	260,000
Intake Channel	600.0	717,200
Tailrace Channel	612.0	603,800
Spillway		
Concrete Structures	119.0	63,700
Approach Channel	128.0	118,000
Discharge Channel	261.0	234,300
TOTAL	-	**1,997,000**

Table C2
Keeyask GS - Stage IV Studies, Axis GR-4
Overbreak Quantity Estimate

Structure	Bedrock Elevation	Base Elevation	Excavation Height (m)	Width (m)		Length (m)		Perimeter Length (m)	Structure Area (m^2)	Horizontal Skew (m)	Vertical Skew (m)	Overbreak Quantity (m^3)
				a	b	c	d					
PH - N. Transition	No excavation required for this structure											
PH - S. Transition	135.0	123.50	11.50	9.1	9.1	14	14	37.1	127.4	0.1	0.2	68
Powerhouse	137.0	112.00	25.00	68	68	212	212	542	14,462	0.1	0.2	4,247
Intake Channel	142.0	135.00	7.00	212	465	469	550	1,019	172,391	0.1	0.2	35,191
Tailrace Channel	137.0	135.00	2.00	212	212	612	612	1,223	129,680	0.1	0.2	26,181
Spillway	147.0	134.90	12.10	37	37	119	119	312	4,373	0.1	0.2	1,252
Approach Channel[(1)]	146.0	140.95	5.05	220	116	274	138	966	34,620	0.1	0.2	7,412
Discharge Channel [(1)]	146.5	139.95	6.55	117	133	356	286	963	40,127	0.1	0.2	8,656
SP - N. Transition	No excavation required for this structure											
SP - S. Transition	No excavation required for this structure											
Service Bay	137.5	123.50	14	45	45	36	36	117	1,620	0.1	0.2	164
Overbreak in Channel Excavations (rounded)												**77,000**
Overbreak in Concrete Structure Excavation (rounded)												**5,700**

Notes:
(1) perimeter length taken from Figure 3, memo GN-4.3.6-Rev02

Table C3
Keeyask GS - Stage IV Studies, Axis GR-4
Pre-Splitting & Line Drilling Estimate

Structure	Area	Section	Elevation		Rock Excavation			Approx. Lift Dimensions			Total	Pre-Shear	Specification - Bill of Quantities Holes			Cost Estimate Areas	
			$B/R_{avg.}$	$Invert_{avg.}$	Depth	Cut Depth	Lifts	Longitudinal	Lateral	Perimeter	Perimeter	Spacing	No. of	Diameter	Length	Sub Total	Total
			(m)	(m)	(m)	(m)		(m)	(m)	(m)	(m)	(m)		(mm)	(m)	(m^2)	(m^2)
A. Pre-Splitting																	
Spillway	Right	A	147.0	137.9	9	6	1.5	-	5.55	6	8	0.450	20	65	118	51	
		B	147.0	134.9	12	6	2.0	-	36.75	37	74	0.450	166	65	994	445	
		C	147.0	138.9	8	6	1.4	-	5.0	5	7	0.450	16	65	96	41	
	Left	A	144.0	137.9	6	6	1.0	-	5.55	6	6	0.450	14	65	81	34	
		B	144.0	134.9	9	6	1.5	-	36.75	37	56	0.450	125	65	749	334	
		C	144.0	138.9	5	5	1.0	-	5.0	5	5	0.450	12	65	62	26	
	U/S	A	147.0	137.9	9	6	1.5	119	-	119	180	0.450	402	65	2,412	1,083	
		B	137.9	134.9	3	3	1.0	119	-	119	119	0.450	265	65	796	357	
	D/S	A	147.0	138.9	8	6	1.4	119	-	119	161	0.450	358	65	2,148	964	
Inlet Channel	Left		145.0	141.0	4	4	1.0	-	274	274	274	0.450	611	65	2,474	1,112	
	Right		145.0	141.0	4	4	1.0	-	138	138	138	0.450	308	65	1,246	559	
Outlet Channel	Left		144.5	140.0	5	5	1.0	-	356	356	356	0.450	791	65	3,601	1,618	
	Rght		144.5	140.0	5	5	1.0	-	286	286	286	0.450	636	65	2,892	1,299	7,921
Spillway North Transition			-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Spillway South Transition			-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Powerhouse	Structure - left		138.7	112.0	27	6	4.4	-	49	49	219	0.450	489	65	2,931	1,316	
	Structure - right		135.1	112.0	23	6	3.9	-	49	49	190	0.450	424	65	2,541	1,141	
	Structure - U/S		134.9	128.0	7	6	1.1	212		212	243	0.450	541	65	3,244	1,457	
	Intake Slab - Left		137.2	128.0	9	6	1.5	-	19	19	29	0.450	66	65	394	175	
	Intake Slab - Right		132.8	128.0	5	5	1.0	-	19	19	19	0.450	43	65	207	91	4180
Intake Channel	Left		138.0	133.0	5	5	1.0	-	469	469	469	0.450	1,042	65	5,211	2,343	
	Right		134.5	131.3	3	3	1.0	-	550	550	550	0.450	1,223	65	3,975	1,788	4130
Tailrace Channel	Left		137.0	128.2	9	6	1.5	-	612	612	895	0.450	1,989	65	11,934	5,368	
	Right		134.0	125.7	8	6	1.4	-	612	612	844	0.450	1,876	65	11,254	5,062	10,429
Service Bay	U/S		137.5	124.0	14	6	2.3	7	-	7	16	0.450	36	65	216	95	
	D/S		137.5	124.0	14	6	2.3	14	-	14	32	0.450	71	65	426	189	
	RIght		137.5	124.0	14	6	2.3	-	45	45	101	0.450	226	65	1,356	608	891
Walls	A		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	B		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	C		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	D		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Powerhouse North Transition			-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Powerhouse South Transition			137.5	123.5	14	6	2.3	14	9	23	54	0.450	121	65	725	323	323
Miscellaneous Small Structures			-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
															Total Estimated Pre-Splitting (m) (rounded)	**62,100**	**28,000**
B. Line Drilling																	
Powerhouse	Draft Tube Noses		128.0	112.5	15.5	6.0	2.6	212		212	547	0.450	1,217	65	7,299	3,282	
															Total Estimated Line Drilling (m) (rounded)	**7,300**	**3,300**

Appendix D

Quantity Estimate – Rock Support

Table D1
Keeyask GS - Stage IV Studies, Axis GR-4
Rock Support Estimate

Location			Safety Mesh			Exposed Rock Face					Bolt Details[4]						
			Exposed Area[1]	No. of Dowels[2]	No. of Mesh Anchors[3]	Cut Length	Max. Height	Avg. Height	Support Height	Facial Area	Bolt Spacing				No. of Bolts	Bolt Length	Total Length
Structure		Rock Face	(m^2)			(m)	(m)	(m)	(m)	(m^2)	(m)		(m)	(m^2)		(m)	(m)
A	Intake Channel	North (w/i 10 m)	90	2	5	10	9.0	9	9	90	4	x	4	16	6	8	48
		South (w/i 10 m)	48	2	3	10	4.8	5	5	48	8	x	5	40	2	6	12
		North (o) - spot	81	2	4	459	9.0	5	5	2,066	6	x	5	30	69	6	414
		South (o) - spot	23	1	1	540	4.8	2	2	1,296	8	x	8	64	0	6	-
B	Service Bay	Upstream	97	2	5	7	14.0	14	14	97	3	x	3	9	11	12	132
		Downstream	190	3	10	14	14.0	14	14	190	6	x	6	36	6	3	18
		North	634	8	32	45	14.0	14	14	634	4	x	4	16	40	6	240
C	Powerhouse	West	4,843	36	243	212	25.0	23	23	4,843	3	x	3	9	539	12	6,468
		North	1,335	12	67	68	25.0	16	20	1,335	4	x	4	16	84	6	504
		South	1,096	12	55	68	15.7	16	16	1,096	8	x	8	64	18	6	108
D	Tailrace Channel	North (w/i 10 m)	208	2	11	10	20.8	21	21	208	3	x	3	9	24	12	288
		South (w/i 10 m)	180	2	9	10	18.0	18	18	180	8	x	8	64	3	6	18
		North (o) - spot	350	4	18	602	18.7	9	9	5,629	6	x	6	36	157	6	942
		South (o) - spot	256	3	13	602	16.0	8	8	4,816	8	x	8	64	76	6	456
E	Spillway	West (U/S)	357	20	18	119	3.0	3	3	357	6	x	3	18	20	3	60
		East (D/S)	476	20	24	119	4.0	4	4	476	6	x	4	24	20	3	60
		North	306	7	16	37	9.1	8	8	306	3	x	3	9	34	12	408
		South	416	7	21	37	12.1	11	11	416	6	x	6	36	12	12	144
F	Approach Channel	North (w/i 10 m)	71	2	4	10	7.1	7	7	71	3	x	3	9	8	12	96
		South (w/i 10 m)	71	2	4	10	7.1	7	7	71	6	x	6	36	2	12	24
		North (o) - spot	50	2	3	264	7.1	4	4	937	6	x	4	24	40	12	480
		South (o) - spot	50	2	3	128	7.1	4	4	454	6	x	4	24	19	12	228
G	Discharge Channel	North (w/i 10 m)	50	2	3	10	5.6	5	5	50	3	x	3	9	6	12	72
		South (w/i 10 m)	50	2	3	10	5.6	5	5	50	6	x	6	36	2	12	24
		North (o) - spot	31	1	2	251	5.6	5	5	1,142	6	x	5	30	39	12	468
		South (o) - spot	31	1	2	251	5.6	5	5	1,142	6	x	5	30	39	12	468
Estimated Total:			11,391	159	577									**Estimated Total:**	1,276		12,180
Anticipated Total:			**11,400**	**160**	**580**									**Anticipated Total:**	**850**		**8,100**

Notes:

(1) For the purposes of this estimate, safety mesh was assumed to only be required on the Channel walls for a distance, from the concrete structures, equivalent to the height of the excavation face at the edge of concrete.

(2) Dowel spacing assumed to be at 6 m centers, and 1.5 m in length

(3) Based on a mesh anchor rate of one anchor per 20 m^2.

(4) Support load from each bolt assumed to be 158 kN

Table D2
Keeyask GS - Stage IV Studies, Axis GR-4
Temporary and Permanent Drainage

Location/Structure		Dimensions			Drain Hole Details								
					Type			Number			Total Lengths		
		Height/ Width	Length	Area of Face	Permanent	Temporary	Spacing (centers)	Permanent	No. of Collars	Temporary	Hole Length	Permanent	Temporary
		(m)	(m)	(m2)			(m)				(m)	(m)	(m)
Service Bay	- Excavation - North Wall	14	45	612	-	0	6	-	-	17	6.0	-	102
	- Excavation - U/S face	14	7	95	-	0	6	-	-	3	6.0	-	18
	- Excavation - D/S face	14	14	189	-	0	6	-	-	6	6.0	-	36
Powerhouse	- Excavation - North Wall	27	68	1,335	-	0	6	-	-	38	6.0	-	228
	- Excavation - South Wall	23	68	1,096	-	0	6	-	-	31	6.0	-	186
	- Excavation - U/S face	6.9	212	1,455	-	0	6	-	-	41	6.0	-	246
	- Draft Tube - Floor	17	212	3,525	0	-	-	424	424	-	6.0	2,541	-
	- Draft Tube - Wall	16	212	3,388	0	-	-	212	212	-	6.0	1,270	-
	Intake Channel - S. Wall	3.3	550	894	0	-	6	25	25	-	6.0	150	-
	Intake Channel - N. Wall	5	469	1,173	0	-	6	33	33	-	6.0	198	-
	Tailrace Channel - S. Wall	8.3	612	2,532	0	-	6	71	71	-	6.0	426	-
	Tailrace Channel - N. Wall	9.3	612	2,838	0	-	6	79	79	-	6.0	474	-
Spillway	- Excavation - North Wall	9	*37*	*394*	-	0	6	-	-	11	6.0	-	66
	- Excavation -South Wall	12	*37*	*536*	-	0	6	-	-	15	6.0	-	90
	- Excavation - U/S face	12	119	1,440	-	0	6	-	-	40	6.0	-	240
	Approach Channel - S. Wall	4	138	280	0	-	6	8	8	-	6.0	48	-
	Approach Channel - N. Wall	4	274	555	0	-	6	16	16	-	6.0	96	-
	Discharge Channel - S. Wall	5	286	651	0	-	6	19	19	-	6.0	114	-
	Discharge Channel - N. Wall	5	356	810	0	-	6	23	23	-	6.0	138	-
							Sum	910	910	202		5,455	1,212
							Estimated Total	**910**	**910**	**200**		**5,500**	**1,200**

Figures

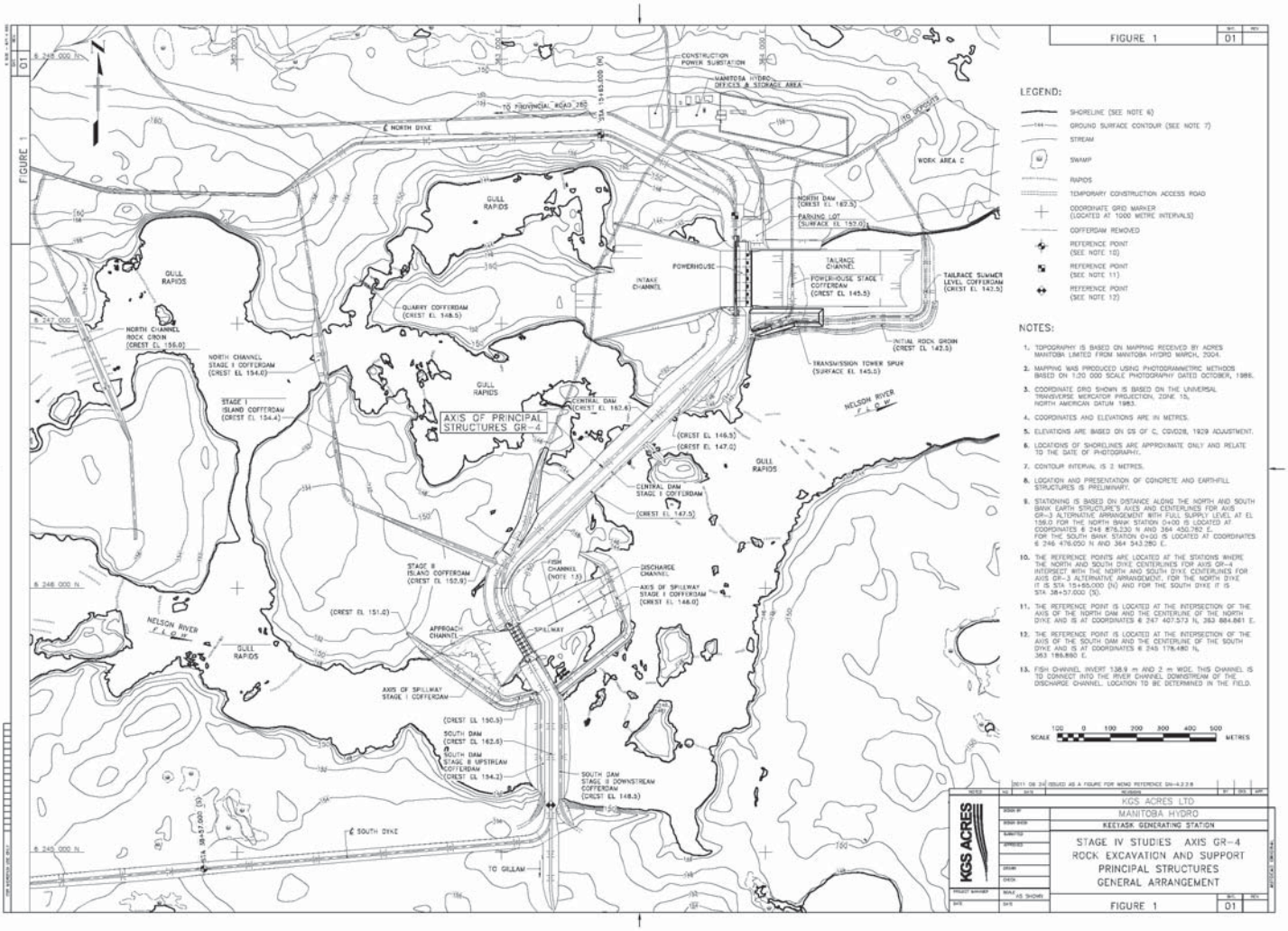
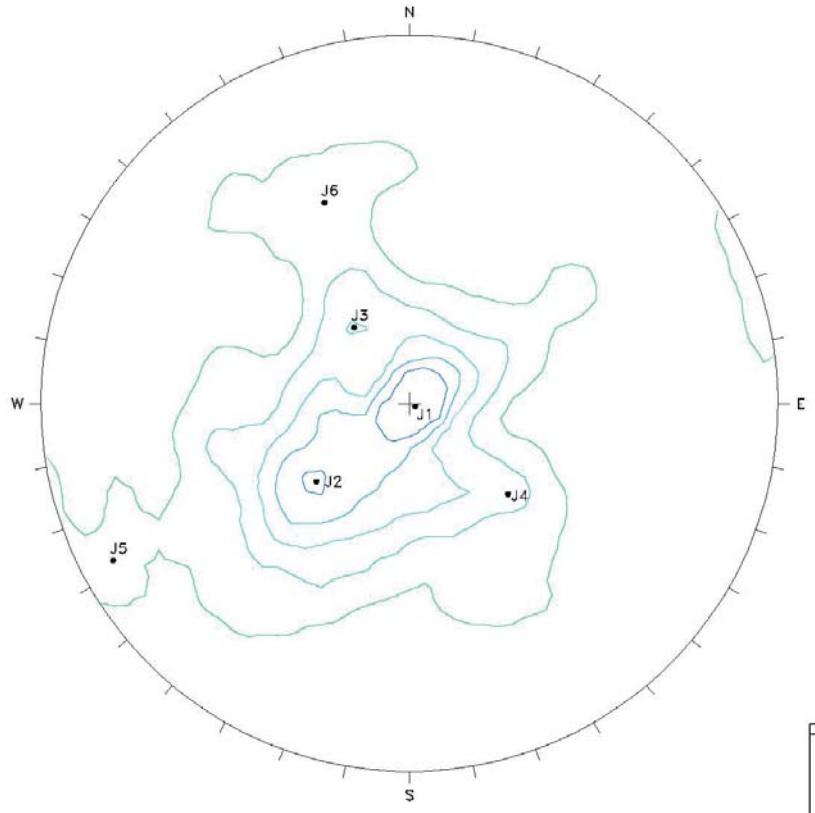


FIGURE 1
01
LEGEND:
SHORELINE (SEE NOTE 6)
GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
TEMPORARY CONSTRUCTION ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
REFERENCE POINT (SEE NOTE 12)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON DS OF C, CGVD28, 1929 ADJUSTMENT.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 158.0 FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES E 246 876.230 N AND 364 450.782 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES E 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES E 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES E 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.
SCALE 100 0 100 200 300 400 500 METRES
AXIS OF PRINCIPAL STRUCTURES GR–4
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO DIESELS & STORAGE AREA
TO PROVINCIAL ROAD 280
℄ NORTH DYKE
TO SPLIDUX
WORK AREA C
GULL RAPIDS
NORTH DAM (CREST EL 162.5)
PARKING LOT (SURFACE EL 157.0)
POWERHOUSE
TAILRACE CHANNEL
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.5)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 143.5)
INTAKE CHANNEL
QUARRY COFFERDAM (CREST EL 148.5)
INITIAL ROCK GROIN (CREST EL 142.5)
TRANSMISSION TOWER SPUR (SURFACE EL 145.5)
NORTH CHANNEL ROCK GROIN (CREST EL 156.0)
NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
STAGE I ISLAND COFFERDAM (CREST EL 154.4)
CENTRAL DAM (CREST EL 162.6)
NELSON RIVER
(CREST EL 146.5)
(CREST EL 147.0)
CENTRAL DAM STAGE I COFFERDAM
(CREST EL 147.5)
STAGE II ISLAND COFFERDAM (CREST EL 152.9)
FISH CHANNEL (NOTE 13)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM (CREST EL 148.0)
(CREST EL 151.0)
APPROACH CHANNEL
SPILLWAY
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
SOUTH DAM (CREST EL 162.6)
SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 154.2)
SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
℄ SOUTH DYKE
TO GILLAM
E 246 000 mE
E 247 000
N 248 000 N
N 245 000 N
STA 15+65.000 (N)
STA 38+57.000 (S)
KGS ACRES
KGS ACRES LTD
MANITOBA HYDRO
KEETASK GENERATING STATION
STAGE IV STUDIES AXIS GR–4
ROCK EXCAVATION AND SUPPORT
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT
FIGURE 1
01
AS SHOWN

FIGURE 2 | SHT. 01 | REV.



LEGEND:

- POLE REPRESENTATION OF JOINT SET

FISHER CONCENTRATIONS % OF TOTAL PER 1.0% AREA

- 1.00 %
- 2.00 %
- 3.00 %
- 4.00 %
- 5.00 %
- 6.00 %
- 7.00 %
- 8.00 %
- > 9.00 %

NO BIAS CORRECTION
MAXIMUM CONCENTRATION = 6.1269%

EQUAL AREA
LOWER HEMISPHERE
708 POLES
708 ENTRIES

NO.	DATE	REVISIONS	BY	DES.	APP.
	2011 06 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

NOTED

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MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
POLAR DENSITY PLOT
POWERHOUSE AREA

DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

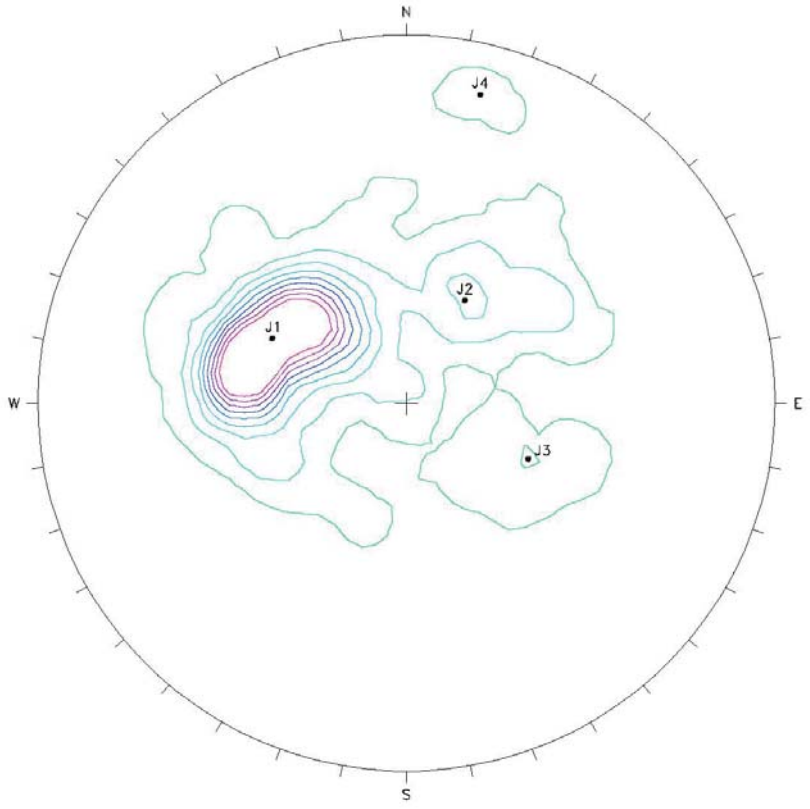
SCALE AS SHOWN

DATE

FIGURE 2 | SHT. 01 | REV.

AUTOCAD ORIGINAL

FIGURE 3 | SHT. 01 | REV.



LEGEND:

- POLE REPRESENTATION OF JOINT SET

FISHER CONCENTRATIONS % OF TOTAL PER 1.0% AREA

	1.00 %
	2.00 %
	3.00 %
	4.00 %
	5.00 %
	6.00 %
	7.00 %
	8.00 %
	> 9.00 %

NO BIAS CORRECTION
MAXIMUM CONCENTRATION = 13.0771%

EQUAL AREA
LOWER HEMISPHERE
364 POLES
364 ENTRIES

NO.	DATE	REVISIONS	BY	DES.	APP.
	2011 06 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
POLAR DENSITY PLOT
SPILLWAY AREA

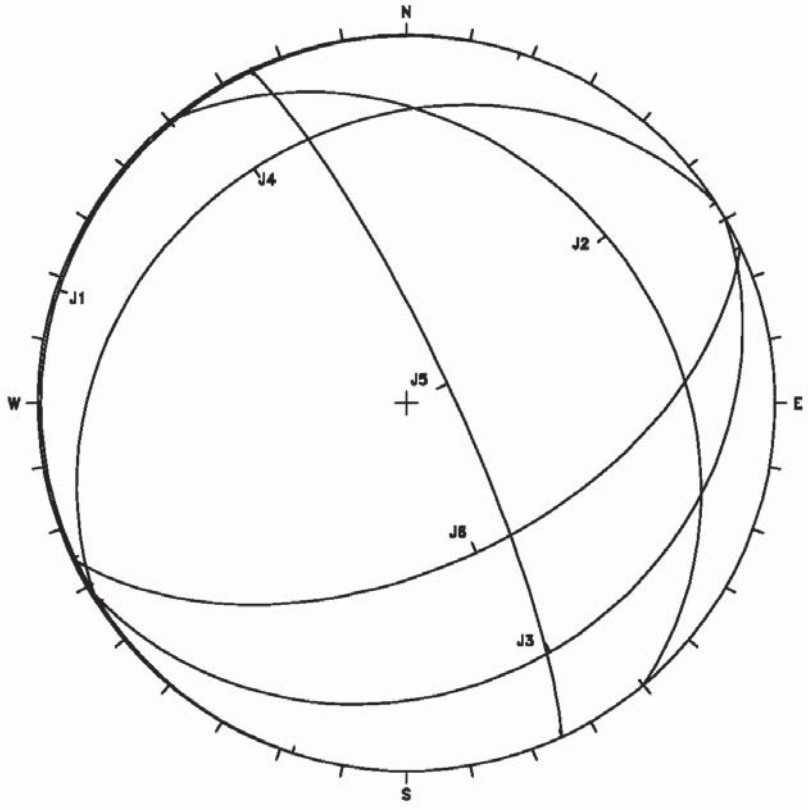
DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE AS SHOWN

FIGURE 3 | SHT. 01 | REV.

AUTOCAD ORIGINAL

FIGURE 4 | SHT. 01 | REV.



ORIENTATIONS

ID	STRIKE/DIP (RIGHT HAND RULE) [DEGREES]
J1	196/01
J2	220/30
J3	060/23
J4	237/29
J5	335/80
J6	065/53

EQUAL AREA
LOWER HEMISPHERE
708 POLES
708 ENTRIES

NOTES	NO.	DATE	REVISIONS	BY	CHK.	APP'D.
	0	2011 06 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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MANITOBA HYDRO
KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
DOMINANT JOINT SETS
POWERHOUSE AREA

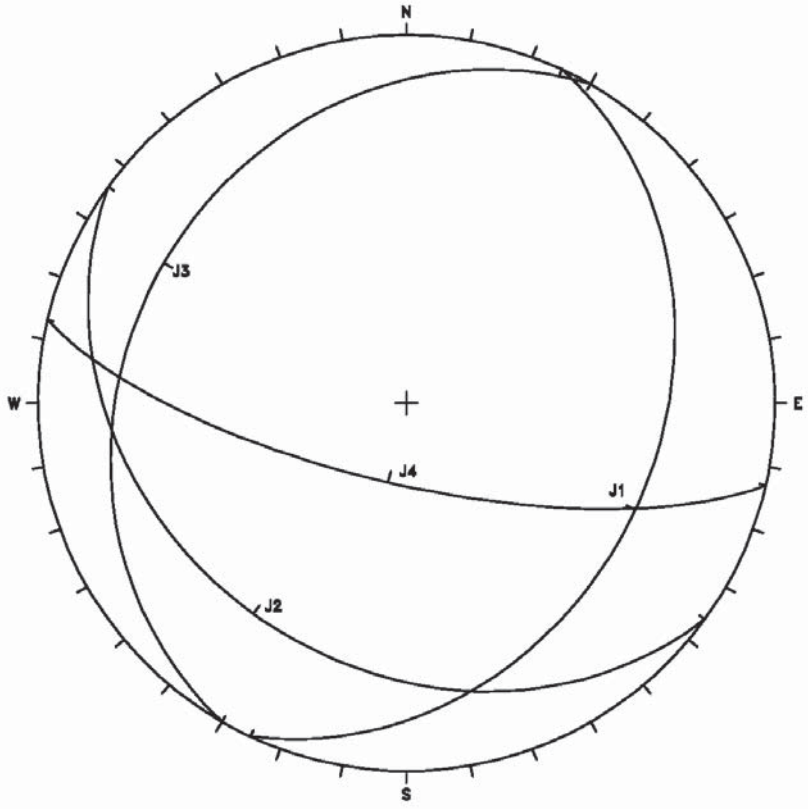
DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE AS SHOWN

FIGURE 4 | SHT. 01 | REV.

AUTOCAD'S ORIGINAL

FIGURE 5 | SHT. 01 | REV.



ORIENTATIONS

ID	STRIKE/DIP (RIGHT HAND RULE) [DEGREES]
J1	025/32
J2	125/30
J3	210/25
J4	103/72

EQUAL AREA
LOWER HEMISPHERE
384 POLES
384 ENTRIES

NO.	DATE	REVISIONS	BY	CHK.	APP'D.
0	2011 06 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

KGS ACRES

KGS ACRES LTD

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
DOMINANT JOINT SETS
SPILLWAY AREA

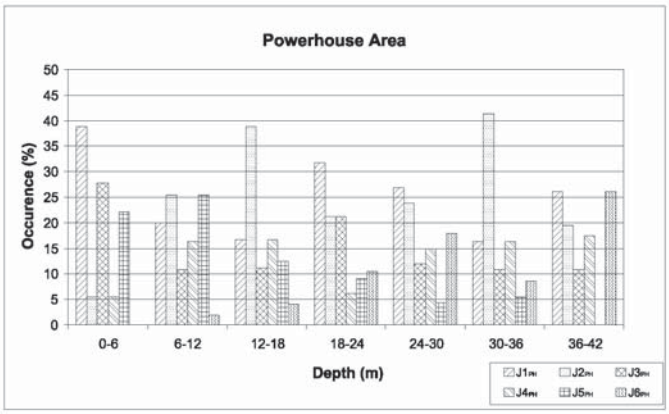
DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

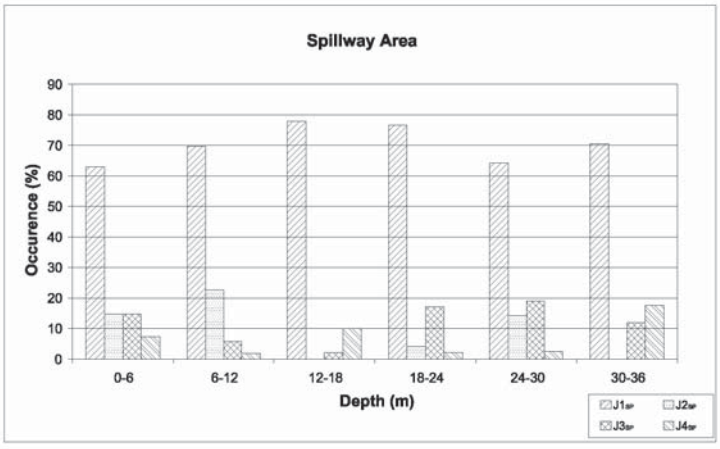
SCALE AS SHOWN

FIGURE 5 | SHT. 01 | REV.

AUTOCAD® ORIGINAL

FIGURE 6 01





2011 08 24 ISSUED AS A FIGURE FOR MDNG REFERENCE DH-4.2.2.8

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KEETASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
JOINT FREQUENCY

FIGURE 6 01

FIGURE 7 | SHT. 01 | REV.

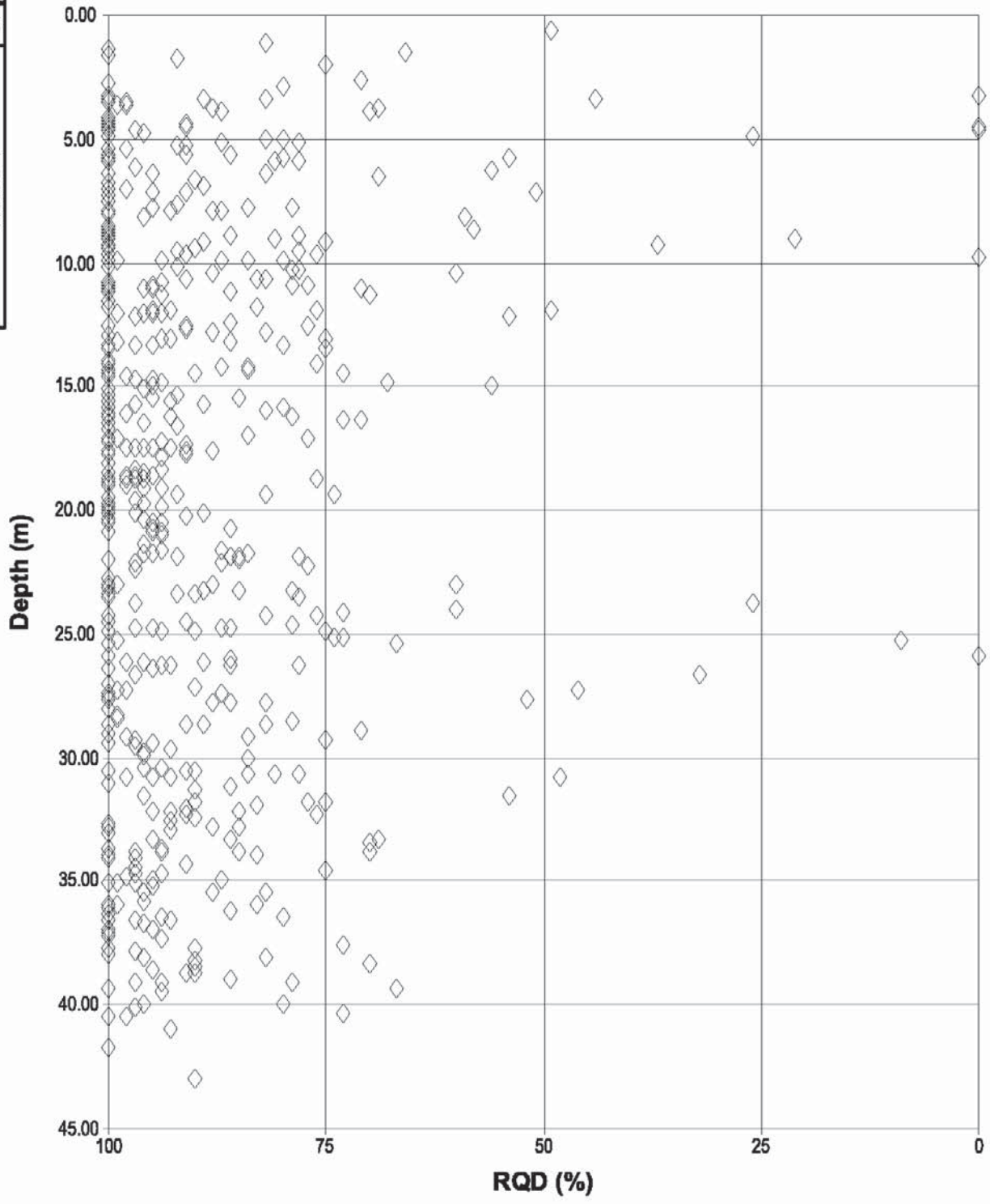

FIGURE 7 01

NOTED	NO.	DATE	REVISIONS	BY	CKD.	APP.
		2011 08 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
RQD VS. DEPTH
POWERHOUSE AREA

DESIGN BY	DESIGN CHECK	SUBMITTED	APPROVED	DRAWN	CHECK	PROJECT MANAGER	DATE

SCALE: AS SHOWN

DATE

FIGURE 7 | SHT. 01 | REV.

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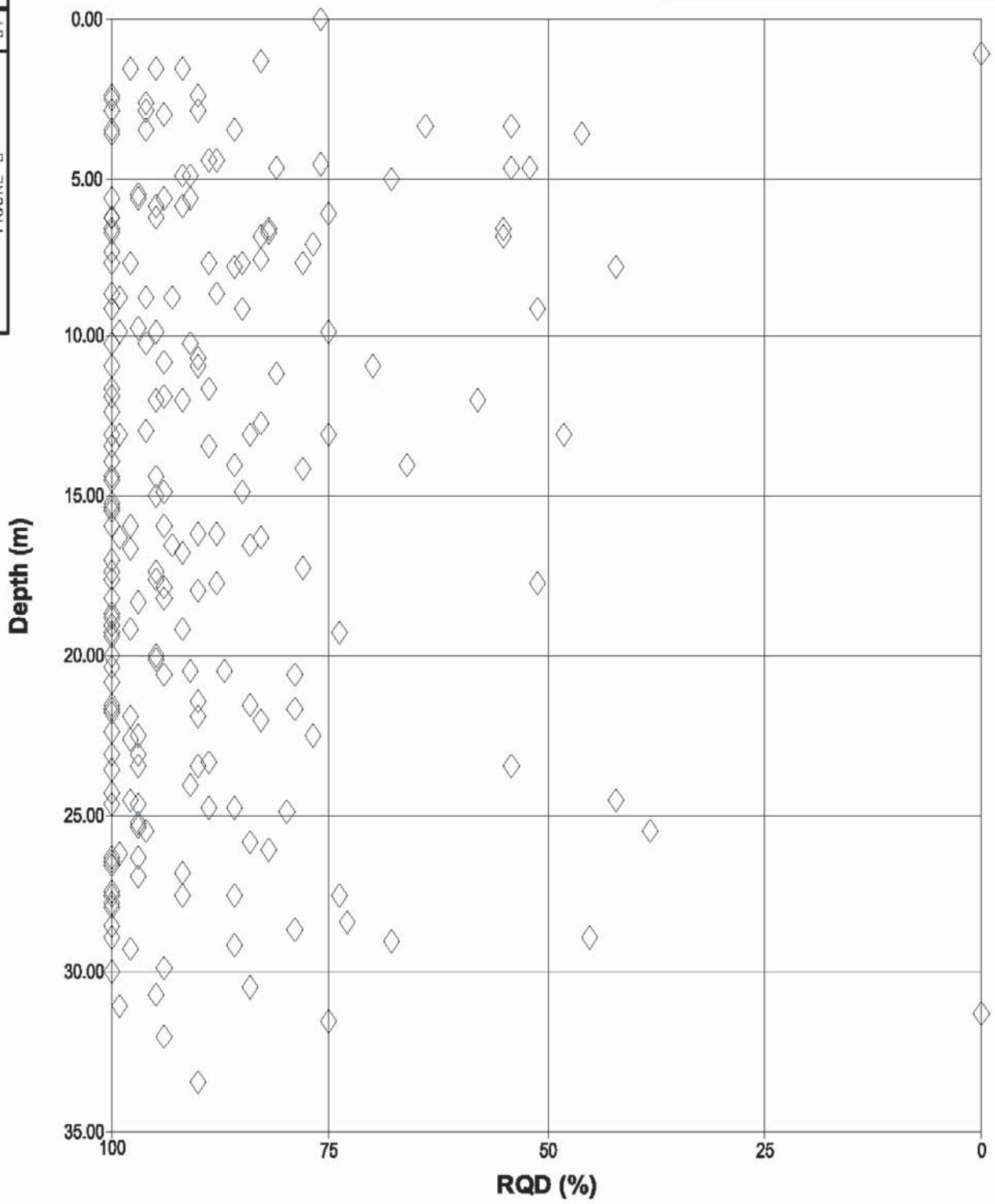
AUTOCAD ORIGINAL

FIGURE 8



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	2011 08 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
RQD vs. DEPTH
SPILLWAY AREA

DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE: AS SHOWN

DATE

FIGURE 8

SHT. 01 REV.

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AUTOCAD ORIGINAL

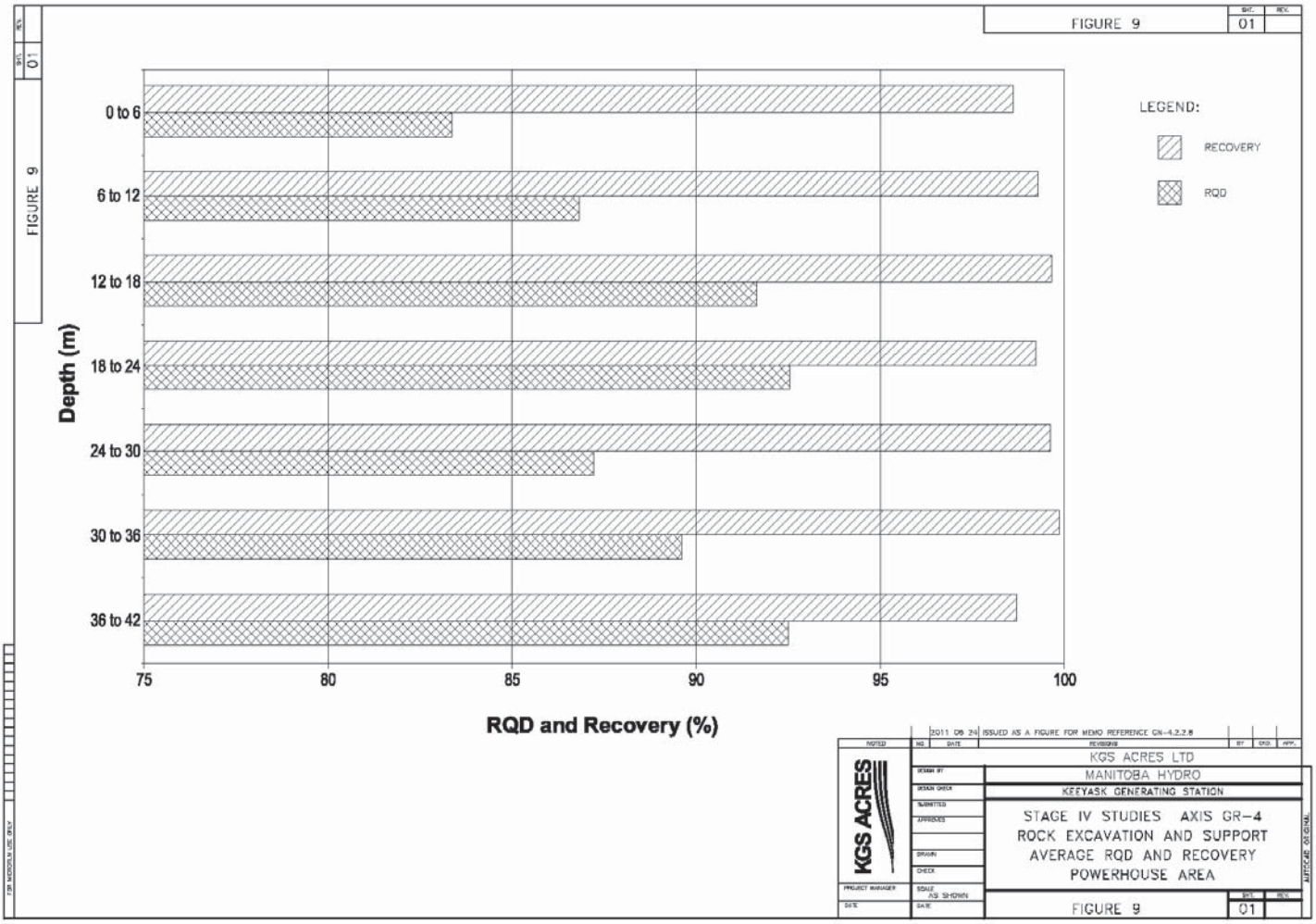
FIGURE 9
01
LEGEND:
RECOVERY
RQD
0 to 6
6 to 12
12 to 18
18 to 24
24 to 30
30 to 36
36 to 42
Depth (m)
75
80
85
90
95
100
RQD and Recovery (%)
2011 06 24 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8
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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
AVERAGE RQD AND RECOVERY
POWERHOUSE AREA
FIGURE 9
01

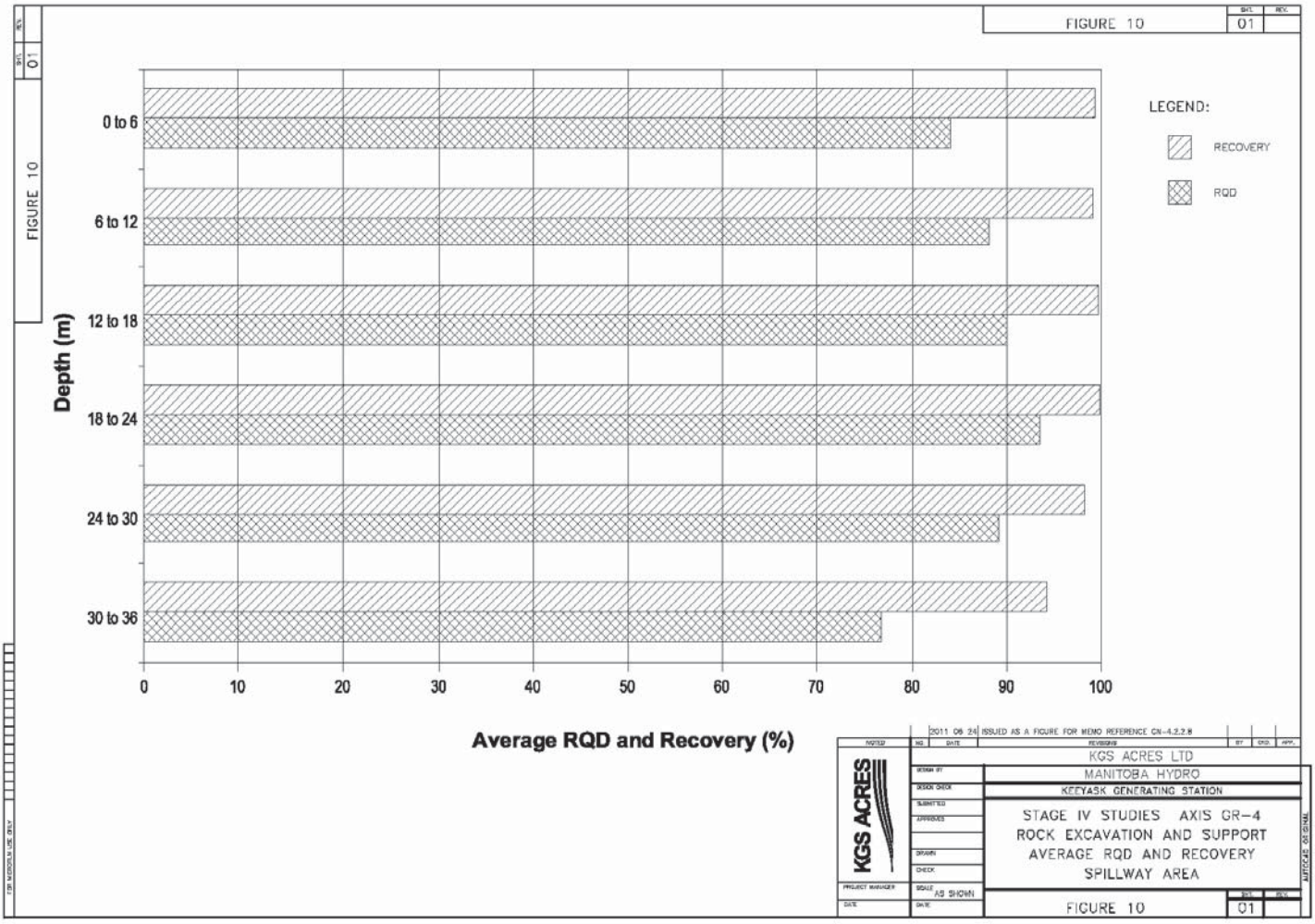
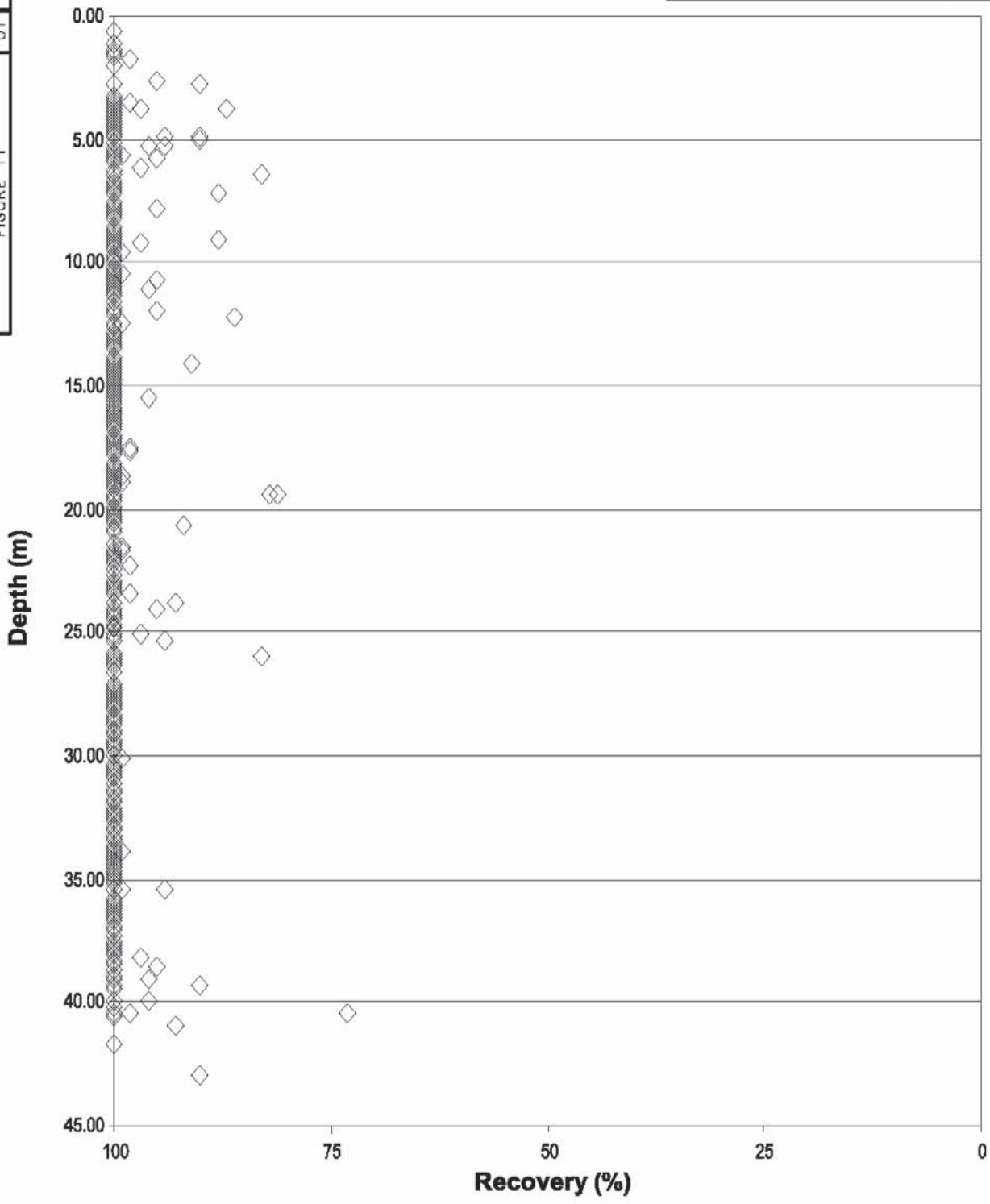


FIGURE 10
01
LEGEND:
RECOVERY
RQD
Depth (m)
0 to 6
6 to 12
12 to 18
18 to 24
24 to 30
30 to 36
0
10
20
30
40
50
60
70
80
90
100
Average RQD and Recovery (%)
2011 06 24 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
AVERAGE RQD AND RECOVERY
SPILLWAY AREA
AS SHOWN
FIGURE 10
01

FIGURE 11 | SHT. 01 | REV.



NOTED	NO.	DATE	REVISIONS	BY	CKD.	APP.
	2011	08 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
PERCENT RECOVERY WITH DEPTH
POWERHOUSE AREA

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SCALE AS SHOWN | DATE

FIGURE 11 | SHT. 01 | REV.

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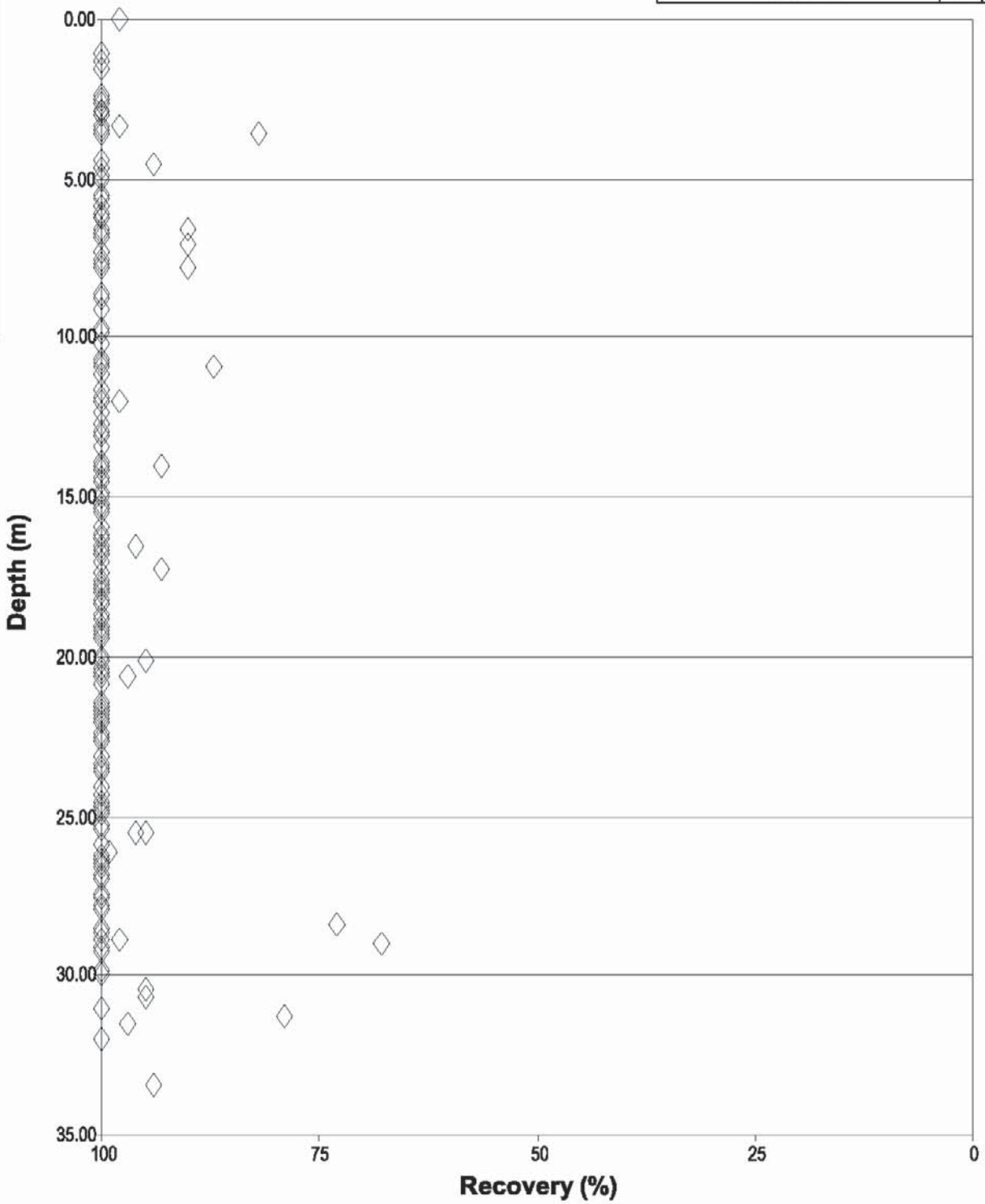
AUTOCAD ORIGINAL

FIGURE 12 | SHT. 01 | REV.



NO.	DATE	REVISIONS	BY	CKD.	APP.
	2011 06 24	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.8			

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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
PERCENT RECOVERY WITH DEPTH
SPILLWAY AREA

DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE AS SHOWN

DATE

FIGURE 12 | SHT. 01 | REV.

FOR MICROFILM USE ONLY

AUTOCAD ORIGINAL

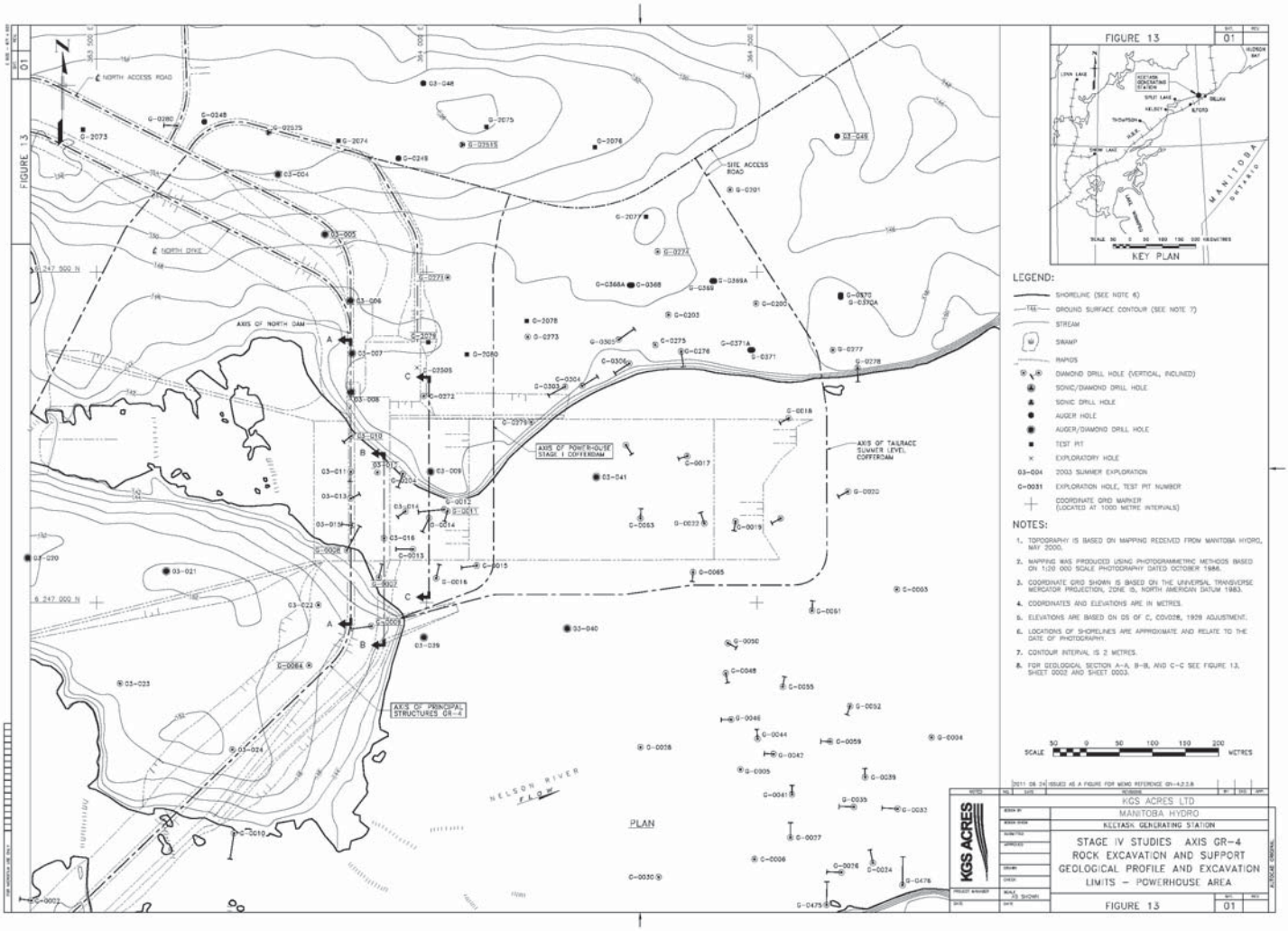
FIGURE 13
01
KEY PLAN
LEGEND:
SHORELINE (SEE NOTE 6)
GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
DIAMOND DRILL HOLE (VERTICAL, INCLINED)
SONIC/DIAMOND DRILL HOLE
SONIC DRILL HOLE
AUGER HOLE
AUGER/DIAMOND DRILL HOLE
TEST PIT
EXPLORATORY HOLE
03-004 2003 SUMMER EXPLORATION
G-0031 EXPLORATION HOLE, TEST PIT NUMBER
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO, MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON GS OF C, CGVD28, 1929 ADJUSTMENT.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. FOR GEOLOGICAL SECTION A-A, B-B, AND C-C SEE FIGURE 13, SHEET 0002 AND SHEET 0003.
SCALE 50 0 50 100 150 200 METRES
2011 08 24 ISSUED AS A FIGURE FOR MEMO REFERENCE GH-4.2.2.8
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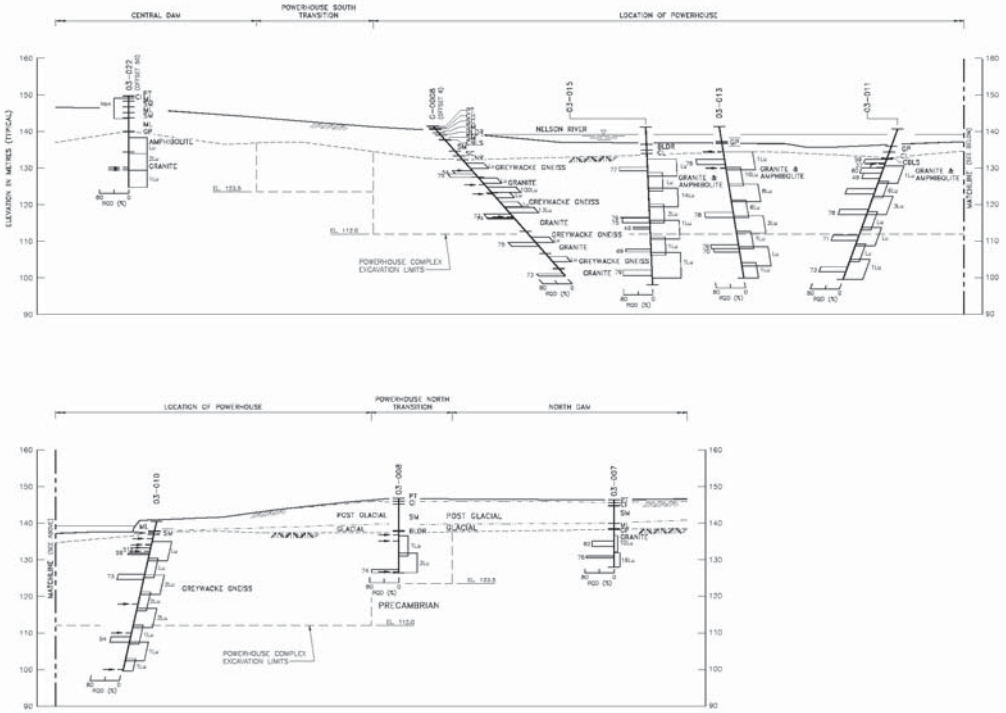
KELTASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION
LIMITS – POWERHOUSE AREA
FIGURE 13
01
PLAN
AXIS OF NORTH DAM
AXIS OF POWER-GLSE STAGE 1 COFFERDAM
AXIS OF TAILRACE SUMMER LEVEL COFFERDAM
AXIS OF PRINCIPAL STRUCTURES GR-4
NELSON RIVER FLOW
SITE ACCESS ROAD
NORTH ACCESS ROAD
NORTH DIKE
6 247 000 N



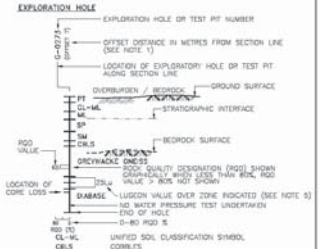
SECTION A-A

NOTES:

1. OFFSET DISTANCE SHOWN IF GREATER THAN 5 METRES FROM SECTION LINE.
2. FOR LOCATION OF GEOLOGICAL SECTION A-A SEE FIGURE 13, SHEET 0001.
3. WHENEVER POSSIBLE THE GROUND SURFACE PROFILES WERE ADJUSTED TO CONFORM TO THE INFORMATION FROM THE DRILL HOLES OR TEST PITS WHICH ARE WITHIN 5 METRES FROM THE SECTION LINES AND THEREFORE THEY MAY DIFFER FROM THAT SHOWN BY THE CONTOURS ON FIGURE 13, SHEET 0001.
4. ELEVATIONS ARE BASED ON DS OF C, CGVD, 1928 ADJUSTMENT.
5. Lu REPRESENTS ZERO LUGEON.

LEGEND: FIGURE 13 02

EXPLORATION HOLE



SOIL CLASSIFICATION SYMBOLS:
BASED UPON THE UNIFIED SOILS CLASSIFICATION SYSTEM

PT	PEAT, MUSKEG, HIGHLY ORGANIC SOILS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
CI	INORGANIC CLAYS OF INTERMEDIATE PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS
OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY
SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
SP	POORLY GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
SW	WELL GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES; LITTLE OR NO FINES
CL-ML, GP-GC, SP-SM, ETC.	TYPICAL BORDER LINE CLASSIFICATION REQUIRING THE USE OF DUAL SYMBOLS

FROZEN GROUND SYMBOLS:

Frozen	SEASONAL FROST
Nf	POORLY BONDED OR FRIABLE ICE
Nbn	WELL BONDED, NO EXCESS ICE
Nbe	WELL BONDED, EXCESS ICE
Vx	INDIVIDUAL ICE CRYSTALS OR INCLUSIONS
Vc	ICE COATINGS ON PARTICLES
Vr	RANDOM OR IRREGULARLY ORIENTED ICE FORMATIONS
Vs	STRATIFIED OR DISTINCTLY ORIENTED ICE FORMATIONS
Ice + UNIFIED SOILS CLASSIFICATION	ICE WITH SOIL INCLUSIONS
Ice	ICE WITHOUT SOIL INCLUSIONS

SCALE 10 0 10 20 30 40 METRES

2011 08 24 ISSUED AS A FIGURE FOR WORD REFERENCE SM-4.2.2.8

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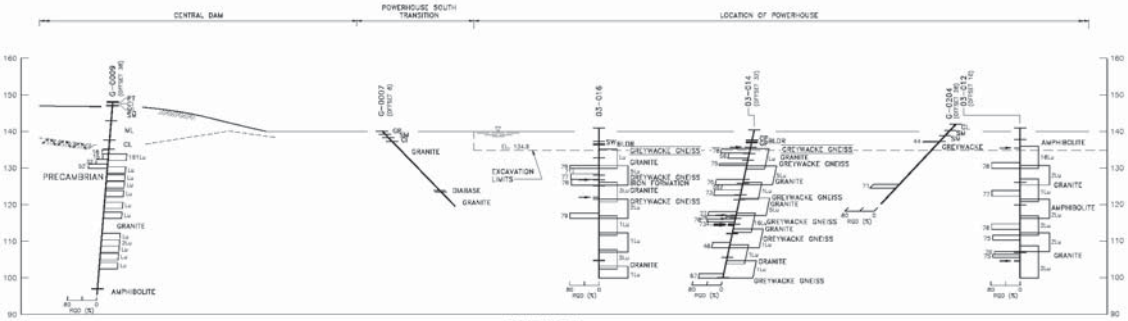
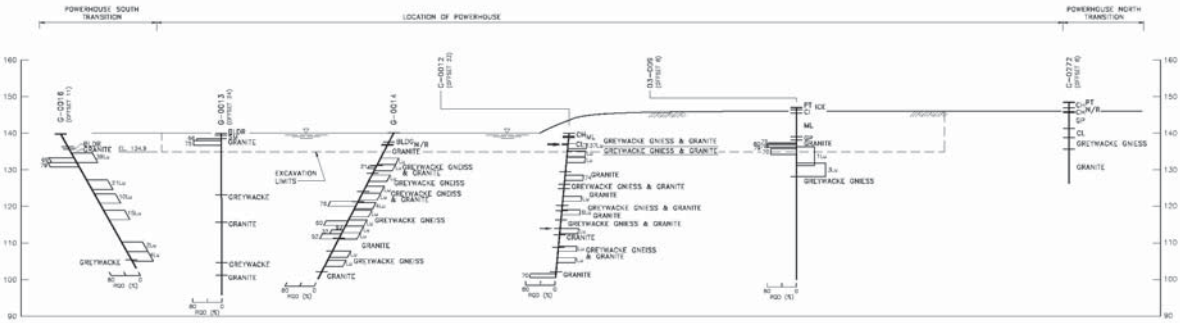
MANITOBA HYDRO
KEETASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION
LIMITS - POWERHOUSE AREA
FIGURE 13 02

FIGURE 13 | 03



SECTION B-B



SECTION C-C

SCALE 10 0 10 20 30 40 METRES

NOTES:

1. FOR LEGEND AND NOTES SEE FIGURE 13, SHEET 0002.
2. FOR LOCATION OF GEOLOGICAL SECTIONS B-B AND C-C SEE FIGURE 13, SHEET 0001.

2011 08 24 ISSUED AS A FIGURE FOR MDMG REFERENCE SM-4.2.2.8

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KEETASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION LIMITS - POWERHOUSE AREA

FIGURE 13 | 03

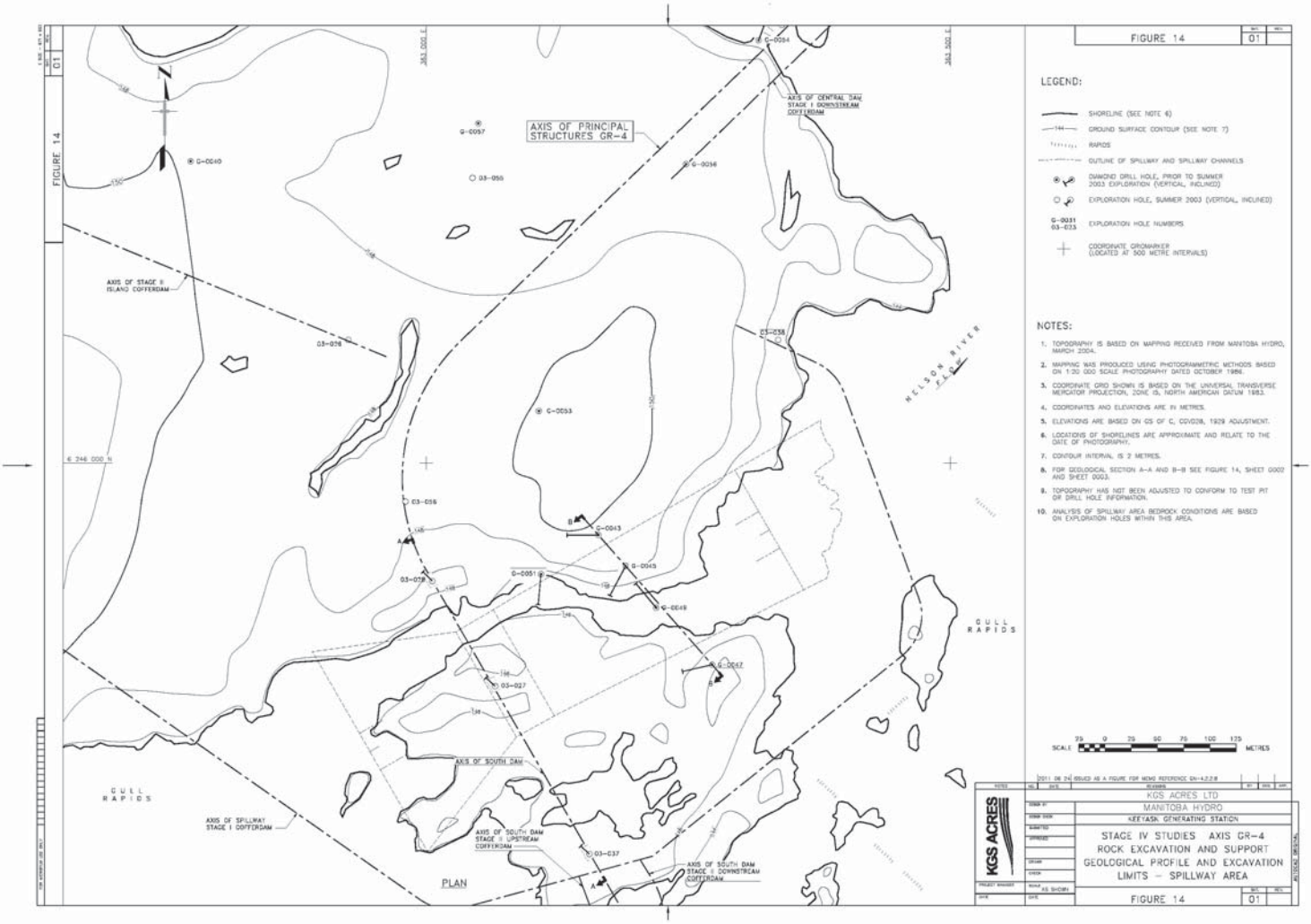
FIGURE 14
01
LEGEND:
SHORELINE (SEE NOTE 6)
144 GROUND SURFACE CONTOUR (SEE NOTE 7)
RAPIDS
OUTLINE OF SPILLWAY AND SPILLWAY CHANNELS
DIAMOND DRILL HOLE, PRIOR TO SUMMER 2003 EXPLORATION (VERTICAL, INCLINED)
EXPLORATION HOLE, SUMMER 2003 (VERTICAL, INCLINED)
G-0031 03-033 EXPLORATION HOLE NUMBERS
COORDINATE GROMARKER (LOCATED AT 500 METRE INTERVALS)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO, MARCH 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON GS OF C, CGVD28, 1929 ADJUSTMENT.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. FOR GEOLOGICAL SECTION A–A AND B–B SEE FIGURE 14, SHEET 0002 AND SHEET 0003.
9. TOPOGRAPHY HAS NOT BEEN ADJUSTED TO CONFORM TO TEST PIT OR DRILL HOLE INFORMATION.
10. ANALYSIS OF SPILLWAY AREA BEDROCK CONDITIONS ARE BASED ON EXPLORATION HOLES WITHIN THIS AREA.
SCALE 25 0 25 50 75 100 125 METRES
2001 08 24 ISSUED AS A FIGURE FOR MDMS REFERENCE SN-4.2.2.8
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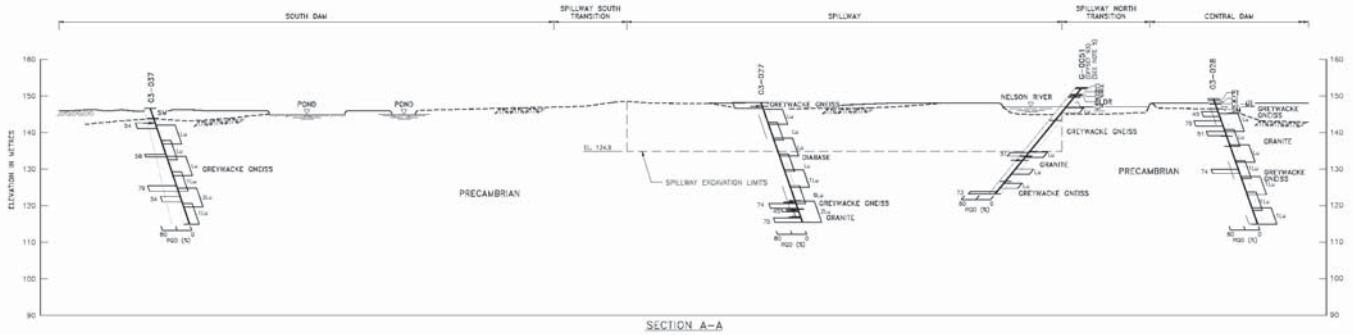
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION
LIMITS – SPILLWAY AREA
FIGURE 14
01
AXIS OF PRINCIPAL STRUCTURES GR-4
AXIS OF CENTRAL DAM STAGE I DOWNSTREAM COFFERDAM
AXIS OF STAGE II ISLAND COFFERDAM
AXIS OF SPILLWAY STAGE I COFFERDAM
AXIS OF SOUTH DAM
AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM
NELSON RIVER
GULL RAPIDS
GULL RAPIDS
PLAN
E 246 000 N
263 000 E
263 500 E
G-0054
G-0057
G-0040
G-0056
03-055
03-058
03-056
G-0053
03-056
G-0043
G-0045
G-0051
03-028
G-0049
G-0047
03-027
03-037

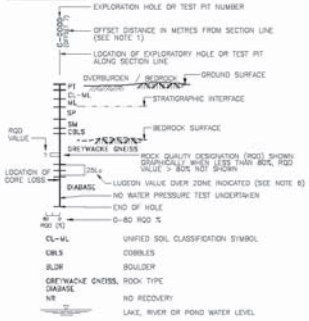
FIGURE 14 02



SECTION A–A

LEGEND:

EXPLORATION HOLE



SOIL CLASSIFICATION SYMBOLS
BASED UPON THE UNIFIED SOILS CLASSIFICATION SYSTEM

PT	PEAT, MUSKEG, HIGHLY ORGANIC SOILS
CH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
MH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
CI	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
OL	INORGANIC CLAYS OF INTERMEDIATE PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS
CL	ORGANIC SILTS AND ORGANIC SILT–CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
SC	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY
SM	CLAYEY SANDS, POORLY GRADED SAND–CLAY MIXTURES
SP	SILTY SANDS, POORLY GRADED SAND–SILT MIXTURES
SW	POORLY GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
GC	WELL GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
GM	CLAYEY GRAVELS, POORLY GRADED GRAVEL–SAND–CLAY MIXTURES
GP	SILTY GRAVELS, POORLY GRADED GRAVEL–SAND–SILT MIXTURES
GW	POORLY GRADED GRAVELS, GRAVEL–SAND MIXTURES, LITTLE OR NO FINES
	WELL GRADED GRAVELS, GRAVEL–SAND MIXTURES; LITTLE OR NO FINES
GL–ML, SP–SC, SP–SM, ETC.	TYPICAL BORDER LINE CLASSIFICATION REQUIRING THE USE OF DUAL SYMBOLS

NOTES:

1. OFFSET DISTANCE SHOWN IF GREATER THAN 5 METRES FROM SECTION LINE.
2. FOR LOCATION OF GEOLOGICAL SECTION A–A SEE FIGURE 14, SHEET 0001.
3. WHENEVER POSSIBLE THE GROUND SURFACE PROFILES WERE ADJUSTED TO CONFORM TO THE INFORMATION FROM THE DRILL HOLES OR TEST PITS WHICH ARE WITHIN 5 METRES FROM THE SECTION LINES AND THEREFORE THEY MAY DIFFER FROM THAT SHOWN BY THE CONTOURS ON FIGURE 14, SHEET 0001.
4. ELEVATIONS ARE BASED ON GS OF C, COVD, 1929 ADJUSTMENT.
5. THE OVERBURDEN IN THE SPILLWAY AREA WAS REMOVED DURING ICE STAGING IN 2005. THEREFORE, BOREHOLES DRILLED PRIOR TO 2005 IN THIS AREA MAY SHOW OVERBURDEN OVERLYING BEDROCK, WHICH HAS SUBSEQUENTLY BEEN REMOVED.
6. Lu REPRESENTS ZERO LUGEONS.

SCALE 10 0 10 20 30 40 METRES

NO.	DATE	REVISION	BY	CHK	APP
		2011 08 24 ISSUED AS A FIGURE FOR MDNR REFERENCE SN-4.2.2.8			

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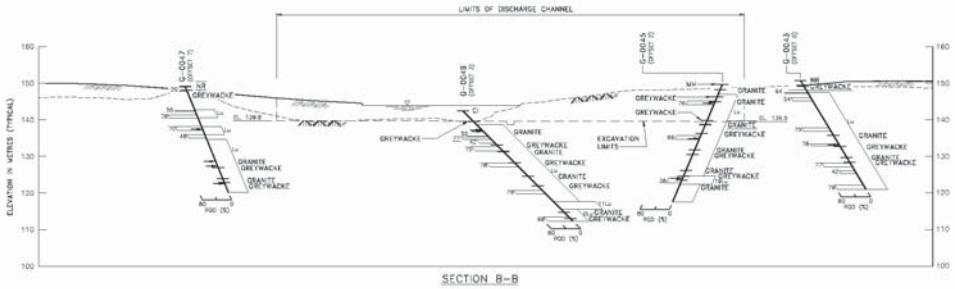
KEETASK GENERATING STATION

STAGE IV STUDIES AXIS GR–4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION
LIMITS – SPILLWAY AREA

FIGURE 14 02

FIGURE 14 | 03



SECTION B–B

NOTES:

1. FOR LEGEND AND NOTES SEE FIGURE 14, SHEET 0002.
2. FOR LOCATION OF GEOLOGICAL SECTION B–B SEE FIGURE 14, SHEET 0001.

SCALE 10 0 10 20 30 40 METRES

2001 06 04 ISSUED AS A FIGURE FOR MEMO REFERENCE SN-4.3.2.8

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STAGE IV STUDIES AXIS GR-4
ROCK EXCAVATION AND SUPPORT
GEOLOGICAL PROFILE AND EXCAVATION
LIMITS – SPILLWAY AREA

FIGURE 14 | 03

AS SHOWN

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PLANNING DEPARTMENT

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.9
Rev. 0, July 29, 2011

Geotechnical Instrumentation Design
Manitoba Hydro File 00195-11621-0002_01

Prepared by _____ (P. R. DANIEL) July 29, 2011
KGS Acres Ltd.

Checked by _____ (R. A. HALIM) JULY 29, 2011
KGS Acres Ltd.

Reviewed by _____ (I. R. Dewar) July 29, 2011
KGS Acres Ltd.

Approved by _____ (N. J. Smith) July 29/11
KGS Acres Ltd.

Accepted by _____ (M. ST. LAURENT) MARCH 2, 2012
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	July 29, 2011
		File No.	P10008(16).19.10
From	P.R. Pantel	**cc**	
Subject	**Keeyask Generating Station** **Stage IV Studies, Axis GR-4** **Geotechnical Instrumentation Design** **Memorandum GN-4.2.2.9, Rev 0** **Manitoba Hydro File 00195-11621-0002_01**		

This design memorandum is being issued Rev 0 with a table summarizing "Quality Review Comments and Responses" inserted following the document sign-off sheet. The table was compiled during the Quality Review Process associated with the finalization of this document and may include follow-up actions to be undertaken in the final design phase of the project, or alternatively, points of clarification pertaining directly to the content of the memorandum. The Quality Review Comments and Responses are considered to be an integral part of the memorandum, and should be read in conjunction with the memorandum.

1 Introduction

The site of the proposed Keeyask Generating Station (GS) is approximately 730 km north of Winnipeg at the head of Stephens Lake on the Nelson River. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is the Gull Rapids site. The proposed axis for the structures crosses the Nelson River at the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam.

This memorandum summarizes the Keeyask GS instrumentation design and proposed instrumentation layout locations, types and estimated quantities for the principal structures.

This memorandum also addresses the Stage IV instrumentation design requirements for which KGS ACRES has a lead role, as defined in a Manitoba Hydro interoffice memorandum [Ref 1]. These requirements include definition of the project conditions, identification of failure modes and parameters to be monitored, definition of specific instrumentation purpose, selection of conceptual instrument location and identification of factors which may influence measured data.

2 General Arrangement

The general arrangement of the proposed Keeyask GS is shown in Figure 1. The layout corresponds to that proposed for Axis GR-4 development, with an FSL at el 159.0 m. The selected arrangement consists of two long riverbank dykes, on the north and south banks, linked to the Principal Structures, which span the Nelson River at Gull Rapids. The Principal Structures consist of a Powerhouse Complex constructed on the north side of the river, which will be connected to the North Dyke through a 99 m long North Dam and a short concrete transition structure.

The Spillway will be located on an island, approximately 1.5 km south of the Powerhouse. A Central Dam will connect the Spillway and the Powerhouse. The Spillway will be connected to the South Dam by means of a short concrete transition structure. The South Dam, approximately 565 m long, will connect to the South Dyke on the south riverbank.

3 Regional Geology

The site for the Keeyask GS is located within the Canadian Shield and is underlain by variable thicknesses of up to 30 m of overburden over competent Precambrian bedrock. In general, the overburden stratigraphy consists of a thin organic cover overlying postglacial lacustrine clay, which in turn overlies deposits of glacial outwash, till or the bedrock directly. Preglacial deposits of sand and silty sand are also occasionally found in bedrock lows.

Two types of postglacial deposits have been identified:

(a) Alluvium

It generally consists of cobbles and boulders overlying sands and gravels and is locally present in the base of present-day stream and river channels.

(b) Lake Agassiz Silts and Clays

A relatively thin layers of clays and silts were deposited on the bottom of glacial Lake Agassiz. The silts and clays form a veneer of up to several metres in thickness over the glacial deposits. These fine-grained deposits are commonly varved and tend to be of greater thickness in the topographic lows. They contain discontinuous permafrost, ranging from non-visible to visible ice segregation (less than 25 mm thick). The postglacial deposit(s) may be absent entirely on ridges and knolls. Peat has formed in the poorly drained flatlands and hollows which remained in the landscape after Lake Agassiz drained to Hudson Bay.

The glacial deposits are widespread and are derived from the several glacial ice sheets which advanced over the Gull Rapids area and deposited till and stratified water lain deposits. The tills contain discontinuous occurrences of permafrost, are generally well graded, compact, have relatively low moisture content, and generally have low ice content when frozen.

Three separate till or till-like horizons have been identified at the Keeyask site. The upper silty sand/sandy silt till unit (Till 1), whose presence is the most widespread over the Keeyask area, generally consists of a light brown horizon (Till 1a) overlying a grey horizon (Till 1b) with essentially identical soil gradations.

Beneath the silty sand/sandy silt units of Till 1, Till 2 and Till 3 consist of grey, low plasticity clays. Not all of the area at the Keeyask GS site encountered these till units.

En-echelon drumlin ridges are aligned east-west at approximate azimuths from 90° to 110° in the Keeyask study area. In addition to the drumlins, there are also some subdued glacial features forming smooth knolls with gentle slopes oriented at an azimuth of 125°. These are made up of both fine grained and granular soils, commonly intermingled.

Superimposed on the drumlins and tills are glaciofluvial deposits. These consist of silts, sands and gravels, which were deposited as serpentine eskers. Where the water velocity reduced in the channels, glaciofluvial silt, sand and gravel deposits formed outwash and flat-topped kame outwash along the chain of discontinuous esker ridges.

The Gull Rapids site is located in an area of discontinuous permafrost. Ice rich peat plateaus are present throughout the area. Thermokarst collapse scars occur throughout the Gull Rapids area.

Discontinuous permafrost is present in the postglacial clays and silts below the peat and locally extends into the underlying tills. Permafrost affected bedrock also occurs locally. The higher and more densely wooded ridge crests and knoll tops are drier and usually contain little if any permafrost. The eskers are also generally free of permafrost.

(c) Precambrian Bedrock

The Keeyask site is located proximal to the Churchill-Superior boundary. The Churchill Superior boundary is a poorly defined zone of cataclastic rocks which resulted from the collision of two tectonic plates.

The Report for the 1991 Summer Subsurface Exploration Program [Ref 1] defined the bedrock conditions at the Keeyask GS Site. During 2003 and 2004, the Manitoba Geological Survey, in collaboration with Manitoba Hydro and the Universities of Alberta and Waterloo, conducted a bedrock mapping program with the objective of documenting the geology of the Keeyask site in great detail [Ref 3]. A result of the 2003-2004 investigation was the understanding that the Churchill-Superior boundary occurs to the east of the Keeyask site.

The Keeyask site is located at the margin of the Superior Province near the Churchill-Superior boundary. The rocks that occur at Keeyask are significantly different compared to the dominantly meta-igneous granulite rocks of the Split Lake Block of the Superior Province to the west and the Kisseynew-type metasedimentary rocks of the Churchill Province to the east.

In brief, the bedrock at the Keeyask site shows a varied degree of metamorphic and deformational texture. Keeyask is a host to a sequence of rocks consisting primarily of Archean supracrustal and intrusive rocks. The supracrustral rock types are identified as Archean amphibolite-grade rocks consisting of amphibolites (metabasalt), and Fe-rich metagreywacke, with interlayered banded oxide-, sulphide and silicate-facies iron formation. Immediately to the west of the Keeyask site is a sequence of granodiorite gneisses.

At the Keeyask site, the bedrock is predominantly greywacke gneiss, with amphibolite predominant in the Powerhouse area. The greywacke gneiss and amphibolites commonly contain bands or sills of granitic material. Diabase dykes crosscut all rock types in the areas, and vary from short narrow features to continuous wider dykes. The contacts between the diabase dykes and the host rocks are typically open near surface, but generally tighten with depth.

All rock types within the Gull Rapids area are typically fresh, strong to very strong, with medium to widely spaced joints. Most of the joints are tight with little or no alteration. Bedrock drilling at the locations of the North, Central and South Dams, Powerhouse, Spillway and Transition Structures has shown no evidence of significant geological weaknesses. The bedrock appears to be competent and suitable for the proposed Principal Structures at this site.

4 Instrumentation Design Considerations

Earthfill embankments, as well as the foundations for these embankments and for the concrete structures, require individual solutions to their instrumentation requirements. The instrumentation system design for the Keeyask GS, therefore, is based on the site specific geotechnical conditions present in the embankments, the structures' foundations and their abutments.

The general purpose of instrumentation is to provide data useful for determining whether or not an embankment or foundation is behaving in accordance with engineering assumptions.

Therefore, instrumentation is required to perform one or more of the following tasks:

- verify the design assumptions and construction techniques
- monitor performance during construction
- monitor long-term post-construction performance of the structures for dam safety assessment
- assist in the analysis of adverse events.

The instrument design for the Keeyask GS was, in general, based on a systematic approach as defined in "Geotechnical Instrumentation for Monitoring Field Performance" [Ref. 2]. The procedure is defined by the following steps:

1) Define the project conditions including subsurface stratigraphy, groundwater conditions, environmental conditions, and planned construction methods.
2) Predict the mechanisms that are likely to control the structures' behavior, based on conditions identified in Step 1. Possible mechanisms may include slope stability, bearing failure, piping, differential settlement, frost penetration, thawing of permafrost in the foundations, etc.
3) Define the conditions to be monitored based on design, construction, or operation phases of the project.
4) Define the purpose of the instrumentation program including benefits of instrumentation during construction and post construction.
5) Select the parameters to be monitored based on identification of which parameters are most significant. Parameters may include pore water pressure, deformation, temperature, etc.
6) Select instruments to be used to monitor the identified parameters in Step 5. Discussions within this memorandum focus on general instrument types, as Manitoba Hydro has indicated that they will undertake the lead role in selection of specific instrument makes and models. Their selection will be based on considerations such as reliability of specific instruments, environment, simplicity of reading, impacts to long-term performance such as seepage pathways, etc.
7) Select conceptual instrument locations at areas of particular concern or at sections where the predicted behaviour is expected to represent the area as a whole.
8) Identify factors which may influence measured data such as construction, geological conditions and environmental factors.
9) List the specific purpose of each instrument to justify their use and installation.

The locations for instruments are determined based on anticipated behavior of the site as well as the type of the instrument. The approach to selecting the instrument locations includes the following:

- identification of critical zones
- identification of typical cross sections where predicted behavior is considered representative of the behavior as a whole.

Typically, instrumented cross sections are located near the maximum height of the embankment. Additional sections are then set at other locations of interest. Preliminary layouts of the instrumentation are discussed in Section 5.

The recommended locations for the slope inclinometers are within the downstream slope where they will intercept the critical slip surfaces calculated during the design process, as well as other potential slip surfaces, as shown in Figure 3. Their location within the filter beneath the downstream slope (instead of within the impervious core) was selected to prevent possible seepage into the core material along the casing and due to difficulty in achieving proper compaction of the impervious core material around the casing.

The crests of the North, Central and South Dams, as well as a small portion of the North Dyke, will eventually form part of a provincial road to Gillam. Therefore, the crest of the dams must not be impeded by instruments which will protrude from the road surface (i.e. slope inclinometers or standpipes). The locations of instruments and the selection of specific instrument types below the crest of these structures, such as temperature monitoring instruments, must also consider traffic loading conditions which may damage the instruments.

The intended purpose for most of the instrumentation is long-term monitoring for dam safety of the structures. However, short-term monitoring during and immediately following impoundment of the reservoir will be valuable in identifying any problems during this critical period. Partial impoundment of the reservoir will commence in October of Year Y, with the reservoir fully impounded by December 1 of Year Y [Ref 3]. Therefore, short-term monitoring is recommended to begin by October of Year Y, prior to the structures being initially subjected to reservoir loading.

5 Instrumentation Design of Keeyask GS

The structures at Keeyask GS consist of Concrete Structures (Spillway and Powerhouse), and earthfill structures (North Dam, Central Dam, South Dam, North Dyke and South Dyke).

The physical data for these structures are summarized in Table 1. This data was obtained from the various design memoranda prepared for the proposed Keeyask GS.

5.1 North Dam

The design of the North Dam is described in Memorandum GN-4.2.2.3 “Design of North Dam” [Ref. 4]. The maximum height of the North Dam is about 24.5 m above the dam’s foundation. The impervious core of the dam will be entirely founded on bedrock, except at a relatively short transition section to the North Dyke that will be founded on till.

The selection of the type and location of instrumentation for the North Dam has been made based on the procedure described in Section 4. The instrumentation design to address each of the potential mechanisms that may control the behavior of the structure is described below. The location of all the instrumentation for the North Dam is shown in Figure 2.

5.1.1 Grout Curtain Effectiveness

The effectiveness of the grout curtain beneath the North Dam is important for the reduction of foundation seepage by sealing the upper portion of the bedrock. The grout curtain will also reduce exit gradients where the North Dam fill material is placed on till or bedrock by extending the seepage path of flow beneath the structure.

To verify the short and long term effectiveness of the grout curtain, monitoring of the pore water pressures beneath the North Dam is recommended. This can be achieved with the installation of piezometers. The piezometers are recommended to be installed in two sections of the North Dam. The first section is located near the Powerhouse structure where the dam is at its maximum height. The second section is located on the north abutment where the dam will be founded on till.

The recommended locations of the piezometers within each instrumented section are as follows: one instrument at a shallow depth in the bedrock upstream of the grout curtain and two instruments at shallow depths in the bedrock downstream of the grout curtain. The pore water pressures measured at each instrument will allow for any changes in pressure across the grout curtain to be calculated and this can be compared to the design values to determine the effectiveness of the curtain.

The monitoring data obtained from the piezometers will be influenced by the grout curtain efficiency at the specific monitoring section. It is recognized that the quality of the grout curtain construction and local bedrock conditions may result in varying effectiveness of the grout curtain over the length of the structure. It may therefore be prudent to allow for flexibility and relocation of the identified locations once foundation conditions become known.

5.1.2 Pore Water Pressures in Till Foundation

To verify that the pore water pressures within the till foundation are as anticipated, monitoring of pore water pressures in this material is recommended. Piezometers are recommended at one section of the North Dam at the north abutment within the till foundation, with one piezometer located upstream of the dam axis and a second downstream of the axis. The recommended instrumentation will determine if the pore pressures beneath the dam are within expected ranges following reservoir impoundment and during long term conditions over the life of the structure.

The monitoring data obtained from the piezometers will be influenced by the permeability of the till.

The installation of an additional piezometer (P10) downstream of the North Dam at the north abutment, as shown in Figure 2, is also recommended to identify any potential ground water flow changes in this area.

5.1.3 Pore Water Pressures Downstream of Impervious Core

The effectiveness of the downstream filter zone can decline during the life of a dam due to clogging with fine grained material which can increase pore water pressure upstream of the filter. Instrumentation is recommended to identify if the downstream filter zone is working effectively during long term conditions.

To verify the effectiveness of the downstream filter it is recommended that a piezometer be installed to monitor pore water pressures. The location of the piezometer (P11) will be within the horizontal portion of Class 3 filter immediately downstream of the impervious core at the section of the dam nearest to the Powerhouse.

5.1.4 Construction Induced Pore Pressures and Long-Term Piezometer Level in Impervious Core

During construction, short term undrained conditions may result in elevated pore water pressures within the impervious core, typically reaching a maximum value by the time fill placement is complete. The recommended instrumentation will determine if the pore water pressures during construction are a concern with respect to the stability of the structure.

To assess the stability of the North Dam during construction, it will be prudent to monitor pore water pressures. This will be achieved by the installation of piezometers. The piezometers can also be used for long term monitoring of the piezometric level within the impervious core during steady-state conditions.

Two sections are recommended for the installation of piezometers, with the first at the highest sections near the Service Bay and the second near the tie-in with the North Dyke on the north abutment. The recommended locations for the piezometers within each section are in the lower portion of the impervious core, with one piezometer installed upstream and one installed downstream of the dam axis as shown in Figure 2.

It is anticipated that pore water pressures generated within the impervious core during construction will vary based on the insitu moisture content of the fill materials at the time of placement and will be influenced by precipitation and the control of moisture during construction (i.e. wetting and drying of fill materials for compaction). The long-term piezometric level in the impervious core is expected to be similar to that calculated from two-dimensional seepage analysis performed for the structure.

5.1.5 Horizontal or Vertical Displacement of the North Dam

Horizontal or vertical displacement of the North Dam is expected to be minimal. The core of the North Dam has been designed to rest on a bedrock foundation, with a short portion founded on till at the north abutment. This design approach is anticipated to result in minimal foundation settlement, however, some internal settlement of fill material is expected. Most of the internal settlement is anticipated to take place during or shortly after construction and reservoir filling. It is recommended that survey monuments be installed on the North Dam to confirm that settlements are within anticipated levels.

The survey monuments would be used for monitoring both short term and long term displacement for dam safety purposes. The monuments are recommended to extend approximately 1.4 m below the crest of the dam. The installation of benchmarks to be used as reference points for the other instrumentation will be required and should extend to a minimum depth of 7.6 m below the ground or fill surface to ensure they will be below the frost line and in till or bedrock.

The survey monuments are recommended to be located at the upstream and downstream crests of the North Dam as well as at mid-height of the downstream slope at the instrumented section near the Powerhouse. An additional monument is recommended at the downstream crest of the instrumented section at the north abutment. It is recommended that one benchmark be installed downstream of the dam as shown in Figure 2.

5.1.6 Frost Penetration

Frost penetration into the upper portion of the North Dam core could result in damage to the core material and allow increased seepage of water.

To monitor the depth of frost penetration, the installation of a thermistor string is recommended. The thermistor string will be located in the impervious core. The thermistor string can also be used in the long term to assist in identifying the seepage regime within the core of the dam.

A thermistor string is recommended to be installed from the top of the core to an approximate depth of 20 m below the crest at the section of the dam near the Powerhouse. The thermistor is recommended to be located between the two instrumented sections discussed in Section 5.1.1. The air temperature at the site will have an effect on the readings of the thermistor strings and should also be monitored to develop relationships between air temperature and frost penetration depth.

5.1.7 Seepage Through the North Dam

Seepage within the foundation and core of the North Dam is expected to be low. To monitor seepage through and beneath the dam, the installation of a V-notch weir is recommended with a piezometer for automated periodic readings. The weir will be located downstream of the dam and any seepage discharge will be conveyed into the tailrace channel. For dam safety considerations, weir operation and data collection is expected to be performed throughout the life of the structure.

It is noted that measured data from the weir will be influenced by seasonal precipitation and snowmelt and hence these input values must be assessed and taken into account during the data analysis. While the weir will not capture all of the seepage through the dam and the foundation, the readings may identify any sudden increase or drop in the seepage flow which would warrant further investigation.

5.2 Central Dam

The design of the Central Dam is described in Memorandum GN-4.2.2.13, "Design of Central Dam" [Ref. 6]. The Central Dam is an approximately 1.6 km long earthfill structure which will span the central channel between the Spillway and Powerhouse structures. The maximum height of the structure will be approximately 27.5 m above the dam's foundation and the core of the dam will be entirely founded on bedrock.

The selection of the type and location of instrumentation for the Central Dam has been made based on potential concerns identified during the design process of the structure. The design of the instrumentation addresses each of the potential mechanisms that may control the behavior of the structure is described below. The location of all the instrumentation for the Central Dam is shown in Figure 3.

5.2.1 Grout Curtain Effectiveness

The effectiveness of the grout curtain beneath the Central Dam is important for the reduction of foundation seepage by sealing the upper portion of the bedrock. The grout curtain will also reduce exit gradients where the Central Dam fill material is placed on the bedrock by extending the seepage flow path beneath the structure.

To verify the short and long term effectiveness of the grout curtain, monitoring of the pore water pressures beneath the Central Dam is recommended. The piezometers are recommended to be installed in three sections along the dam. The first section is located near the Powerhouse structure at the highest section of the dam; the second section is

located within the river portion of the dam to represent a primary instrumented section where the performance is considered to be representative of the river section of the dam; and the third section is located near the Spillway structure which represents a primary instrumented section which is representative of the southern portion of the dam.

The recommended locations of the piezometers within each instrumented section are as follows: two instruments at shallow depths in the bedrock upstream of the grout curtain and two instruments at shallow depths in the bedrock downstream of the grout curtain. The pore water pressures measured at each instrument will allow for any changes in pressure across the grout curtain to be calculated and this can be compared to the design values to determine the effectiveness of the curtain.

The monitoring data obtained from the piezometers will be influenced by the grout curtain efficiency at the specific monitoring section. It is recognized that the quality of the grout curtain construction and local bedrock conditions may result in varying effectiveness of the grout curtain over the length of the structure. Therefore, it may be prudent to allow for flexibility and relocation of the identified locations once foundation conditions become known.

5.2.2 Construction Induced Pore Pressures and Long-Term Piezometric Level in Impervious Core

During construction, short term undrained conditions may result in elevated pore water pressures within the impervious core, typically reaching a maximum value by the time fill placement is complete. The recommended instrumentation will determine if the increased pore water pressures during construction are a concern with respect to the stability of the structure.

To assess the stability of the Central Dam during construction, it will be prudent to monitor pore water pressures. This will be achieved by the installation of piezometers. The piezometers can also be used for long term monitoring of the piezometric level within the impervious core during steady-state conditions.

The sections recommended for the installation of the piezometers are at the three sections identified above in Section 5.2.1. The recommended locations for the piezometers within each section are in the lower portion of the impervious core, with one piezometer installed upstream and one installed downstream of the dam axis as shown in Figure 3.

It is anticipated that pore water pressures generated within the impervious core during construction will vary based on the insitu moisture content of the fill materials at the time of placement and will be influenced by precipitation and the control of moisture during construction (i.e. wetting and drying of fill materials for compaction). The long-term piezometric level in the impervious core is expected to be similar to that calculated from two-dimensional seepage analyses performed for the structures.

5.2.3 Pore Water Pressures Downstream of Impervious Core

The effectiveness of the downstream filter zone can decline during the life of a dam due to clogging with fine grained material which can increase pore water pressure upstream of the filter. Instrumentation is recommended to determine if the downstream filter zone is working effectively during long term conditions.

To verify the effectiveness of the downstream filter it is recommended that two piezometers will be installed to monitor pore water pressures. The location of the piezometers will be within the horizontal portion of Class 3 filter immediately downstream of the impervious core at each of the sections of the dam nearest to the Powerhouse and Spillway structures.

5.2.4 Lateral Deformation of Downstream Slope

The design of the Central Dam was developed to provide upstream and downstream

slopes which met or exceeded minimum factor of safety requirements as defined by the Canadian Dam Association's Dam Safety Guidelines [Ref. 5]. To monitor any potential slope movement, the installation of a slope inclinometer is recommended within the downstream slope of the instrumented section located near the Powerhouse. The slope inclinometer is recommended to be installed within the slope and be embedded into the bedrock foundation.

The instrument is intended for long term dam safety monitoring following construction of the dam.

5.2.5 Horizontal or Vertical Displacement of the Central Dam

Horizontal or vertical displacement of the Central Dam is expected to be minimal. The core of the Central Dam has been designed to rest on a bedrock foundation. This design approach is anticipated to result in minimal foundation settlement, however, some internal settlement of fill material is expected. Most of the internal settlement is anticipated to take place during or shortly after construction and reservoir filling. It is recommended that

survey monuments be installed on the Central Dam to confirm that settlements are within anticipated levels.

The survey monuments would be used for monitoring both short term and long term displacement for dam safety purposes. The monuments are recommended to extend approximately 1.4 m below the crest of the dam. The installation of benchmarks to be used as reference points for other instrumentation will be required and should extend to a minimum depth of 7.6 m below the ground or fill surface to ensure they will be below the frost line and tie in to till or bedrock.

The survey monuments are recommended at 100 m spacing which alternate between the upstream and downstream crests. Survey monuments are recommended at two additional sections within the river portion of the dam with monuments installed on the upstream and downstream crest as well as two monuments on the downstream slope. One benchmark will be installed near the Spillway and one benchmark will be installed near the Powerhouse.

5.2.6 Frost Penetration

Frost penetration into the upper portion of the Central Dam core could result in damage to the core material and allow increased seepage of water.

To monitor the depth of frost penetration, the installation of thermistor strings is recommended. The instruments will provide long term monitoring for dam safety. The thermistor string can also be used in the long term to assist in identifying the seepage regime within the core of the dam.

A total of two thermistor strings are recommended to be installed from the top of the core to an approximate depth of 20 m below the crest to provide redundancy, with the first located near the Powerhouse and the second at the river section of the dam. The air temperature at the site will have an effect on the readings of the thermistor strings and should also be monitored to develop relationships between air temperature and frost penetration depth.

5.2.7 Seepage Through the Central Dam

Seepage within the foundation and core of the Central Dam is expected to be low. To monitor seepage through and beneath the dam, the installation of two V-notch weirs is recommended with piezometers for automated periodic readings. The weirs will be located within the ditch at the downstream toe of the dam and any seepage discharge will be conveyed into the tailrace channel. The first weir will be located at the north island to monitor seepage near the Powerhouse and the second weir will be located at the south island to monitor seepage near the Spillway. For dam safety considerations, weir

operation and data collection is expected to be performed throughout the life of the structure.

It is noted that measured data from the weirs will be influenced by seasonal precipitation and snowmelt and hence these input values must be assessed and taken into account during the data analysis. While the weir will not capture all of the seepage through the dam and to foundation, the readings may identify any sudden increase or drop in the seepage flow which would warrant further investigation.

5.3 South Dam

The design of the South Dam is described in Memorandum GN-4.2.2.4 "Design of South Dam" [Ref. 7]. The South Dam will be a zoned earthfill embankment with a maximum height of about 21.5 m above the riverbed. The impervious core of the dam will be entirely founded on bedrock, except at the south abutment section which transitions to the South Dyke, which will be founded on till.

The selection of the type and location of instrumentation for the South Dam has been made based on the procedure described in Section 4. The instrumentation design to address each of the potential mechanisms that may control the behavior of the structure is described below. The location of all the instrumentation proposed for the South Dam is shown in Figure 4.

5.3.1 Grout Curtain Effectiveness

The effectiveness of the grout curtain beneath the South Dam is important for the reduction of foundation seepage by sealing the upper portion of the bedrock. The grout curtain will also reduce exit gradients where the South Dam fill material is placed on till or bedrock by extending the seepage path of flow beneath the structure.

To verify the short and long term effectiveness of the grout curtain, monitoring of the pore water pressures beneath the South Dam is recommended. The piezometers are recommended to be installed in two sections of the dam. The first section is located at the north end of the South Dam. The second section is located in the river channel area at the highest section of the dam.

The recommended locations of the piezometers within each instrumented section are as follows: two instruments at shallow depth in the bedrock upstream of the grout curtain and two instruments at shallow depth in the bedrock downstream of the grout curtain. The pore water pressures measured at each instrument will allow for any changes in piezometric level across the grout curtain to be calculated and this can be compared to the design values to determine the effectiveness of the curtain. The installation of two piezometers within the

bedrock foundation at the South Abutment (P47 and P48) are also recommended, with both piezometers to be located downstream of the grout curtain.

The monitoring data obtained from the piezometers will be influenced by the grout curtain efficiency at the specific monitoring section. It is recognized that the quality of the grout curtain construction and local bedrock conditions may result in varying effectiveness of the grout curtain over the length of the structure. Therefore, it may be prudent to allow for flexibility and relocation of the identified locations once foundation conditions become known.

5.3.2 Pore Water Pressures in Till Foundation

To verify that the pore water pressures within the till foundation are as anticipated, monitoring of pore water pressures in this material is recommended. Two piezometers are recommended at one section of the South Dam at the south transition within the till foundation, with both piezometers installed downstream of the axis. The recommended instrumentation will determine if the pore pressures within the dam's abutment are within expected ranges following reservoir impoundment and during long term conditions over the life of the structure.

The monitoring data obtained from the piezometers will be influenced by the permeability of the till.

The installation of an additional piezometer (P111) downstream of the South Dam at the south abutment is recommended to identify any potential ground water flow changes in this area.

5.3.3 Pore Water Pressures Downstream of Impervious Core

The effectiveness of the downstream filter zone can decline during the life of a dam due to clogging with fine grained material which can result in increased pore water pressures upstream of the filter. Instrumentation is recommended to identify if the downstream filter zone is working effectively during long term steady-state conditions.

To verify the effectiveness of the downstream filter it is recommended that piezometers be installed to monitor pore water pressures. The location of the piezometer will be within the base of the Class 3 filter immediately downstream of the impervious core at the north end section of the dam. One piezometer is also recommended at each of the south abutment and south transition sections within the downstream Class 5 filter.

5.3.4 Construction Induced Pore Pressures and Long-Term Piezometric Level in Impervious Core

During construction, short term undrained conditions may result in elevated pore water pressures within the impervious core, typically reaching a maximum value by the time fill placement is complete. The recommended instrumentation will identify if the pore water pressures during construction are a concern with respect to stability of the structure.

To assess the stability of the South Dam during construction, it will be prudent to monitor pore water pressures. This will be achieved by the installation of piezometers. The piezometers can also be used for long term monitoring of the piezometric level within the impervious core during steady-state conditions.

The instrumented sections of the piezometers for the impervious fill are at the two sections identified above in Section 5.3.1. The recommended locations of the piezometers within each section are in the lower portion of the impervious core, with one piezometer installed upstream and one installed downstream of the dam axis. It is also recommended that piezometers be installed within the impervious core at the tie-in area of the South Dam to the Spillway Stage I Cofferdam, with one piezometer installed upstream and two installed downstream of the dam axis.

It is anticipated that pore water pressures generated within the impervious core during construction will vary based on the insitu moisture content of the fill materials at the time of placement and will be influenced by precipitation and the control of moisture during construction (i.e. wetting and drying of fill materials for compaction). The long-term piezometric level in the impervious core is expected to be similar to that calculated from two-dimensional seepage analyses performed for the structure.

5.3.5 Lateral Deformation of Downstream Slope

The design of the South Dam was developed to provide upstream and downstream slopes which meet or exceed minimum factor of safety requirements as defined by the Canadian Dam Association's Dam Safety Guidelines [Ref. 5]. To monitor any slope movement, the installation of a slope inclinometer is recommended at the instrumented section located at the north end of the structure. The slope inclinometer is recommended to be embedded into the bedrock foundation.

The instrumentation is intended for long term dam safety monitoring following construction of the dam.

5.3.6 Horizontal or Vertical Displacement of the South Dam

Horizontal or vertical displacement of the South Dam is expected to be minimal. The core of the South Dam has been designed to rest on a till or bedrock foundation. This design approach is anticipated to result in minimal foundation settlement, however, some internal settlement of fill material is expected. Most of the internal settlement is anticipated to take place during or shortly after construction and reservoir filling. It is recommended survey monuments be installed on the South Dam to confirm that settlements are within anticipated levels.

The survey monuments would be used for monitoring both short term and long term displacement for dam safety purposes. The monuments are recommended to extend approximately 1.4 m below the crest of the dam. The installation of benchmarks to be used as reference points for the other instrumentation will be required and should extend to a minimum depth of 7.6 m below the ground or fill surface to ensure they will be below the frost line and tied into till or bedrock.

The survey monuments are recommended to be located at alternating upstream and downstream crest locations of the South Dam with a 100 m spacing. In addition, five monuments will also be installed at each of the Spillway Stage I Cofferdam tie-in and the river sections on the upstream slope, upstream crest, downstream crest, downstream slope and edge of the Downstream Stage II South Dam Cofferdam. It is recommended that one benchmark be installed at the south transition section of the dam.

5.3.7 Frost Penetration

Frost penetration into the upper portion of the South Dam core could result in damage to the core material and allow increased seepage of water.

To monitor the depth of frost penetration, the installation of a thermistor string is recommended. The instrument will provide long term monitoring of the frost penetration depth within the core for dam safety purposes. The thermistor string can also be used in the long term to assist in identifying the seepage regime within the core of the dam to confirm piezometric readings.

Two thermistor strings are recommended to be installed from the top of the core to an approximate depth of 20 m below the crest. The first thermistor string is recommended to be located at the instrumented river section of the dam and the second at the south transition section. The air temperature at the site will have an effect on the readings of the thermistor strings and should also be monitored to develop relationships between air temperature and frost penetration depth.

5.3.8 Seepage Through the South Dam

Seepage within the foundation and core of the South Dam is expected to be low. To monitor seepage through the southern portion of the dam, the installation of a V-notch weir is recommended with ultrasonic sensors for automated periodic readings. The weir will be located within the ditch at the downstream toe of the dam and any seepage discharge will be conveyed into the tailrace channel.

For dam safety considerations, weir operation and data collection is expected to be performed throughout the life of the structure. The long term monitoring of the piezometers discussed earlier within the instrumented sections can be used with the monitoring data from the weirs to compare the readings of each device based upon theoretical values obtained from seepage analyses.

It is noted that measured data from the weir will be influenced by seasonal precipitation and snowmelt and hence these input values must be assessed and taken into account during the data analysis.

5.4 North and South Dykes

The design of the North and South Dykes are described in Memorandum GN-4.2.2.2 "Design of North and South Dykes" [Ref. 8]. The North Dyke is approximately 11.7 km long, and is divided into four different sections: the Zoned Impervious Core Dyke, the Granular Dyke, the Freeboard Dyke, and the Road Section Dyke. The Freeboard and Road Section Dykes are relatively low structures, and located above the FSL. The Road Section Dyke will be located in those areas where the ground surface is above the elevation required for the passage of the design flood and will consist of road topping over minimal thickness of granular base material. Consequently, the instrumentation will be concentrated in the Zoned Impervious Core and Granular Dyke sections. The Zoned Impervious Core sections consist of an impervious core, with granular filters and transitions, and rockfill outer shells. The Granular Dyke sections are located within the North Dyke, between Sta. 78+50 and 79+80 and Sta. 96+85 and 97+40. The Granular Dykes will be constructed over permafrost affected foundations.

The South Dyke is approximately 11.1 km long, and is divided into three different sections: the Zone Impervious Core Dyke, the Freeboard Dyke, and the Road Section Dyke. All of the major instrumentation will be located in the Zoned Impervious Core Dyke sections.

The selection of the type and location of instrumentation for the North and South Dykes has been made based on the procedure described in Section 4. The instrumentation is designed to address each of the potential mechanisms that may control the behavior of the structures, as described below. The location of instrumentation selected for the North and South Dykes is shown in Figures 5 and 6.

5.4.1 Pore Water Pressures in Overburden Foundation

To verify that the pore water pressures within the overburden foundation are as anticipated, monitoring of pore water pressures in this zone is recommended. Piezometers are recommended at tentative locations along the North Dyke which may be adjusted based on specific conditions observed during construction. These locations are at Stas. 10+00, 13+00, 18+00, 23+00, 43+00, and 99+00. On the South Dyke, the locations are at Stas. 26+00, 57+00, 70+00, and 98+00, as shown in Table 3. The piezometers should be located beneath the Class 2a filter zone on the downstream portion of the dyke. The recommended instrumentation will determine if the pore pressures beneath the dyke are within expected ranges following reservoir impoundment and during long term conditions over the life of the structure.

The monitoring data obtained from the piezometers will be influenced by the permeability of the overburden.

5.4.2 Pore Water Pressures in Downstream Filter

The effectiveness of the downstream filter zone can decline during the life of a structure due to clogging with fine grained material, which can result in increased pore water pressures upstream of the filter.

To verify the effectiveness of the downstream filter within the North and South Dykes it is recommended that piezometers be installed to monitor pore water pressures. The locations of the piezometers will be at the same sections described in Section 5.4.1, and should monitor pore water pressures within the Class 2a pervious granular fill located downstream of the impervious core. Additional piezometers are recommended on the North Dyke at Stas. 79+00 and 97+00 within the Granular Dyke, as shown in Table 2, to monitor pore water pressures within the Class 2a pervious granular fill.

5.4.3 Construction Induced Pore Pressures and Long-Term Piezometer Level in Impervious Core

During construction, short term undrained conditions may result in elevated pore water pressures within the impervious core of the Zoned Impervious Core Dykes, typically reaching a maximum value by the time fill placement is complete.

To assess the stability of the North and South Dykes during construction and at the end of construction, it will be prudent to monitor the increase in pore water pressures. This will be achieved by the installation of piezometers. The piezometers can also be used for long term monitoring of the piezometric level within the impervious core during steady-state conditions.

On the North Dyke, the instrumented sections of the piezometers for the impervious fill are tentatively located at Stas. 10+00, 13+00, 18+00, 23+00, 43+00, and 99+00, as shown in Table 2. The recommended locations for the piezometers within each section are in the lower portion of the impervious core, upstream of the dyke centreline. A second piezometer is also recommended at each location above except Stas. 23+00 and 99+00 to provide redundancy.

On the South Dyke, the instrumented sections of the piezometers for the impervious fill are tentatively located at Stas. 26+00, 57+00, 70+00, and 98+00, as shown in Table 3. The recommended locations for the piezometers within each section are in the lower portion of the impervious core, upstream of the dyke centreline. A second piezometer is also recommended at each location to provide redundancy.

It is anticipated that pore water pressures generated within the impervious core during construction will vary based on the insitu moisture content of the fill materials at the time of placement and will be influenced by precipitation and the control of moisture during construction (i.e. wetting and drying of fill materials for compaction). The long-term piezometric level in the impervious core is expected to be similar to that calculated from two-dimensional seepage analyses performed for the dyke.

5.4.4 Thawing of Permafrost Beneath Granular Dykes

It is anticipated that following reservoir impoundment, the permafrost affected foundation of the Granular Dykes will slowly thaw due to seepage of reservoir water beneath the dykes. Monitoring of pore water pressures beneath the Granular Dyke is recommended to determine the effectiveness of the sand drains beneath the structure. Temperature measurement within the foundation is also recommended to identify thawing of the permafrost. It is anticipated that the combination of pore water pressure and temperature measurements along with settlement measurements from monitoring pins will provide a reasonably clear understanding of foundation conditions during this critical period.

On the North Dyke, the installation of piezometers at approximately Stas. 79+00 and 97+00 are recommended within the overburden foundation beneath the structure, as shown in Table 2. A second piezometer is recommended to be installed at Sta. 97+00 for redundancy. It is also recommended that thermistor strings be installed at each of these two locations and extend to a minimum of 1.5 m into the till. The final installation locations may be adjusted in the field based on actual conditions during construction. There is no instrumentation required to monitor thawing of permafrost along the South Dyke since there are no Granular Dyke sections along the dyke.

5.4.5 Lateral Deformation of Downstream Slope

The design of the North Dyke was developed to provide upstream and downstream slopes which will meet or exceed minimum factor of safety requirements as defined by the Canadian Dam Association's Dam Safety Guidelines [Ref. 5]. To monitor potential lateral deformation of the downstream slope, the installation of a slope inclinometer is recommended within the Zone Impervious Core Dyke at each of Stas. 10+00 and 79+00 on the North Dyke, and at Sta. 70+00 on the South Dyke. The location of Sta. 10+00 on the North Dyke was selected since this is where the dyke will be at its greatest height. The locations at Sta. 79+00 on the North Dyke and Sta. 70+00 on the South Dyke are intended to represent typical granular and zoned impervious core sections along each dyke, respectively. The slope inclinometers are recommended to be installed within the slope and be embedded a minimum of 3 m in dense till or bedrock.

The instrumentation is intended for long term dam safety monitoring following construction of the dam.

5.4.6 Horizontal or Vertical Displacement of the North and South Dykes

Horizontal or vertical displacements of the North and South Dykes are expected to be minimal for the Zoned Impervious Core dyke sections and have been accounted for in the Granular dyke sections by an increase in the crest elevation. It is recommended to install survey monuments on the North and South Dykes to confirm that settlements are within anticipated levels. The installation of six benchmarks along each dyke to be used for survey control are also recommended.

The survey monuments would be used for monitoring long term movement for dam safety. The monuments are recommended to extend approximately 1.4 m below the crest of the dam. The benchmarks should extend to a minimum depth of 7.6 m below the ground or fill surface to ensure they will be below the frost line and tie in to till or bedrock.

On the North Dyke, the survey monuments are recommended to be located at Stas. 16+00, 21+00, 26+00, 51+00, and 97+00, alternating at upstream and downstream crest locations of the North Dyke. In addition, six monuments will also be installed across each dyke (at the upstream shoulder of the crest, downstream shoulder of the crest, mid-slope of downstream slope, toe of downstream slope, downstream shoulder of the downstream bench, and toe of downstream bench) section at Stas. 11+00, 46+00, and 79+00. Six benchmarks are recommended installed along the North Dyke at approximate Stas. 12+00, 21+50, 36+00, 54+00, 84+00, and 104+00. These will be used as reference points for the survey monuments and be located downstream of the dyke axis.

On the South Dyke, the survey monuments are recommended to be located at Stas. 32+00, 37+00, 47+00, 72+00, 77+00, 97+00, 102+00, 107+00, and 130+00, alternating at upstream and downstream crest locations. In addition, six monuments should also be installed across each dyke section at Stas. 27+00 and 57+00. Six benchmarks will be installed along the South Dyke at approximate Stas. 30+00, 68+00, 86+00, 94+00, 103+00, and 120+00.

5.4.7 Frost Penetration

Frost penetration into the core of the Zoned Impervious Dyke sections of the North Dyke could result in damage to the core material and allow increased seepage of water. To monitor the depth of frost penetration, we recommend the installation of thermistor strings. The instruments will provide long term monitoring for dam safety.

On the North Dyke, four thermistor strings are recommended to be installed from the dam crest to the base of the postglacial clays at Stas. 10+00, 79+00, 97+00, and 99+00. On the South Dyke, two thermistor strings are recommended to be installed from the dam crest to the base of the post glacial clay at Stas. 26+00 and 98+00. The selected locations are intended to provide an indication of temperature variation along the dyke, as well as within both the Zoned Impervious Core and Granular Dyke sections. The air temperature at the site will have an effect on the readings of the thermistor strings and should also be monitored to develop relationships between air temperature and frost penetration depth.

5.4.8 Seepage Through the North and South Dykes

Seepage within the foundation and core of the dykes is expected to be low. To monitor seepage along sections of the dykes, the installation of V-notch weirs is recommended with a piezometer at each for automated periodic readings at specific locations where seepage has been identified following construction. The weirs will be located at the downstream toe of the dyke and any seepage discharge will be conveyed into natural water areas downstream of the dyke.

For the North Dyke, the preliminary locations of the weirs are at Stas. 9+49, 21+85, 34+40, 46+55, 78+00, 98+40 and 125+00. For the South Dyke, the preliminary locations of the weirs are at Stas. 26+00, 42+10, 71+00, 95+00,106+20, 122+40 and 131+80. For dam safety considerations, weir operation and data collection is expected to be performed throughout the life of the structure. It is noted that measured data from the weirs will be influenced by seasonal precipitation and snowmelt and hence these input volumes must be assessed during the data analysis.

5.5 Transmission Tower Spur

The design of the Transmission Tower Spur is described in Memorandum GN-4.2.2.6 "Transmission Tower Spur Design" [Ref 9]. This will be a granular fill and rockfill structure which will support the transmission line towers to allow the transmission lines to cross the Nelson River, from the Powerhouse Complex to the south bank.

Horizontal and vertical displacement of the Transmission Tower Spur is expected to be minimal. The structure will be founded on a thin foundation of granular alluvium overlying bedrock which is anticipated to result in minimal foundation settlement. A small amount of internal settlement of the granular fill material is expected, however, most of this will take place during or shortly after construction. It is recommended that survey monuments be installed on the Transmission Tower Spur to confirm that settlements are within anticipated levels.

The survey monuments would be used for monitoring both short term and long term displacement for dam safety purposes. A total of eight monuments are recommended to be installed with four along each of the north and south shoulders of the spur that extend approximately 1.4 m below the crest of the structure.

5.6 Concrete Structures

Design of the Concrete Structures is described in Memorandum GN-4.2.1.2 "Concrete Structures" [Ref. 3]. The concrete structures for the Keeyask GS include a 7-bay Spillway and 7-bay Powerhouse. The concrete structures will be constructed on bedrock foundations.

Selection of the types and locations for instrumentation for the Concrete Structures have been made based on the procedure described in Section 4. The instrumentation design to address each of the potential mechanisms that may control the behavior of the structure is described below.

5.6.1 Uplift of Spillway and Powerhouse Structures

It is recommended that instrumentation be installed to identify that the performance of the Spillway and Powerhouse Structures are satisfactory following the reservoir impoundment and during long term conditions.

It is recommended that a total of 18 piezometers be installed beneath the Concrete Structures to monitor potential uplift. Nine piezometers are to be installed within the bedrock at shallow depths below the Spillway, with three piezometers in each of Bays 2, 4 and 6 as shown in Figure 7. Nine additional piezometers are to be installed within the bedrock at shallow depths below the Powerhouse Structure with three piezometers at each of Units 2, 4 and 6 as shown in Figure 7. The piezometers will allow for measurements of

uplift pressures along the length and section of the structures and provide redundancy in case of potential malfunctioning or failure of individual instruments.

It is anticipated that the piezometers within the shallow bedrock will be installed during the construction of the structures. The installation procedure will be developed during Final Design. The intent of developing installation procedures is to minimize any potential damage to the instruments due to construction of the structures. Recorded measurements from the piezometers may be influenced by the effectiveness of the grout curtain and presence of joints which may extend beneath the grout curtain in the bedrock foundation.

5.6.2 Displacement of Concrete Structures

While displacement of the Powerhouse and Spillway structures is expected to be minimal, displacement monitoring will be used in assessing the performance of the structures following reservoir impoundment and over the life of the structures.

The parameters which require monitoring for this condition are vertical and horizontal deformation. The recommended instruments to measure these parameters are survey monuments, which can be surveyed to determine movements.

It is recommended that a total of 48 survey monuments be installed on the Powerhouse and Spillway. The installation of a permanent benchmark at the site is also required to ensure the measured values from the survey monuments are established from a known location.

To prevent potential disturbance or damage to the survey monuments, it is recommended that they be installed post-construction, but prior to full impoundment to obtain measurements during this period, as discussed in Section 4.

6 Special Considerations

To ensure satisfactory performance of an instrument, it should be protected prior to installation. Some instruments or their cables degrade in direct sunlight, while others should be protected from extreme temperature, humidity, and water. Cables and tubes should be protected from nicking, bending, and kinking. The manufacturers usually provide recommendations in regards to conditions that are unsuitable for their devices.

All instruments should be handled carefully, and should be kept free from dirt and dust.

Instruments and associated equipment should be robust and capable of long-term operation in an adverse environment.

Adequate compaction should be obtained around instrumentation.

The cables and tubes should be "snaked" to allow for extension due to settlement.

Scheduling of cable installation must be coordinated with other activities to avoid damage to cables and instrumentation by construction equipment. Instruments should be protected and located with highly visible survey stakes and flagging.

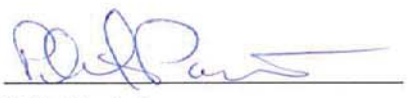
Instruments themselves can cause problems. The installation of instruments, even under the best circumstances, can introduce discontinuities into the embankment and foundations. Horizontal or vertical protective risers and/or cables leading from the instruments can result in areas of poor compaction and preferential paths for seepage. Therefore, the design must avoid the use of horizontal conduits in the primary direction of seepage flow.

7 Summary of Instrumentation Requirements

A summary of the instrumentation requirements is shown in Table 4. In addition to the items listed in Table 4, the instrumentation installation will also require the following items:

- approximately 750 m of drilling embankments, fill and overburden for installation of piezometers, slope indicators and thermistors
- approximately 280 m of drilling in bedrock for piezometer installations in bedrock under the embankments, and drilling in concrete for piezometers in the Spillway structure
- approximately 3400 m of trenching for installation of vibrating wire piezometers
- approximately 3400 m of wiring for vibrating wire piezometers
- approximately 100 m length of slope indicator casing for installation of slope indicators
- approximately 250 m length of thermistor string, with beads spaced at 1.5 m.

Prepared By



P.R. Pantel

Attach
References
Tables 1 to 4
Figures 1 to 7

PRP:spa

References

1. Manitoba Hydro, Interoffice Memorandum, “Keeyask Instrumentation Design for Dam Safety”. Manitoba Hydro file CED-00195-0002, January 28, 2009.

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3. Bohm, C.0., Bowerman, M.S. and Downey, M.W. 2006: Bedrock Geology of the Gull Rapids Area, Manitoba (part of NTS 54D6): Manitoba Science, Technology, Energy and Mines, Manitoba Geological Survey, Open File Report OF2006-32.

4. KGS Acres Ltd., Memorandum GN-4.2.1.2, “Keeyask Generating Station – Stage IV Studies – Concrete Structures”. Manitoba Hydro File 00195-20000-0005.

5. KGS Acres Ltd., Memorandum GN-4.2.2.3, “Keeyask Generating Station – Stage IV Studies – Design of North Dam”. Manitoba Hydro File 00195-21200-0009.

6. Canadian Dam Association, “Dam Safety Guidelines”, Canadian Dam Association, 2007.

7. KGS Acres Ltd., Memorandum GN-4.2.2.13, “Keeyask Generating Station – Stage IV Studies – Design of Central Dam”. Manitoba Hydro File 00195-21200-0008.

8. KGS Acres Ltd., Memorandum GN-4.2.2.4, “Keeyask Generating Station – Stage IV Studies – Design of South Dam”. Manitoba Hydro File 00195-21200-0007.

9. KGS Acres Ltd., Memorandum GN-4.2.2.2, “Keeyask Generating Station – Stage IV Studies – Design of North and South Dykes”. Manitoba Hydro File 00195-21200-0006.

10. KGS Acres Ltd., Memorandum GN-4.2.2.6, “Keeyask Generating Station – Stage IV Studies – Transmission Tower Spur Design”. Manitoba Hydro File 00195-21200-0010.

Tables

Tables 1 to 4

Table 1
Keeyask GS - Stage IV Studies, Axis GR-4
Physical Data of Structures

Name of Structure	Parameter	Description
Concrete Structures	Length	7-bay Spillway: 134.00 m. 7-unit Powerhouse(1): 247.74 m
	Crest Elevation	Spillway: el 162.5 m. Powerhouse - intake deck at el 161.5 m. Tailrace deck at el 152.0 m
	Foundation	Bedrock
	Maximum Height	Spillway: 33 m, Powerhouse: 65 m
North Dam	Length	Approx 99 m
	Crest Elevation	162.6 m
	Foundation	River Section: Bedrock
		North Abutment Section (transition to North Dyke): entire dam on till
	Maximum Height	Approx 24.5 m
Central Dam	Length	Approx 1606 m
	Crest Elevation	el 162.6 m
	Foundation	Bedrock
	Maximum Height	27.5 m
South Dam	Length	Approx 565 m
	Crest Elevation	el 162.6 m
	Foundation	River Section: impervious core on bedrock, the remaining parts on river bed surface. South Abutment Section (transition to South Dyke): Entire dam on till.
	Maximum Height	Approx 21.5 m
North Dyke	Length	Approx 11,655 m
	Crest Elevation	Zoned Impervious Core dyke section: el 161.8 to el 162.2 m Granular dyke section: el 163.0 m Freeboard dyke section: el 161.8 to el 162.2 m
	Foundation	Zoned Impervious Core dyke sections: primarily on till overburden, occasionally on bedrock. Granular and freeboard dyke sections: overburden (postglacial or till).
	Maximum Height	Approx 18 m (zoned impervious core dyke section)
South Dyke	Length	Approx 11,132 m
	Crest Elevation	Zoned impervious core dyke section: el 162.0 m to el 162.2 m Freeboard dyke section: el 162.0 m to el 162.2 m
	Foundation	Zoned impervious core dyke sections: primarily on till overburden, occasionally on bedrock. Freeboard dyke sections: overburden (postglacial or till).
	Maximum Height	Approx 18 m (zoned impervious core dyke section)

Note:

(1) Including Service Bay Structure

Table 2
Keeyask GS - Stage IV Studies, Axis GR-4
Tentative Location of Piezometers (North Dyke)

Type of Dyke	Approx Location of Piezometers	Section (see Figure 5)	Number of Piezometers
Zoned Impervious Core Dyke	Sta 10+00	K-K	4
	Sta 13+00	L-L	4
	Sta 18+00	M-M	4
	Sta 23+00	N-N	3
	Sta 43+00	P-P	4
	Sta 99+00	S-S	3
Granular Dyke	Sta 79+00	Q-Q	2
	Sta 97+00	R-R	3

Table 3
Keeyask GS – Stage IV Studies, Axis GR-4
Tentative Location of Piezometers (South Dyke)

Type of Dyke	Approx Location of Piezometers	Section (see Figure 5)	Number of Piezometers
Zoned Impervious Core Dyke	Sta 26+00	T-T	4
	Sta 57+00	U-U	4
	Sta 70+00	V-V	4
	Sta 98+00	W-W	4

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of Instrumentation Requirements

Structure	Type of Instrument	Location	Total Qty	Quantity and Remarks
Concrete Structures	Survey Monument	Spillway	48	24 monuments alternating U/S and D/S of structures axis
		Powerhouse		24 monuments alternating U/S and D/S of structures axis
	Piezometer	Spillway Structure Powerhouse Station	9 9	3 piezometers at each of the Bays 2, 4 & 6 3 piezometers at each of Units 2, 4 & 6
North Dam	Piezometer	North Abutment Section	11	2 in till, 2 in core, 1 within in-situ D/S foundation
		Main Section		3 in B/R, 2 in core, 1 D/S toe of dam
	Survey Monument	Central portion of dam	4	3 across dyke line section 1 near North Dam and North Dyke transition
	Thermistor	Approx midpoint between Main and North Abutment Section	1	At D/S of dam's crest. Approx. 20 m depth, probes spread at 1.5 m
	Weir	D/S toe of dam	1	
	Benchmark	North end	1	1 at D/S of dam/dyke transition
Central Dam	Piezometer	Central Dam near Powerhouse Section	20	4 in B/R, 2 in core 1 D/S toe of dam
		Central Dam with Remnant Stage I D/S CD (River Section)		4 in B/R, 2 in core
		Central Dam near Spillway Section		4 in B/R, 2 in core, 1 D/S toe of dam
	Slope Indicator	Central Dam near Powerhouse Section	1	1 at D/S slope
	Survey Monument	Every 100 m, alter-natively at U/S and D/S edge of dam's crest.	21	15 at 100 m spacing
		Across the dam near River Section		2 sets of 3 (i.e. 6 additional pins) along the D/S leg between instrumentation sections
	Thermistor	Central Dam near Powerhouse Section	1	Each at D/S of dam's crest. Approx 20 m depth, probes spaced at 1.5 m.
		Central Dam with remnant Stage I D/S CD (River Section)	1	
	Weir	Between Central Dam near PH section and Central Dam with	2	D/S toe of dam

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of Instrumentation Requirements

Structure	Type of Instrument	Location	Total Qty	Quantity and Remarks
		Remnant Stage I D/S CD section		
	Benchmark	North end of Central Dam near Powerhouse Section South end of Central Dam near Spillway Section	2	1 at D/S toe near Powerhouse structure 1 at D/S toe near Central Dam and Spillway transition
South Dam	Piezometer	North End Section Tie-in to Stage I Spillway Cofferdam River Section South Abutment Section South Transition Section	22	4 in B/R, 2 in core, 1 in filter zone 3 in core 4 in B/R, 2 in core 2 in B/R, 1 in filter zone (toe) 2 at till fdtn, 1 in filter zone (toe)
	Slope Indicator	North End Section	1	1 at D/S slope
	Survey Monument	Every 100 m, alter-natively at U/S and D/S edge of dam's crest across the dam near River Section and at tie-in to Stage I Spillway Cofferdam	13	5 at 100 m spacing 4 additional at two sections (1 at U/S above water line, 3 D/S slope and toe)
	Thermistor	South Transition Section River Section	2	D/S of dam's crest. Approx 20 m depth, probes spaced at 1.5 m.
	Weir	South Abutment Section	1	D/S toe of dam
	Benchmark	South Transition Section	1	1 at D/S of dam near dam/dyke transition

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of Instrumentation Requirements

Structure	Type of Instrument	Location	Total Qty	Quantity and Remarks
North Dyke	Piezometer	Sta 10+00 (Sect K-K)	27	4
		Sta 13+00 (Sect L-L)		4
		Sta 18+00 (Sect M-M)		4
		Sta 23+00 (Sect N-N)		3
		Sta 43+00 (Sect P-P)		4
		Sta 79+00 (Sect Q-Q)		2
		Sta 97+00 (Sect R-R)		3
		Sta 99+00 (Sect S-S)		3
	Slope Indicator	Sta 10+00 Sta 79+00	2	At D/S slope
	Survey Monument	Sta 16+00 Sta 21+00 Sta 26+00 Sta 51+00 Sta 97+00	5	One monument alternatively at U/S and D/S crest of dam.
		Sta 11+00 Sta 46+00 Sta 79+00	18	Six monuments across each dyke section
	Thermistor	Sta. 10+00 Sta. 79+00 Sta. 97+00 Sta. 99+00	4	Each at D/S of the dyke's crest. Approx 20 m depth each, probes spaced at 1.5 m.
	Weir	Approx Sta 9+49 Approx Sta 21+85 Approx Sta 34+40 Approx Sta 46+55 Approx Sta 78+00	7	D/S toe of dyke
		Approx Sta 98+40 Approx Sta 125+00		
	Benchmark	Approx Sta 12+00 Approx Sta 36+00 Approx Sta 54+00 Approx Sta 84+00 Approx Sta 104+00	6	All at D/S side of dyke

Table 4
Keeyask GS – Stage IV Studies, Axis GR-4
Summary of Instrumentation Requirements

Structure	Type of Instrument	Location	Total Qty	Quantity and Remarks
South Dyke	Piezometer	Sta 26+00 (Sect. T-T) Sta 57+00 (Sect. U-U) Sta 70+00 (Sect. V-V) Sta 98+00 (Sect. W-W)	16	4 at each station
	Slope Indicator	Sta 70+00	1	At D/S slop.
	Survey Monument	Sta 32+00 Sta 37+00 Sta 47+00 Sta 72+00 Sta 77+00 Sta 102+00 Sta 107+00 Sta 130+00	9	One monument alternatively at U/S and D/S edge of dam's crest.
		Sta 27+00 Sta 57+00	12	Six monuments across each dyke section.
	Thermistor	Sta. 26+00 Sta. 98+00	2	Each at D/S edge of the dyke's crest. Approx 20 m depth each, probes spaced at 1.5 m.
	Weir	Approx Sta 26+00 Approx Sta 42+10 Approx Sta 71+00 Approx Sta 95+00 Approx Sta 106+20 Approx Sta 122+40 Approx Sta 131+80	7	D/S toe of dyke
	Benchmark	Approx Sta 30+00 Approx Sta 68+00 Approx Sta 86+00 Approx Sta 94+00 Approx Sta 103+00 Approx Sta 120+00	6	All at D/S side of dyke
Transmission Tower Spur	Survey Monument	Along north and south shoulders at 50 m spacing.	8	

Figures

Figures 1 to 7

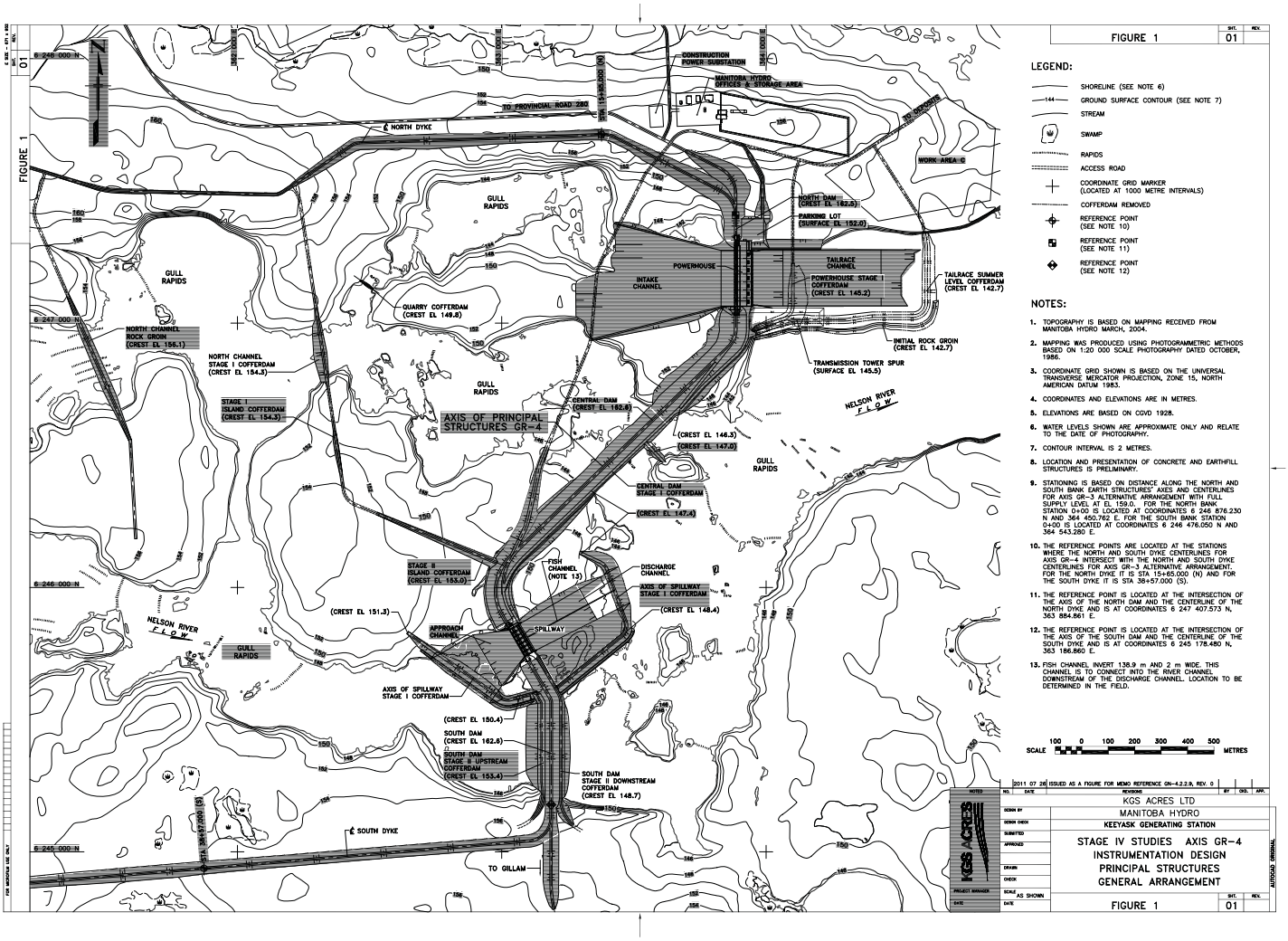

FIGURE 1

LEGEND:

- SHORELINE (SEE NOTE 6)
- 144 GROUND SURFACE CONTOUR (SEE NOTE 7)
- STREAM
- SWAMP
- RAPIDS
- ACCESS ROAD
- COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
- COFFERDAM REMOVED
- REFERENCE POINT (SEE NOTE 10)
- REFERENCE POINT (SEE NOTE 11)
- REFERENCE POINT (SEE NOTE 12)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURES' AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0. FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 854.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.

SCALE 100 0 100 200 300 400 500 METRES

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
INSTRUMENTATION DESIGN
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT

DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE AS SHOWN

SHT. 01 REV.

FIGURE 1

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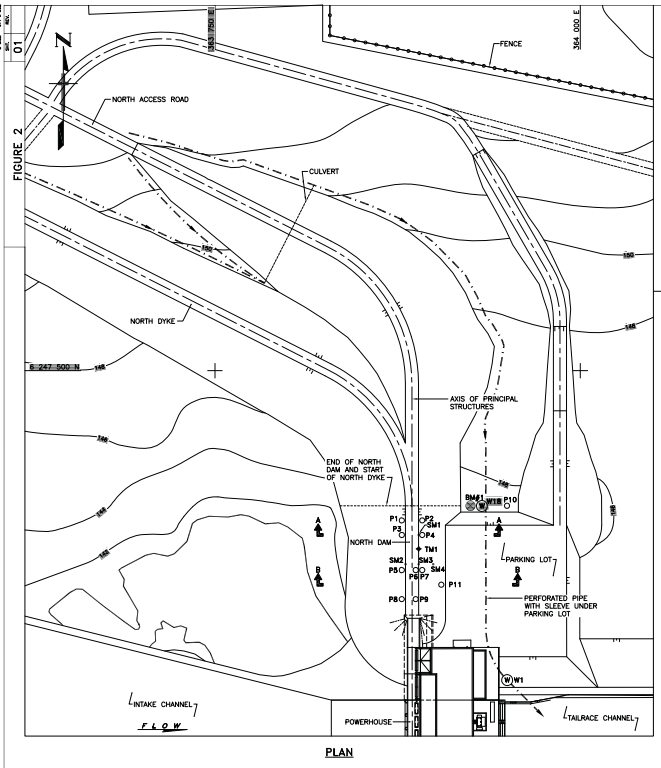
THIRD ANGLE PROJECTION



PLAN

FIGURE 2

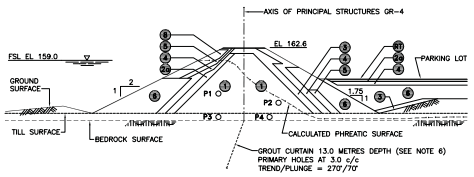
SHT. 01 REV.

LEGEND:

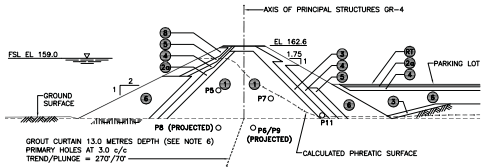
1 IMPERVIOUS FILL	SHORELINE	P1 PIEZOMETER
2A PERVIOUS GRANULAR FILL	142 GROUND SURFACE CONTOUR	BM BENCHMARK
3 FINE FILTER	DRAINAGE	SM SURVEY MONUMENT
4 TRANSITION – CRUSHED ROCK/ COARSE FILTER	WATER LEVEL SURFACE	W1 WEIR
5 RIPRAP BEDDING	COORDINATE GRID MARKER (LOCATED AT 250 METRE INTERVALS)	
6 ROCKFILL	TM1 TEMPERATURE MONITORING (PLAN)	
8 RIPRAP		
RT ROAD TOPPING		

NOTES:

1. TOPOGRAPHY IN THIS AREA BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. THE COORDINATE GRID SHOWN IS BASED ON UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
5. SURVEY MONUMENTS ARE NOT SHOWN ON SECTIONS FOR CLARITY.
6. DEPTH MEANS THE VERTICAL DEPTH, IT IS NOT THE LENGTH ALONG THE AXIS OF THE BOREHOLE.



SECTION A–A
NORTH DAM NORTH ABUTMENT SECTION



SECTION B–B
NORTH DAM MAIN SECTION

SCALE 20 0 20 40 60 80 100 METRES
PLAN

SCALE 10 0 10 20 30 40 METRES
SECTION

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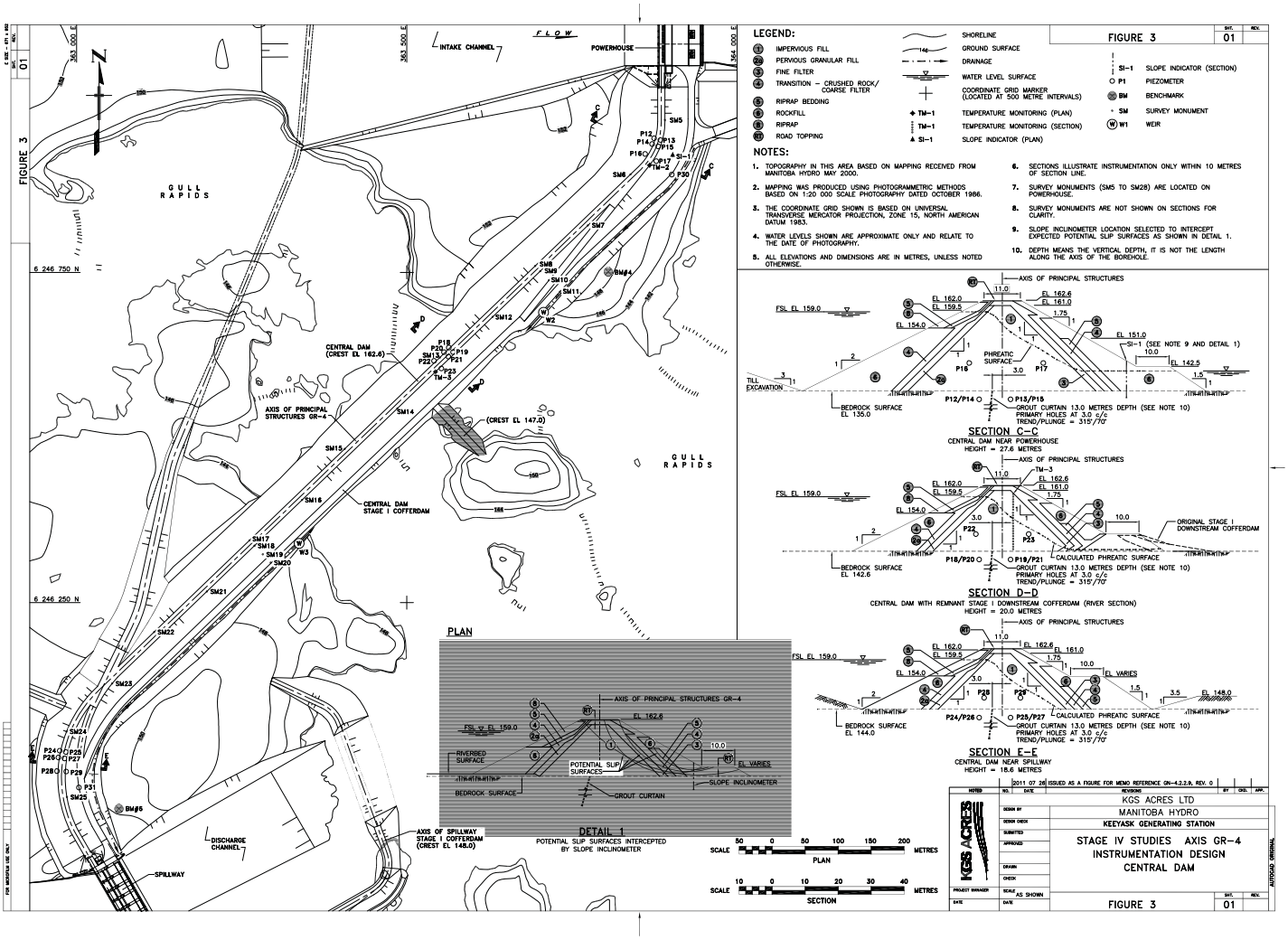
DESIGN CHECK	MANITOBA HYDRO
SUBMITTED	KEEYASK GENERATING STATION
APPROVED	STAGE IV STUDIES AXIS GR-4 INSTRUMENTATION DESIGN NORTH DAM
DRAWN	
CHECK	
PROJECT MANAGER	SCALE AS SHOWN
DATE	DATE

SHT. 01 REV.
FIGURE 2

AUTOCAD ORIGINAL

FOR ACCEPTANCE BY: DATE



LEGEND:

1	IMPERVIOUS FILL		SHORELINE
2A	PERVIOUS GRANULAR FILL	146	GROUND SURFACE
2B	FINE FILTER		DRAINAGE
2C	TRANSITION – CRUSHED ROCK/ COARSE FILTER		WATER LEVEL SURFACE
			COORDINATE GRID MARKER (LOCATED AT 500 METRE INTERVALS)
5	RIPRAP BEDDING		
6	ROCKFILL	TM-1	TEMPERATURE MONITORING (PLAN)
8	RIPRAP	TM-1	TEMPERATURE MONITORING (SECTION)
RT	ROAD TOPPING	SI-1	SLOPE INDICATOR (PLAN)
		SI-1	SLOPE INDICATOR (SECTION)
		P1	PIEZOMETER
		BM	BENCHMARK
		SM	SURVEY MONUMENT
		W1	WEIR

NOTES:

1. TOPOGRAPHY IN THIS AREA BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. THE COORDINATE GRID SHOWN IS BASED ON UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
5. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES, UNLESS NOTED OTHERWISE.
6. SECTIONS ILLUSTRATE INSTRUMENTATION ONLY WITHIN 10 METRES OF SECTION LINE.
7. SURVEY MONUMENTS (SM5 TO SM28) ARE LOCATED ON POWERHOUSE.
8. SURVEY MONUMENTS ARE NOT SHOWN ON SECTIONS FOR CLARITY.
9. SLOPE INCLINOMETER LOCATION SELECTED TO INTERCEPT EXPECTED POTENTIAL SLIP SURFACES AS SHOWN IN DETAIL 1.
10. DEPTH MEANS THE VERTICAL DEPTH. IT IS NOT THE LENGTH ALONG THE AXIS OF THE BOREHOLE

SECTION C–C
CENTRAL DAM NEAR POWERHOUSE
HEIGHT = 27.6 METRES

SECTION D–D
CENTRAL DAM WITH REMNANT STAGE I DOWNSTREAM COFFERDAM (RIVER SECTION)
HEIGHT = 20.0 METRES

SECTION E–E
CENTRAL DAM NEAR SPILLWAY
HEIGHT = 18.6 METRES

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SUBMITTED	KEEYASK GENERATING STATION
APPROVED	STAGE IV STUDIES AXIS GR-4 INSTRUMENTATION DESIGN CENTRAL DAM
DRAWN	
CHECK	
PROJECT MANAGER	SCALE AS SHOWN
DATE	DATE

FIGURE 3 — SHT. 01 REV.

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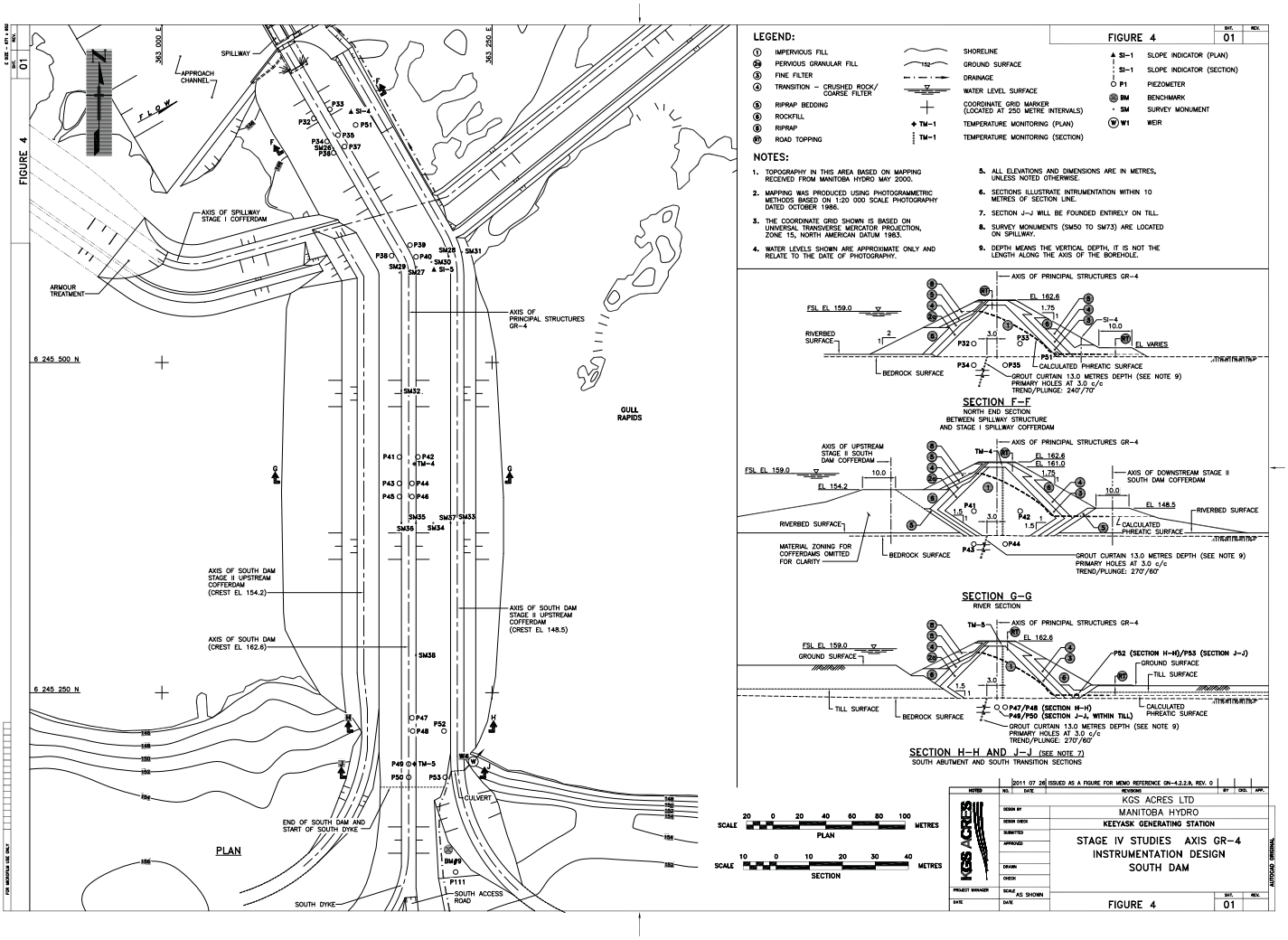

LEGEND:

① IMPERVIOUS FILL	SHORELINE	SI-1	SLOPE INDICATOR (PLAN)
②a PERVIOUS GRANULAR FILL	152 GROUND SURFACE	SI-1	SLOPE INDICATOR (SECTION)
③ FINE FILTER	DRAINAGE	P1	PIEZOMETER
④ TRANSITION – CRUSHED ROCK/ COARSE FILTER	WATER LEVEL SURFACE	BM	BENCHMARK
⑤ RIPRAP BEDDING	COORDINATE GRID MARKER (LOCATED AT 250 METRE INTERVALS)	SM	SURVEY MONUMENT
⑥ ROCKFILL	TM-1 TEMPERATURE MONITORING (PLAN)	W1	WEIR
⑧ RIPRAP	TM-1 TEMPERATURE MONITORING (SECTION)		
⑰ ROAD TOPPING			

NOTES:

1. TOPOGRAPHY IN THIS AREA BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. THE COORDINATE GRID SHOWN IS BASED ON UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
5. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES, UNLESS NOTED OTHERWISE.
6. SECTIONS ILLUSTRATE INTRUMENTATION WITHIN 10 METRES OF SECTION LINE.
7. SECTION J–J WILL BE FOUNDED ENTIRELY ON TILL.
8. SURVEY MONUMENTS (SM50 TO SM73) ARE LOCATED ON SPILLWAY.
9. DEPTH MEANS THE VERTICAL DEPTH, IT IS NOT THE LENGTH ALONG THE AXIS OF THE BOREHOLE.

SECTION F–F
NORTH END SECTION BETWEEN SPILLWAY STRUCTURE AND STAGE I SPILLWAY COFFERDAM

SECTION G–G
RIVER SECTION

SECTION H–H AND J–J (SEE NOTE 7)
SOUTH ABUTMENT AND SOUTH TRANSITION SECTIONS

SCALE 20 0 20 40 60 80 100 METRES
PLAN

SCALE 10 0 10 20 30 40 METRES
SECTION

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
INSTRUMENTATION DESIGN
SOUTH DAM

FIGURE 4 — SHT. 01 — REV.

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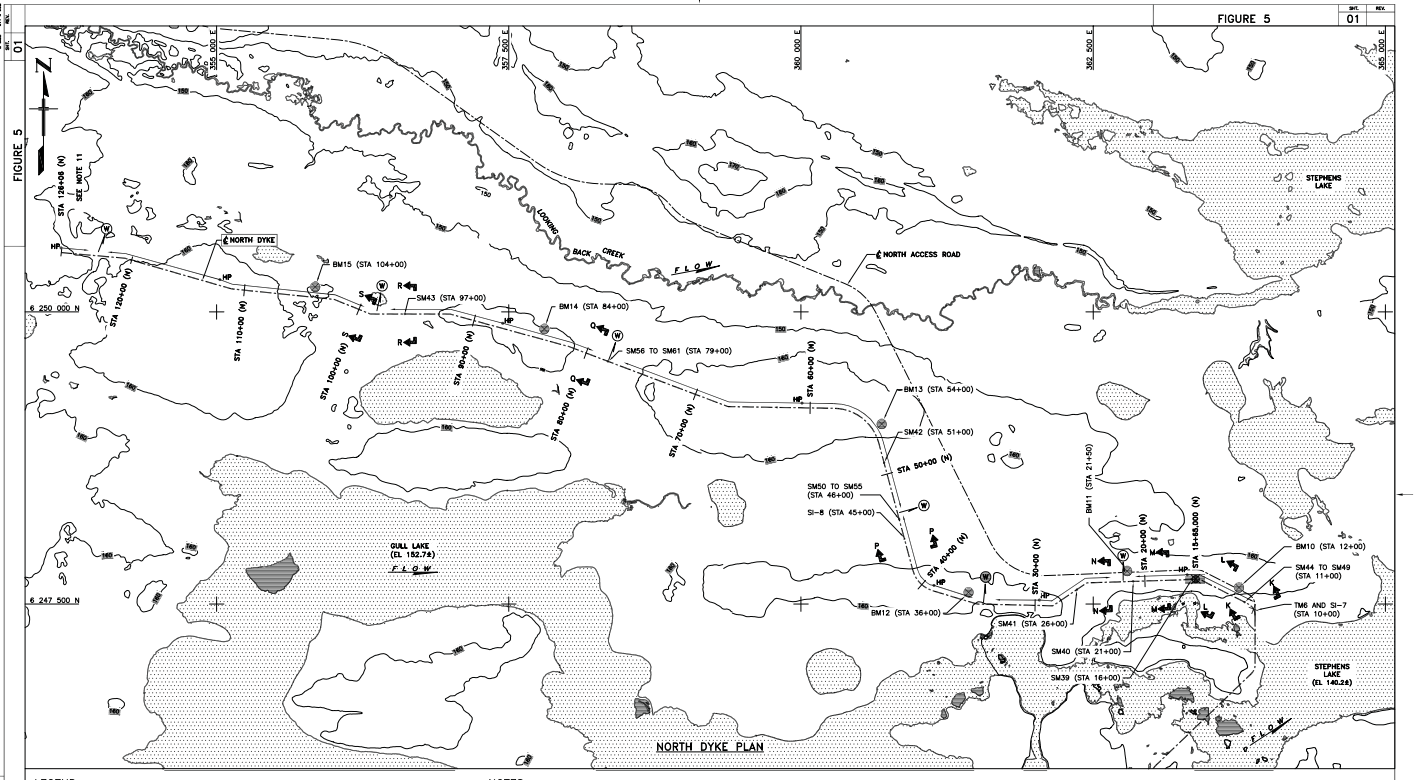
FIGURE 5



LEGEND:

- SHORELINE (SEE NOTE 6)
- GROUND SURFACE CONTOUR (SEE NOTE 7)
- COORDINATE GRID MARKER (LOCATED AT 2500 METRE INTERVALS)
- REFERENCE POINT (SEE NOTE 10)
- BM — BENCHMARK
- SM — SURVEY MONUMENT
- W — WEIR
- HP — HIGH POINT
- DIRECTION OF DRAINAGE
- OUTLET TO MARSHES, CREEKS OR PONDS
- TM — TEMPERATURE MONITORING
- SI — SLOPE INDICATOR
- STA 50+00 (N) — STATIONING ALONG AXIS AND CENTERLINE OF NORTH BANK EARTH STRUCTURES (SEE NOTE 9)
- STA 50+00 (S) — STATIONING ALONG AXIS AND CENTERLINE OF SOUTH BANK EARTH STRUCTURES (SEE NOTE 9)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. THE COORDINATE GRID IS BASED ON UNIVERSAL TRANSVERSE MERCATOR SYSTEM, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES, STATIONING AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. WATER SURFACE ELEVATION AND LOCATIONS OF SHORELINES ARE APPROXIMATE AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 10 METRES.
8. DYKE CENTERLINES SHOWN ARE FOR THE AXIS GR-4, WITH FULL SUPPLY LEVEL (FSL) AT EL 159.0 m.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURES' AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0. FOR THE NORTH BANK, STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK, STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT.
11. THE LAST STATION SHOWN FOR THE NORTH AND SOUTH DYKES IS BASED ON THE POINT WHERE THE FREEBOARD DYKE CREST DESIGN ELEVATION INTERSECTS WITH THE GROUND SURFACE INTERPRETED FROM TOPOGRAPHIC MAPPING AND THEREFORE IS APPROXIMATE.
12. SEE TABLE 4 IN MEMORANDUM GN-4.2.2.9 FOR COMPLETE INSTRUMENTATION REQUIREMENTS.
13. TYPICAL SECTIONS ARE SHOWN IN FIGURE 6.

SCALE 250 0 250 500 750 1000 1250 METRES

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STAGE IV STUDIES AXIS GR-4
INSTRUMENTATION DESIGN
NORTH AND SOUTH DYKES
PLANS

DESIGN BY	
DESIGN CHECK	
SUBMITTED	
APPROVED	
DRAWN	
CHECK	
PROJECT MANAGER	SCALE AS SHOWN
DATE	DATE

FIGURE 5 — SHT. 01 REV.

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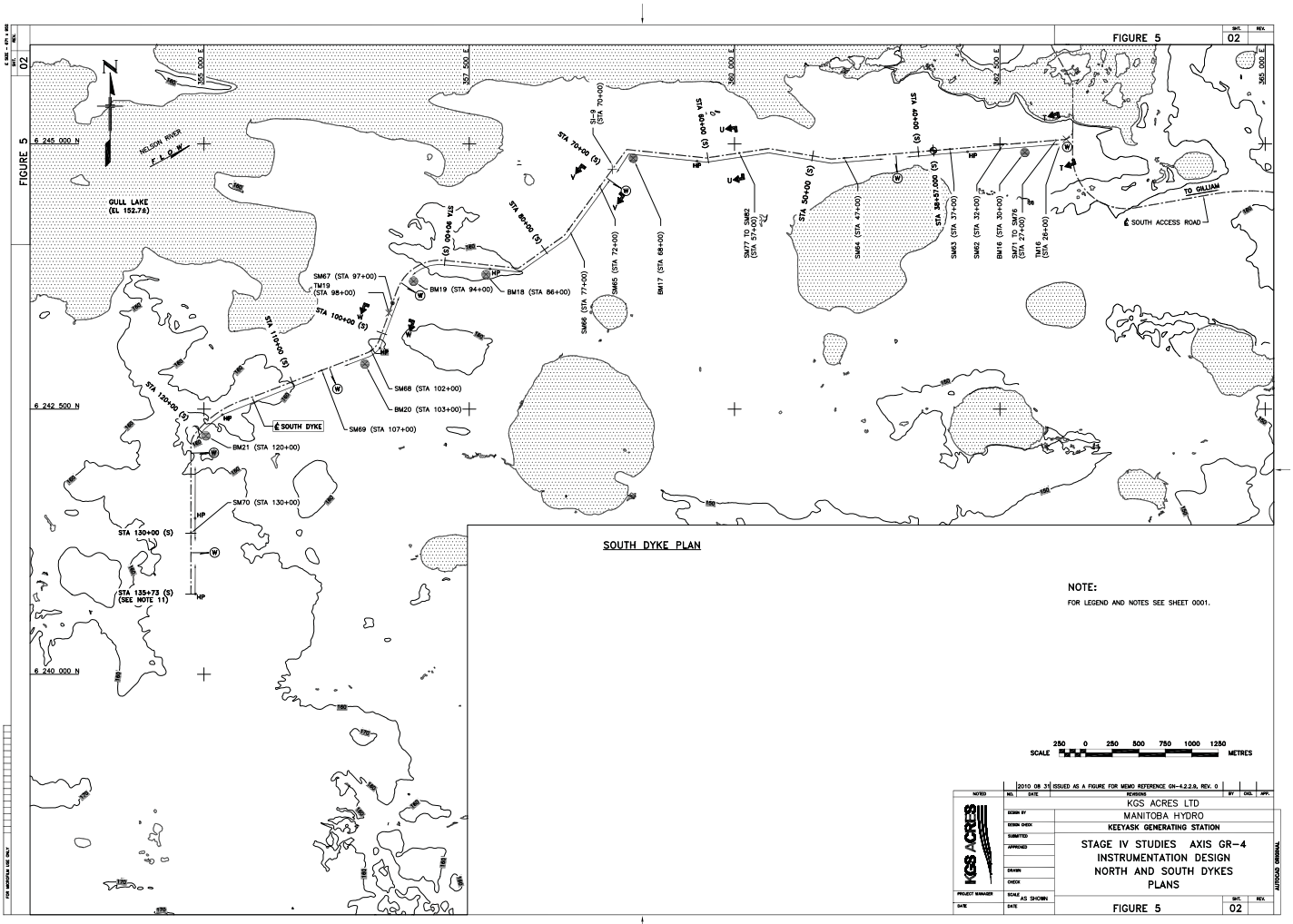
FIGURE 5
02
N
355 000 E
357 500 E
360 000 E
362 500 E
365 000 E
6 245 000 N
6 242 500 N
6 240 000 N
NELSON RIVER
GULL LAKE
(EL 152.7±)
STA 70+00 (S)
STA 80+00 (S)
STA 100+00 (S)
STA 110+00 (S)
STA 120+00 (S)
STA 130+00 (S)
STA 135+73 (S)
(SEE NOTE 11)
STA 00+08 (S) VIA
STA 00+08 VIA
STA 00+40 VIA
STA 00+40 (S) VIA
STA 50+00 (S)
STA 38+57.00 (S)
BM-3
(STA 70+00)
SM67 (STA 97+00)
TM19
(STA 98+00)
BM19 (STA 94+00)
BM18 (STA 86+00)
SM66 (STA 77+00)
SM65 (STA 72+00)
BM17 (STA 69+00)
SM68 (STA 102+00)
BM20 (STA 103+00)
SM69 (STA 107+00)
BM21 (STA 120+00)
SM70 (STA 130+00)
SM77 TO SM82
(STA 7+00)
SM64 (STA 47+00)
SM63 (STA 37+00)
SM62 (STA 32+00)
BM16 (STA 30+00)
SM71 TO SM76
(STA 27+00)
BM15
(STA 26+00)
TO GILLAM
℄ SOUTH ACCESS ROAD
℄ SOUTH DYKE
HP
W
U
T
SOUTH DYKE PLAN
NOTE:
FOR LEGEND AND NOTES SEE SHEET 0001.
250 0 250 500 750 1000 1250
SCALE METRES
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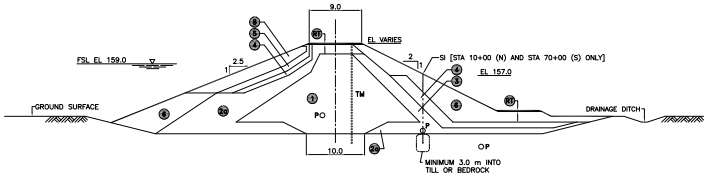
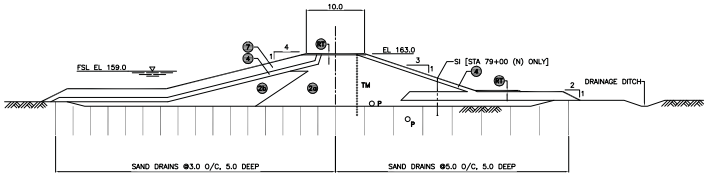
INSTRUMENTATION DESIGN
NORTH AND SOUTH DYKES
PLANS
FIGURE 5
02
AS SHOWN



ZONED IMPERVIOUS CORE DYKE

NORTH DYKE SECTIONS K–K, L–L, M–M, N–N, P–P, S–S (SEE NOTE 2)
SOUTH DYKE SECTIONS T–T, U–U, V–V, W–W (SEE NOTE 2)



GRANULAR DYKE

SECTION Q–Q, R–R (SEE NOTE 3)

LEGEND:

1	IMPERVIOUS FILL
2A	PERVIOUS GRANULAR FILL
2B	SEMI–PERVIOUS GRANULAR FILL
3	FINE FILTER
4	TRANSITION – CRUSHED ROCK/COARSE FILTER
5	RIPRAP BEDDING
6	ROCKFILL
7 & 8	RIPRAP
11	ROAD TOPPING
	WATER LEVEL SURFACE
O P	PIEZOMETER
TM	TEMPERATURE MONITORING
SI	SLOPE INDICATOR

NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. SECTIONS K–K, L–L, M–M, P–P, T–T, U–U, V–V AND W–W WILL HAVE AN ADDITIONAL BACK UP PIEZOMETER INSTALLED ADJACENT TO THE EXISTING PIEZOMETER LOCATED WITHIN THE CORE OF THE DYKE.
3. SECTION R–R WILL HAVE AN ADDITIONAL BACK UP PIEZOMETER INSTALLED ADJACENT TO THE EXISTING PIEZOMETER LOCATED WITHIN THE DYKE'S FOUNDATION.
4. SEE TABLE 4 IN MEMORANDUM GN–4.2.2.9 FOR COMPLETE INSTRUMENTATION REQUIREMENTS.

SCALE 5 0 5 10 15 20 25 METRES

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STAGE IV STUDIES AXIS GR–4
INSTRUMENTATION DESIGN
NORTH AND SOUTH DYKES
TYPICAL SECTIONS

DESIGN BY | DESIGN CHECK | SUBMITTED | APPROVED | DRAWN | CHECK | PROJECT MANAGER | DATE

SCALE AS SHOWN

FIGURE 6 | SHT. 01 | REV.

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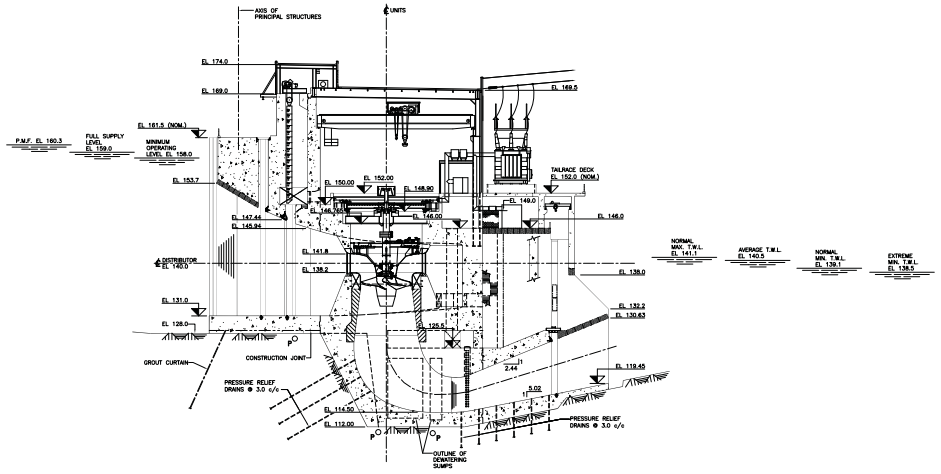
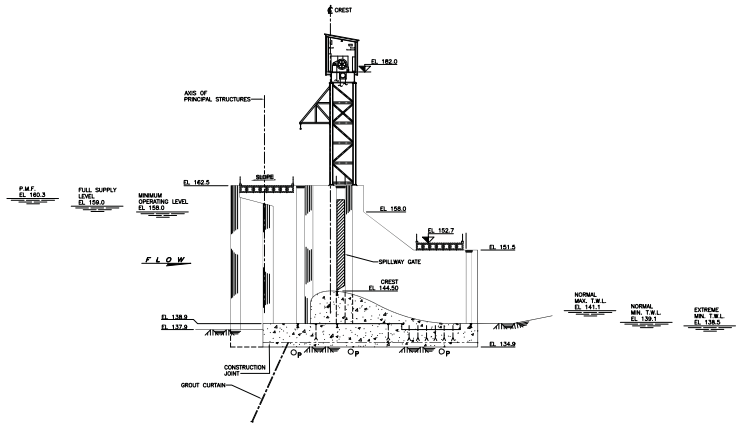
FIGURE 7 | SHT. 01 | REV.



POWERHOUSE SECTION AT ℄ UNIT



SPILLWAY SECTION – FINAL CONFIGURATION

LEGEND:

OP PIEZOMETER

SCALE 5 0 5 10 15 20 25 METRES

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		2010 08 31 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.9, REV. 0			

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DRAWN	
CHECK	
PROJECT MANAGER	SCALE AS SHOWN
DATE	DATE

FIGURE 7 | SHT. 01 | REV.

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Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.6
Rev. 0, July 29, 2011

Earthworks Structures Transmission Tower Spur Design
Manitoba Hydro File 00195-21200-0010_01

Prepared by _____ (P. PANTEL) July 29, 2011
KGS Acres Ltd.

Checked by _____ (D.M.R. Fuchs) July 29, 2011
KGS Acres Ltd.

Reviewed by _____ (I.R. Dewar) July 29, 2011
KGS Acres Ltd.

Approved by _____ (N.J. Smith) July 29, 2011
KGS Acres Ltd.

Accepted by _____ (M. ST. LAURENT) MAR. 2, 2012
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	September 7, 2010
		File No.	P10008(16).27.07
From	B.D. Garinger, P.R. Pantel	**cc**	
Subject	**Keeyask Generating Station Stage IV Studies – Axis GR-4 Earthworks Structures Design of Central Dam Memorandum GN-4.2.2.13, Rev 0 Manitoba Hydro File 00195-21200-0008_01**		

1 Introduction

The site of the proposed Keeyask Generating Station (GS) is approximately 730 km north of Winnipeg, at the head of Stephens Lake, on the Nelson River. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is the Gull Rapids site. The proposed GR-4 for the structures crosses the Nelson River at the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam.

This memorandum documents the design of the Central Dam at the proposed Keeyask GS site based on the Axis GR-4 Arrangement. The dam will be located across the central channel, and will be approximately 1.6 km long.

2 General Arrangement

The general arrangement of the proposed Keeyask GS is shown in Figure 1. The layout corresponds to the Axis GR-4, with a Full Supply Level (FSL) of el 159.0 m. The selected arrangement consists of two long river bank dykes, on the north and on the south, linked to the Principal Structures which span the Nelson River at Gull Rapids. The Principal Structures consist of a Powerhouse Complex constructed on the north side of the river, which will be connected to the North Dyke through a 99 m long North Dam. The Spillway will be located on an existing island to the south of the Powerhouse (South Island) and will be connected to the South Dyke through a 565 m long South Dam. A Central Dam, approximately 1.6 km in length, and having a maximum height of 27.5 m, will connect the Spillway and the Powerhouse across the two main islands of the Gull Rapids.

3 Regional Geology

The site for the Keeyask GS is located within the Canadian Shield and is underlain by variable thicknesses of up to 30 m of overburden over competent Precambrian bedrock. In general, the overburden stratigraphy consists of a thin organic cover on postglacial lacustrine clay, which overlies deposits of glacial outwash, till or the bedrock directly. Preglacial deposits of sand and silty sand are also occasionally found in the bedrock lows.

Two types of postglacial deposits have been identified:

(a) Alluvium

It generally consists of cobbles and boulders overlying sands and gravels and is locally present in the base of present day stream and river channels.

(b) Lake Agassiz Silts and Clays

A relatively thin layer of silts and clays was deposited on the bottom of glacial Lake Agassiz. The silts and clays form a veneer of up to several metres in thickness over the glacial deposits. These fine-grained deposits are commonly varved and tend to be of greater thickness in the topographic lows. They contain discontinuous permafrost, ranging from non-visible to visible ice segregation (less than 25 mm thick). The postglacial deposit(s) may be absent entirely on ridges and knolls. Peat has formed in the poorly drained flatlands and hollows which remained in the landscape after Lake Agassiz drained to Hudson Bay.

The glacial deposits are widespread and are derived from the several glacial ice sheets which advanced over the Gull Rapids area and deposited till and stratified water lain deposits. The tills contain discontinuous occurrences of permafrost, are generally well-graded, compact, have a relatively low moisture content, and have a low ice content when frozen.

Three separate till or till-like horizons have been identified at the Keeyask site. The upper silty sand/sandy silt till unit (Till 1), whose presence is the most widespread over the Keeyask area, generally consists of a light brown horizon (Till 1a) overlying a grey horizon (Till 1b) with essentially identical soil gradations. Beneath the silty sand/sandy silt till units, Till 2 and Till 3 consist of grey, low plasticity clays. Not all of the area at the Keeyask GS site encountered these till units. The till units may be separated by discontinuous intertill units, especially in areas of bedrock lows or in drumlin features.

En-echelon drumlin ridges are aligned east-west at approximate azimuths from 90° to 110°in the Keeyask study area. In addition to the drumlins, there are also some subdued glacial features forming smooth knolls with gentle slopes oriented at an azimuth of 125°. These are made up of both fine grained and granular soils, commonly intermingled.

Superimposed on the drumlins and some of the morainal features are glaciofluvial deposits. These consist of silts, sands and gravels, which were deposited in the ice walled channels to form narrow, serpentine eskers. Where the ice channels widened sufficiently to lower water velocity, glaciofluvial silt, sand and gravel deposits formed deltas and flat-topped kame deltas along the chain of discontinuous esker ridges.

The Gull Rapids site is located in an area of discontinuous permafrost. Ice rich peat plateaus are present throughout the area. Collapse scars (thermokarst topography) also occur throughout the Gull Rapids area, where permafrost has thawed.

Discontinuous permafrost is present in the postglacial clays and silts below the peat and locally extends into the underlying tills. Permafrost affected bedrock also occurs locally. The higher and more densely wooded ridge crests and knoll tops are drier and usually contain little if any permafrost. The eskers are also generally free of permafrost.

The bedrock within the Gull Rapids area shows a varied degree of metamorphic and cataclastic texture. The site is located within the Canadian Shield physiographic region, near the boundary between the Churchill and Superior provinces. The Churchill-Superior boundary zone is poorly defined, and contains cataclastic rocks which resulted from collision of the two plates during the late Hudsonian orogeny. Downstream, or to the east, the bedrock consists of metasedimentary rocks and igneous intrusive rocks, the same type as in the foundations for the Kettle GS. West and upstream of the site, and occasionally within the site area, the bedrock consists of a different group of metasedimentary and igneous intrusive rocks. At the site itself, the bedrock is primarily a greywacke gneiss. Diabase dykes cross-cut all rock types in the area, varying from short narrow features to continuous wider lineations.

4 Foundation Conditions Within the Central Dam Area

4.1 Site Investigations

The investigations of the Keeyask GS site commenced in the early 1960s and continued in 1988, 1990, winter 1990/1991, summer 1991, winter 1991/1992, winter 1999/2000, and 2003. The location of the boreholes and test pits in the vicinity of the Central Dam are shown in Figure 2. Selected stratigraphic sections illustrating the subsurface conditions within the same area are presented in Figure 3.

4.1.1 Overburden

The thickness of the overburden deposits along the axis of the Central Dam is variable, ranging from zero up to approximately 14 m (near the Powerhouse). Discontinuous permafrost is present in the postglacial clays and silts and locally extends into the underlying tills.

The Central Dam traverses several ridges and topographic lows, across the central section of Gull Rapids. The ridge located on the main central islands is covered by a 0.5 m to 1.5 m thick layer of peat, underlain by low to medium plasticity silty clays up to a depth of 6 m below the ground surface. The postglacial clays contain, at various depths, boulders, sand layers, visible ice lenses, and occasional permafrost.

On the north island, the surficial peat layer is thinner, less than 0.5 m thick, and overlays deposits of permafrost affected postglacial clays of up to 2 m in depth. A silty sand/sandy silt till deposit extends for between 5 m and 11 m below the base of the postglacial clays. Intertill deposits consisting of silts, sands and gravels can be found in bedrock lows near the Powerhouse location, occasionally containing cobbles and boulders.

4.1.2 Bedrock

Along the Central Dam alignment, the bedrock is generally encountered at depths greater than 5 m, except between Station 6+50 to Station 11+20 (i.e., across the rapids) as well as at Station 12+40. At these locations, the overburden layer is relatively thin or absent. Bedrock lows are encountered at Station 11+60 (el 140.4 m), at Station 15+10 (el 135.0 m), and at Station 15+70 (el 135.0 m).

In general, the bedrock consists of metasedimentary rocks and igneous intrusive rocks, primarily greywacke gneiss and granites, with both gabbro and diabase dykes cross-cutting all rock types. The rock mass can be characterized as strong to very strong, with RQD values typically in the 55 to 85 range. The blocky structure is characterised by tight or healed joints. The vast majority of the water pressure tests revealed very low permeability for the bedrock, with values typically in the 0 to 2 Lugeon range, although isolated zones with higher values are present.

4.2 Groundwater Conditions

Based on the available groundwater data from the boreholes drilled in the area, the groundwater table in the island portions of the Central Dam's foundation is generally within 3 m to 5 m below the ground surface (approximately el 145.5 m).

5 Construction Materials for the Central Dam

The impervious fill (Class 1) to be used in the construction of the core will consist of silty sand to sandy silt, which may be obtained from Deposits N-5 along the north bank. The granular fill materials for the Central Dam will be obtained from granular Deposits G-1 and G-3 on the north side of the river. The rockfill materials will be obtained from the required rock excavations for the Intake Channel, Powerhouse, and Spillway. Filter materials will consist of processed material from the granular deposits or from the rock quarries.

The gradation limits for the material to be utilized in the construction of the Central Dam are given in Table 1, and are shown in Figure 4. The properties of the fills to be used for the construction of the Central Dam are summarized in Tables 2 and 3. They comply with the criteria set forth in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref 1]. These values have been used for the stability and seepage analyses of the Central Dam.

Descriptions for cofferdam materials (Classes A, B and C) are provided in Memorandum GN-4.2.2.11, "Earthwork Structures - Cofferdams Arrangement and Design" [Ref 2].

Impervious Fill (Class 1)

Class 1 fill shall be a well graded unfrozen sandy silt/silty sand till that will compact into a homogeneous impervious fill. Class 1 fill is intended for use as the impervious sections in the Central Dam, and will be compacted. Moisture contents will be controlled within specified minimum and maximum levels to assure adequate compaction, trafficability and to limit the magnitude of construction pore pressures in the fill. Limited processing, involving moisture conditioning, reworking at the borrow site or during placement to break down large "lumps" in the material, and selective exploitation to remove oversize materials may be required. It is also anticipated that discing will be necessary to ensure proper bonding between layers.

Pervious Granular Fill (Class 2a)

Class 2a fill shall be a uniform to well graded pit run sandy gravel with less than 7% (by weight) passing 0.075 mm. Class 2a fill is intended to be a pervious granular fill for the upstream shell of the Central Dam.

Fine Filter (Class 3)

Class 3 fill shall be a well graded, free draining gravelly sand, with less than 5% finer (by weight) than 0.075 mm, having a coefficient of uniformity greater than 3, and a maximum particle size of 19 mm. Limited processing and/or selective exploitation to remove oversize materials may be required. Class 3 filter shall be used as a filter for the downstream shell of the Class 1 impervious fill and to provide adequate drainage.

Coarse Filter (Class 4)
Class 4 shall be a clean gravel or gravel sized crushed rock, processed as required so that the gradation limits are satisfied. Class 4 fill will be used as a filter layer adjacent to Classes 2a, 3 and 5, as bedding for the Class 6 on the upstream and downstream slopes and as a surficial erosion resistant layer on the surface of the downstream slope.

Riprap Bedding (Class 5)
Class 5 fill shall be a crushed or fragmented blasted rock. Class 5 will be used as riprap bedding or coarse filter fill. Processing involving crushing or screening of blasted rock may be required to obtain the desired gradation.

Rockfill (Class 6)
Rockfill Class 6 shall be well graded rock obtained from the required rock excavations and/or quarries, with a maximum particle size of 500 mm. Processing may not be required, other than selective removal of oversized pieces.

Riprap (Class 8)
Riprap Class 8 will be selected, narrowly graded, broken or blasted, sound, durable rock to be used for riprap protection of the upstream slopes of the North Dam. The riprap sizing will be based on Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref 3] requirements.

Road Topping (Class RT)
Road topping shall be processed sands and gravels with a trace to some fines, or gravel sized crushed rock with some sand and a trace to some fines. Road topping will be used for topping of the Central Dam.

The details of filter design are provided in Section 6.4.2.

6 Design of the Central Dam

6.1 Design Water Levels

The reservoir levels applicable for the design of the proposed Keeyask GS are presented in Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref 3]. Freeboard requirements due to the wind generated waves are provided in Memorandum GN-4.3.1, "Freeboard Requirements" [Ref 4]. The following operating range of reservoir levels were used for the preliminary design of crest and impervious core elevations at the Central Dam:

Full Supply Level (FSL)	el. 159.0 m
Minimum Operating Level (MOL)	el. 158.0 m
Level During Probable Maximum Flood (PMF)	el. 160.3 m

6.2 Crest and Impervious Core Elevations

The freeboard requirements discussed herein for the embankment structures follow the recommendations of the CDA Guidelines [Ref. 5] and the Geotechnical Design Criteria for Keeyask GS [Ref. 1]. The Freeboard Requirements Design Memorandum [Ref. 4] indicates that as a minimum, the crest of the Central Dam will be set at a level which satisfies the more critical of the following two conditions:

- FSL plus wind effects for the 1:1000 year wind event.
- PMF level plus wind effects for the 1:2 year wind event.

The freeboard requirements for the Central Dam are long term conditions, and therefore must also take into account any potential settlement of the embankment core, and foundations of the structure.

The wind effects determining the crest elevation of the dam includes provisions for wave run up and wind setup. The top of core elevation of the Central Dam is determined by similar criteria, except that the wave run-up is ignored. The wave run-up is excluded from the top of core calculation, as the core will be protected by the outer shells of the Central Dam.

The minimum end of construction elevations of the crest and top of the impervious core were determined based on a Full Supply Level indicated in Subsection 6.1 and a Probable Maximum Flood Level for the forebay. Based on the requirements contained in the design criteria, the minimal acceptable crest elevation is el 162.6 m and the top of core elevation is el 161.0 m (see Table 4). Further details are discussed and provided in Memorandum GN-4.3.1, “Freeboard Requirements” [Ref. 4].

6.3 Frost Protection

The design thickness of the granular materials overlying the top of the impervious core of the Central Dam is 1.5 m. In addition, the height of the core materials above the FSL is 2 m. Therefore, under average normal operating conditions, the phreatic water level within the impervious core of the dam is generally greater than 3.5 m below the crest of the dam.

Based on the apparent acceptable experience with the structures at the Kettle GS, where the core is composed of similar materials and the cover over the winter water levels is approximately 3.0 m, the proposed 3.5 m cover to the water line for Keeyask GS is assumed to be adequate to protect the core of the dam from damage due to frost penetration. Consequently, no additional insulation will be required between the top of the core and the overlying granular fill forming the crest of the dam. More refined analyses to better define the frost protection requirements of the impervious core should be carried out in the Final

Design Phase. It may also be worthwhile to undertake some explorations at Kettle GS to confirm the condition of the fill in the upper portion of the core.

6.4 Central Dam Cross-Section

The Central Dam will be a zoned earthfill and rockfill dam consisting of the following zones:

- impervious core
- pervious granular fill
- filter zone and transition filter materials
- rockfill outer shells
- rip rap and bedding materials
- road topping.

The description and typical gradation limit requirements for each of the above zones is provided in Section 5.

6.4.1 Impervious Core

The minimum width of the core is designed to be 5 m at the crest, which is based on practical construction considerations. The minimum elevation of the core at the end of construction was set at el 161.0 m, which is based on the analysis in Table 4. To minimize the cost and to satisfy the stability, seepage and design criteria requirements, both upstream and downstream slopes of the impervious core were set at 1H:1V. This provides seepage paths through the core much longer than the minimum requirement of 0.5 times the hydraulic head stipulated in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref. 1]. Typical cross-sections for the Central Dam are shown in Figure 6.

6.4.2 Filter Design

The filters included within the dam's cross-section provide for the structure's long term functionality, while accommodating structural movements. The granular chimney filter drains, located downstream of the impervious core, are designed to maintain the phreatic surface below the base of the downstream shell, thus maintaining a low free water surface throughout the dam's body.

The filters placed in the Central Dam were designed with two purposes:

- To intercept water flow and block the movement of eroding soil particles into the filter should cracks or openings develop in a base soil.
- To intercept water flowing through the pores of the base soil, allowing passage of the water while preventing movement of base soil particles.

The procedure for fine and coarse filter design follows the sequence proposed by United States Department of Agriculture, Chapter 26 Gradation Design of Sand and Gravel Filters, Part 633 National Engineering Handbook, October 1994 [Ref. 8] to ensure the designed filter will seal a crack. In general, the gradation of the fine and coarse drainage filters shall be based on the following:

- filter material shall be well graded, no gap graded material will be allowed
- maximum particle size shall be <75 mm
- filter shall have a maximum of 5% finer than 0.075 mm
- filter material shall not contain any plastic fines
- the D15 of filter shall be <4 times the D15 of the base soil, but no smaller than 0.1 mm.

As a generally accepted practice, the required minimum thickness of filters for high and medium risk embankment dams is 0.9 m. For the Keeyask GS, this requirement was accepted as a minimum cumulative thickness of the fine and coarse filters.

6.4.3 Slope Protection

Slope protection is required to protect the slopes of the dam against damage from wave erosion, weathering, ice damage and damage from floating debris and from both the Powerhouse and Spillway approach flow velocities.

6.4.3.1 Upstream Slope Protection

The upstream slope protection of the Central Dam will consist of Class 8 riprap. The riprap's gradation is based on the design weight as determined using the procedure given in Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref. 3] and Memorandum GN-4.1.3, Geotechnical Design Criteria" [Ref. 1]. Riprap shall be placed on the upstream slope from el 154.2 up to el 162.0 m. The minimum thickness of riprap perpendicular to the slope is set at 2.5 times the minimum stone size for Class 8 material.

In addition, riprap will extend to the toe of the upstream slopes adjacent to the Powerhouse South Transition and the Spillway North Transition. The extent and size of the erosion protection must be addressed in final design.

The riprap will be placed to provide a well-integrated mass with minimum void spaces so that the underlying bedding cannot be washed out.

6.4.3.2 Downstream Slope Protection

It is desirable that riprap be placed onto the remnant impervious fill of the Central Dam Cofferdam (see Section 11), to protect against wave action from tailwater levels.

6.5 Optimization of Central Dam

6.5.1 Type of Embankment

The design of the Central Dam as a zoned structure was reviewed and optimized in order to minimize the volume and the cost of the dam, while maintaining its stability and seepage requirements.

Two types of embankment were reviewed:

- a granular (Class 2a) shell section
- a rockfill (Class 6) section.

The Class 6 is available from sources in the vicinity of the Central Dam, making it more economical than the Class 2a granular fill. Consequently an embankment having rockfill shells was adopted and the use of Class 2a fill was minimized.

This reduced the overall cost, while at the same time allows for steepening the downstream slope of the Central Dam from 2H:1V to 1.75H:1V, meeting the design criteria requirements for stability, and improving drainage conditions in the dam shoulders.

6.5.2 Optimization of Dam Alignment

Two different alignments were studied for the layout of the Central Dam, and are presented in Figure 2:

- **The straight alignment,** under which the Central Dam axis follows the shortest route between the Powerhouse and the Spillway.

- **The arched alignment,** under which the Central Dam axis follows a curved axis across the rapids at a slightly upstream location from the straight alignment. Although of a greater length, the arched alignment was considered because it offers a slightly higher foundation elevation and smaller cofferdam. At the time of review, there was believed to be a hydraulic benefit related to the approach flows to the Powerhouse associated with the arched alignment. Subsequent analysis has since indicated this benefit to be insignificant.

Preliminary designs (discussed in Subsection 6.5.3) revealed that the typical cross-section for the Central Dam should have a 2H:1V upstream slope and 1.75H:1V downstream slope when founded on bedrock. Using this typical cross-section, the two alternatives were reviewed. The following observations were noted:

- The arched alignment requires approximately 9% more overburden excavation, due to its greater length, even though the average thickness of the overburden deposits across the two islands is the same.
- The construction of the Central Dam following the arched alignment requires a slightly higher fill volume than the dam built along the straight alignment.
- Contact area underneath the impervious core which requires foundation preparation, would be greater by approximately 6% under the arched alignment. Foundation grouting will also be greater by 11% under the arched alignment.
- The fill volume required in the construction of the Central Dam Stage I Downstream Cofferdam would be smaller by 28 000 m3 (or almost 50%) under the arched alignment.
- Cost comparison of both alternatives for a bedrock foundation determined that the straight alignment was more economical by about $[redacted]
- Cost comparison of both alternatives was also undertaken for typical sections founded on till with 2.5H:1V upstream slopes and 2H:1V downstream slopes. The cost difference between the two alignments was similar. It was determined that the straight alignment was more economical by about $[redacted]

Based on the results of the above mentioned comparative analyses, the straight alignment was selected as the preferred alternative, due to the overall cost savings it offers.

6.5.3 Cross-Section Optimization

The proposed cross-section of the Central Dam consists of a zoned structure with an impervious (Class 1) core, with filter zones on both sides, and outer rockfill shells. Along the selected straight alignment, the height of Central Dam ranges from a minimum of 13.0 m (at Station 12+40) to a maximum of 27.6 m near the Powerhouse (at Station 15+10), with a weighted average height of 19 m.

The Central Dam will be founded mostly on bedrock over an estimated 70% of its length, where the ground surface consists of exposed or relatively shallow bedrock. For the remaining length of the dam, four cross-section options were studied as follows:

(i) embankment founded entirely on till

(ii) embankment founded on till with a cut-off trench extending to bedrock

(iii) embankment with the upstream shell and impervious core founded on till with the downstream shell on bedrock

(iv) embankment founded entirely on bedrock.

Preliminary design was carried out for each of these possible cross-sections in order to assess the suitability of each option. These preliminary designs are described as follows.

Option 1 - Foundation on Till

Under this option the remaining length of the Central Dam would be founded entirely on till. Stability analyses were calculated with outer slopes of 2H:1V. Critical slip surfaces developed at the contact between the impervious core and the filter layers, and extended into the till foundation. Results of the stability analyses (Figure 5, Sheet 0001) show that the safety factors do not meet the minimum requirements set in the Geotechnical Design Criteria [Ref. 1], suggesting that the outer slope should be flatter than 2H:1V.

Option 2 - Impervious Core on Bedrock

The granular shells for the remaining portion of the dam would be founded on till, while the impervious core would be founded directly on bedrock (Figure 5, Sheet 0001). This option was considered in order to improve the dam's stability relative to Option 1, minimize seepage, and eliminate requirements for any potential overburden grouting beneath the impervious core. This option would reduce the amount of excavation beneath the dam and outside the footprint of the impervious core relative to Option 4, particularly in areas where the overburden cover thickness is greater than 2 m. The results of the stability analysis conducted on the preliminary cross-section indicated that the downstream slopes require a small berm to meet with the stability requirements (Figure 5, Sheet 0001).

Option 3 - Partial Foundation on Bedrock

Option 3 is based on the results of the stability analyses of Option 2. The upstream slope was flattened to 2.5H:1V and the downstream outer rockfill shell was founded on bedrock to meet the stability requirements (Figure 5, Sheet 0002). Factor of safety of the critical slip surfaces was calculated to be greater than 1.65, which shows that they are well above the required 1.5.

Option 4 - Foundation on Bedrock

Under this alternative, the entire footprint of the dam would be placed on bedrock with a 2H:1V upstream slope and 1.75H:1V downstream slope. Results of the stability analyses (Figure 5, Sheet 0002) show that the safety factors for this design exceed the minimum requirements set forth in the design criteria. In addition, this option eliminates the long term concerns related to the potential seepage through the overburden beneath the impervious core (as would be the case with Option 3), as well as any potential settlement from thawing of overburden under the dam due to the presence of permafrost affected soils.

6.5.4 Selected Cross-Section for the Central Dam

The overburden at the proposed dam site contains permafrost, occasionally with visible ice lenses, in the post glacial clays and till deposits. If the dam were to be founded on these materials, thawing might potentially result in significant differential settlements. Consequently, due to stability considerations and to avoid differential settlements, the Central Dam's impervious core will be founded entirely on the bedrock's surface. Foundation settlements for dams which are founded on sound Precambrian bedrock will be negligible.

The placement of the impervious core of the Central Dam over a prepared bedrock surface will also further minimize any seepage concerns at the dam's foundation.

Based on the above, the following rationale and conclusions were drawn:

- As a minimum requirement to minimize any seepage potential along the unknown contact area between the till and the bedrock, the footprint of the impervious fill for the Central Dam would be founded directly on top of bedrock. This would also allow for foundation inspection and preparation to be carried out prior to the placement of the impervious fill, to ensure that a complete cutoff of the impervious core to the bedrock will be made. As a consequence, Option 1 discussed above would not be utilized as an acceptable cross-section for any portion of the Central Dam.
- From Station 0+40 to Station 3+00 (near the Spillway structure), the till thickness that would be left in place after removal of organic and permafrost affected materials is in the order of 1.5 m to 3 m thick. A cost comparison between Options 2 and 3 at this location showed no economic advantages of one solution versus the other. The first 40 m of the Central Dam will be founded entirely on the bedrock, as the adjacent Spillway structure will also be built on bedrock. Therefore, Option 4 (foundation on bedrock) was selected from Station 0+00 to 3+00 because of shallow bedrock condition and proximity to the Spillway structure.

- Between Station 6+50 and Station 11+50, the dam will be founded entirely on bedrock, as the bedrock is mostly exposed over this area (across the Central Channel area). Option 4 was also selected for this section.
- From Station 12+50 to Station 15+50 (adjacent to the Powerhouse structure), till thickness ranges between 4 m and 9 m. At several borehole locations the till was permafrost affected (G-0009 and G-0064), and thick layers of coarse granular materials were encountered (G-0064 and 03-022). At this location, the Powerhouse Stage I Downstream Cofferdam as well as the Transmission Tower Spur are tying into the Central Dam, at its maximum height (i.e., 27.5 m to bedrock). Based on the results of the subsurface investigation program and all other considerations, it was deemed prudent to remove the till and found the Central Dam directly on bedrock in this section (Option 4).
- The remaining section from Station 15+50 to Station 16+06 will also utilize Option 4, due to its proximity to the Powerhouse concrete structure. The concrete structure will be founded on bedrock.
- The remaining section between Station 3+00 and Station 6+50 (350 m length), and between Station 11+50 to Station 12+50 (100 m length) the dam would utilize Option 2, with impervious core footprint founded on bedrock. Transition areas between Option 2 and Option 4 cross-sections would be required to bring smooth transition along the length of the dam. In these transition sections, the slope cut in the till beyond the footprint of the impervious core would be set at 1V:4H.

The proposed Central Dam was therefore selected to be a zoned earthfill dam founded on bedrock for up to approximately 70% of the dam's length (up to 1,150 m length), with a 2H:1V upstream slope and 1.75H:1V downstream slope. The remaining 30% (up to 450 m length) of the dam's length will have a central core trench to the bedrock surface with the upstream and downstream shells of the dam being founded on the surface of the till.

6.6 Material Quantities

A summary of the estimated quantities of the materials required for the construction of the Central Dam is presented in Table 5. The total volume of fill material to be used in the construction of the Central Dam is estimated at approximately 1.5 million m3.

7 Stability Analyses

7.1 General

Stability analyses were carried out using Version 5.2 of the XSTABL slope stability analysis computer program or Slope/W 2007. These computer programs generate potential circular,

irregular, or block slip surfaces for two-dimensional limit equilibrium analyses. The stability analysis for the preliminary cross-sections were undertaken using XSTABL using Bishop's and/or Janbu's method of circular analysis. The subsequent stability analyses for the adopted cross-sections were undertaken using Slope/W with the Morgenstern-Price method.

Slope stability analyses were carried out during the various stages of the Central Dam design on each of the cross-section alternatives under consideration (Figure 5). This included the cross-section of maximum height (27.5 m) located near the Powerhouse South Transition structure.

The stability of the Central Dam was also analyzed for earthquake loading under pseudostatic conditions, using horizontal and vertical seismic coefficients. The pseudostatic method of analysis is considered suitable for the site conditions at Keeyask GS by the Canadian Dam Association (CDA), Dam Safety Guidelines [Ref. 5].

7.1.1 Earthquake Resistance

The seismic hazard values intended for the 2005 National Building Code of Canada (NBCC) indicate that the Peak Ground Acceleration (PGA) value for the Gull Rapids area is approximately 0.059g for a probability of exceedance of 2% in 50 years (1:2,500 year event).

The selected PGA value, which is presented in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref. 1] and outlined below, is the recommended PGA value for the stability analysis of the Keeyask GS earth structures.

PGA value = 0.059g

In selecting seismic coefficients for the stability analysis, a reduction factor is typically applied to the PGA values. The psuedostatic seismic coefficients selected are as follows:

- Horizontal Seismic Coefficient ah = 0.05
- Vertical Seismic Coefficient av = 0

7.1.2 Factor of Safety Requirements

The minimum acceptable factor of safety for various conditions are presented in Memorandum GN-4.1.3, "Geotechnical Design Criteria" [Ref. 1] and outlined in Table 6.

The extreme rapid drawdown condition refers to a case where the reservoir water level is assumed to drop instantaneously from the FSL to any lower elevation which is equal to or above the crest of the Spillway rollway at el 146.5 m. In these cases, the piezometric level in the impervious core is assumed to remain unchanged from the prevailing level just

before the commencement of the drawdown, i.e., no excess pore water pressure dissipation was considered within the impervious core.

The Central Dam was evaluated to withstand ground motions associated with a Maximum Design Earthquake (MDE), without release of the reservoir. The minimum acceptable factor of safety against slope instability was selected in order to ensure reservoir containment while allowing damage to be repaired. A target value for the minimum acceptable factor of safety for the MDE in this evaluation has therefore been set as 1.0.

7.1.3 Design Water Levels

Based on Memorandum GN-4.1.2, "Hydraulic Design Criteria" [Ref. 3], the reservoir water level at FSL will be at el 159.0 m, with the Normal Minimum Supply Level being el 158.0 m. The Design Flood Level (DFL) will be at el 160.3 m.

The water levels used in the stability analyses are summarized in Table 7.

7.1.4 Piezometric Levels

Piezometric conditions adopted for the design of the Central Dam are as follows.

(a) End of Construction

- ru = 0.3 in impervious fill
- No excess pore water pressures in granular filter/transition and rockfill zones.

(b) Long Term Steady State

- Piezometric level in the impervious core was based on flownets with an upstream water level of el 159.0 m (FSL) and a downstream water level of el 142.5 m.
- Piezometric levels in the granular filter/transition and rockfill zones are based on the assumption that these materials are free draining.

(c) Normal Drawdown, Extreme Rapid Drawdown and Seismic Loading Cases

- No drainage of pore water pressure in the impervious core (i.e. pore water pressures in the core remain unchanged from steady state conditions described above).
- Piezometric levels in the granular filter/transition and rockfill zones are based on the assumption that these materials are free draining.

7.2 Results of Stability Analyses

7.2.1 Stability of Cross-Section Options

During the cross-section optimization process (described in Subsection 6.5), stability analyses were performed for the various options considered and at different locations along the Central Dam, i.e., near the Spillway structure, in the riverbed and near the Powerhouse structure. Graphical representation of several circular slip surfaces and their corresponding factors of safety are shown in Figure 5. The stability analyses shown in Figure 5 cover the different options considered, i.e.:

1) foundation on till
2) core on bedrock
3) partial foundation on till
4) foundation on bedrock.

7.2.2 Stability of Adopted Cross-Sections

The results of the stability analysis for the adopted cross-sections (Options 2 and 4) are summarized in Tables 8 and 9.

For the analysis of the seismic loading, only the End of Construction and Long Term Steady Seepage design cases were analyzed, as it is not likely that an earthquake loading will occur simultaneously with drawdown conditions.

In addition, the Central Dam was further analyzed under seismic loading, using a horizontal seismic coefficient of 0.10. This analysis was carried out as part of the screening process to determine the sensitivity of the analyses under more severe seismic loading conditions. For a seismic loading of 0.10g the calculated minimum factor of safety is 1.0.

The calculated minimum factors of safety are equal to or slightly higher than the minimum required values. The factors of safety for the upstream slope are at or above the minimum design requirements, and it is the extreme rapid drawdown case which governs the stability. A stabilizing berm is required at the upstream toe in areas where the dam is founded on till (i.e. Option 2).

The stability analysis of the downstream slope indicates that it is the End of Construction case that governs the stability of the dam.

8 Seepage Analyses

A two-dimensional seepage analysis was carried out to estimate the seepage gradients and quantities for the Central Dam and its foundation. The analysis undertaken for a typical cross-section of the Central Dam was conducted using SEEP/W, which is a two dimensional finite element software program widely used to model saturated and unsaturated flows. For the purposes of the Stage IV Studies, only steady state conditions (excluding the grout curtain) were simulated. The average coefficient of permeability of the fill materials used in the analyses is presented in Table 2. The results are summarized below.

The total seepage through the Central Dam under normal operating conditions is estimated to be in the order of 3800 L/min (or 2.5 L/min/m). This estimate is based on an average 19.5 m high and 1554 m long Central Dam (excluding the two transition zones). The actual dam height varies between 13.0 m and 27.6 m.

The anticipated seepage quantity is relatively low and can easily be handled by the drainage zones within the dam and by collection ditches to the downstream, as required.

9 Foundation Preparation and Grouting

9.1 Foundation Preparation

As discussed in Subsection 6.5 the impervious cut-off of the Central Dam will extend to the bedrock surface. As part of the foundation preparation, all overburden materials, weathered rock, loose rock, and other deleterious materials remaining after excavation and completion of grouting shall be removed from the rock surface. Cavities, faults, potholes, and exposed open joints will be cleaned by barring, hand excavation, and jetting with air and/or water. Any overhanging or near vertical rock surfaces will be trimmed to a slope of 1H:1V to permit thorough compaction of the earth fill. Holes and depressions underneath the impervious core will be filled with dental concrete. Where rock surfaces are fractured, a slush grout mixture will be used to fill in all the fractures. No consolidation grouting is anticipated to be required.

9.2 Grout Curtain

The permeability of the bedrock along Axis GR-4 is generally less than 1 Lugeon, which suggests that the bedrock is relatively tight. However, a grout curtain should be installed beneath the dam. The curtain will be installed along the dam axis, underneath the impervious core and is intended to reduce the seepage in the upper portion of the bedrock.

For the Central Dam, the grout curtain will be a single-row curtain, comprised of inclined grout holes at 315 /70 trend/plunge and a minimum 15 m in length (approximately 13 m measured vertically). The grout curtain will be developed by advancing Primary, Secondary and Tertiary grout holes (P.S.T holes) as a minimum. Primary holes will be spaced at 12 m on center, secondary holes will be advanced midway between primary holes and tertiary holes will be advanced midway between the secondary and primary holes. All grout holes will be grouted to the refusal criteria and will follow the split spacing principle. Therefore, if necessary, additional grout holes (Quatenary and Quinary holes) will be advanced following the split spacing principle to close off the grout curtain.

Details of the grouting requirements and of the split spacing principle are presented in Memorandum GN-4.2.2.7, "Grouting and Pressure Relief" [Ref. 6].

10 Tie-In Section

This section discusses the Tie-In Section for the Central Dam earthworks. This "Tie-In" is identified as Tie-In 5 and it is essential for maintaining the continuity of the impervious zone between the Central Dam and the Stage II Island Cofferdam.

10.1 Tie-In 5

Tie-In 5 will form part of a continuous impervious connection between the Central Dam and the Stage II Island Cofferdam. This will allow for construction within the central and northern channels to proceed in-the-dry during Stage II Diversion.

At the location of this connection, the Stage II Island Cofferdam will be founded on overburden, while the Central Dam will be founded on bedrock. Therefore, the Central Dam must be constructed up to the footing elevation of the Stage II Island Cofferdam prior to commencing the construction of the Tie-In. The Central Dam's impervious core will be connected to the impervious zone of the Stage II Island Cofferdam. Similarly, the rockfill and transition zones for this portion of the Central Dam will tie into the Stage II Island Cofferdam.

The Stage II Island Cofferdam will remain in place after impoundment and will become an integral part of the Central Dam.

11 Central Dam Stage I Downstream Cofferdam

The Central Dam Cofferdam will remain in place and will be incorporated as an integral part of the permanent Central Dam, acting as downstream toe berm.

Portions of the impervious and transition materials used in the construction of the Central Dam Stage I Downstream Cofferdam will be removed following completion of the Central Dam to permit seepage to exit at the lowest possible elevation. This will ensure in the long term that the seepage from the chimney drains of the Central Dam will not be obstructed by impervious fill at the downstream end. In addition, the remaining impervious blanket of the downstream cofferdam will be protected against erosion, as required, with riprap underlain with riprap bedding. This will minimize erosion of the fine-grained materials and its transportation further downstream into the Nelson River system. This should minimize the long term maintenance costs and improve the visual impact of the downstream side of the structure.

The partial removal of the cofferdam will be accomplished by use of a backhoe.

The Stage II Island Cofferdam which will tie into the Central Dam will remain in place and be submerged (as discussed in Subsection 10).

12 Disposal of Spoil Materials

Unclassified overburden materials that will be excavated to form the dam's foundation, as well as the cofferdam materials that will be removed, as discussed in Section 11, will both be disposed of in designated disposal areas either inside or outside of the forebay limits. The preferred disposal alternatives shall be finished to safe, stable lines, each to their set maximum elevation.

Prepared By

P.R. Pantel

Attach
References
Tables 1 to 9
Figures 1 to 6

B.D. Garinger

PRP:BDG:spa

References

1. KGS Acres Manitoba Limited, Memorandum GN-4.1.3, "Keeyask Generating Station – Stage IV Studies - Geotechnical Design Criteria". Manitoba Hydro File 00195-06310-0002.

2. Acres Manitoba Limited, Memorandum GN-4.2.2.11, "Keeyask Generating Station - Stage IV Studies - Earthworks Structures - Cofferdam Arrangement and Design". Manitoba Hydro File 00195-21600-0004.

3. KGS Acres Ltd., Memorandum GN-4.1.2, "Keeyask Generating Station – Stage IV Studies - Hydraulic Design Criteria". Manitoba Hydro File 00195-6330-0004.

4. KGS Acres Ltd., Memorandum GN-4.3.1, "Keeyask Generating Station – Stage IV Studies - Freeboard Requirements". Manitoba Hydro File 00195-11420-0016.

5. Canadian Dam Association, "Dam Safety Guidelines", Canadian Dam Association, Edmonton, January 1999.

6. KGS Acres Ltd., Memorandum GN-4.2.2.7, "Keeyask Generating Station – Stage IV Studies - Axis GR-4 - Grouting and Pressure Relief", Acres Manitoba File 00195-20100-0002.

7. United States Department of Agriculture, 210-VI-NEH, "Part 628 Dams National Engineering Handbook, Chapter 45 Filter Diaphragm", January, 2007.

8. United States Department of Agriculture, 210-VI-NEH, "Part 633 National Engineering Handbook, Chapter 26 Gradation Design of Sand and Gravel Filters", October, 1994.

Tables

Tables 1 to 9

Table 1
Keeyask Generating Station - Axis GR-4
Central Dam Construction Materials - Gradation Limits

US Standard Sieve Size	Limits - Percent Passing (by weight)									
	Class 1	Class 2a	Class 2b	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class RT
(1100 mm)									100	
(1000 mm)									50-100	
(900 mm)								100	30-50	
(800 mm)								75-100	0-20	
(760 mm)									0	
(650 mm)								35-50		
(600 mm)								20-40		
(550 mm)								0-15		
(500 mm)						100	100	0		
(400 mm)										
(300 mm)						70-100	50-100			
(200 mm)						60-80	30-100			
(150 mm)	100									
(100 mm)							0-80			
75 mm	98-100	100	100		100					
37.5 mm	95-100									
25 mm										100
19 mm	90-100	90-100	90-100	100	90-100	10-35	0-35			85-100
12.5 mm						0-27				70-100
9.5 mm				90-100	60-100					
4.75 mm	80-100	70-100	70-100	80-100	40-85	0-10	0-10			35-60
2 mm	70-100	55-100	50-100	60-100	15-60					25-45
0.85 mm					0-35					
.425 mm	50-92	10-55	0-80	15-50	0-15					10-25
.15 mm	38-80			0-10	0-7					
.075 mm	30-70	0-5	0-10	0-5						4-10
0.01 mm	10-40									
0.002 mm	0-20									

Table 2
Keeyask Generating Station - Axis GR-4
Central Dam - Construction Material Properties

Fill	Moist Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	Effective Angle of Shearing Resistance ϕ' (deg)	Effective Cohesion c' (kPa)	Hydraulic Conductivity k (cm/s)	Pore Pressure Parameter (r_u)
Central Dam						
Impervious Fill (Class 1)	19.9	20.3	33	0	10^{-5}	0.3
Granular Fill (Class 2a)	21.2	22.1	35	0	10^{-3}	0
Fine Filter (Class 3)	21.2	22.1	35	0	10^{-2}	0
Coarse Filter (Class 4)	22.2	22.7	35	0	10^{-1}	0
Riprap Bedding (Class 5)	18.7	21.1	45	0	Free Draining	0
Rockfill (Class 6)	20.1	21.4	45	0	Free Draining	0
Riprap (Class 7 and 8)	18.7	21.1	45	0	Free Draining	0
Road topping (Class RT)	22.2	22.7	35	0	Free Draining	0
Stage I/II Cofferdam						
Impervious fill (Class A), compacted above water level	19.3	19.8	30	0	10^{-4}	0.3
Impervious fill (Class A), dumped in water	17.4	17.4	22	0	10^{-4}	0.5
Filter (Transition Fill – Class B), compacted above water level	20.8	21.9	35	0	10^{-3}	0
Filter (Transition Fill – Class B), dumped in water	16.7	20.4	30	0	10^{-3}	0
Rockfill (Class C)	19.5	21.3	45	0	Free Draining	0

Table 3
Keeyask Generating Station – Axis GR-4
Central Dam – Filter Summary

Material Class	Fill			Filter			
	Description	D_{85}	D_{15}	Filter	Hydraulic Conductivity k	Minimum Thickness Required (1),(2)	Design Thickness (6)
		(mm)	(mm)	Class	(cm/s)	(m)	(m)
Class 1	Impervious Fill	0.25	<0.001	2a	10^{-3}	0.6	1.0
				3	10^{-2}	0.6	1.5
Class 2a	Pervious Granular Fill	1.18	0.10	4	10^{-1}	0.3	Varies
Class 3	Fine Filter	1.18	0.17	4	10^{-1}	0.3	Varies
Class 4	Coarse Filter	5	0.50	5	--	--	Varies
Class 5	Riprap Bedding	230	4.2	8	--	--	2.0
				--	--	--	--
Class 6	Rockfill	120	4.2	--	--	--	--
Class 8	Riprap	700	550	--	--	--	--

Notes:

(1) US Department of Agriculture, Natural Resources Conservation Service, National Engineering Handbook, Part 628 Dams, Chapter 45 Filter Diaphragm [Ref 7].

(2) Minimum thickness requirement is 0.9 m for large and high to moderate hazard embankments. For two-stage filters, the fine filter (i.e. Class 2 or 3) is to have a minimum thickness of 0.6 m and the coarse filter (i.e. Class 4) is to have a minimum thickness of 0.3 m.

(3) D_{85} based on the lower band limits of the gradation envelope.

(4) D_{15} based on the upper band limits of the gradation envelope.

(5) The thickness is measured perpendicular to the slope.

(6) The selected design for the filter thickness was determined based on construction practicality.

Table 4
Keeyask Generating Station - Axis GR-4
Central Dam - Crest and Top of Core Requirements

	Full Supply Level (FSL)[1]	Probable Maximum Flood Level (PMF)[2]	Selected for Design
Still-water elevation	159.0	160.30	-
Design Wind (AEP)	1:1000	1:2	-
Wave run-up (m)[3]	2.98	1.68	-
Wind setup (m)[3]	0.38	0.17	-
Reservoir tilt (m)[3]	0.00	0.00	-
Settlement allowance (m)	0.19	0.19	-
Foundation settlement (m)	0.0	0.0	-
Crest elevation (m)	162.55[1]	162.34[2]	**162.6**
Top of core elevation (m)	159.57	160.66	**161.0**

Notes:
(1) FSL plus wind effects (1:1000-yr wind) plus settlement allowance.
(2) PMF plus wind effects (1:2-yr wind) plus reservoir tilt and settlement allowance.
(3) Reference Memorandum GN-4.3.1, "Freeboard Requirements" [Ref 4].
(4) The elevations shown will be used as nominal elevations for the Central Dam. Additional camber will be provided in areas where the Central Dam is the highest.

Table 5
Keeyask Generating Station - Axis GR-4
Central Dam – Material Quantities for Construction

Item	Description	Unit	Quantity
Excavation	Overburden (in-the-dry)	m^3	595 200
Foundation preparation	Bedrock	m^2	54 200
Dam fill	Class 1 - Impervious Fill	m^3	653 100
	Class 2a – Pervious Granular Fill	m^3	42 500
	Class 3 - Fine Filter	m^3	82 600
	Class 4 - Coarse Filter	m^3	102 100
	Class 5 - Riprap Bedding	m^3	38 300
	Class 6 – Rockfill	m^3	544 000
	Class 8 – Riprap	m^3	55 200
	Class RT - Road Topping	m^3	5 500
	Total Central Dam Fill[1]	**m^3**	**1 523 300**
Cofferdam removal	In-the-wet (Classes A and B)	m^3	300

Note:
(1) Total Central Dam fill volume (in the above table) does not include 32 400 m^3 of Class C material from the Stage I Downstream Cofferdam.

Table 6
Keeyask Generating Station - Axis GR-4
Required Minimum Factors of Safety

Description/Condition	Required Minimum Factor of Safety
End of construction	1.3
Long term steady state at FSL	1.5
Long term steady state at DFL (1)	1.3
Normal drawdown (to 1 m below FSL)	1.4
Extreme rapid drawdown	1.1
Seismic Loading	1.0

Note:
(1) The Design Flood Level (DFL) is the PMF level selected for the design.

Table 7
Keeyask Generating Station - Axis GR-4
Water Levels in Stability Analyses

Description/Conditions	Central Dam	
	Upstream Water Elevation (m)	Downstream(3) Water Elevation (m)
End of Construction	149.5	142.5
Long Term Steady State at FSL	159.0	142.5
Steady Stage at DFL	160.3	142.5
Normal Drawdown (to 1 m below FSL)	158.0	142.5
Extreme Rapid Drawdown (1)	143.0	142.5
Seismic Loading (2)	159.0	142.5

Notes:
(1) The upstream elevation shown is the upstream water elevation which results in the lowest factor of safety.
(2) Seismic Loading is discussed in Subsection 7.1.1.
(3) Assumed Tailwater elevation. (Actual values will vary between el 140.1 and el 142.5 m depending on Stephens Lake elevation and whether or not flow is passing through the Spillway.)

Table 8
Keeyask Generating Station - Axis GR-4
Central Dam – Section Founded on Bedrock
Results of Slope Stability Analyses
(Morgenstern-Price Method)

Slope	Condition	Minimum FOS Required	Calculated Minimum FOS	Remarks
Upstream 2H:1V	End of construction	1.3	1.8	With FSL at el 159.0 m
	Steady state at FSL	1.5	1.7	--
	Steady state at DFL	1.3	1.8	DFL at el 160.3 m (= PMF)
	Normal drawdown	1.4	1.8	Drawdown from el 159.0 to el 158.0 m
	Extreme rapid drawdown	1.1	1.6	Drawdown from el 159.0 to el 146.5 m
	Seismic Loading	1.0	1.4	Seismic coefficients set at a_h = 0.05 and a_v = 0
		n/a (1)	1.2	Seismic coefficients set at a_h = 0.10 and a_v = 0
Downstream 1.75H:1V	End of construction	1.3	1.5	With FSL at el 159.0 m
	Steady state at FSL	1.5	1.6	--
	Steady state at DFL	1.3	1.6	--
	Seismic Loading	1.0	1.4	Seismic coefficients set at a_h = 0.05 and a_v = 0
		n/a (1)	1.3	Seismic coefficients set at a_h = 0.10 and a_v = 0

Notes:
(1) Not applicable, for screening only
(2) ah - horizontal seismic coefficient
(3) av - vertical seismic coefficient
(4) FOS - Factor of Safety
(5) DFL - Design Flood Level
(6) PMF - Probable Maximum Flood

Table 9
Keeyask Generating Station - Axis GR-4
Central Dam – Section Founded Till
Results of Slope Stability Analyses
(Morgenstern-Price Method)

Slope	Condition	Minimum FOS Required	Calculated Minimum FOS(7)	Remarks
Upstream 2H:1V	End of construction	1.3	1.7	With FSL at el 159.0 m
	Steady state at FSL	1.5	1.6	--
	Steady state at DFL	1.3	1.7	DFL at el 160.3 (= PMF)
	Normal drawdown	1.4	1.4	Drawdown from el 159.0 to el 158.0 m
	Extreme rapid drawdown	1.1	1.1	Drawdown from el 159.0 to el 146.5 m **Stabilizing berm included**
	Seismic Loading	1.0	1.3	Seismic coefficients set at a_h = 0.05 and a_v = 0
		n/a (1)		Seismic coefficients set at a_h = 0.10 and a_v = 0
Downstream 1.75H:1V	End of construction	1.3	1.3	With FSL at el 159.0
	Steady state at FSL	1.5	1.5	--
	Steady state at DFL	1.3	1.5	--
	Seismic Loading	1.0	1.1	Seismic coefficients set at a_h = 0.05 and a_v = 0
		n/a (1)	1.0	Seismic coefficients set at a_h = 0.10 and a_v = 0

Notes:
(1) Not applicable, for screening only
(2) a_h - horizontal seismic coefficient
(3) a_v - vertical seismic coefficient
(4) FOS - Factor of Safety
(5) DFL - Design Flood Level
(6) PMF - Probable Maximum Flood
(7) FOS without upstream stabilizing berm unless noted otherwise

Figures

Figures 1 to 6

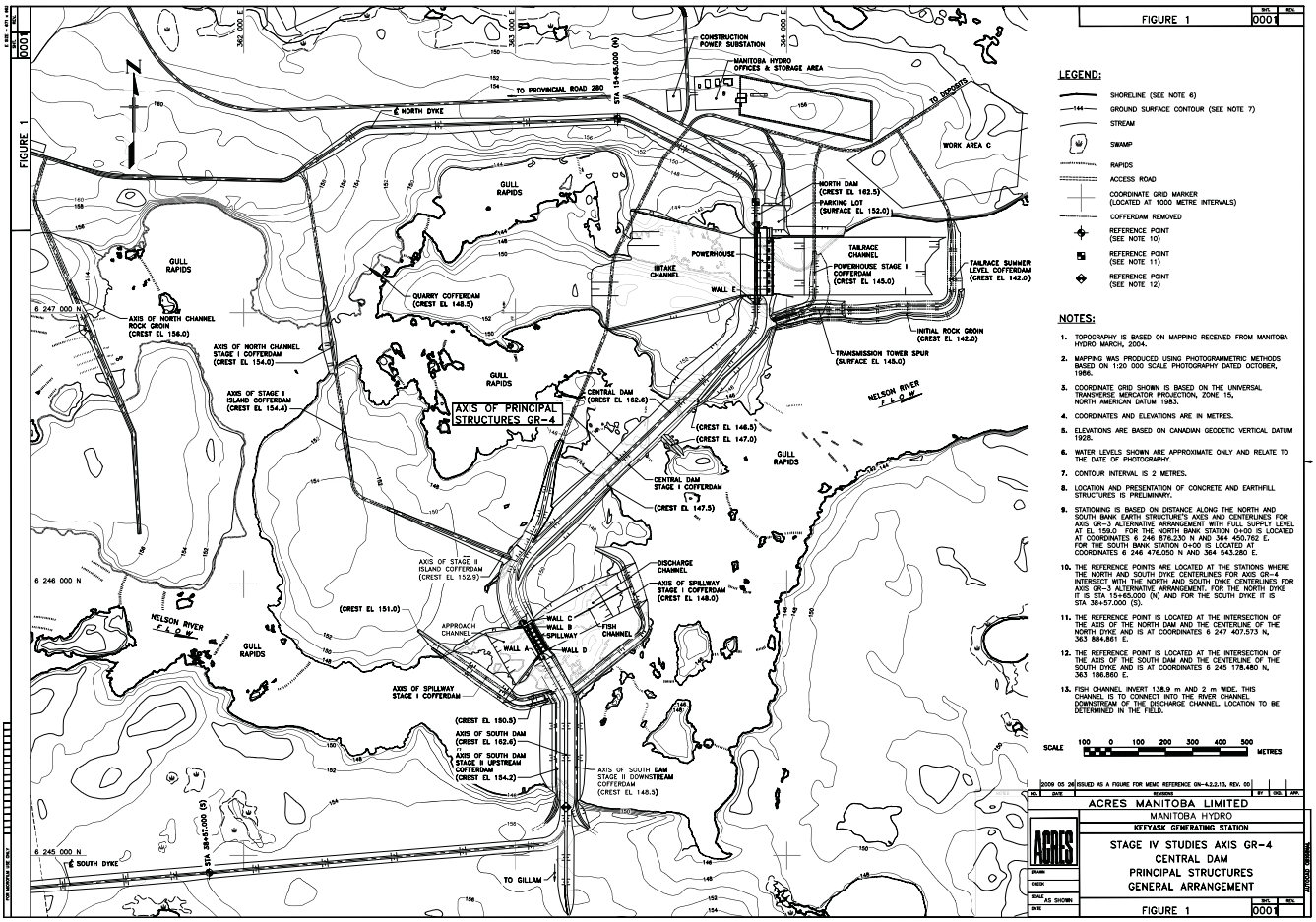


FIGURE 1 — 000

LEGEND:

- SHORELINE (SEE NOTE 6)
- GROUND SURFACE CONTOUR (SEE NOTE 7)
- STREAM
- SWAMP
- RAPIDS
- ACCESS ROAD
- COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
- COFFERDAM REMOVED
- REFERENCE POINT (SEE NOTE 10)
- REFERENCE POINT (SEE NOTE 11)
- REFERENCE POINT (SEE NOTE 12)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0 FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.782 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.

SCALE 100 0 100 200 300 400 500 METRES

NO.	DATE	REVISIONS	BY	CHD	APP'D
0	2009 05 26	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13, REV. 00			

ACRES MANITOBA LIMITED

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
CENTRAL DAM
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT

DRAWN | CHECKED | SCALE AS SHOWN | DATE

FIGURE 1 — 000

DO NOT SCALE DRAWING — AUTOCAD DRAWING

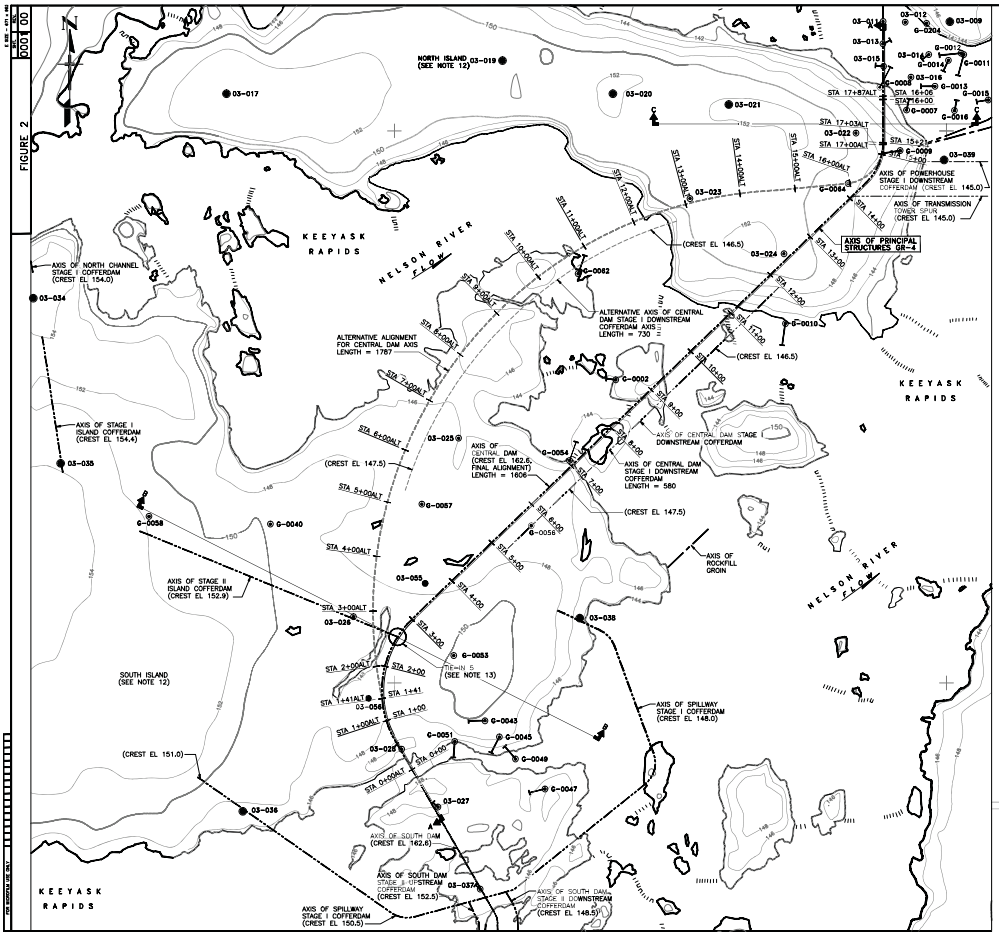


FIGURE 2 — 000 | 00

LEGEND:

- SHORELINE (SEE NOTE 6)
- 144 GROUND SURFACE CONTOUR (SEE NOTE 7)
- STREAM
- SWAMP
- RAPIDS
- DIAMOND DRILL HOLE (VERTICAL, INCLINED)
- SONIC/DIAMOND DRILL HOLE
- SONIC DRILL HOLE
- AUGER HOLE
- AUGER/DIAMOND DRILL HOLE
- TEST PIT
- × EXPLORATORY HOLE
- 03-004 2003 SUMMER EXPLORATION (SEE NOTE 8)
- G-0031 EXPLORATION HOLE, TEST PIT NUMBER
- COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
- STA 7+00 CENTRAL DAM STAIONING (SEE NOTE 11)

NOTES:

1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MAY 2000.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES, ELEVATIONS AND LENGTHS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. SUMMER 2003 EXPLORATIONS ARE SHOWN AT THEIR PROPOSED LOCATIONS.
9. EXPLORATORY HOLES HAVE BEEN REPOSITIONED USING A COMMON TOPOGRAPHIC FEATURE AS AN INSERTION POINT, ON TO THE NORTH AMERICAN DATUM 1983 COORDINATE GRID FROM THEIR ORIGINALLY PLOTTED LOCATION ON THE NORTH AMERICAN DATUM 1927 COORDINATE GRID. AS A RESULT, THE HOLES MAY HAVE SLIGHTLY DIFFERENT COORDINATES THAN THE FIELD SURVEYED COORDINATES.
10. FOR GEOLOGICAL SECTION A–A SEE FIGURE 3.
11. CENTRAL DAM STATIONING IS MARKED IN HUNDREDS OF METRES, AND IS MEASURED STARTING FROM SOUTH END OF THE SPILLWAY NORTH TRANSITION STRUCTURE.
12. FOR IDENTIFICATION ONLY, SEE TEXT.
13. TIE–IN 5 IS THE IMPERVIOUS TIE–IN OF THE STAGE II ISLAND COFFERDAM TO THE CENTRAL DAM.

SCALE 1:2500 — 50 0 50 100 150 200 250 METRES

NO.	DATE	REVISIONS	BY	CHK	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN–4.2.2.13	RMR	PRP	NJS

ACRES MANITOBA LIMITED

MANITOBA HYDRO

KEEYASK GENERATING STATION

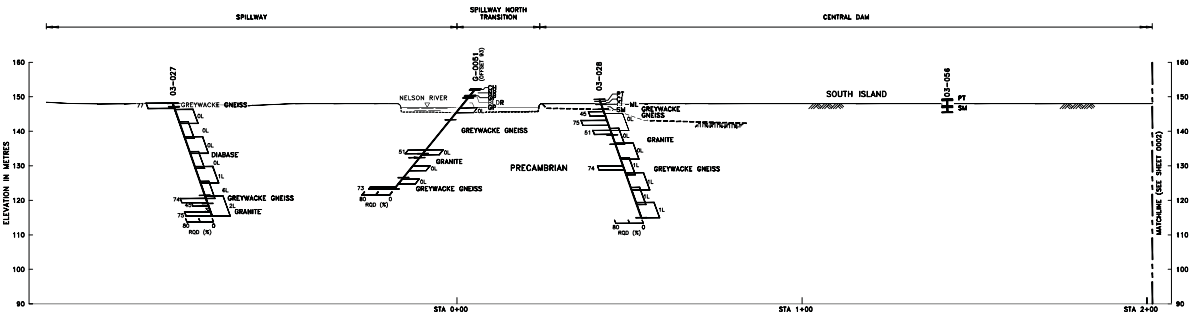
STAGE IV STUDIES – AXIS GR–4
CENTRAL DAM
LOCATION OF EXPLORATIONS AND GEOLOGICAL SECTIONS

DESIGN	RR
CHECK	RMR
SCALE	AS SHOWN
DATE	2009 02 06

FIGURE 2 — 000 | 00

DO NOT SCALE THIS DRAWING

FIGURE 3 — REV. 001 00



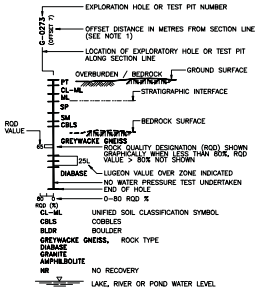
SECTION A–A

NOTES:

1. OFFSET DISTANCE SHOWN IF GREATER THAN 5 METRES FROM SECTION LINE.
2. FOR LOCATION OF GEOLOGICAL SECTIONS SEE FIGURE 2.
3. FOR LOCATION OF CENTRAL DAM STATIONING SEE FIGURE 2.
4. WHENEVER POSSIBLE THE GROUND SURFACE PROFILES WERE ADJUSTED TO CONFORM TO THE INFORMATION FROM THE DRILL HOLES OR TEST PITS WHICH ARE WITHIN 5 METRES FROM THE SECTION LINES AND THEREFORE THEY MAY DIFFER FROM THAT SHOWN BY THE CONTOURS ON FIGURE 2.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.

LEGEND:

EXPLORATION HOLE



- EXPLORATION HOLE OR TEST PIT NUMBER
- OFFSET DISTANCE IN METRES FROM SECTION LINE (SEE NOTE 1)
- LOCATION OF EXPLORATORY HOLE OR TEST PIT ALONG SECTION LINE
- GROUND SURFACE
- STRATIGRAPHIC INTERFACE
- BEDROCK SURFACE
- ROCK QUALITY DESIGNATION (ROD) SHOWN GRAPHICALLY WHEN LESS THAN 80%, ROD VALUE > 80% NOT SHOWN
- LUGEON VALUE OVER ZONE INDICATED
- NO WATER PRESSURE TEST UNDERTAKEN
- END OF HOLE
- 0–80 ROD %
- CL–ML — UNIFIED SOIL CLASSIFICATION SYMBOL
- CBLS — COBBLES
- BLDR — BOULDER
- GREYWACKE GNEISS, DIABASE, GRANITE, AMPHIBOLITE — ROCK TYPE
- NR — NO RECOVERY
- LAKE, RIVER OR POND WATER LEVEL

SOIL CLASSIFICATION SYMBOLS:
BASED UPON THE UNIFIED SOILS CLASSIFICATION SYSTEM

PT	PEAT, MUSKEG, HIGHLY ORGANIC SOILS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
CI	INORGANIC CLAYS OF INTERMEDIATE PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS
OL	ORGANIC SILTS AND ORGANIC SILT–CLAYS OF LOW PLASTICITY
CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS WITH SLIGHT PLASTICITY
SC	CLAYEY SANDS, POORLY GRADED SAND–CLAY MIXTURES
SM	SILTY SANDS, POORLY GRADED SAND–SILT MIXTURES
SP	POORLY GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
SW	WELL GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES
GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL–SAND–CLAY MIXTURES
GM	SILTY GRAVELS, POORLY GRADED GRAVEL–SAND–SILT MIXTURES
GP	POORLY GRADED GRAVELS, GRAVEL–SAND MIXTURES, LITTLE OR NO FINES
GW	WELL GRADED GRAVELS, GRAVEL–SAND MIXTURES; LITTLE OR NO FINES
CL–ML, GP–GC, SP–SM, ETC.	TYPICAL BORDER LINE CLASSIFICATION REQUIRING THE USE OF DUAL SYMBOLS

FROZEN GROUND SYMBOLS:

Frozen	SEASONAL FROST
Inf	POORLY BONDED OR FRIABLE ICE
Nbn	WELL BONDED, NO EXCESS ICE
Nbe	WELL BONDED, EXCESS ICE
Vx	INDIVIDUAL ICE CRYSTALS OR INCLUSIONS
Vc	ICE COATINGS ON PARTICLES
Vr	RANDOM OR IRREGULARLY ORIENTED ICE FORMATIONS
Vs	STRATIFIED OR DISTINCTLY ORIENTED ICE FORMATIONS
Ice + UNIFIED SOILS CLASSIFICATION	ICE WITH SOIL INCLUSIONS
Ice	ICE WITHOUT SOIL INCLUSIONS

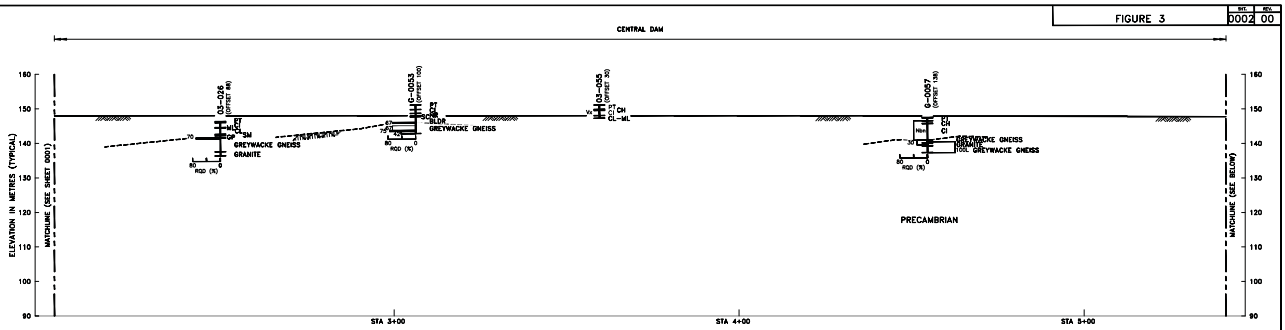
SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	BY	CHD	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RMR	PRP	NJS

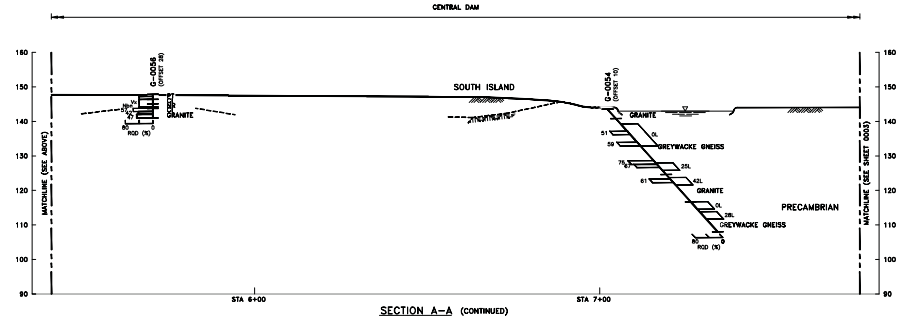
ACRES MANITOBA LIMITED
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES – AXIS GR-4
CENTRAL DAM
GEOLOGICAL SECTIONS

DRAWN RR/RCA
CHECK RMR
SCALE AS SHOWN
DATE 2009 02 06

FIGURE 3 — REV. 001 00



SECTION A–A (CONTINUED)



SECTION A–A (CONTINUED)

NOTE:

1. FOR LEGEND AND NOTES SEE SHEET 1.

SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	BY	CHD	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RWR	PRP	NJS

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MANITOBA HYDRO

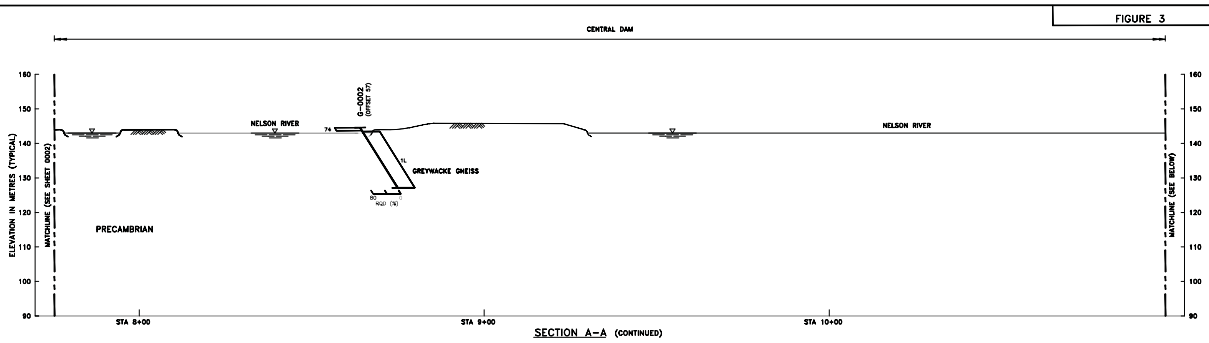
KEEYASK GENERATING STATION

STAGE IV STUDIES – AXIS GR-4
CENTRAL DAM
GEOLOGICAL SECTIONS

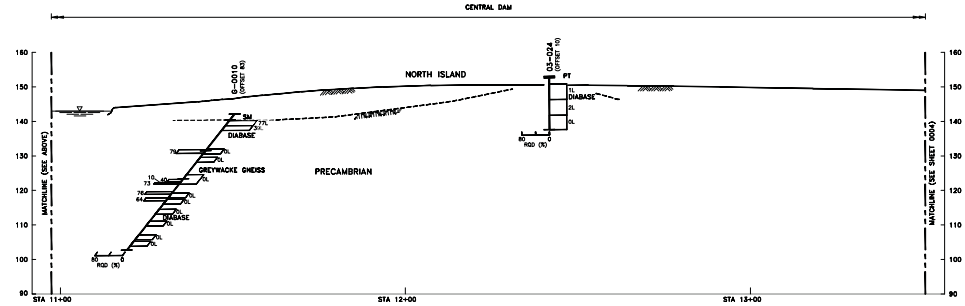
DRAWN RR/RCA
CHECK RWR
SCALE AS SHOWN
DATE 2009 02 06

FIGURE 3 — DWG. 0002 — REV. 00

FIGURE 3 — 0003 00



SECTION A–A (CONTINUED)



SECTION A–A (CONTINUED)

NOTE:

1. FOR LEGEND AND NOTES SEE SHEET 1.

SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	BY	CHK	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	PRP	NJS

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MANITOBA HYDRO

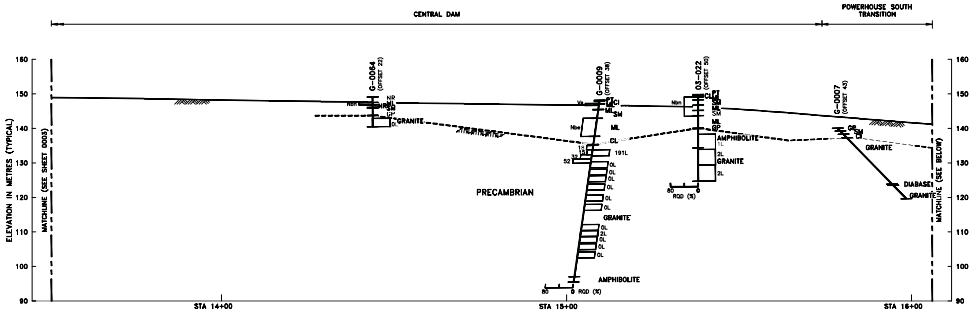
KEEYASK GENERATING STATION

STAGE IV STUDIES – AXIS GR-4
CENTRAL DAM
GEOLOGICAL SECTIONS

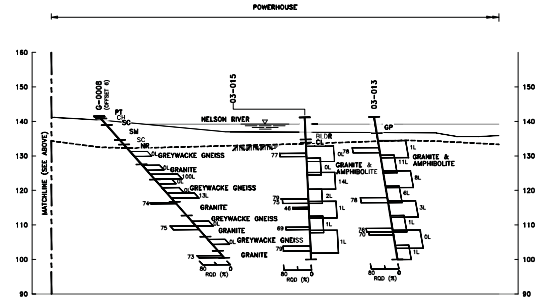
DRAWN RR/RCA
CHECK RAR
SCALE AS SHOWN
DATE 2009 02 06

FIGURE 3 — DWG. 0003 — REV. 00

FIGURE 3 | DOC. 0004 | REV. 00



SECTION A–A (CONTINUED)



SECTION A–A (CONTINUED)

NOTE:

1. FOR LEGEND AND NOTES SEE SHEET 1.

SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	BY	CHD	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	PRP	NJS

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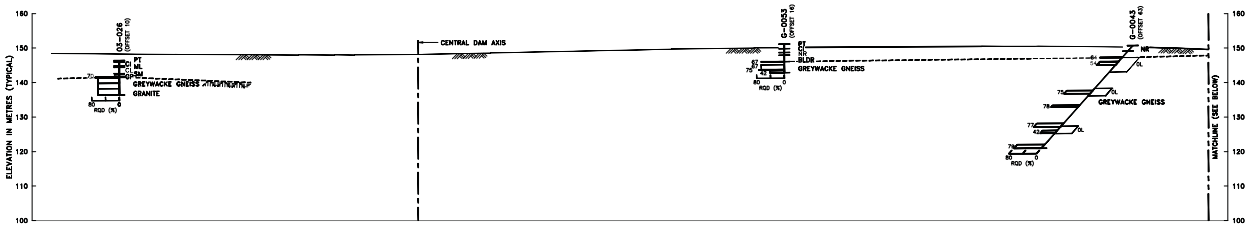
KEEYASK GENERATING STATION

STAGE IV STUDIES – AXIS GR-4
CENTRAL DAM
GEOLOGICAL SECTIONS

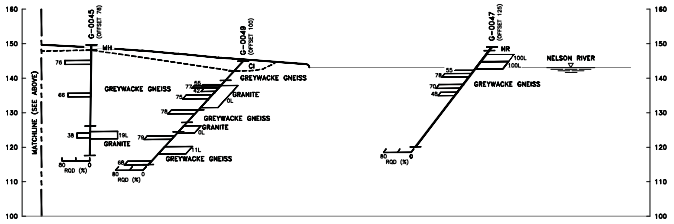
DESIGN	RR/RCA
CHECK	RAR
SCALE	AS SHOWN
DATE	2009 02 06

FIGURE 3 | DOC. 0004 | REV. 00

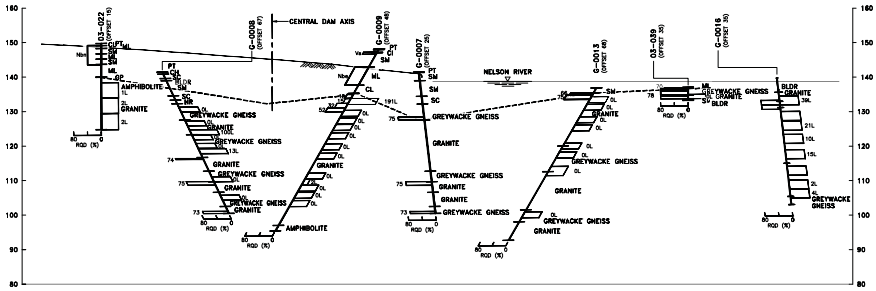
FIGURE 3 | REV. 0005 00



SECTION B–B



SECTION B–B (CONTINUED)



SECTION C–C

NOTE:

1. FOR LEGEND AND NOTES SEE SHEET 1.

SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	BY	CHK	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	PRP	NJS

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KEEYASK GENERATING STATION

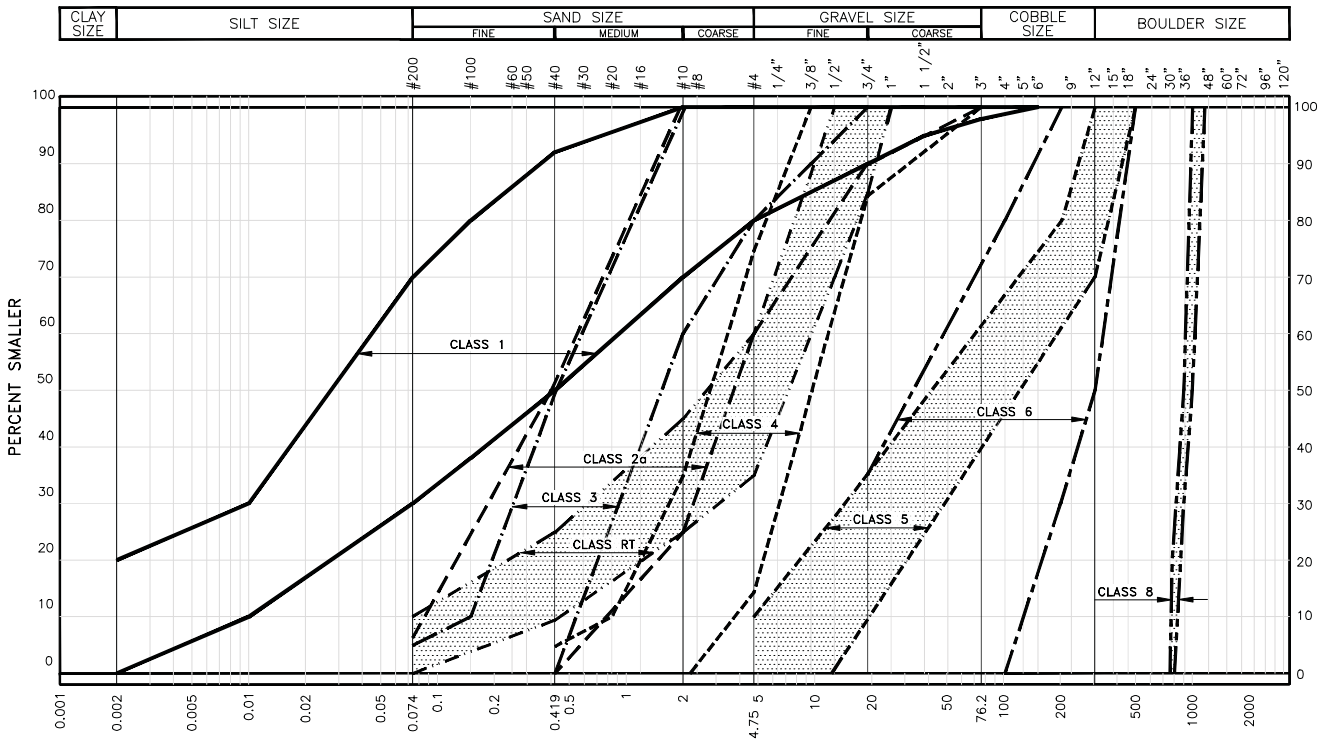
STAGE IV STUDIES – AXIS GR-4
CENTRAL DAM
GEOLOGICAL SECTIONS

DESIGN	RR/SSZ/RCA
CHECK	RAR
SCALE	AS SHOWN
DATE	2009 02 06

FIGURE 3 | REV. 0005 00

FIGURE 4

SHT.	REV.
0001	00



LEGEND:

- CLASS 1
- CLASS 2a
- CLASS 3
- CLASS 4
- CLASS 5
- CLASS 6
- CLASS 8
- CLASS RT

NO.	DATE	REVISIONS	BY	CKD.	APP.
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	PRP	NJS

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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
CENTRAL DAM
CONSTRUCTION MATERIALS
GRADATION LIMITS

DRAWN	RR
CHECK	RAR
SCALE	AS SHOWN
DATE	2009 02 06

FIGURE 4

SHT.	REV.
0001	00

AUTOCAD ORIGINAL

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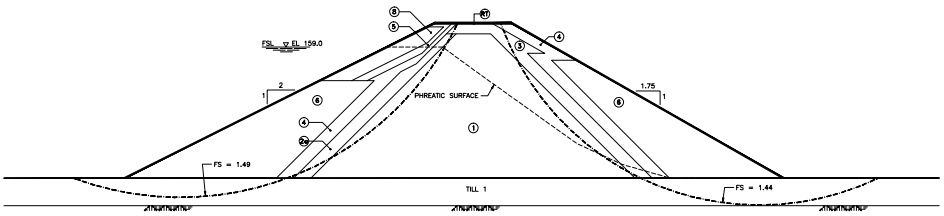
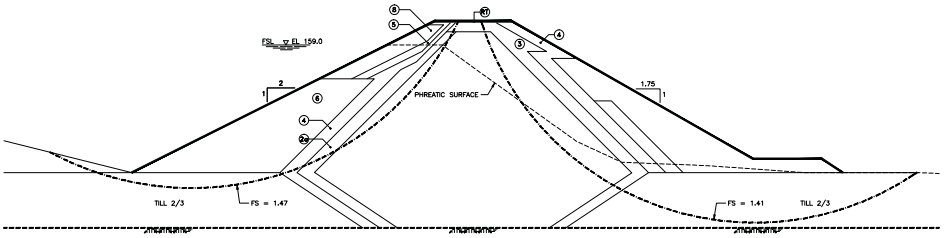
FIGURE 5 — 000 | 00



A) OPTION 1 – FOUNDATION ON TILL



B) OPTION 2 – IMPERVIOUS CORE ON BEDROCK

LEGEND:

1	IMPERVIOUS FILL
2a	GRANULAR FILL
3	FILTER
4	TRANSITION FILL
5	RIPRAP BEDDING
6	ROCKFILL
8	RIPRAP
RT	ROAD TOPPING
FS	FACTOR OF SAFETY
FSL	NORMAL FULL SUPPLY LEVEL
	BEDROCK SURFACE

STRENGTH PARAMETERS SUMMARY

MATERIAL	UNIT WEIGHT MOIST	UNIT WEIGHT SATURATED	COHESION (kPa)	FRICTION ANGLE (DEGREES)
TILL 1	18.1	18.1	0	33
TILL 2/3	18.1	18.1	0	30
CLASS 1	19.9	20.3	0	33
CLASS 2a	21.2	22.1	0	35
CLASS 3	21.2	22.1	0	35
CLASS 4	22.2	22.7	0	35
CLASS 5	18.7	21.1	0	45
CLASS 6	20.1	21.4	0	45
CLASS 8	18.7	21.1	0	45
CLASS RT	22.2	22.7	0	35

NOTES:

1. STABILITY ANALYSES FOR ALL OPTIONS SHOWN ARE FOR STEADY STATE CONDITIONS AT FULL SUPPLY LEVELS, FOR WHICH A MINIMUM SAFETY FACTOR OF 1.5 IS REQUIRED FOR BOTH UPSTREAM AND DOWNSTREAM SLOPES.
2. ELEVATIONS ARE IN METRES.
3. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.

SCALE 1:200 — 5 0 5 10 15 20 METRES

NO.	DATE	REVISIONS	BY	CHD	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RWR	PRP	NJS

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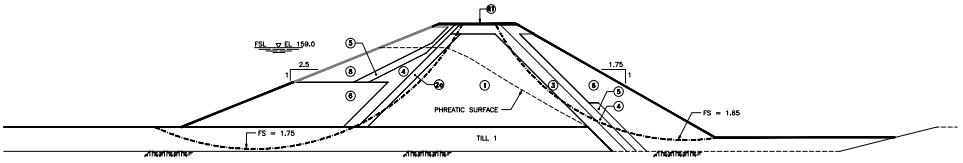
KEEYASK GENERATING STATION

STAGE IV STUDIES
AXIS GR-4
CENTRAL DAM
RESULTS OF STABILITY ANALYSES

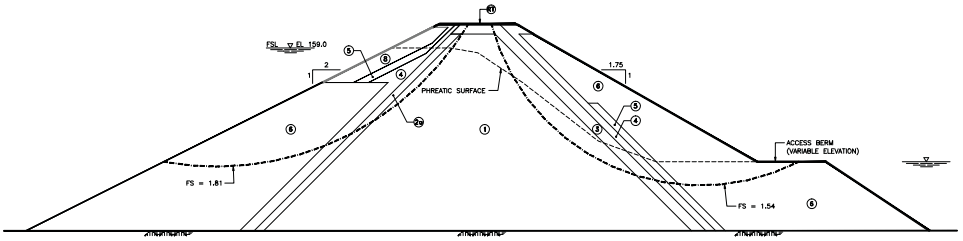
FIGURE 5 — 000 | 00

DESIGN LMA/WR
CHECK RWR
SCALE AS SHOWN
DATE 2009 02 06

FIGURE 5 0002 00



C) OPTION 3 – IMPERVIOUS CORE AND UPSTREAM SHELL ON TILL



D) OPTION 4 – FOUNDATION ON BEDROCK

LEGEND:

①	IMPERVIOUS FILL
②a	GRANULAR FILL
③	FILTER
④	TRANSITION FILL
⑤	RIPRAP BEDDING
⑥	ROCKFILL
⑧	RIPRAP
⑪	ROAD TOPPING
FS	FACTOR OF SAFETY
	BEDROCK SURFACE

NOTE:

1. FOR NOTES SEE SHEET 0001.

SCALE 1:500 — 5 0 5 10 15 20 METRES

NO.	DATE	REVISIONS	BY	CHD	APP
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	PRP	NJS

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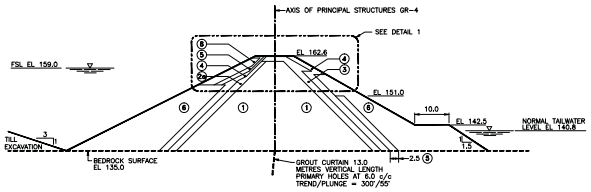
KEEYASK GENERATING STATION

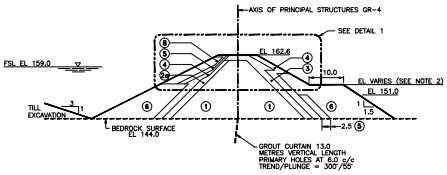
STAGE IV STUDIES
AXIS GR-4
CENTRAL DAM
RESULTS OF STABILITY ANALYSES

FIGURE 5 — DWG 0002 00

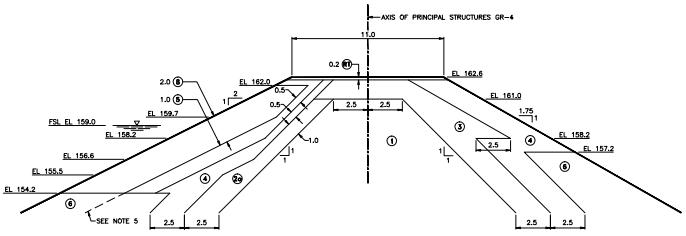
DESIGN LMA/SSZ
CHECK RAR
SCALE AS SHOWN
DATE 2009 02 06



CENTRAL DAM NEAR POWERHOUSE
SCALE 1:400



CENTRAL DAM NEAR SPILLWAY
SCALE 1:400



DETAIL 1
SCALE 1:100

LEGEND:

1	IMPERVIOUS FILL
2A	PERVIOUS GRANULAR FILL
3	FINE FILTER
4	TRANSITION – CRUSHED ROCK/COARSE FILTER
5	RIPRAP BEDDING/FINE ROCKFILL
6	ROCKFILL
8	RIPRAP
RT	ROAD TOPPING
	WATER LEVEL
FSL	NORMAL FULL SUPPLY LEVEL

NOTES:

1. WHEN POSTGLACIAL CLAYS ARE 3 METRES THICK OR LESS USE 1 VERTICAL TO 3.5 HORIZONTAL AND, WHEN POSTGLACIAL CLAYS ARE GREATER THAN 3 METRES THICK USE 1 VERTICAL TO 4 HORIZONTAL.
2. DOWNSTREAM TOE BERM TO BE USED FOR ACCESS TO SPILLWAY BRIDGE DECK.
3. ELEVATIONS AND DIMENSIONS ARE IN METRES.
4. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
5. CLASS (8) PLACED ALONG FULL LENGTH OF UPSTREAM SLOPE AT CONCRETE STRUCTURES TIE-INS.

SCALE 1:100 — 2 0 2 4 6 8 10 METRES

SCALE 1:400 — 10 0 10 20 30 40 METRES

NO.	DATE	REVISIONS	INIT.	PREP	AUS
00	2009 02 06	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.13	RAR	BP	AUS

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MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES
AXIS GR-4
CENTRAL DAM
TYPICAL SECTIONS

DRAWN	RR
CHECK	RAR
SCALE	AS SHOWN
DATE	2009 02 06

FIGURE 6 — DWG 0001 REV 00

FOR MANITOBA HYDRO USE ONLY

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.16
Rev. 0, April 19, 2012

South Side Containment Dykes
Manitoba Hydro File 00195-21200-0013_01

Prepared by J.J. Rogers (J. ROGERS) APRIL 19, 2012
KGS Acres Ltd.

Checked by P. Pantel (P. PANTEL) April 19, 2012
KGS Acres Ltd.

Approved by I.K. Dewar (I. K. Dewar) April 19, 2012
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	April 19, 2012
		File No.	P10008(16).36
From	D.E. Ranta	**cc**	P. Pantel
Subject	**Keeyask Generating Station** **Stage IV Studies – Axis GR-4** **South Side Containment Dykes** **Memorandum GN-4.2.2.16, Rev 0** **Manitoba Hydro File 00195-21200-0013_01**		

1 Introduction

The South Side Containment Dykes discussed herein are structures that may need to be constructed depending upon river flows and field observations made during construction.

The dykes have been identified as potentially being required as the top elevation of mineral soils in the affected areas may be below the water levels in Gull Lake. The purpose of the dykes would be to block potential subsurface seepage which is expected after the North Channel Rock Groin is built. The water levels in Gull Lake will rise approximately 0.86 to 1.1 m relative to their natural elevation following construction of the North Channel Rock Groin.

Three areas of concern for potential seepage, or overtopping, have been identified on the south bank of the river. These areas are designated as, “Area 1”, “Area 2” and “Area 3”. Within Area 1 the locations for two (2) potential containment dykes are identified. One (1) potential containment dyke is identified for Area 2, and one (1) in Area 3. The proposed containment dykes (4 in total) are shown on Figure 1.

This memorandum summarizes the preliminary design and first order construction cost estimates for four containment dykes located on the south bank of the Nelson River (i.e. South Side Containment Dykes). In addition, we have incorporated and provided comments on the results of the field exploration program undertaken in August 2011.

2 Design Parameters and Assumptions

The North Channel Rock Groin is to be constructed to raise water levels upstream of the site to facilitate the Ice Boom in forming an ice cover, as well as to allow for construction of the North Channel Stage I Cofferdam. This increase in water level on the downstream end of Gull Lake may require the construction of containment dykes along the south shore of the lake. One-dimensional numerical Hec-Ras models originally developed for the Stage I and Stage II Diversion were utilized to predict water levels in the downstream end of Gull Lake for the design flow of a 1:20 year peak daily discharge. Forecasts for 1:50 year and 1:100 year flows were also modelled, to provide a sensitivity of water levels to changes in flows. The water levels for the three areas of interest along the South Shore of Gull Lake are provided in Figure 1.

The South Side Containment Dykes are assumed to have maximum height of 1.5 m above anticipated water levels for a 1:20 return period [Ref 1]. The design crest elevations for the containment dykes vary between el 156.65 m and 157.10 m depending on the anticipated water level for each respective location. The crest width has been set at 6 m with an upstream slope of 6:1 and a downstream slope of 5:1.

For Area 1-Dyke 1 and Area 1-Dyke 2, fill material can be obtained locally, adjacent to the dykes. For Area 2-Dyke 1 and Area 3-Dyke 1, the fill material can be obtained from the currently proposed impervious deposits, Deposits S-18 and S-17, respectively.

Note:

During the fall of 2011, the project site experienced flow conditions on the Nelson River near the 1:20 year design flood. Although peak flows in late August of 2011 were very close to the 1:40 year return of 6538 cms, no adverse effects were observed under these conditions. With the North Rock Groin in place under the same flow conditions water levels could be up to 1.0 m higher.

Review of Depth of Peat

The estimated depth to mineral soil or bedrock at the containment dyke locations, as utilized in this analysis, is based on 50^{th} and 95^{th} percentile values as presented in a memorandum by ECOSTEM Ltd., dated May 31, 2011 titled, "Preliminary Analysis of Peat and Water Thickness in the Containment Dyke Areas" [Ref 2].

For the purposes of this memorandum, the average of the above two values (50^{th} and 95^{th} percentiles) has been used for estimating the volume of peat at the dyke locations. The 50^{th} percentile depths were used in calculating the depth of stripping required at the deposit locations. This assumption is considered appropriate at this stage of design.

The depths and volume of peat for removal is noted in Table 1. It should be noted that if the 95th percentile peat depth is encountered for the full area of the dykes, the construction costs will increase significantly as a result of the additional excavation and fill required.

The above information on the possible depth of peat was further augmented by the results of the 2011 field investigation program. The field investigation program was undertaken based on recommendations made in the report titled "Draft Southside Containment Borehole and LIDAR Summary – Review and Conclusion" [Ref 3] and in subsequent meetings with Manitoba Hydro, KGS Acres and ECOSTEM. The field exploration program was completed in August/September of 2011. A total of 45 hand auger holes were drilled. Thirty-seven (37) hand auger holes were advanced along the proposed alignments of the containments dykes and eight (8) hand auger holes were advanced to define a possible source of mineral soils for construction of the containment dykes in Area 1.

The following is a brief summary of the findings of the hand auger holes advanced at each dyke location.

Area	Depth of Peat Observed - Summer 2011				
	Minimum		Maximum		Average Thickness (m)
	Hole #	Thickness (m)	Hole #	Thickness (m)	
Area 1 Dyke 1	HA-12	0.14	HA-15	0.68	0.43
Area 1 Dyke 2	HA-20	0.20	HA-21	1.70	0.87
Area 2 Dyke 1	HA-34	0.30	HA-31	3.62	0.98
Area 3 Dyke 1	HA-40	0.30	HA-43 HA-44	1.90 1.80	0.88

The profile of the top of mineral soil for each area is illustrated in Graphs 1 to 4 in Appendix A. The graphs show the ground surface, top of mineral soils, the end of the hand auger hole and the four water levels; 1:20 existing, and the associated water levels for the 1:20, 1:50 and 1:100 peak daily discharges with the North Channel Rock Groin in place.

Based on the results of the field investigations and the estimated water levels for the various peak daily discharges, discussed above, the 1:20 year water levels with the North Rock Groin in place are lower than the ground surface elevation. The top of mineral soils

in 3 of the sites was observed to dip below the water level associated with the peak daily discharge for the 1:20 year flow with the rock groin in place. A summary of the field data collected is provided below:

- For all areas the top of ground surface was observed to be above the peak daily water level for the 1:20 year flow with the North Channel Rock Groin in Place. Generally, the top of mineral soil was observed to be above the peak daily average water level for the 1:20 year flow. However, the hand auger hole, HA-31, advanced in Area 2 Dyke 1, shows the top of mineral soil to be near the peak daily average water level for the 1:20 year case (see Graph 3).
- Area 1 Dyke 1 (Graph 1) - The top of mineral soil observed generally rests above the water level for the 1:100 year flow with the North Channel Rock Groin in place. The exception being the observed top of mineral soil in HA-15 which rests above the 1:50 year water level (see Graph 3). Based on the field observation at Area1 Dyke 1, this area is not anticipated to be problematic.
- Area 1 Dyke 2 (Graph 2) - The top of mineral soil has a U-shaped profile which dips below the water level for the 1:20 year flow with the North Channel Rock Groin in place. The maximum depth of water is estimated to be in the order of 0.8 m at HA-21.
- Area 2 Dyke 1 (Graph 3) – The mineral soil profile dips sharply at HA-31 and HA-33. The top of the mineral soils at HA-31 was observed at el 154.4 m which is slightly below the 1:20 year flow for the existing conditions. The top of mineral soil at HA-33 is at or above the 1:20 year flow with the North Channel Rock Groin in place. For HA-31, the maximum depth of water for the 1:20 year flow with the North Channel Rock Groin in place is in the order of 1.1 m.
- Area 3 Dyke (Graph 4) – The mineral soil profile dips below the water level for the 1:20 year flow with the North Channel Rock Groin. The maximum depth of water for the 1:20 year flow with the North Channel Rock Groin in place is in the order of 0.5 m.
- The hand auger holes were space on average about 50 m apart along the axis of each containment dyke. We note that it is possible that some low points may exist between the hand auger holes advanced. However, the trend of the top of the mineral soils appears to follow the trend of the ground surface in most cases, the exception being at Area 2 Dyke 2 and Area 3 Dyke 1 (see Graphs 3 and 4).

Fill Materials

If, during construction, the containment dykes are deemed to be necessary the work would be undertaken in the winter months, when the fill materials will be frozen. Thus, settlement and consolidation of both the embankment and foundation is expected to some

extent. For the estimates presented herein, a factor of 10% was added to the calculated volumes to account for the expected settlement and a factor of 30% was added for consolidation, as noted in Table 2. Similar to the peat removal, the average of the 50th and 95th percentile values was used in calculating the volume of fill material required.

A monitoring program and some maintenance to top up the structures is anticipated to be required. The 10% consolidation factor noted above was also considered to allow for the expected maintenance in the first year of operation as there would be no access for equipment during this time period. The South Access Road could be utilized in the remaining years for equipment access should any settlement occur after the first year of operation.

3 Construction Methodology

The methodology for constructing the subsurface dykes takes into consideration access to the dyke locations and the working conditions when constructing the dykes.

There would be no road access to the dyke locations on the south shore prior to the requirement for the permanent dykes to be constructed. Thus, mobilization of equipment to the site would need to be by winter road from the north shore over the river ice, overland on a winter road on the south shore from the Butnau Dam, barge during summer months, or helicopter transport.

The containment dykes would be constructed by placing unclassified mineral soils sourced in the vicinity of the dykes. Road topping, granular filters and riprap armouring would not be required.

There are two approaches to constructing the subsurface dykes. One approach is to excavate a trench and backfill with impervious materials. The second is to build an embankment on top of the peat, thereby causing the peat to become compressed, to minimize and possibly cut off any seepages. Both options are discussed below.

3.1 Option 1 – Cut-off Trench

Option 1 consists of excavating a cut-off trench and backfilling it with impervious soils. The base width of the cut-off trench has been set to 3 m with a cut slope of 2:1. The base width for the cut-off trench was selected as the minimum desirable core contact on the foundation to mitigate against seepage using impervious mineral soils. However, the cut slope as well as the base width should be re-evaluated during construction.

Construction of the dykes is assumed to require excavating peat to an average depth ranging from 0.75 m to 2.8 m over the four dyke locations. Peat, by nature, is a soft material that will excavate relatively easily as compared to a granular material. However, the equipment loads on the peat will cause significant settlement from the movement of

the equipment and will hinder the productivity of the excavation process. The equipment could become stuck due to the settlement of the peat. To mitigate the settlement a granular road/pad could be constructed as a firm base for the equipment. However, this comes at a considerable expense. A reasonable alternative to address site access and a stable working platform would be to perform the work during the winter months when the peat is frozen and the settlement due to equipment loads is expected to be minimal.

The excavation of the peat from the dyke locations and the stripping of peat from the borrow locations is proposed to be done simultaneously. For the Area 1 – Dyke 1 and Area 2 – Dyke 2 locations, the borrow location of Deposit S-18, and adjacent to Area 1-Dyke 2, the peat is proposed to be side-cast during the initial excavation. After the completion of the dykes the peat would be transported to locations above the high water level south of the South Dyke location. The peat excavated from Area 2 – Dyke 1 and Area 3 – Dyke 1 locations and deposit location S-17a could be set above the high water level south of the South Dyke location without additional trucking to move the material.

A typical cross-section of the preliminary design of the South Side Containment Dykes is presented in Figure 2.

3.2 Option 2 – Earthfill Embankment Dyke to Compress the Peat

Option 2 consists of constructing an earthfill embankment on top of the peat to compress it in an effort to minimize and possibly cut off any seepage. This option is the preferred approach for constructing the containment dykes in the spring, summer or early fall. It is noted that this option is not the preferred approach for winter conditions, as the ground will be frozen and therefore the peat will likely not compress until the ground thaws, which may not occur until late spring, or summer.

Construction of the dykes under this option does not require any peat excavation. Earthfill material would be placed on top of the peat by advancing the dyke along the proposed alignment. The dyke would also serve as a relatively stable working platform for the construction equipment. Peat, by nature, is a soft material that is readily compressible. Hence, mineral soil materials would be placed continuously until the settlement of the embankment has stabilized. Equipment loads in addition to the placed earthfill would cause significant consolidation of the peat.

3.3 Construction Schedule

Scenario 1 – Proactive measures

For this scenario, a decision would be made to construct the containment dykes in the winter months prior to the construction of the North Channel Rock Groin, although it is recognized that this may be problematic with respect to the date which construction of the GS is scheduled to be approved.

Scenario 2 – Monitor and construct dykes if required

For this scenario, the approach would be to monitor the sites during construction. The monitoring program should start prior to the advancement of the North Channel Rock Groin. If the monitoring program indicates that water seepage through any of these areas is problematic then a field crew can be mobilized to the south side to construct the containment dykes. The construction technique would consist of either Option 1 or Option 2 as discussed in Subsection 3.1 and 3.2. The decision on which option to utilize would depend on the site conditions and the time of year the work needs to be undertaken.

3.4 Monitoring Program

The following discussion presents a first order monitoring plan. The primary objective of the monitoring plan is to obtain representative groundwater levels and lake levels of interest to the sites discussed herein. Field data collected from the monitoring program will assist the project team during construction in assessing the site conditions at various stages throughout construction and provide valuable data for determining if and when containment dykes are needed.

A properly designed monitoring plan, complete with the design of the monitoring wells and instrumentation should be developed during final design. The monitoring program should provide the following: groundwater level, ability to collect representative samples of groundwater that can be analyzed to determine physical properties and compare it to the physical properties of the water from the Nelson River (optional, if deemed required), measurement of the Gull Lake water levels immediately in the vicinity of the areas of interest, data on head loss between monitoring wells from the upstream side to the downstream side of the areas of interest.

The monitoring plan should consider the following:

- The use of monitoring wells. The location of these wells should be planned out in consultation with Manitoba Hydro, KGS Acres and ECOSTEM. Ideally, the wells should be installed at locations where the greatest depths to the top of the mineral soils have been observed. There are currently six (6) sites of interest for installation of monitoring wells. These wells should be installed adjacent to, or in the vicinity of, the lowest point at the top of mineral soils observed in the 2011 field investigation program:
    - Area 3 Dyke 1 – Auger Hole HA-39 and HA-44
    - Area 2 Dyke 2 – Auger Hole HA-31 and HA-33
    - Area 1 Dyke 1 – Auger Hole HA-15
    - Area 1 Dyke 2 – Auger Hole HA-21

- Consider installing monitoring wells on both the upstream and downstream sides of the dyke alignment for all four sites. Locate the wells in areas where they will not be disturbed by construction activities for both the containment dykes and the South Dyke.
- Consider advancing hand auger holes spaced 25 m on either side of the of the 2011 hand auger holes where the greatest depth to top of mineral soil was observed. These can be advanced at the time the monitoring wells are being installed and they will provide valuable data on the depth to mineral soils on either side of the lowest points observed. They will also confirm the trend of the mineral soil profile in the low areas.
- All monitoring wells should be installed prior to the advancement of the North Channel Rock Groin, and baseline groundwater levels should be recorded. During the advancement of the rock groin, observations should be made daily for the first week and then an assessment can be made to space out the readings weekly for the next few weeks. The monitoring plan should be adaptable so that it can be adjusted based on the forecasted Nelson River flows and observed water levels at the time of construction. More frequent measurements should be made for larger flows in the Nelson River.
- Monitoring of Gull Lake levels at Area 1, 2 and 3. Lake levels should be recorded within 1 hour of measurements taken at the monitoring wells.

Developing a proper monitoring well and monitoring program will improve the ability of the monitoring wells to provide representative hydraulic data. The additional time and money spent on this important step will minimize the potential for damaging equipment and in-situ sensors, and increase the probability that groundwater data are representative.

4 Cost Estimate

5 Conclusions

Based on the results of the 2011 field investigation program the ground surface was observed to be above the water levels for the 1:20 year flow with the North Channel Rock Groin in place. In discussions held with Manitoba Hydro, ECOSTEM and KGS Acres the project team reached a consensus that the potential for the river to erode the peat and to run-out is very low.

Based on the results of the field investigation and the information available, the preferred approach is to proceed without constructing the containment dykes prior to the Stage I Diversion, and that three of the four areas studied should be monitored during construction.

- Area 1 Dyke 1 (Graph 1) - Based on the field observation this area is not anticipated to be problematic. Monitoring not required.
- Area 1 Dyke 2 (Graph 2) - This area shows a U-shaped profile for the top of mineral soil that dips below the water level for the 1:20 year flow with the rock groin. The maximum depth of water is estimated to be in the order of 0.8 m at HA-21. This area should be monitored during construction. If a 1:20 year flow is observed during construction the head difference between Area 1 Dyke 2 and the Nelson River located about 300 m northwest of the dyke would be in the order of 2.5 to 3.0 m. This area is not deemed to be critical to the construction activities as any seepage/flow through this area would simply end run back into the Nelson River. However, from an environmental perspective, there is potential for erosion contributing to the TSS levels estimated from shoreline erosion. Hence, a decision would need to be made to assess if the containment dyke is required for this area.
- Area 2 Dyke 1 (Graph 3) – shows two locations where the mineral soil profile dips sharply at HA-31 and HA-33. The top of the mineral soils at HA-31 was observed at el 154.4 m which is slightly below the 1:20 year flow for the existing conditions, while the top of mineral soil at HA-33 is at or above the 1:20 year flow with the rock groin in place. For HA-31, the maximum depth of water for the 1:20 year flow with the rock groin in place is in the order of 1.1 m. This area should be monitored during construction.
- Area 3 Dyke (Graph 4) – shows two locations where the mineral soil profile dips below the water level for the 1:20 year flow with the rock groin. The maximum depth of water for the 1:20 year flow with the rock groin in place is in the order of 0.5 m. This area should be monitored during construction.

Should the monitoring program determine that containment dykes are needed, a construction crew can be mobilized to the south bank of the Nelson River to construct the containment dykes.

6 Recommendations

The following are specific recommendations for final design:

- Develop a detailed monitoring plan complete with monitoring wells and instrumentation.
- Review the proposed sources of materials.
- The need and the size of potential containment dykes are to be re-assessed if they are deemed required during the monitoring program.
- Review the impacts of the containment dykes on the regulatory process.

The final design of the containment dykes should also consider the following items:

- Embankment settlement and consolidation
- Foundation settlement
- Stability concerns
- Seepage and ground water control during construction of the South Dyke.

Prepared by:

D.E. Ranta, P.Eng.
for

P.R. Pantel, P.Eng.

Attach
Tables 1 to 5
Figures 1 and 2

DER:PRP:spa

References

1. KGS Acres Ltd., Memorandum GN-2.8 “Keeyask Generating Station - Stage IV Studies - River Management During Construction”, Manitoba Hydro File 00195-10420-0003.
2. ECOSTEM Ltd., Memorandum “Preliminary analysis of peat and water thickness in the containment dyke areas”, May 31, 2011
3. KGS Acres Ltd., “Draft Southside Containment Borehole and LIDAR Summary – Review and Conclusion”, Manitoba Hydro File 00195-21600-0005_00.

Tables

Tables 1 to 5

Table 1
Peat Removal Summary Table - Option 1 Cut Off Trench

		Depths and percentage of peat at 50% percentile[2]					Avg. depth	Depths and percentage of peat at 95% percentile[2]					Avg. depth	Avg. Depth Observed in 2011 Field Investigation	Peat Volume for removal			
															50% + 95% avg depth	Hatch [1]	50% percentile [2]	95% percentile [2]
							mm						mm	mm	mm	m^3	m^3	m^3
Area 1 Dyke 1	depth (mm)	500					**500**	1,000					**1,000**	**500**	**750**	-	1,706	3,329
	percentage at depth	100%						100%										
Area 1 Dyke 2	depth (mm)	500	1,200				**967**	1,000	2,000				**1,667**	**900**	**1,317**	1,236	2,768	4,424
	percentage at depth	33.3%	66.7%					33.3%	66.7%									
Area 2 Dyke 1	depth (mm)	500	1,000	2,000	2,100	5,000	**1,872**	1,000	1,800	3,600	4,000	6,000	**2,762**	**100**	**2,317**	3,263	13,176	23,777
	percentage at depth	37%	24%	10%	7%	22%		37%	24%	10%	7%	22%						
Area 3 Dyke 1	depth (mm)	500	1,000	3,000	3,100		**1,959**	1,000	1,800	5,600	5,700		**3,662**	**900**	**2,811**	1,996	15,484	45,216
	percentage at depth	35%	9%	42%	14%			35%	9%	42%	14%							
																6,496	**33,134**	**76,746**

Notes:

1) Values based on field investigation data
2) Values based on preliminary data by Ecostem
3) Average of the 50% and 95% volumes are used in the construction costs estimate for excavation at dyke locations
4) 50% volumes are used in the construction costs estimate for stripping of borrow areas

Average 50% and 95% Percentile [3]	**54,940**
50% Percentile [4]	**33,134**

Table 2
Impervious Fill Summary Table - Option 1 Cut Off Trench

	Fill Volume					
	Measured volume below grade [1]	Measured volume above grade	Total above and below grade	Add 30% for compaction	Add 10% for settlement	Total
	m^3	m^3	m^3	m^3	m^3	m^3
Area 1 Dyke 1	-	-	**-**	-	-	**-**
Area 1 Dyke 2	1,236	915	**2,151**	645	215	**3,011**
Area 2 Dyke 1	3,263	-	**3,263**	979	326	**4,569**
Area 3 Dyke 1	1,996	149	**2,145**	643	214	**3,002**
	6,496	**1,063**	**7,559**	**2,268**	**756**	**10,583**

Note:

1) Volumes based of field investigation data are used in the construction costs estimate.

Table 3
Impervious Fill Summary Table - Option 2 Compress Peat

	Fill Volume					
	Measured volume below grade [1]	Measured volume above grade	Total above and below grade	Add 30% for compaction	Add 10% for settlement	Total
	m^3	m^3	m^3	m^3	m^3	m^3
Area 1 Dyke 1	-	-	**-**	-	-	**-**
Area 1 Dyke 2	1,212	915	**2,127**	638	213	**2,977**
Area 2 Dyke 1	1,888	-	**1,888**	566	189	**2,643**
Area 3 Dyke 1	1,182	149	**1,330**	399	133	**1,862**
	4,281	**1,063**	**5,345**	**1,603**	**534**	**7,483**

Note:

1) Volumes based of field investigation data are used in the construction costs estimate.

Table 4
Option 1 Construction Cost Estimate
(Year 2012 dollars)

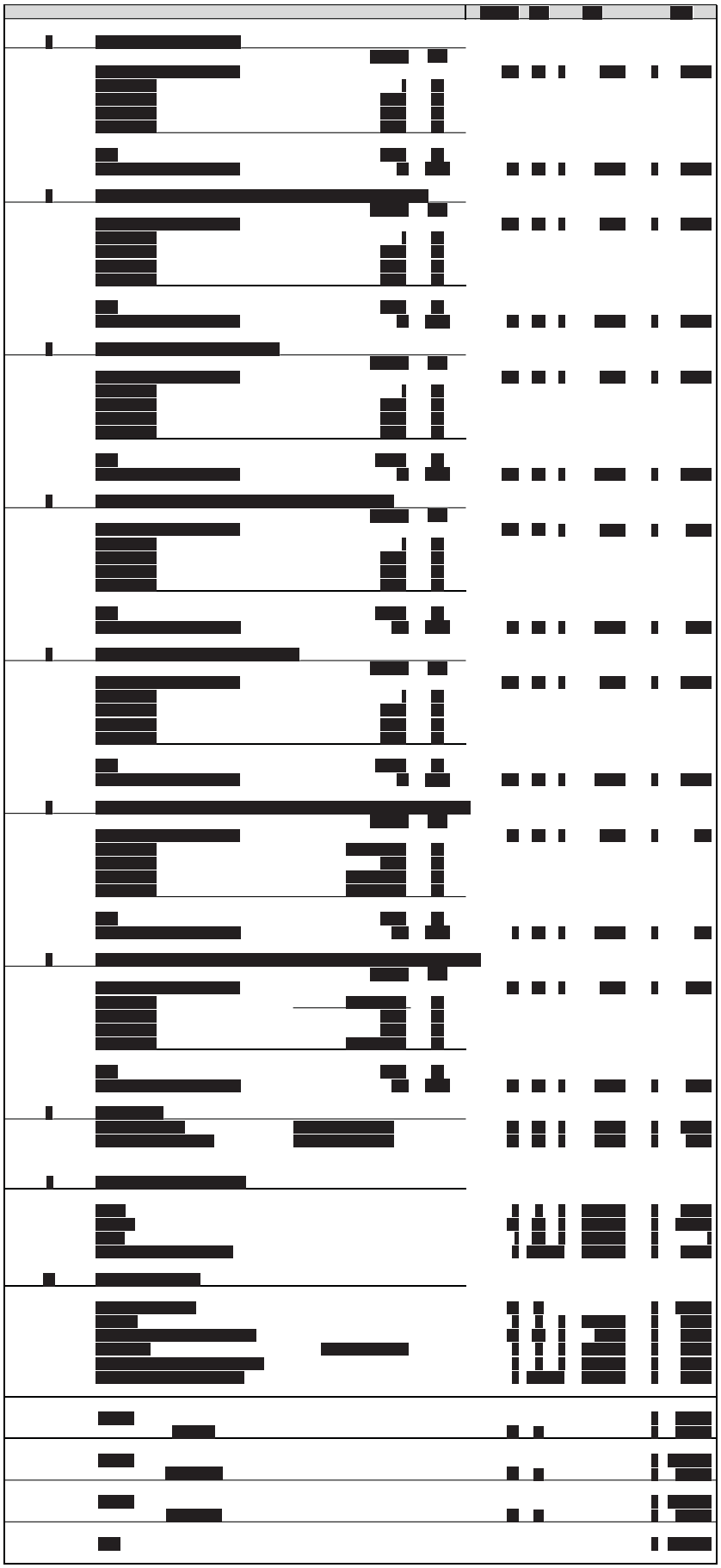
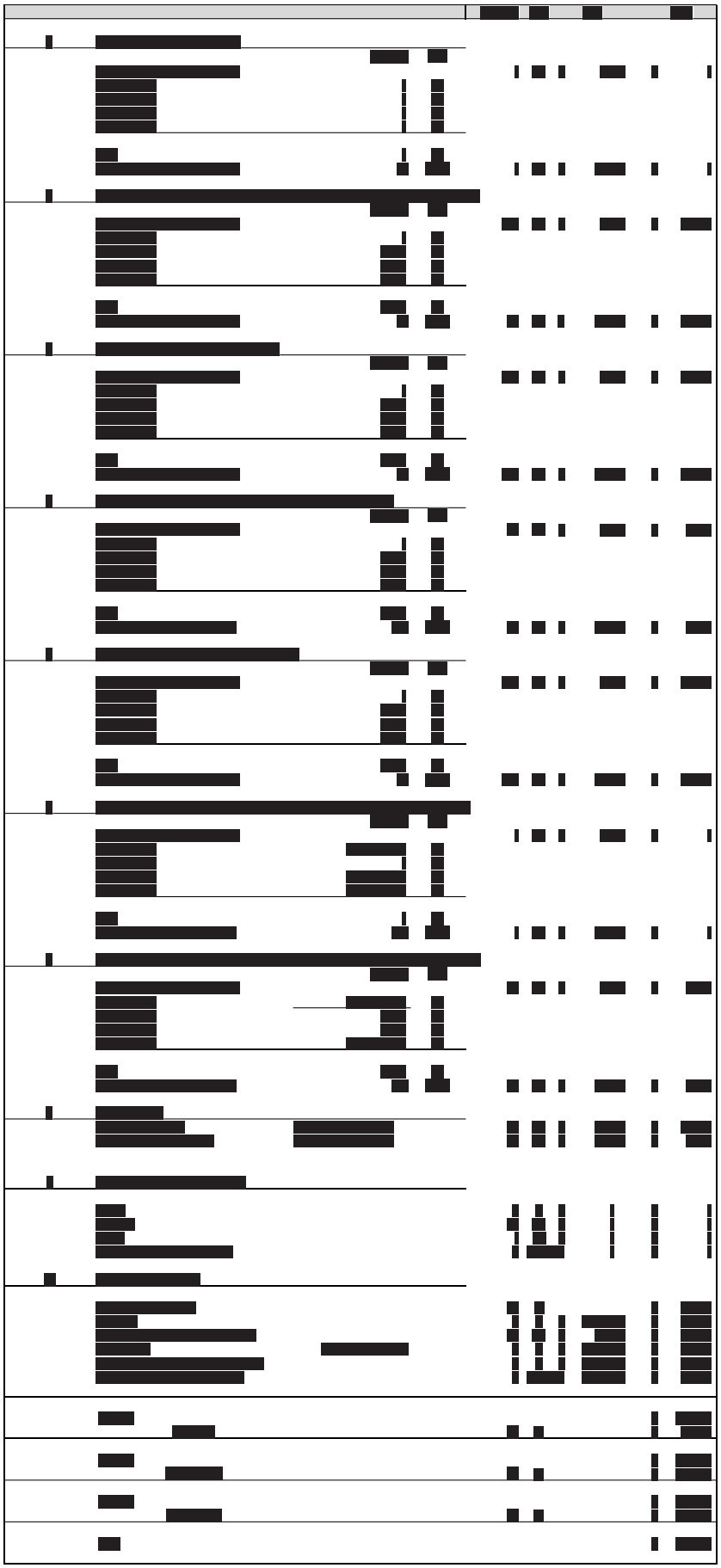
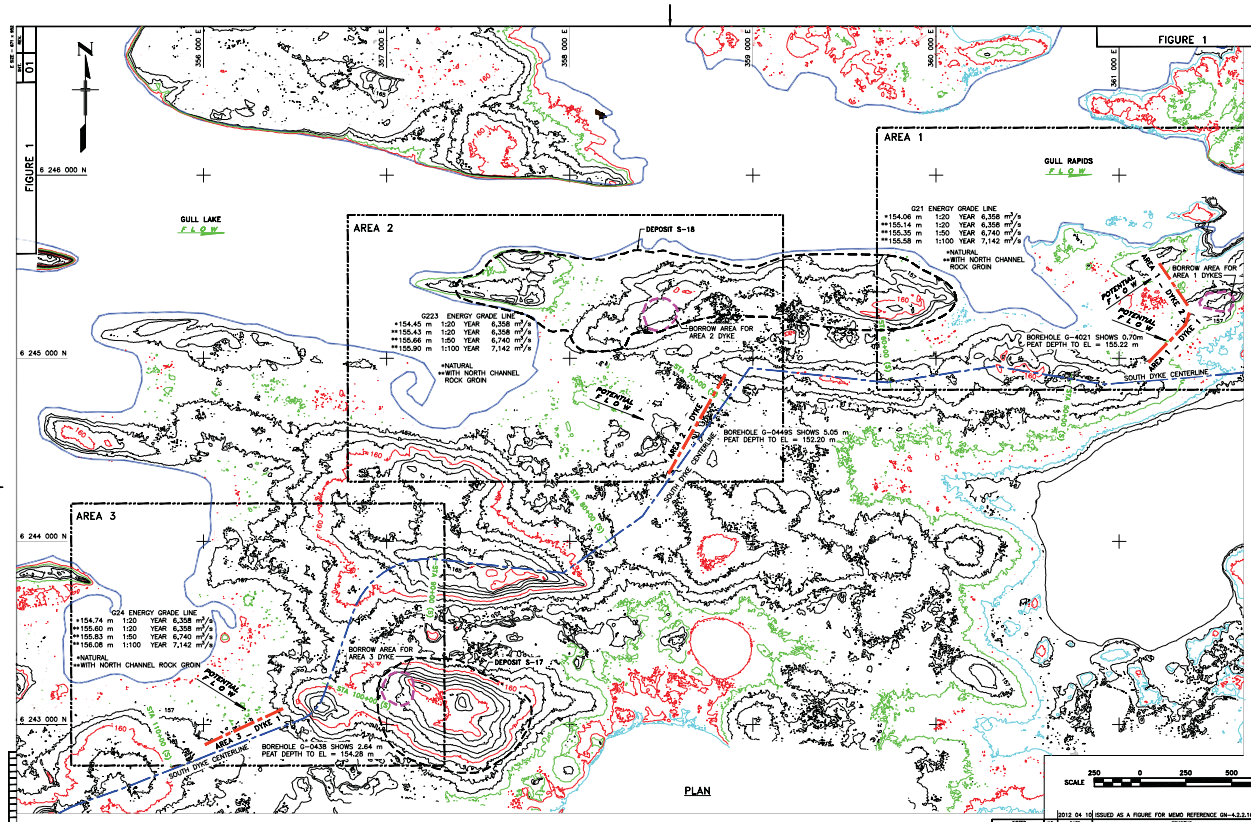


Table 5
Option 2 Construction Cost Estimate
(Year 2012 dollars)



Figures

Figures 1 and 2



LEGEND:

- SHORELINE
- GROUND SURFACE CONTOUR
- EL 154 CONTOUR
- EL 155 CONTOUR
- EL 156 CONTOUR
- CONTAINMENT DYKE
- COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)

NOTES:

1. TOPOGRAPHY IS BASED ON 2010 LIDAR DATA PROVIDED BY MANITOBA HYDRO.
2. COORDINATE GRID IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
3. ELEVATIONS ARE BASED ON GS OF C, CGVD28, 1929 ADJUSTMENT.

2012 04 10 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.16

KGS ACRES LTD

MANITOBA HYDRO

KEEYASK GENERATING STATION

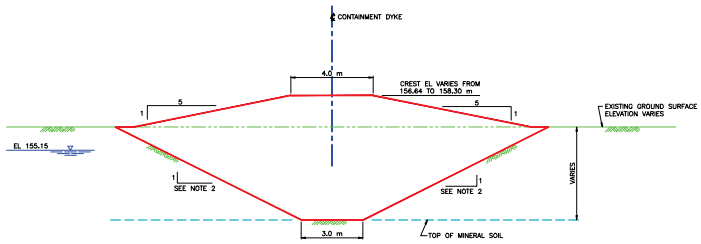
STAGE IV STUDIES AXIS GR-4
LOCATIONS FOR POTENTIAL
SOUTH SIDE CONTAINMENT DYKES

FIGURE 1 — REV. 01

FIGURE 2 | REV. 01

NOTES:

1. ELEVATIONS ARE BASED ON GS OF C, CGVD28, 1929 ADJUSTMENT.
2. EXCAVATED SLOPES MAY VARY TO ENSURE A STABLE SLOPE.



OPTION 1: CUT-OFF TRENCH
N.T.S.

NOTED	NO.	DATE	REVISIONS	BY	CHD.	APP.
	01	2012 04 10	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.2.2.16			

KGS ACRES LTD

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
POTENTIAL SOUTH SIDE
CONTAINMENT DYKES
TYPICAL SECTION

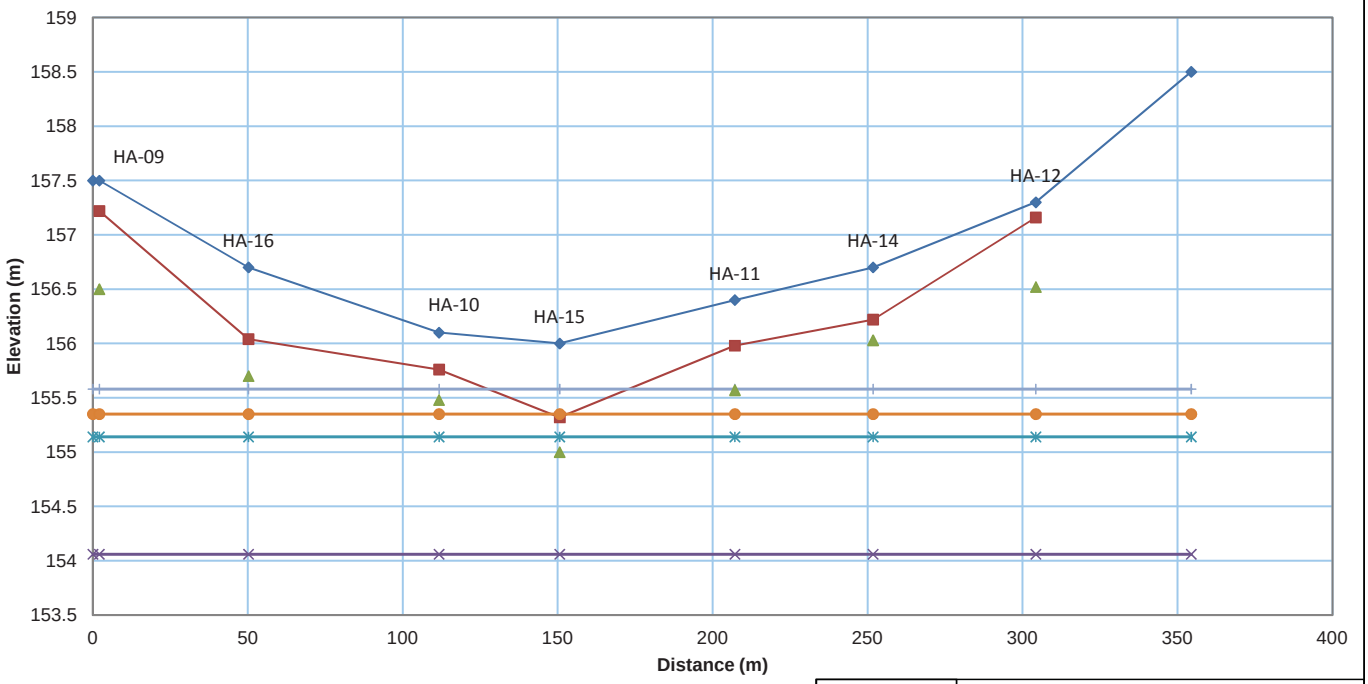
DESIGN BY	DESIGN CHECK	SUBMITTED	APPROVED	DRAWN	CHECK	SCALE	DATE
						N.T.S.	

PROJECT MANAGER

DATE

FIGURE 2 | DWG. 01 | REV.

Appendix A



KGS ACRES

KGS ACRES

MANITOBA HYDRO

KEEYASK RIVER MANAGEMENT

STAGE 4 STUDIES
AREA 1 DYKE 1

STAGE 4 STUDIES

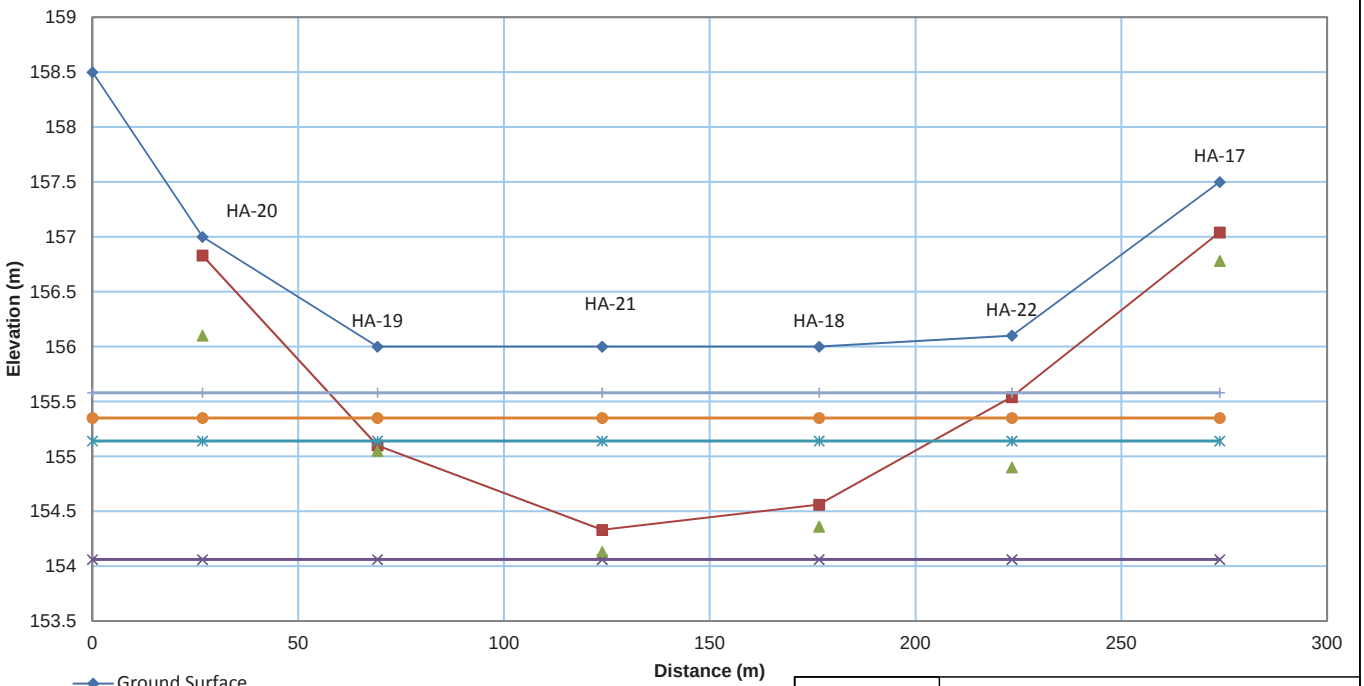
PROJECT
H-210008

SCALE
NTS

DATE
2012 03 05

REFERENCE TO MEMO NO.
GN 4-2.2.16

GRAPH 1



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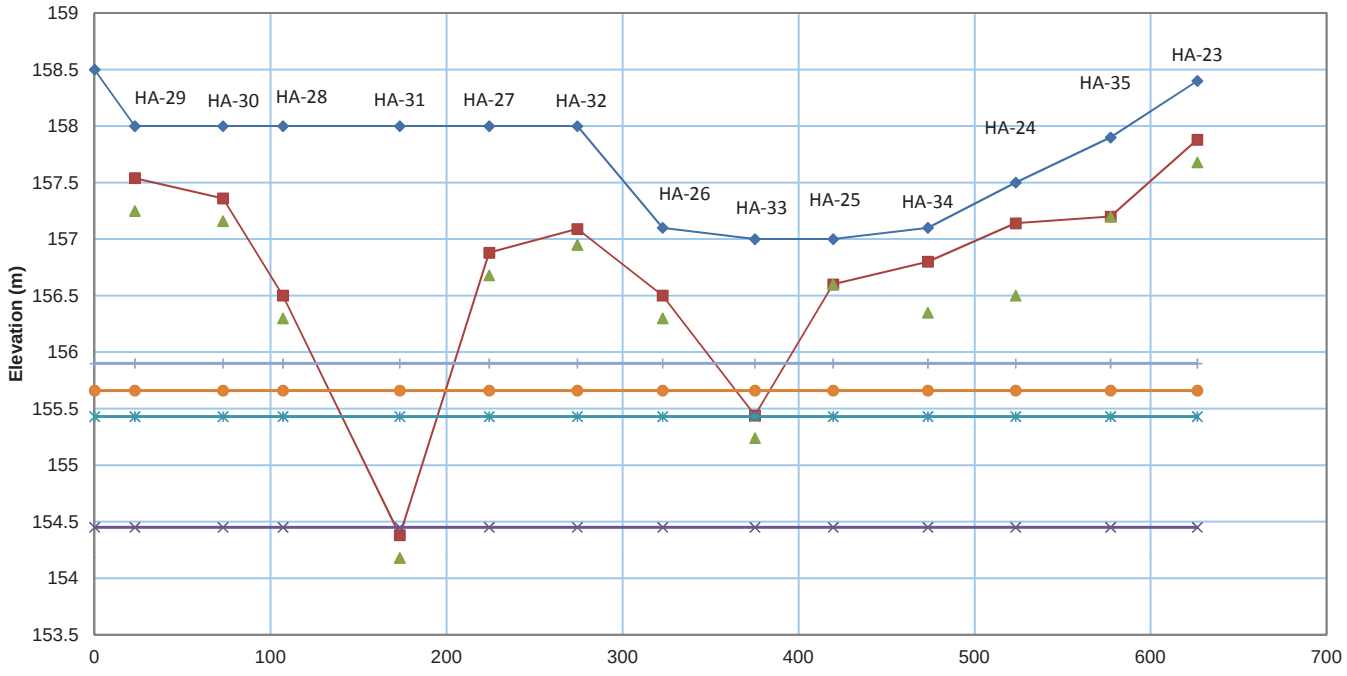
MANITOBA HYDRO

KEEYASK RIVER MANAGEMENT

STAGE 4 STUDIES
AREA 1 DYKE 2

PROJECT	SCALE	DATE	REFERENCE TO MEMO NO.	
H-210008	NTS	2012 03 05	GN 4-2.2.16	GRAPH 2



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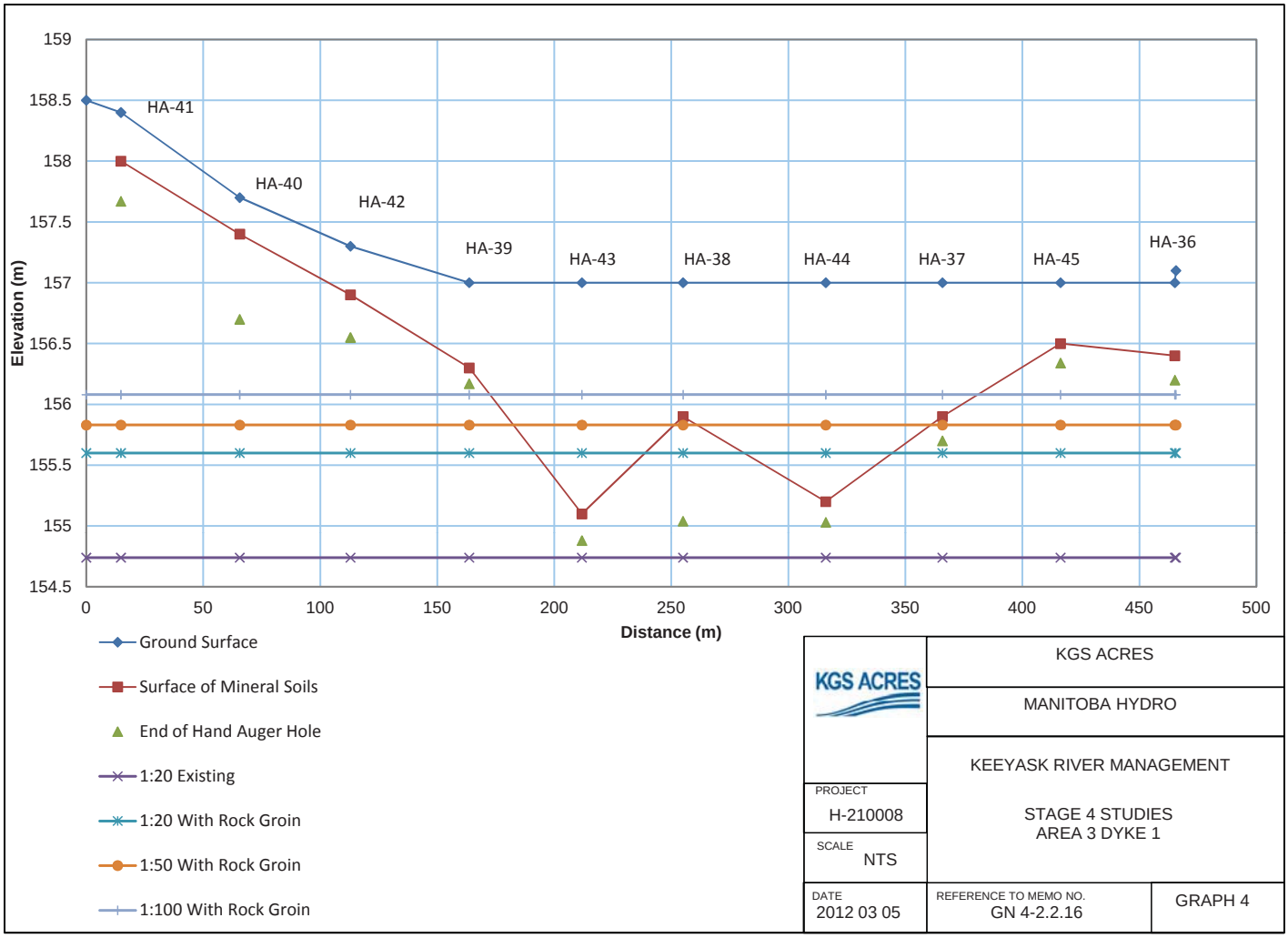
MANITOBA HYDRO

KEEYASK RIVER MANAGEMENT

STAGE 4 STUDIES
AREA 2 DYKE 1

PROJECT	SCALE	DATE	REFERENCE TO MEMO NO.	
H-210008	NTS	2012 03 05	GN 4-2.2.16	GRAPH 3



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KEEYASK RIVER MANAGEMENT

STAGE 4 STUDIES
AREA 3 DYKE 1

PROJECT
H-210008

SCALE
NTS

DATE
2012 03 05

REFERENCE TO MEMO NO.
GN 4-2.2.16

GRAPH 4

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.2.2.18

Rev. 0, December 5, 2011

Prepare Plan Views of North and South Dykes

Manitoba Hydro File 00195-21200-0015_00

Prepared by _____ (A. Lupu) Dec 5/11
KGS Acres Ltd.

Checked by _____ (EDWARD SIKORA) DEC 5/11
KGS Acres Ltd.

Reviewed by _____ (P. PANITCL) Dec 5, 2011
KGS Acres Ltd.

Approved by _____ (I. R. Dewar) Dec 5, 2011
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	December 5, 2011
		File No.	P10008(16).27.07
From	E. Sikora/A.M. Lupu	**cc**	P. Pantel
Subject	**Keeyask Generating Station** **Stage IV Studies - Axis GR-4** **Memorandum GN-4.2.2.18, Rev 0** **Prepare Plan Views of North and South Dyke (ESCA 55)** **Manitoba Hydro File 00195-21200-0015_00**		

1 Introduction

Manitoba Hydro requested KGS ACRES (KA) to prepare 3D line work for the North and South Dykes in AutoCAD and for a geo-database in GIS. This memorandum provides a brief summary of the scope of work completed. The extent of the line work described in this memorandum is illustrated in the attached Sketches No. 1 and 2.

2 Scope of Work

2.1 AutoCAD Work

The assignment was to develop 3D AutoCAD line work for the North and South dykes to show crest details and slopes. All work has been based on the design of the North and South Dykes as defined in deliverable GN 4.2.2.2, Rev 0, issued on June 30, 2010.

KA was to create polygons for each section of the dykes (in AutoCAD), i.e., for all Road Sections, Freeboard Dyke Sections, Granular Dyke Sections, and Zoned Impervious Core Dyke Sections.

KA was to prepare 2D polygons for perimeter drains along both dyke lines in AutoCAD (additional details are provided in the comment section below). Polygons were to include the footprint area for ditch turnouts. As requested by Manitoba Hydro (MH), KA prepared a small section of the perimeter drain in AutoCAD and GIS format for MH's review. MH reviewed the data and advised KA that the drain detail met expectations and that KA was given approval to proceed with producing all files.

2.2 GIS Work

KA developed GIS polygon shape files in a single feature class where each section of dyke is a separate attributed polygon. One of the attributes is the type of dyke or roadway section. One attribute indicates north vs south dyke. One attribute includes volume quantities for the dykes.

3 Reason for This Engineering Scope Change

MH required a plan view of the North and South Dykes to establish the footprint as required for the EIS. The origins and validity of the previous versions of the GIS shape files of the north and south dykes in both MH's and KA's Geo-databases could not be established. The project team had not produced plan views of the dykes to their full extent in their most current configuration. In addition, the preparation of polygons for each change in the dyke sections along the dyke lines provided insufficient information. More detailed information had been developed and was available in the GIS geo-database.

4 Comments

The scope of work described above applies to the full length of both the North and South Dykes.

The preparation of the line work and polygons for the perimeter drains has been prepared in 2D. This is because the preparation of 3D line work for the drains could not be started at this time as the design of the ditch grades was not undertaken in the Stage IV Studies. It is understood that these details will be addressed during Final Design.

The 2010 LiDAR contours were not used in completing this work. The drawings are based on the 2004 topographic mapping data. MH has advised that depending on the requirements and licensing, there may be a need to use the 2010 LiDAR data in future revisions.

There was some discussion during the initial development of the scope of work for this assignment on preparing a "Viz" file for the dykes for use in the June 2011 3D Model. Subsequently, during the coordination meetings for the development of the 3D Model, it was agreed that new AutoCAD and GIS files for the dykes would not be incorporated into the 2011 3D model. MH had indicated that they would proceed with the current 3D model of the dykes as presented in the 2007 3D Model, with some small changes to the North Dyke to accommodate the North Access Road.

4.1 AutoCAD Drafting

North and South Dykes – 3D Lines

The 3D lines for the North and South Dykes, including crest details and slopes, were developed using AutoCAD Civil3D. The sources of information used were:

- Ground Surface Topography:
  - GN-4.2.2.2 Figure 2.
  - GN-4.2.2.2 Figure 3.
  - GN-4.2.2.2 Figure 4.
- Dyke Alignment:
  - GN-4.2.2.2 Figure 10.
- Dyke Sections:
  - GN-4.2.2.2 Figure 5.
  - With the assumptions shown in Figure 1 attached herein.
- Limits of Dyke Sections:
  - Design Memorandum GN-4.2.2.2, Tables 4 & 5.
- Transitions Between Sections:
  - Simple linear transition based on dyke outlines.
  - Transitions occur over a distance of approximately 25 m.

The steps followed in developing the 3D lines were:

- Create Civil3D surface of ground surface topography using contours in the source figures given above.
- Extract "Strips" of topography along the dyke lines. Strips extend 100 m upstream and 100 m downstream of the dyke center line.
- Create upstream-downstream ground surface profiles along the dyke lines at 5 m station intervals.
- Determine the dyke section, including crest, slope break lines, and toe of fill line at each 5 m station interval.
- Join the crest, slope break and toe of fill lines at each section to create the required 3D lines.

The resulting 3D lines were checked by:

- Creating Civil3D surfaces of the dykes based on the 3D lines created above.
- Extracting sections of the dyke "surfaces" at 1 m intervals.
- Visually assessing the extracted sections.

Perimeter and Ditch Turn-outs – 2D Line Work

Perimeter and Ditch turn-outs were developed in 2D, as required by ESCA No. 55. In creating the ditch line work the following assumptions were made:

- Ditches were offset 8 m from the downstream toe of the dyke/road.
- Ditches were assumed to be 1 m deep with 1V:3H side slopes and an invert width of 2 m.
- Ditch turn-outs were to extend to a point 1m lower than the elevation of their starting point at the perimeter ditch.

The ditch turn-outs were located at the low points of the perimeter ditches and the alignment from the perimeter ditch to the discharge point was determined using the water-drop utility in Civil3D.

4.2 GIS Processing

The drafting department at KA developed a sample of the 3D line work for conversion to GIS polygons. The sample GIS work was delivered to MH for comments and approval. After approval of the sample by MH, the drafting department updated the AutoCAD line work for the GIS department for conversion to GIS polygons and Shapefiles.

The first line work to be developed for conversion to GIS was "Disturbed Areas" for both the North and South Dykes. These lines were based on the assumption that disturbed areas would extend up to 15 m all around the dykes and ditches. The AutoCAD line work was converted to GIS line work and then divided into individual features to create polygons. All polygons were merged into one file for the North Dyke and one file for the South Dyke. The attribute table was then populated with information for each polygon and metadata for the Shapefile was then developed.

The AutoCAD line work for the North and South Dykes depicting section specific details was developed and used for conversion to GIS polygons. The process of conversion was the same as described above. The North and South Dyke files depicted sections of the dyke, ditch, ditch turnouts and dyke crest road. Once the files were finalized they were delivered to MH for comments and approval.

MH requested that the misleading data from the disturbed areas be deleted from the files. GIS data was submitted to MH in geo-database and Shapefile Formats. The attached two sketches illustrate the GIS data that was derived from AutoCAD line work submitted to MH.

Prepared by:

A.M. Lupu,
Senior Environmental Planner

Attach
Sketches
CD Rom

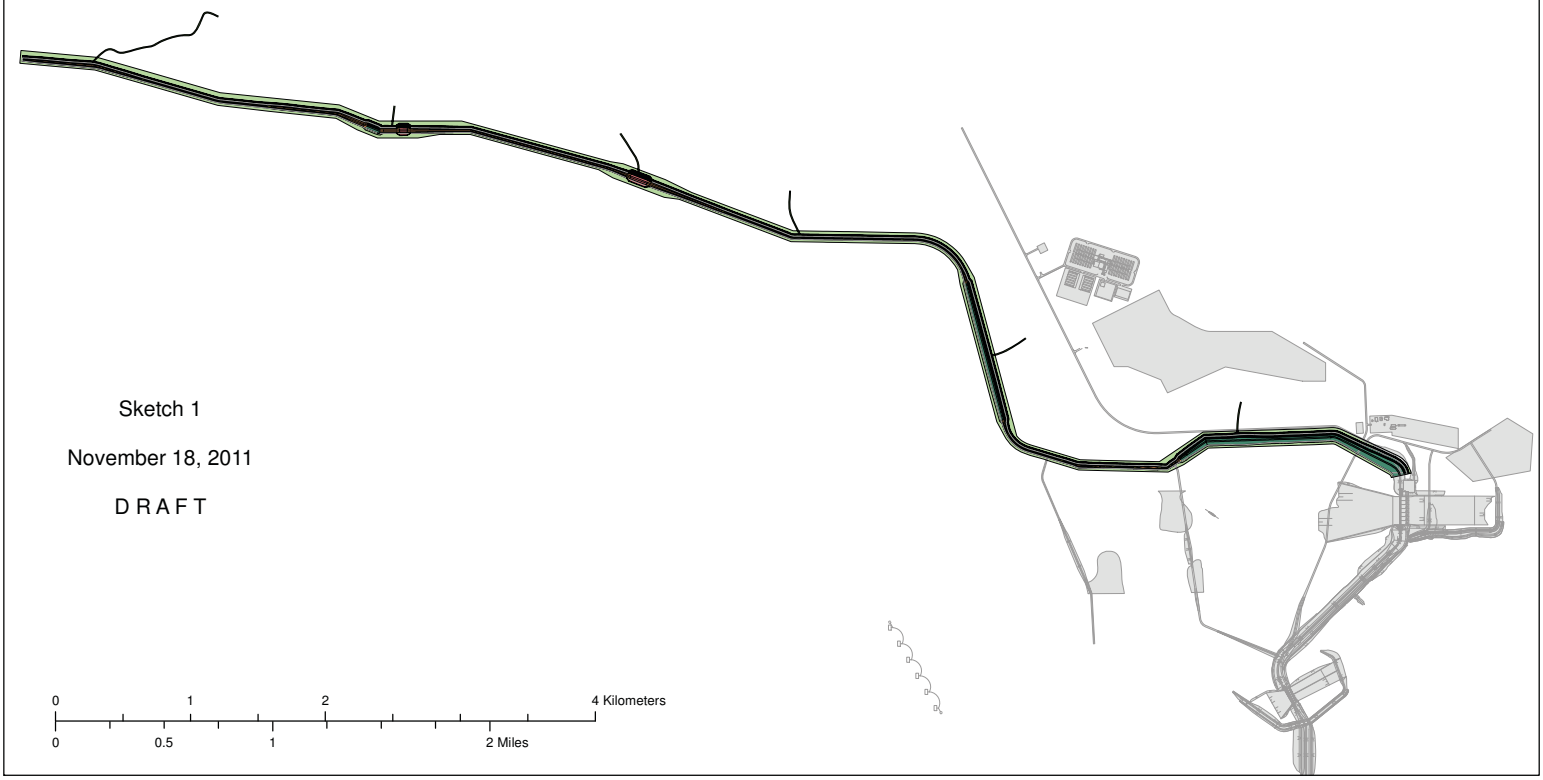
E. Sikora, P. Eng.

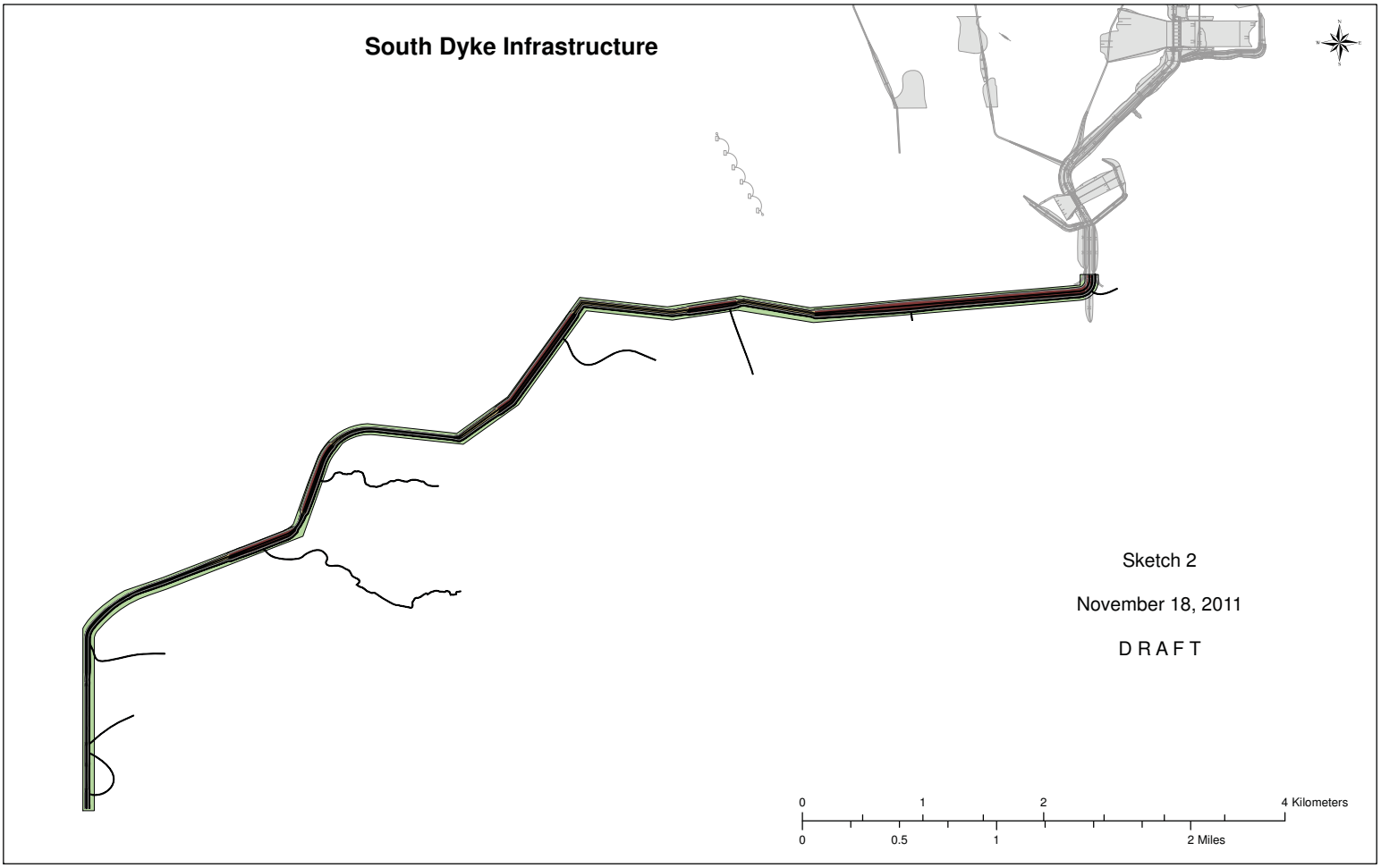
AML:ES:PRP:cl

P.R. Pantel, P. Eng.

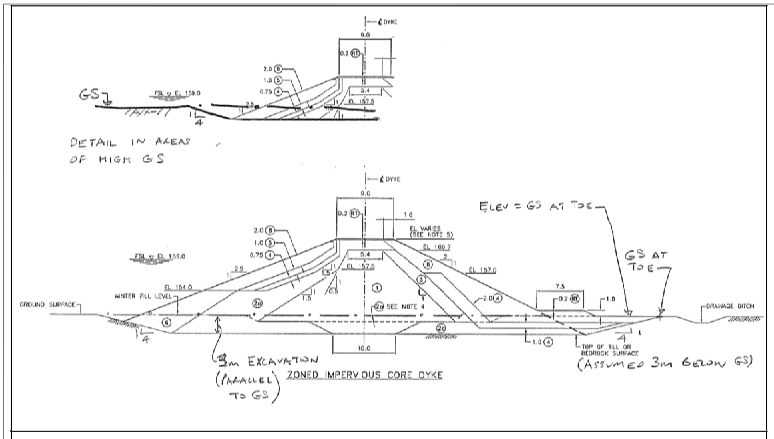
Sketches

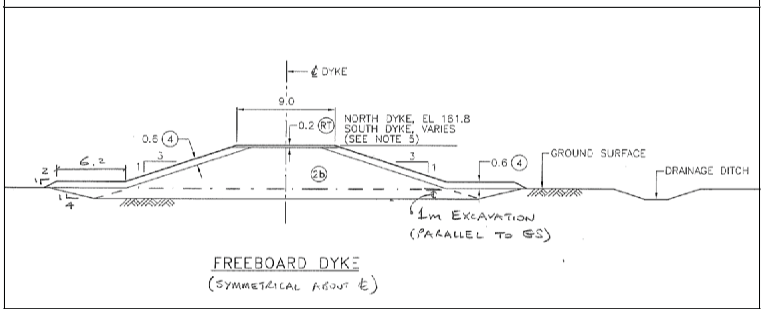
North Dyke Infrastructure

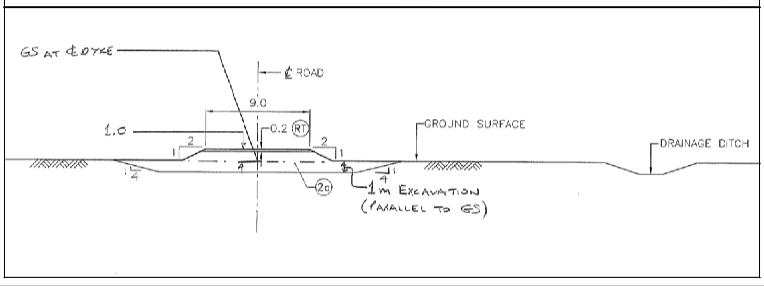



South Dyke Infrastructure
Sketch 2
November 18, 2011
D R A F T
0
1
2
4 Kilometers
0
0.5
1
2 Miles

Figure







NO.	DATE	REVISIONS
A1	2011.11.14	ISSUED AS FOR INFORMATION TO MANITOBA HYDRO

KGS ACRES

KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION

ESCA 55
NORTH & SOUTH DYKES
3D LINE WORK
TYPICAL DYKE SECTIONS

SCALE		SHEET	REV.
AS SHOWN	FIGURE 1	01 of 01	D1

AUTOCAD ORIGINAL

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Manitoba Hydro
Hydro Power Planning Department
Power Planning and Development

Gull Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.3.17

Revision 0 June 3, 2004

Physical Hydraulic Model Studies

Manitoba Hydro File 00195-11400_0024_01

Prepared by R.D. Wolger (R.D. Wolger) June 3, 2004
for D.G. Murray
Acres Manitoba Limited

Checked by R.D. Wolger (R.D. Wolger) June 3, 2004
Acres Manitoba Limited

Reviewed by I.R. Dewar (I.R. Dewar) June 3, 2004
Acres Manitoba Limited

Approved by N.J. Smith (N.J. Smith) June 3, 2004
Acres Manitoba Limited

Noted by O.S. Magnusson (O.S. Magnusson) Nov 23/04
Manitoba Hydro

Accepted by B.J. Osiowy (B.J. Osiowy) May 3/2005
Manitoba Hydro

Acres Manitoba Limited
Winnipeg, Manitoba

Acres Office Memorandum

To	I. R. Dewar	**Date**	June 3, 2004
		File	P10008(16).26
From	D. G. Murray	**cc**	R. D. Woolgar

Subject **Gull Generating Station**
Stage IV Studies – Axis GR-4
Physical Hydraulic Model Studies
Memorandum GN-4.3.17 Revision 0
Manitoba Hydro File 00195-11400-0024_01

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1 Introduction

The proposed Gull Generating Station (GS) will be located 730 km north of Winnipeg, on the Nelson River, at the upstream end of Stephens Lake. The development will have a full supply level of el 159.0 m and will be located along Axis GR-4. The proposed generating station will develop approximately 18 m of the 27 m total head between Split Lake and Stephens Lake (Kettle GS Forebay) at a rated plant output of 4000 m^3/s. Seven generating units are proposed with a total capacity of 675 MW.

The model testing program has been devised to address the main hydraulic concerns of the project. The program has been influenced by the model and prototype experience of similar features at Kettle, Long Spruce, Limestone and the proposed Conawapa Generating Stations. In addition, extensive numerical modeling has been undertaken on many of the features. A number of the tests in the program are therefore considered confirmatory in nature and are not required to be as extensive as would be required without this experience.

Two separate models are proposed; the first would comprise a comprehensive model study of the river closure, river diversion works and Spillway discharge facility at the GR-4 axis, the second would comprise a sectional model study of the Spillway.

2 Comprehensive Model Study

2.1 Purpose

The purpose of the comprehensive model study would be to confirm the proposed scheme for river management during construction of the Gull GS and to confirm the overall performance and discharge carrying capacity of the proposed Spillway facility. A two-stage program is proposed to divert the Nelson River and allow the construction of the project along the GR-4 axis. The first stage involves blocking off the north and central channels to facilitate construction of the Central Dam and Powerhouse Cofferdam. Also included, in the first stage, is the construction of a U-shaped cofferdam (Spillway Cofferdam) in the south channel which will divert the river towards the south bank and permit construction of the Spillway structure, and the Spillway Approach and Discharge Channels. The second stage of the diversion will involve removal of the Spillway Cofferdam to allow flow to pass through the partially completed Spillway and permit construction of the South Dam Cofferdam across the southern portion of the Nelson River.

Although detailed numerical modeling studies have been undertaken to develop the proposed scheme for river management, confirmation of the results from comprehensive physical model tests would significantly reduce the construction risk to be assumed by the Contractor during river closure. This would result in significant cost savings at time of tender; cost savings that would likely more than offset the costs of the physical model studies. The results from the physical model may also prove to be beneficial in relation to the environmental hearings and licensing process, whereas the results of the less commonly used numerical modeling might be challenged if not verified by a physical model.

2.2 General Description

During the first year of construction, a U-shaped cofferdam will enclose the construction area for the Spillway, and associated Approach and Discharge Channels, which are located on the north bank of the south channel. A small cofferdam at the upper end of the central channel, in combination with an L-shaped Powerhouse cofferdam and the Central Dam and North Channel Cofferdams, will allow construction of the Powerhouse and associated Intake Channel. The Tailrace Channel will be constructed behind the Tailrace Channel Summer Level Cofferdam. The layout of the Stage I Diversion will direct the entire flow of the Nelson River to the southern portion of the river, allowing the construction of the following structures in the dry:

- Powerhouse Intake Channel

- Powerhouse Intakes
- Powerhouse Tailrace Channel
- Spillway (Stage II Diversion Structure)
- Spillway Approach Channel
- Spillway Discharge Channel.

The alignment of the Stage I Spillway Cofferdam is designed to provide sufficient working space around the Spillway wing walls.

The cofferdam crest elevations are designed to provide protection during passage of the Construction Design Flood. The recommended crest elevations are based on the larger of the energy grade line during either open water or winter conditions. The energy grade line is used because at any given section in a channel the water surface can range between the hydraulic grade line (high velocities) and the energy grade line (negligible velocities). The energy grade line represents the highest possible water level for a particular section in areas of lower velocity. The cofferdams considered for this study typically are located in areas in the channel of lower velocities; therefore, the water level adjacent to the cofferdams will most likely be closer to the energy grade line.

Table 1, below, summarizes the list of significant flow conditions to be modeled during the construction phase, as well as during the operation phase of the generating station.

Table 1
List of Significant Flow Conditions

Description	Discharge (m^3/s)	Remark
Construction Design Flood	6 240	1:20 year flood
Full Powerhouse Discharge	4 000	
Long-Term Average River Flow	3 130	
River Closure Flow	5 270	1:20 August flood
Probable Maximum Flood (PMF) Flow	12 700	
Speed-no-Load Discharge	1 400	

The normal reservoir operating level is el 159.0 m with a minimum operating level of el 158.0 m. The maximum surcharge elevation during passage of the PMF has been estimated to be el 160.3 m.

2.3 Model Description

Previous models of this nature have been built at a scale ranging from 1:70 to 1:100. The model boundaries have typically extended from two river widths upstream of the axis to three river widths downstream of the axis. For tendering purposes, the laboratories will be instructed to assume that the model will be constructed to a 1:100 scale and extend from 1200 m upstream of the Spillway axis to 1200 m downstream of the Spillway axis.

The construction and testing of the model will generally follow the same construction sequence as that of the prototype. Tailwater levels in the model will be controlled by a tilting weir or equivalent. Riverbed roughness in the model will be adjusted as required to produce a reasonable simulation of known river water levels in the area.

The model is proposed to be constructed with a fixed-bed for calibration of bed roughness for natural conditions and for simulation of the Stage I Cofferdam construction. After calibration and Stage I Cofferdam tests are complete, the channels and structures will be modeled. The channels are to be modeled with a fixed-bed and must be capable of convenient modification to investigate alternative geometries.

The Spillway will be modeled in Plexiglas or equivalent, and the construction must be devised to permit convenient conversion/modification of the diversion invert level and the final rollways. The geometry of these elements may be revised based upon the test results from the Sectional Model.

2.4 Test Program

The construction of the Stage I Spillway Cofferdam from the left bank is not expected to create significant problems. Nevertheless, tests are included in the program to determine minimum stone sizes required and the flow characteristics to be expected during cofferdam advancement. The fixed-bed model will be used for all of the Stage I Spillway Cofferdam tests since scour of the riverbed during this construction phase is not considered to be of concern.

The Stage I Spillway Cofferdam will be advanced by end dumping and pushing by bulldozer. Simulation of the granular and impervious material placement will not be necessary. Other Stage I Cofferdams are not considered for testing as these structures will be constructed in areas of relatively low velocities. Although there is a concern for placement of rock along the Transmission Tower Spur and Tailrace Channel Summer Level Cofferdam, these cofferdams are not located within the boundaries of the physical model. It is not deemed feasible to extend the model to this location as there is insufficient bathymetric data available in the reach between the Spillway location and the

Powerhouse location. Flow-3D simulations have been prepared for these cofferdams and have been used to assess the impact of velocities on stone sizes and placement.

Closure of the Nelson River by a cofferdam is considered to be one of the more difficult construction activities in the project. The Stage II Diversion Structure will consist of low level sluiceways through the partially completed Spillway. Initially, tests will be carried out at a modeled discharge of 6240 m^3/s with the Stage II Diversion Structure fully opened and with the upstream leg of the Stage II Cofferdam closed. These tests will rate the structure against the upstream water level as measured near the upstream leg of the cofferdam. If there is a significant departure from the assumed theoretical rated value, modifications to the structure or Approach Channel may be required.

An initial test of a single upstream leg advance will be undertaken at a modeled discharge of 5270 m^3/s with the Stage II Diversion Structure fully opened. Criteria for recommended rockfill size distributions will be provided. The locations at which excessive losses occur will be noted and the next larger rockfill size will be used to permit further advancement. At a modeled discharge of 5270 m^3/s it may not be possible to achieve river closure even with the largest sized rock specified. In this case, the discharge will be reduced and the test continued. If necessary, the discharge will continue to be reduced until closure is achieved.

During the test, the following will be recorded at regular intervals and at points where it is necessary to increase rock size or to reduce discharge:

- total discharge
- estimated split in discharge between the structure and the closure gap
- estimated rate of rock loss as a percentage of material placed
- water levels upstream and downstream of the cofferdam
- velocities at 0.2 and 0.8 of depth at the leading edge of the cofferdam
- general flow patterns.

The above test will be repeated simulating the simultaneous coordinated advancement of both the upstream and downstream legs of the Stage II Cofferdam. The purpose is to split the head differential and hence, reduce rock losses. The same observations as above will be recorded.

2.5 Closure of the Stage II Diversion Structure

Towards the end of project construction, the diversion sluiceways will be closed over a period of several months to allow impoundment of the reservoir. Closure of the sluiceways will be undertaken singularly or in pairs depending on flow conditions and on the construction status of the embankment structures. Following the closure of each sluiceway, the permanent rollway will be completed. During this closure period, the structure will consist of various combinations of open sluiceways and completed rollways.

The discharge capacity of these various combinations of openings will have been estimated from empirical formulae, from the earlier Stage II Cofferdam closure tests and from numerical modeling tests. Tests will be carried out in the hydraulic model to verify these capacities.

2.6 Preferred Spillway Operation

Release of flow through the Spillway can be achieved by many possible combinations of bays and gate openings. However, some of these combinations may cause unacceptable hydraulic conditions. Model tests are required to explore alternatives, and to identify the ramifications of deviating from the preferred sequence.

A preferred sequence of bay usage and gate opening will be developed for the complete range of discharges that can pass through the Spillway with the forebay at its full supply level of el 159.0 m. In undertaking these tests, a minimum Spillway gate opening will be considered. Gate openings should avoid as much as possible, any large air-entraining vortices which may cause gate vibration. Any increase in potential for scour along permanent structures should be avoided if possible.

The discharge capacity of the Spillway at full gate opening will be rated from the forebay level at the ogee crest to the forebay level at el 160.3 m.

In addition, flow velocities along the critical areas of the central dam will be monitored and tests of performance will also be undertaken under elevated tailwater conditions.

3 Sectional Spillway Model Study

3.1 Purpose

The main purposes of the sectional Spillway model will be to:

- determine uplift and downpull forces on the gate and to detect any tendency for gate vibration
- determine the pressures on adjacent piers at selected locations, for structural design purposes, and on the rollway surface to confirm that excessive subatmospheric pressures do not occur for a variety of gate openings
- determine water surface profiles for selection of pier height geometry
- verify the estimated discharge capability of the diversion sluiceway and final rollway.

In lieu of the physical sectional Spillway model studies, more comprehensive numerical model studies could be undertaken. However, one significant shortfall of the numerical model studies is the inability to determine the performance of the gates as regards to the uplift and downpull forces and the tendency for any gate vibration. Otherwise, it is expected that excellent confirmation will be obtained between the numerical model and the sectional model study results which will add to the credibility of the design arrangement.

3.2 Model Description

A model scale of approximately 1:50 is envisaged. This model will represent one or possibly two full bays with one-half bay on each side constructed within the confines of a flume. The ogee section will be modeled in Plexiglas or equivalent, and the construction will be devised to permit convenient conversion/modification of the various Spillway elements.

A number of pressure measurements will be required in the test program. No mobile bed tests are anticipated. The Approach and Tailrace Channels will be modeled in a fixed bed. Tailwater levels must be adjustable within a given range.

When the final geometries of the water passages have been determined, discharge measurements will be made at partial gate openings. Measurements will be made at a range of headwater/tailwater/gate openings so that discharge coefficients can be determined and applied to develop continuous rating curves. A frictionless gate apparatus will be devised to test for vibration. Unacceptably high uplift/downpull forces

or any evidence of vibration tendency may require modifications to the gate design and retesting of the model.

4 Schedule for Model Tests

The schedule for the model construction, calibration and testing is estimated to be 14 months, including an allowance of four months for report preparation by the laboratory. This duration assumes there will be no major changes in structure layouts.

Because of the importance of these tests in the design of the major structures, and the need for timely delivery of model results, it is considered that the best approach is to retain a reputable commercial laboratory with a depth of experience in such work. As the models are dependent on one another and would run in parallel, it would be preferable to have the same laboratory doing both studies.

The proposed procedure would be the preparation of detailed test programs for each model and drawings of the structures, river topography, etc. These would then be sent to the appropriate laboratories in the form of a request for proposal. A laboratory would then be selected on the basis of these proposals and the reputation/expertise of the companies involved.

5 Cost

DGM:llk

Robert Woodgate
for D. G. Murray

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.3.19

Rev. 0, November 15, 2011

Comparison of Numerical and Physical Hydraulic Model Analyses

Manitoba Hydro File 00195-11400-0073_00

Prepared by David Fuchs (D.M.R. Fuchs) Nov 15, 2011
KGS Acres Ltd.

Checked by David Fuchs (D.M.R. Fuchs) for J.L. Groeneveld Nov 15, 2011
KGS Acres Ltd.

Approved by (I.R. Dewar) Nov 15, 2011
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	November 15, 2011
		File No.	P10008(16).26.04
From	D.M. Fuchs	**cc**	J.L. Groeneveld D.M. Murray
Subject	**Keeyask Generating Station Stage IV Studies – Axis GR-4 Memorandum GN-4.3.19, Rev 0 Comparison of Numerical and Physical Hydraulic Model Analyses Manitoba Hydro File 00195-11400-0073_00**		

1 Background

The engineering assessment of the conceptual design for the Keeyask hydroelectric facility necessitated developing an understanding of the very complex behaviour of moving water. In designing each component of this facility, the engineer was required to efficiently and reliably verify the adequacy of the hydraulic design under all plant operating conditions. To accomplish this, two types of modeling assessments were undertaken. A Computational Fluid Dynamics (CFD) model and two physical models were set up to analyze flows under a variety of conditions. The models were used to assess river management strategies during the construction of the project, and also for the design of the Spillway and Diversion Structures [Ref 1].

2 Introduction

This memorandum presents a comparison of the results between two physical models and the FLOW-3D (CFD) modeling. A number of conference and journal papers have been written discussing the FLOW-3D and physical modeling of Keeyask Generating Station (GS). These papers are provided in Appendix A and are referenced. The river management and Spillway design memoranda GN-2.8 and GN-4.3.6, respectively also present a comparison of the FLOW-3D and physical modeling results.

3 Set-Up and Calibration

3.1 FLOW-3D

The FLOW-3D model was selected and utilized for all of the CFD modeling. The FLOW-3D Program is distributed and supported by Flow Science Inc., located in Sante Fe, New Mexico. This program simulates the dynamic behaviour of fluid in three

dimensions through a solution of the complete Navier-Stokes equations simulating free surface flows, including transition between supercritical and subcritical flow within a single model set-up [Ref 2]. FLOW-3D modeling was undertaken to develop conceptual designs for Keeyask GS.

3.1.1 Set-Up

For river management as discussed in GN 2.8 Rev 0, the FLOW-3D model was set-up using available topographic and bathymetric data over an area of approximately 3.3 km x 2.7 km. Simulations were conducted to represent flow conditions in the area of the Spillway Stage I Cofferdam Structure during passage of the Construction Design Flood. FLOW-3D was also used to confirm the hydraulic performance of both the Spillway and Stage II Diversion Structure [Ref 3].

3.1.2 Calibration

To facilitate direct comparison with the physical model, the CFD model was revised using the same bathymetric data that was used in the physical model. The model boundaries were also adjusted to cover the same area as the comprehensive model. The CFD model was calibrated against field derived stage discharge curves at the gauge locations. Some modifications were made to the model at the downstream location by the rapids [Ref 4]. The calibration results for different gauging stations are presented in Appendix A.

3.2 Physical Model

A physical model study was undertaken by the LaSalle Consulting Group Inc. to support the design of Keeyask GS [Ref 2]. These model studies are well documented in the papers attached in Appendix A of this memorandum, and in the Keeyask GS Stage IV Comprehensive Model Study [Ref 5] and Keeyask GS Stage IV Section Model Study [Ref 6].

3.2.1 Set-Up

The comprehensive model was built to an undistorted scale of 1/120. It covered a 3 km stretch of the Nelson River, centred on the proposed Spillway area. This model included the full Spillway structure (including all seven Spillway bays) and was built to permit simulation of various aspects of diversion, including the Stage I Spillway Cofferdam construction, river closure, operation of the diversion sluices and operation of the final rollways. In addition, a 1:50 scale Sectional Model was constructed at LaSalle Consulting Group Inc's laboratory. This model represented full bays in Bay 4 and 5 and half bays in Bay 3 and 6. The Spillway structure was

built in component modules to facilitate representation of both the diversion structure and the final rollway configuration [Ref 3].

3.2.2 Calibration

The physical model was calibrated against stage–discharge curves at all gauge locations where prototype measurements were available. The intial results showed that the downstream water levels were too high and the upstream water levels were too low. Therefore, bed contours were modified to better match the prototype rating curves. The final match obtained was very good [Ref 7]. The calibration results for different gauging stations are shown in Appendix A.

3.3 Comparison of Stage I Test Results

With the Spillway Stage I Cofferdam and Central Dam Stage I Cofferdam in place, FLOW-3D predicted velocities under a flow of 3,930 m^3/s at the upstream corner of the cofferdam to be approximately 4.5 m/s. This compares well with the 4.0 m/s to 4.8 m/s velocities recorded for the same case in the physical model. For a flow of 6,240 m^3/s, surface velocities at the upstream corner of the cofferdam are in the order of 6.5 m/s to 7.5 m/s. The physical model study showed an average velocity of 6.6 m/s in approximately the same area – very similar to the numerical model test results.

Comparative tests were also undertaken for a condition in which the Stage I Spillway Cofferdam had been advanced by 450 m, during a flow of 3,930 m^3/s. For this case, velocities near the end of the advancing cofferdam in the physical model ranged from 4.0 m/s to 7 m/s while for FLOW-3D the velocities ranged from 3.5 m/s to 6.8 m/s. This was again considered to be a good match.

3.4 Comparison of Stage II Test Results

Additional tests were performed comparing the CFD and physical model results for a case in which the Stage IV Studies South Dam Stage II Upstream Cofferdam had been advanced by 250 m during a flow of 5,270 m^3/s. Measured velocities at the head of the advancing cofferdam varied between 3.6 m/s and 6.6 m/s in the physical model, and computed velocities varied between 3.5 m/s to 5.5 m/s in the FLOW-3D model results.

With the South Dam Stage II Upstream Cofferdam advanced by 300 m under a flow of 4,500 m^3/s, velocities within the physical model varied between 2.2 m/s and 6.5 m/s. These same velocities varied between 1.8 m/s and 6.8 m/s in the FLOW-3D model.

Comparative runs were also made between the numerical and physical models for the hydraulic performance of the Stage II Diversion Structure. Structure discharge capacities were predicted for forebay levels of 148 m, 149 m, 150 m, 151 m and 152 m. The overall

difference between the FLOW-3D results and physical model study results is about ± 200 m^3/s for a 3,000 m^3/s of flow (approximately a 6 % difference). The upstream water level was also computed and compared for the flows of 3,838 m^3/s, 4,487 m^3/s, 5,170m^3/s, 5,884m^3/s and 6,628m^3/s. The difference in water level between the two models was about ± 0.2 m.

3.5 Comparison of Final Spillway Test Results

The comparison between the two models for the hydraulic design of the Spillway rollway, considered the performance of the final Spillway structure when fully open. For this test the seven 13 m wide bays were assumed to be fully open.

Flows were estimated using both models for forebay energy level of 151 m, 155 m, 158 m, 159 m and 160.3 m. The FLOW-3D results were then compared to those of the physical model. The differences in estimated discharge varied by ± 200 m^3/s for a 3,000 m^3/s discharge. The water level was also compared for flows of 2,855 m^3/s, 6,088 m^3/s, 8,952 m^3/s, 9,959 m^3/s and 11,313 m^3/s. The difference in the upstream water level between the two models is about ± 0.3 m.

4 Summary

In this memorandum both numerical and physical model test results have been compared when testing the hydraulic design of the Spillway structure and river management strategies proposed for the Keeyask GS. The results of Keeyask GS for both types of models have been analyzed from the papers and memoranda attached to this document. It can be concluded that the difference between the results extracted from the FLOW-3D and physical modeling are minimal. As stated in the papers attached, CFD modeling has been found to be suitable in determining the final conceptual configuration for the facility in a relatively short timeframe. It provides the flexibility in being able to quickly change the conceptual details and configuration. On major design projects, it is recommended that both modeling techniques be integrated during design to provide the optimum hydraulic engineering solution.

Prepared by:

D.M. Fuchs, P.Eng.

Attach
Appendix A
DMF:spa

References:

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Appendix A

Optimization of Hydraulic Design Using Computational Fluid Dynamics

Optimization of Hydraulic Design Using Computational Fluid Dynamics

By Joe L. Groeneveld, Kevin M. Sydor and David M. Fuchs, Acres Manitoba Ltd, Winnipeg, Manitoba, Canada, and Efrem Teklemariam and Brian W. Korbaylo, Manitoba Hydro, Winnipeg, Manitoba, Canada

ABSTRACT

In the past, the only viable means to analyze complex hydraulic design issues was through the undertaking of physical model studies. However, Computational Fluid Dynamics (CFD) has emerged as a powerful alternative to assist in the analysis of such complex design problems.

CFD analysis involves the solution of the governing equations for fluid flow at thousands of discrete points on a computational grid, giving the analyst a full three dimensional representation of the fluid flow domain. This numerical simulation capability gives engineers the power to quickly and inexpensively explore different design options and often demonstrates how a water resource project can be constructed and or operated more efficiently. It can be used early in the design phase to provide initial insight into critical design issues, and can also be used to compliment any planned physical model studies.

In keeping with these technological advancements, engineers at both Manitoba Hydro and Acres Manitoba Ltd have been using a commercially available CFD model, the FLOW-3D model, to assist in the advanced hydraulic design of many different components of a number of hydroelectric facilities. The purpose of this paper is to present a select number of examples in which the FLOW-3D model has been successfully used to assist in solving practical hydraulic design problems, particularly as related to hydropower plant design. Initially, examples are presented in which the numerical model was utilized to replicate a number of actual physical model studies, with good success. Following this, examples are presented in which the model was used to assist in spillway design, river management studies, ice formation studies, and Intake design.

1 Introduction

The design and engineering assessment of hydroelectric facilities involves developing an understanding of the very complex behavior of moving water. In designing each component of a proposed facility, the engineer must be able to efficiently and reliably verify the adequacy of the hydraulic design under all plant operating conditions.

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To accomplish this, the engineer must develop a thorough understanding of the complexities of fluid flow phenomena - complexities which are often highly two and three dimensional in nature. In early years, physical model studies would have been the only practical medium available to gain insight into the three-dimensional and time-dependent nature of fluid flow. However, physical modeling is typically only undertaken during the final stages of design, and can be costly to execute. With the advancements in computing power made since the 1980's, CFD analysis has emerged as a powerful alternative design tool, and can be used to provide insight into hydraulic design at all levels of study.

Manitoba Hydro and Acres Manitoba Ltd., both of Winnipeg, Manitoba, Canada, have undertaken an extensive 3-dimensional modeling program as a part of pre-commitment level studies for three proposed hydropower projects in Northern Manitoba. These three projects are the Gull Generating Station (675 MW) on the Nelson River, near Gillam, Manitoba, and the Notigi (117 MW) and Wuskwatim (206 MW) Generating Stations, both on the Burntwood River, near Thompson, Manitoba. A sophisticated computer model, FLOW-3D, was carefully reviewed and tested prior to its use as a design tool. Following this test phase, the 3-dimensional model was used to evaluate a number of complex hydraulic design issues associated with these three projects.

This paper presents selected examples where Acres Manitoba and Manitoba Hydro have applied CFD analysis in the design of proposed hydroelectric facilities in Northern Manitoba.

2 Description of the FLOW-3D Model

The FLOW-3D model [1], developed by Flow Science Incorporated of Los Alamos, New Mexico, USA, is a Computational Fluid Dynamics (CFD) model capable of simulating the dynamic and steady state behavior of liquids and gases in one, two or three dimensions. It does so through solution of the complete Navier Stokes equations of fluid dynamics. FLOW-3D is designed to produce simulations of fluid flows influenced by a wide variety of physical processes (e.g., heat conduction, solidification, surface tension, cavitation). Because the program is based on the fundamental laws of mass, momentum and energy conservation, it is applicable to almost any type of flow process. For this reason the program is referred to as a “general purpose” solver. Unlike some three dimensional models, the FLOW-3D model is capable of simulating free surface flows, and the program utilizes specialized algorithms to track the location of the water surface over large and small spatial and temporal variations. It is robust, handling transitions between subcritical and supercritical flow within a single model setup. These capabilities make the model well suited for simulating the varied and complex flow conditions, which typically occur in a variety of hydraulic design and analysis problems.

3 Confirmation of Model

Prior to relying on CFD analysis to assist in the hydraulic design of the proposed hydropower projects, considerable effort was made to verify the performance of the model under a number of real world applications. The model had been used with good success by engineers at Manitoba Hydro and Acres Manitoba Ltd. over a period of approximately 3 years to study a variety of complex hydraulic design issues. These past projects included the modeling of:

- Two and three dimensional dambreak studies
- Hydroelectric Intake design
- Channel Improvement studies on a large northern river
- Replication of past physical model studies to verify model performance
- Fishway design
- Trash boom design
- Dilution studies of a contaminated lake
- Spillway design

But perhaps what most impressed the users of the model over this period of time was the ability of the model to replicate the results of past physical model studies, with very good success. Two such example applications are described below.

3.1 Simulation of Conawapa Diversion Port

Physical model study tests were previously carried out to refine estimates of discharge through the 8 m high diversion ports of the proposed 1390 MW Conawapa Generating Station on the Nelson River. Flow conditions through the ports are highly three dimensional in nature, and in the past physical model studies were the only practical medium available to develop discharge estimates for these large structures. To establish confidence in the FLOW-3D model's ability to predict discharge capacity through a diversion port structure, the model was configured to replicate the Conawpa physical model study [2]. Figure 1 illustrates a section view through the diversion port, as setup within the FLOW-3D model.

The physical dimensions of the diversion port simulated with the 3-D model were based on the prototype dimensions used to configure the actual physical model. Fluid properties in the numerical model (viscosity, density, etc) were set to standard properties for water at a temperature of 20 degrees Celsius. Two test cases were then simulated representing operation of the fully open port at headpond elevations covering the anticipated operating range during diversion.

A comparison of the FLOW-3D results, shown in Figure 1, confirmed the FLOW-3D model's accuracy in matching the physical model test results.

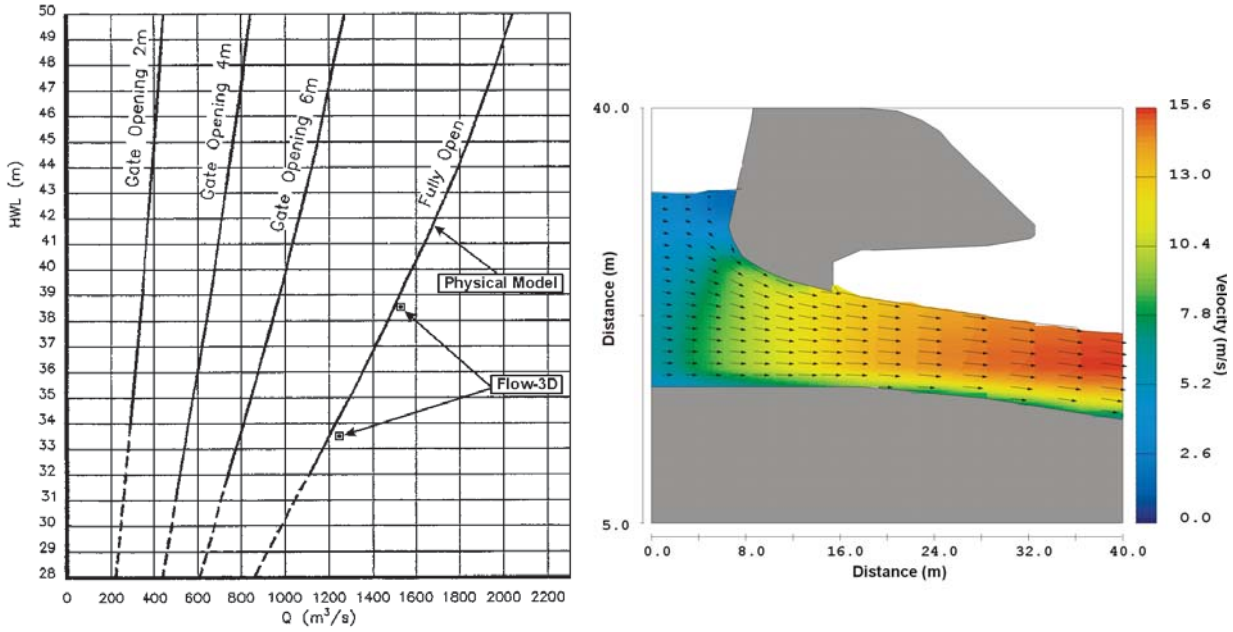

Figure 1: This figure presents a comparison of the CFD results with results from a physical model study for a fully open diversion port (see left) and a velocity magnitude profile of the FLOW-3D results (see right).

3.2 Simulation of Conawapa Stage II Diversion

A second simulation was undertaken which tested the model's ability to replicate complicated flow patterns. For this case Manitoba Hydro and Acres Manitoba developed a simulation of a diversion scheme proposed to be used during the construction of the Conawapa hydropower project. In earlier design studies employing physical model studies, engineers found that one particular diversion scheme resulted in the formation of some very interesting flow patterns in the model. As illustrated in Figure 2, taken from the actual model study report [3], flow was being diverted through the partially completed powerhouse and the completed spillway structure. The partially removed Stage I cofferdam created a significant barrier to flow, particularly as it approached the north bank powerhouse location. As shown in Figure 2, the FLOW-3D model closely reproduced both the flow patterns and the magnitudes of flow velocity observed in the physical model.

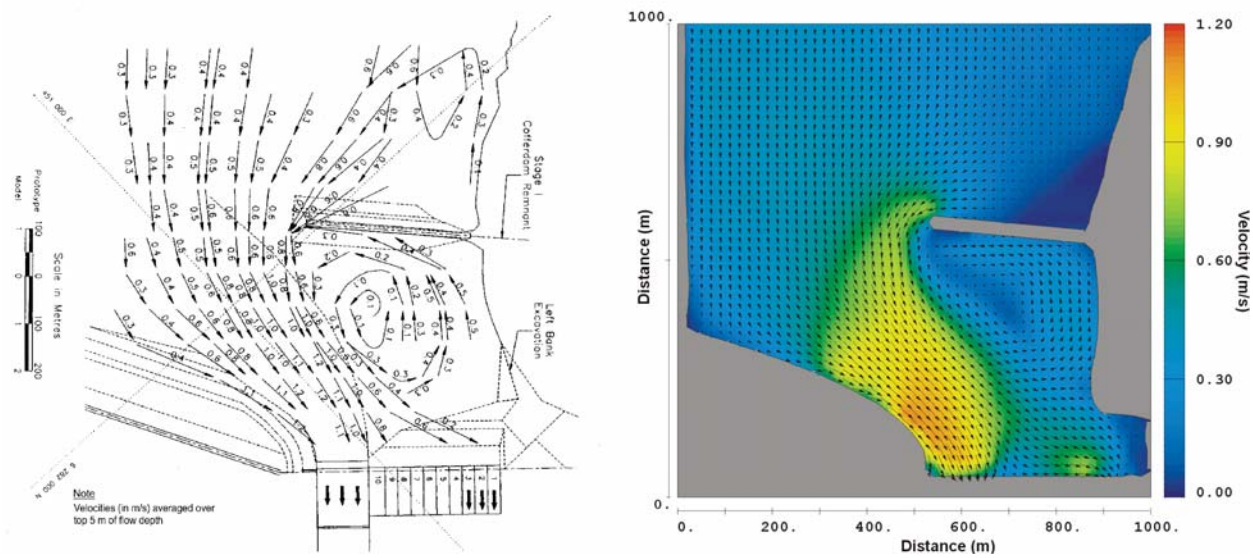

Figure 2: Manitoba Hydro initially used a physical model to analyze flow behavior at a diversion structure for the proposed Conawapa hydro project (see left). Then a computational fluid dynamics model was applied to the same diversion design, in a simulation that closely reproduced both the patterns and magnitudes of flow velocity observed in the physical model study (see right).

4 Current Design Applications

After gaining confidence in the numerical model's capabilities, engineers at Acres and Manitoba Hydro then used the model to assist in pre-commitment level design studies for the proposed Wuskwatim and Gull Generating Stations in Northern Manitoba. The model was setup and used to provide guidance on a number of technical design issues, including but not limited to:

- Confirmation of spillway discharge estimates
- Simulation of three dimensional flow patterns in the project's forebay area
- Refinement of cofferdam layouts
- Refinement of powerhouse intake design

Each of these examples is discussed in more detail below.

4.1 Gull Spillway Design

The spillway structure proposed for the Gull project consists of eight 13 m wide bays, each constructed with an elliptical crest rollway. In order to divert river flows during construction of the project's main dam, each bay will initially be constructed as a low level diversion port. The FLOW-3D program was used to simulate the hydraulic performance of these final rollways and diversion ports. The results of the 3-D modeling were used to verify discharge capacities, and evaluate water surface profiles, velocities and pressures through the spillway structure.

4.1.1 Final Rollway Design

The FLOW-3D model was used to confirm the discharge capacity of the elliptical crest rollways proposed for the project. Simulations were undertaken to assess total discharge capacity at the facility's normal full supply level of 159.0 m, and also during passage of the project's design flood in which the forebay is surcharged by 1.3 m to el 160.3 m. With a crest elevation of 146.5 m, the spillway is designed to pass the design flood under a total head of approximately 13.8 m. Initially, discharge estimates for the spillway were made using standard empirical techniques. Following this, a three dimensional numerical model was developed of the spillway structure. The numerical simulation matched closely to the empirically estimated discharge rating curve, as shown in Figure 3.

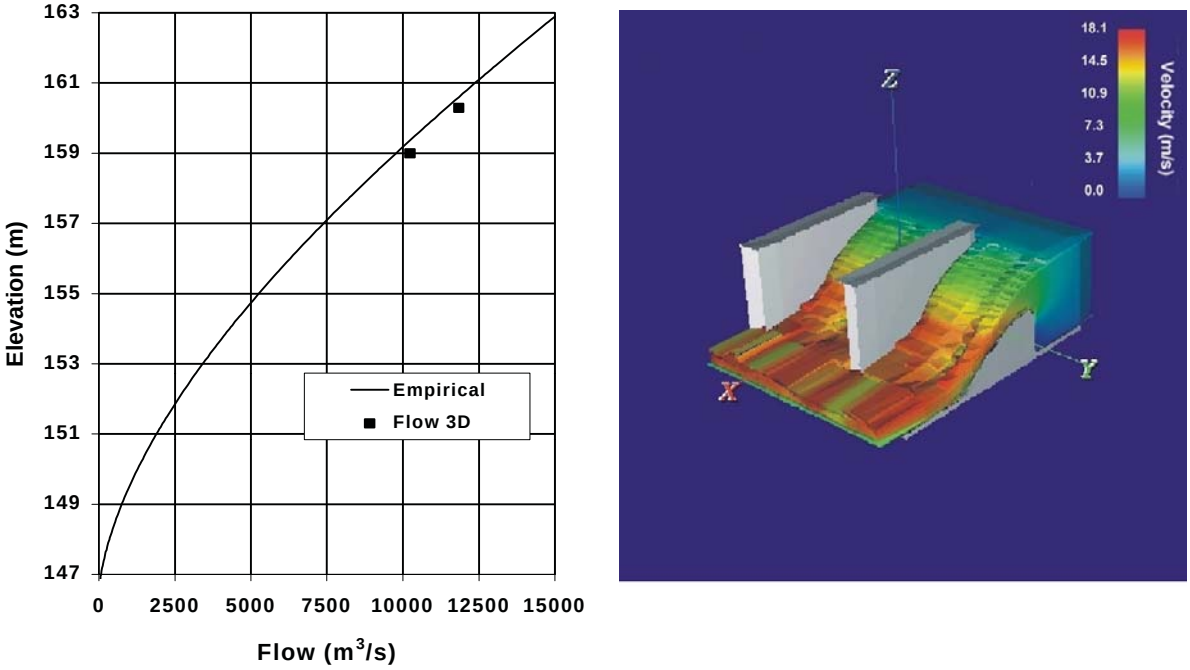

Figure 3: This figure presents a comparison of FLOW-3D discharge estimates with empirical estimates for the Gull final spillway (see left) and a 3-D plot of velocity magnitude contours for the Gull spillway (see right).

4.1.2 Performance of Diversion Ports

During construction, the river will be diverted through 8 low level ports constructed within the partially completed spillway. These ports are each 6.5 m high, and a full 13 m wide. The ports have been designed with an elliptical soffit entrance to minimize potential negative pressures on the port roof. The prediction of discharge capacity through low level diversion ports can be a difficult task, particularly under flow conditions in which the ports may be partially submerged by a higher downstream tailwater level. Initially the discharge capacity for each port was estimated based on solution of the standard momentum and energy equations. Following this, a three dimensional

numerical model was developed in order to confirm/refine the theoretical discharge capacity estimates. The numerical model confirmed the theoretical capacity estimates including the transition between submerged and free-flow operating ranges, as shown in Figure 4.

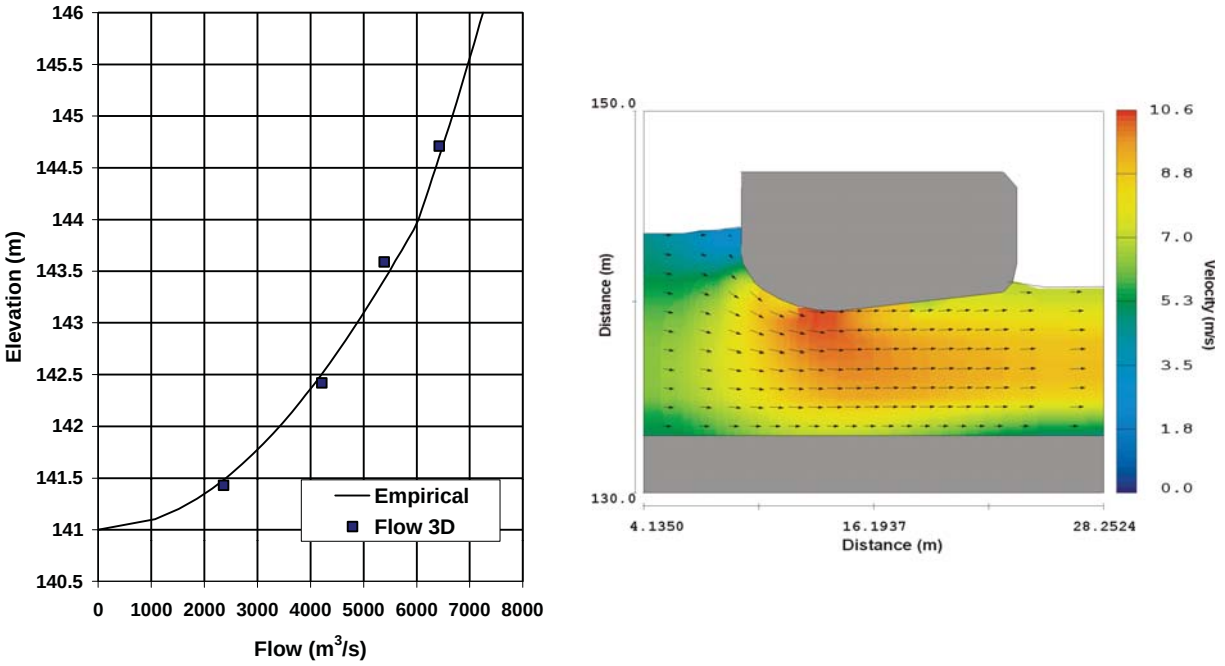

Figure 4: This figure compares the FLOW-3D discharge estimates for the Gull diversion ports against theoretical design calculations (see left) and shows a velocity magnitude profile through a diversion port (see right).

4.2 Gull Stage I Cofferdam Alignment

The FLOW-3D program was also used to assist in planning the general layout of the Stage I cofferdam for the proposed Gull project previously described. The principal structures for the proposed development would be constructed within the confines of a large earthen cofferdam built out in to the main river channel. The cofferdam would restrict the river to approximately one third of its natural width, resulting in a substantial increase in the main channel flow velocities. Traditionally, at this level of study analyses would be conducted with a standard one dimensional backwater model in order to assess the maximum water levels and velocities along the cofferdam legs. However, given the alignment of the cofferdam, shown in Figure 5, it was considered that flow around the structure was highly 2 and 3 dimensional in nature. Therefore it was considered important to analyze flow conditions in a full three dimensional environment.

Accordingly, a three dimensional numerical model was developed of the area based on existing topographic and bathymetric data. The model covered a 1.5 km by 1.5 km square area around the Stage I cofferdam. Simulations were conducted to represent

flow conditions around the cofferdam structure during passage of its design flow condition, the 1:20 year flood. The left side of Figure 5 represents resulting flow velocities around the cofferdam for one of the initial alignments tested. As shown in the figure, velocities along the face of the cofferdam are relatively high for this design condition. Velocities are greater than 4 m/s over a length of some 300 m, and approach 6.5 m/s near the north west face of the structure. In order to reduce the quantity of large rock required to protect the cofferdam over this length, additional configurations were tested in which small spur groins were added at strategic locations along the length of the cofferdam. This form of protection has been successfully utilized in earlier Nelson River hydro projects. The purpose of the spur groins is to deflect and therefore reduce velocities along the river leg of the cofferdam. The right side of Figure 5 illustrates the resulting flow distributions and velocities with the addition of two spur groins. As shown, velocities along the cofferdam have been reduced by 1.5 to 2 m/s over a substantial length of the cofferdam. The added protection provided by the spur groins would allow substantially smaller rock sizes to be adopted, resulting in an overall cost saving to the project.

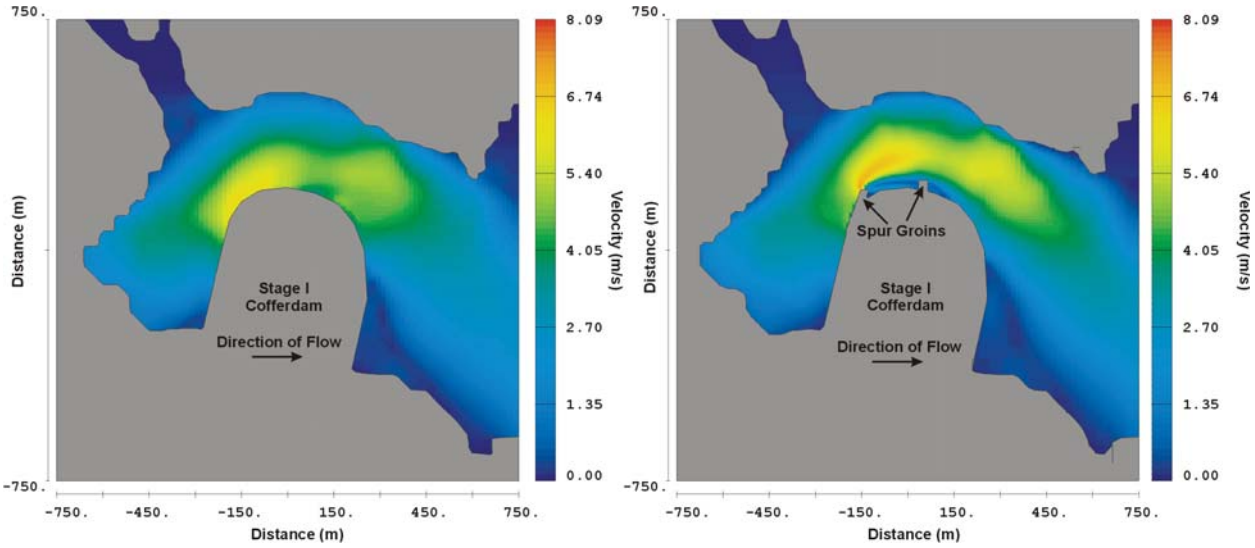

Figure 5: This figure presents predicted velocity magnitudes under design flow conditions during Stage I diversion at the Gull Generating Station without groins (see left) and with groins (see right).

4.3 Gull Forebay Modeling

The FLOW-3D program was also used to simulate flow conditions within the Gull forebay during normal plant operation. In particular, the model was used to evaluate velocities over a section of causeway located approximately 1.5 km upstream of the plant, shown in Figure 6. Construction of a causeway across the river is required in order to provide early access to both sides of the river during construction of the project's forebay dikes. However, following impoundment, the causeway would be inundated, and there was concern that the resulting flow restriction at this area may

result in locally high velocity zones. If the velocities in these areas exceed approximately 0.7 m/s, it may compromise the ability of the reservoir to form a competent ice cover. To address this concern, a three dimensional model was developed of the forebay area.

The model covered a reach of the Nelson River, which extended some 4.5 km upstream of the proposed powerhouse location. The causeway was included in the model setup, with a crest elevation of 153 m. Plant operation was simulated for a condition in which the Gull powerhouse was discharging at it's full plant capacity of 4,000 m^3/s, with an upstream forebay level of 158 m. This is the lower end of the proposed forebay operating range for this development.

Figure 6 presents a plan view of predicted velocity magnitude contours within the Gull forebay for this operating condition. As shown, velocities are greatest over the north causeway section and former islands but are less than 0.7 m/s. Accordingly the decision was made to leave the causeway in place, without a requirement for it's partial removal.

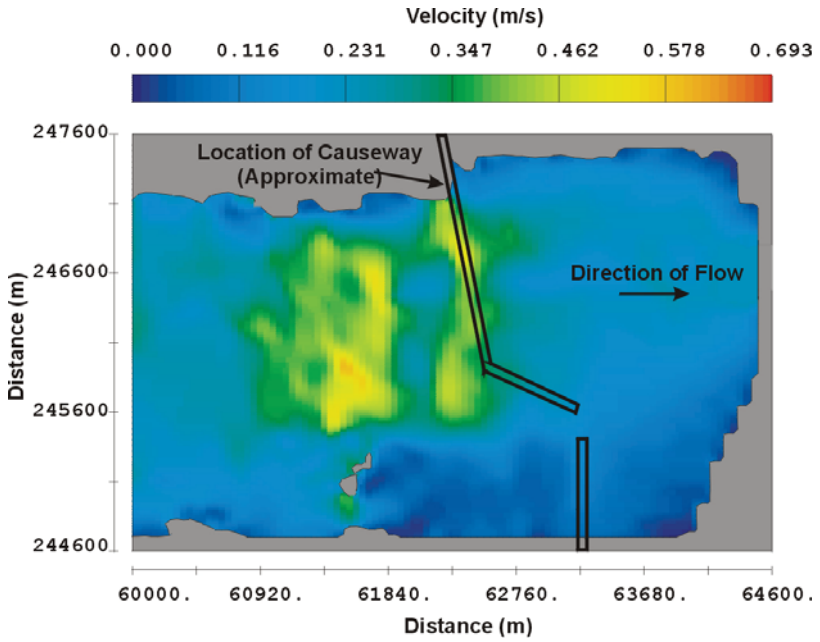

Figure 6: This figure presents the predicted surface velocities under normal operating conditions for the Gull forebay.

4.4 Wuskwatim Intake Design Comparison

The proposed Wuskwatim powerhouse is equipped with three units, and has a maximum plant capacity of 1100 m^3/s. As a part of the powerhouse design, the FLOW-3D model was used to evaluate the overall hydraulic performance of a number of powerhouse intake configurations. Two of these alternatives are shown in Figure 7. The use of a 3-D numerical model allowed design engineers to quickly visualize resultant flow fields, and evaluate alternative designs early in the design process.

Based on the simulation results, it was concluded that each of the tested alternatives produced a relatively streamlined flow field, minimizing the potential for formation of air entraining vorticies. Each alternative was also tested to ensure that cross flow velocities within the intake were minimized, and that there was an equal distribution of flow between all three units. The primary difference between alternatives was in the vertical distribution of flow at the Intake entrance, as shown in Figure 7. Of the designs tested, the alternative shown in the right side of Figure 7 exhibited the most even distribution of velocity, resulting in the lowest estimated trashrack losses at the entrance to the Intake.

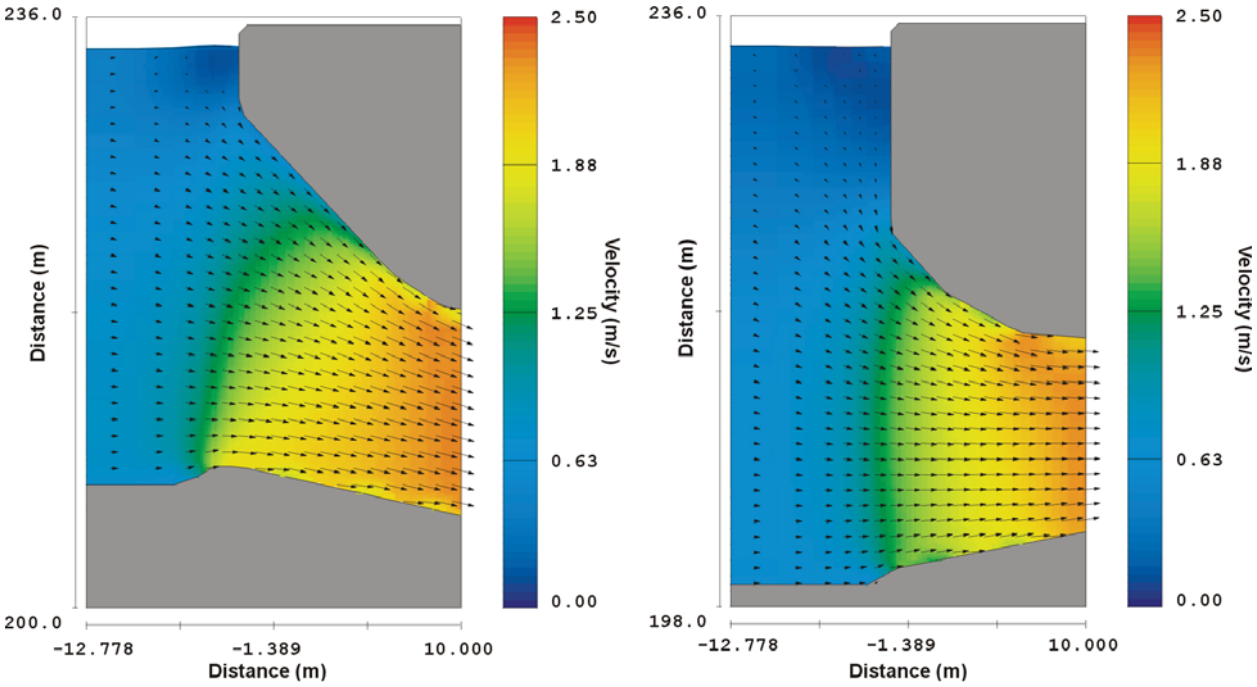

Figure 7: 3-D Model of velocity profiles for two alternative intake designs for the Wuskwatim project.

4.5 Wuskwatim 3-D Spillway Model

Three dimensional numerical simulations were also undertaken to assist in the hydraulic design of the Wuskwatim spillway structure. The proposed spillway for this project is comprised of three gated sluiceways, each nine meters wide, separated by intermediate piers. The spillway is designed to handle the probable maximum flood (PMF) of 2700 m^3/s at a design head of 15.8 m. The approach channel to the spillway consists of a relatively deep channel excavated into rock. In order to maximize spillway efficiency, designers aligned the approach channel side walls to coincide closely with the spillway entrance wall – a feature which was expected to reduce abutment contraction losses for the structure.

A 3-D model was setup and utilized to confirm the anticipated hydraulic performance of the proposed arrangement. The results indicated lateral contraction effects, and therefore abutment losses, would be substantially decreased for the proposed design, and as a result an approximate 5% increase in spillway capacity at the design head was realized. This in turn allowed the designers to increase the spillway invert, and reduce overall construction costs.

The model was also utilized to assess the need for downstream stoplogs and guides for the spillway structure. The spillway discharges into a relatively steep channel, and as a result it was anticipated that little would be required to protect against tailwater effects in the event that one of the bays were to be closed for maintenance, while other bays continued to operate. To test this assumption, a 3-D model was setup of the spillway in which a single bay was assumed to be closed, while the other two bays operated with the gates fully raised. Figure 8 shows the results of the simulation. As shown, the momentum of the water being discharged from the operating bays prevents water from entering the "dry" bay from downstream. Accordingly, downstream stoplog requirements for the structure were reduced.

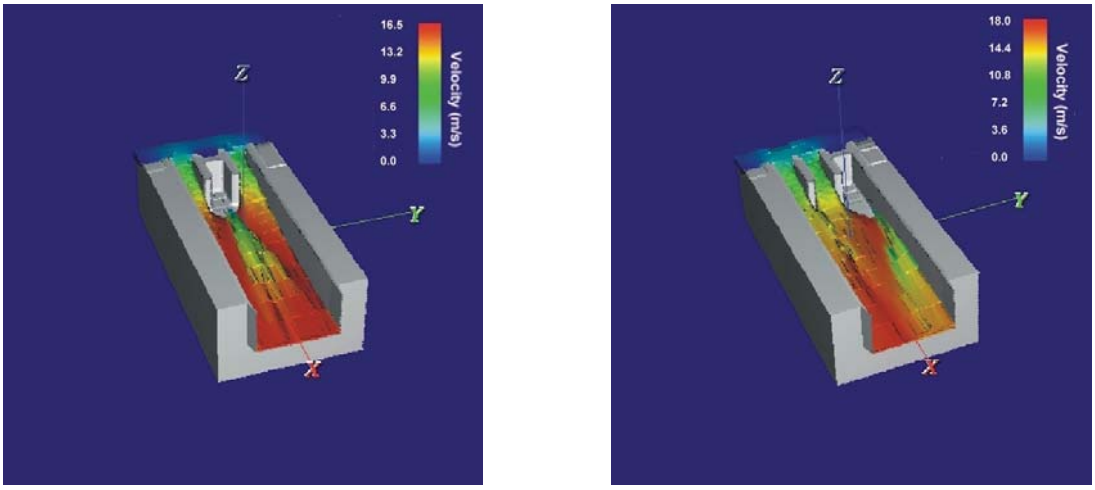

Figure 8: 3-D velocity magnitude plots of Wuskwatim spillway during the construction design flood, with north bay closed (see left) and with center bay closed for maintenance (see right).

5 Conclusions

In summary, CFD analysis has been successfully used to provide important design support for advanced hydropower design. The use of this tool has allowed engineers at Manitoba Hydro and Acres Manitoba to provide early input to critical design issues, and refine hydraulic design to a level normally achieved only through the undertaking of physical model studies.

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Authors

Efrem Teklemariam, P. Eng., is Hydrotechnical Studies Engineer at Manitoba Hydro. He was responsible for implementing the 3-dimensional modeling program for the studies, as well as providing direction and input to technical issues.

Brian W. Korbaylo, P. Eng., is Head of the Hydraulic Engineering Section at Manitoba Hydro. He was responsible for the study management of the proposed projects and review of this paper.

Joe L. Groeneveld, P. Eng., is Head of the Hydraulic Department of Acres Manitoba Ltd, Winnipeg. He was responsible for the overall direction of the hydraulic model studies. As a recognized specialist in hydraulic modeling, he was involved in selection, confirmation and application of the FLOW-3D model, and provided advice and expertise throughout the studies.

Kevin M. Sydor, P. Eng., is an Intermediate Hydraulic Engineer at Acres Manitoba Ltd. He was responsible for carrying out hydraulic studies and design of the proposed hydroelectric projects presented in this paper, and was involved in coordinating and applying the FLOW-3D model.

David M. Fuchs, EIT, is an Engineer-In-Training at Acres Manitoba Ltd. He was involved in the detailed model setup, running the model, obtaining results, preparation of figures and also participated in the hydraulic design for the proposed hydroelectric projects presented in this paper.

Mr. Teklemariam may be contacted at Manitoba Hydro, 820 Taylor Avenue, P.O. Box 815, Winnipeg, Manitoba, R3C 2P4 Canada; (204) 474-3174; E-mail: eteklemariam@hydro.mb.ca

Mr. Groeneveld may be contacted at Acres Manitoba Limited, 6th Floor, 500 Portage Avenue, Winnipeg, Manitoba R3C 3Y8 Canada; (204) 786-8751; (888) 824-0441 Fax: (204) 786-2242; E-mail: jgroeneveld@acres.com

Combining Computational and Physical Modeling to Design the Keeyask Station

Combining Computational and Physical Modeling to Design the Keeyask Station

To develop a design for the proposed 675-MW Keeyask station, Manitoba Hydro integrated computational fluid dynamics modeling and physical modeling. Using CFD modeling to determine the conceptual configuration of the facility and spillway, then verifying with physical modeling, revealed good agreement between the two methods.

By Efrem Teklemariam, Bernie Shumilak, Don Murray, and Graham K. Holder

To validate the proposed design of the 675-MW Keeyask Generating Station on the Nelson River in Manitoba, Canada, Manitoba Hydro performed computational fluid dynamics (CFD) modeling and then physical modeling. Combining results from these two modeling techniques allowed Manitoba Hydro to validate the proposed design of the Keeyask station, while providing further confidence in the use of CFD modeling for optimizing hydro plant design.

To validate the proposed design for the Keeyask station, Manitoba Hydro first used three-dimensional (3D) CFD modeling to visualize flow fields and evaluate alternative designs for the station.

Second, personnel carried out two physical model studies:

— A comprehensive study of the river closure, diversion works, and spillway. The goals were to confirm the plans for managing river flow during construction and to confirm the overall performance and discharge carrying capacity of the proposed spillway; and

— A sectional study of the spillway. The goals were to determine uplift and downpull forces on the spillway gates and detect any tendency for gate vibration; determine the pressure along adjacent spillway piers and on the rollway surface to ensure that excessive negative pressures do not occur for a variety of gate openings; determine water surface profiles to aid in the selection of pier height geometry; and verify the estimated discharge capacity of the diversion sluiceway and final rollway.

Background on Keeyask

The site of the Keeyask station is 730 kilometers north of Winnipeg, Manitoba, Canada, at the head of Stephens Lake (see Figure 1 on page 2). Stephens Lake is the reservoir for Manitoba Hydro's 1,220-MW Kettle station. The proposed axis for the spillway, powerhouse, and main dam crosses the Nelson River at the foot of Gull Rapids. The proposed facility will develop 18 meters of the available 27 meters of head between the tailwater of Manitoba Hydro's 224-MW Kelsey station and Stephens Lake. Seven generating units are to be installed, with a total capacity of 675 MW at a plant discharge of 4,000 cubic meters per second (cms).

The arrangement selected during the planning stage of the design for the Keeyask station consists of:

— A 3-kilometer-long dike on the north river bank;

— A powerhouse on the north side of the river, connected to the dyke via the north dam and the powerhouse north transition (a concrete gravity section);

— A spillway on an existing island, about 1.5 kilometers south of the powerhouse, equipped with seven fixed roller vertical lift gates;

— A powerhouse south transition, central dam, and spillway north transition to connect the spillway and powerhouse; and

— A 2-kilometer-long dike on the south river bank, connected to the spillway by a south dam and the spillway south transition.

River diversion will take place in two stages. Stage I involves construction of a series of cofferdams to allow construction of the principal structures in the dry. Stage II involves closing off the south channel of the river and building two parallel cofferdams to divert flow through the open sluices of the partially completed spillway. During this stage, the south dam will be constructed in the dry.

CFD modeling

The model selected to perform these assessments is FLOW-3D from Flow Science in Santa Fe, New Mexico.[1] This CFD model is capable of simulating the

Efrem Teklemariam, P.Eng., senior hydrotechnical studies engineer at Manitoba Hydro, supervised and coordinated the hydrotechnical studies, including computational fluid dynamics (CFD) modeling, for the environmental impact statement (EIS) for 675-MW Keeyask. Bernie Shumilak, P.Eng., special studies hydrotechnical engineer at Manitoba Hydro, coordinated the physical modeling of the Keeyask EIS. Don Murray, P.Eng., senior hydraulic engineer at Hatch Acres, supervised and coordinated the hydraulic studies for the Keeyask stage IV studies, including the CFD and physical model studies. Graham Holder, P.Eng., senior engineer and president of the LaSalle Consulting Group, was responsible for the physical hydraulic model studies.

☑ **Peer Reviewed**
This article has been evaluated and edited in accordance with reviews conducted by two or more professionals who have relevant expertise. These peer reviewers judge manuscripts for technical accuracy, usefulness, and overall importance within the hydroelectric industry.

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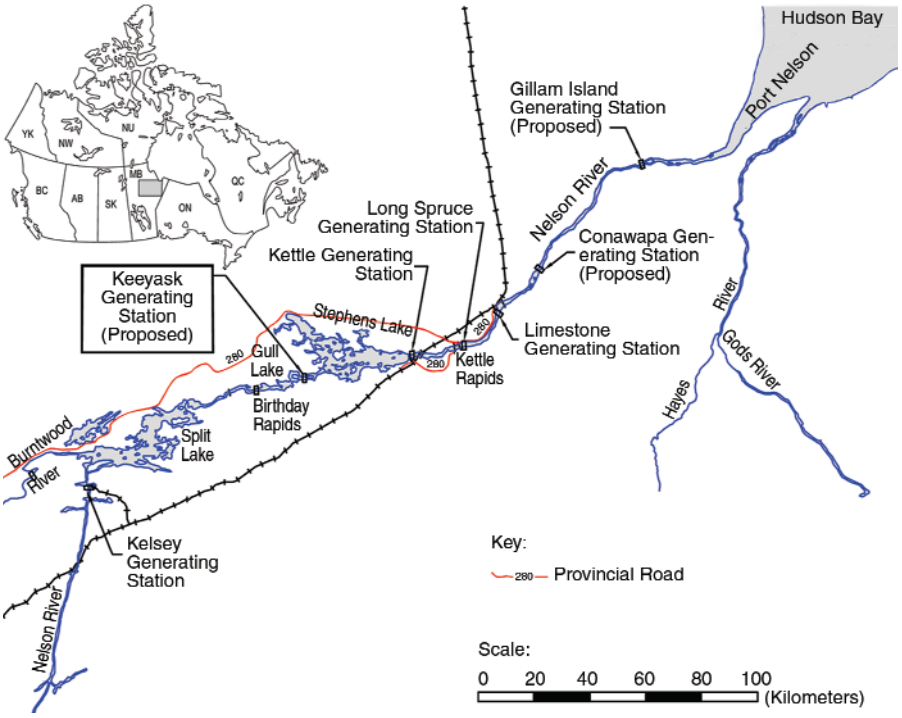

Figure 1: The 675-MW Keeyask Generating Station will be located on the Nelson River, between the existing 1,220-MW Kettle and 224-MW Kelsey stations.

dynamic and steady-state behavior of liquids and gases in one, two, or three dimensions. The model can simulate free surface flows and can handle transitions between subcritical and supercritical flow within a single model setup. These capabilities make it well-suited for simulating the varied and complex flow conditions associated with most water resource projects.

Setting up and calibrating the model

The CFD model covered an area about 3.3 kilometers long by 2.7 kilometers wide. Simulations were conducted to represent flow conditions near the cofferdams during passage of the construction design flood. The initial maximum tailwater level used for the CFD model was 141.1 meters at Stephens Lake, which represents the normal maximum lake level.

The model was later revised using updated bathymetric data used during the physical modeling. The model extent was adjusted to cover the same approximate area as the comprehensive model — 3 kilometers long by 2 kilometers wide — centered on the spillway structure. The boundary conditions were: a velocity boundary to control flow into the upstream end and a continuative outflow boundary at the downstream end (which was controlled by the natural river bed contours).

The model was set up to use the renormalized group turbulent model and the generalized minimum residual method implicit pressure-velocity solver. The renormalized group method was chosen for its robustness. The generalized minimum residual method, although requiring more memory than other methods available in the CFD model, was selected for computational speed. The mesh was set up in terms of Cartesian coordinates (x, y, z) and used nested mesh blocks to refine the grid in areas that required finer meshing. The area around the spillway structure had a grid spacing of 1 meter by 1 meter by 1 meter. This spacing was necessary to resolve the shapes of the spillway piers, abutments, and spillway crest.

Once the CFD model was developed, it was calibrated against stage-discharge curves at gage locations where measurements were available. Some modifications to the downstream portion in the rapids section were required to calibrate the model. Because limited cross-sectional data was available in the rapids section of the river, some changes to the model were made based on photographs taken during lower flows.

Modeling the proposed design

The next step was to construct the cofferdams, spillway structure, abutments, and piers using AutoCAD software. These solid geometric objects then were saved as stereolithographical (STL) files that were brought into the CFD model.

The CFD model was intended to address the principal hydraulic concerns. After gaining confidence in the model's capabilities, it was used to assist in the pre-commitment design studies for the Keeyask station. The model was used to provide guidance on several technical design issues, including:

— Spillway design, including confirmation of spillway discharge estimates;

— Simulation of 3D flow patterns in the forebay and tailrace;

— Refinement of cofferdam layouts;

— Refinement of powerhouse intake design;

— Channel design; and

— Refinement of river management strategies, including river closure.

For the spillway, 3D numerical simulations were undertaken to confirm the anticipated hydraulic performance of the proposed arrangement. Results indicated that pier and abutment losses would be very low. As a result, an approximate 5 percent increase in spillway capacity at the design head could be realized. This allowed designers to raise the spillway invert, thereby reducing overall construction costs.

The river diversion for this project will take place in two stages. The first stage involves blocking the north and central channels to build the central dam and powerhouse cofferdam. The second stage involves removing the spillway cofferdam, to allow construction of the south dam cofferdam across the southern portion of the river. Flow-3D and a one-dimensional backwater model supplied by Flow Science were used to estimate water levels and velocities under open water conditions at various locations during construction of these cofferdams.

The resulting velocity magnitudes were used to determine the size of the stone needed for the rockfill used for construction of the cofferdam, for selected river discharges. It is important to have a good estimate of the velocities and hydraulic forces acting on the rockfill over time. Appropriate rock sizes must be selected to resist drag forces, which will move individual boulders downstream. These design rock sizes vary depending on the degree of advancement of the cofferdam and the local bathymetry along the closure line.

Researchers performed both CFD simulation and physical modeling for a case in which the cofferdam is nearly complete near the right bank closure site. Although velocities calculated

This partial model of the spillway for the proposed 675-MW Keeyask station was developed to confirm rating curves for full and partial gate openings for both diversion and final rollway conditions, as well as to verify pressures and gate behavior.

using the CFD model and those measured using the physical model were similar, the rock size required for closure in the physical model was less than that recommended using theoretical calculations based on the measured velocities. In this case, the theoretical calculations provide a conservative estimate of the required stone size. On the other hand, the physical model allows actual testing of the stone size necessary for closure.

A small project with a limited budget may benefit from the more economical and conservative option of using only CFD modeling. However, for large or potentially sensitive projects, using physical modeling in addition to CFD modeling can provide advantages.

Physical modeling

The LaSalle Consulting Group constructed two physical hydraulic models — a comprehensive model and a partial model — in its laboratory. These models were used to confirm and refine the spillway structure design and to address potential problems during construction of the river diversion cofferdams.

Setting up the models

The comprehensive model, built to scale of 1:120, encompassed a 3-kilometer stretch of the Nelson River, centered on the projected spillway zone. Model boundaries were defined by concrete block perimeter walls, waterproofed on the inside and laid on the laboratory floor. Riverbed contours were drawn on the floor inside the walls using projection slides taken off the bathymetric and topographic charts. Once the contours were traced, aluminum rods were bent to take the individual contour line shape and were then carefully adjusted to the correct level on top of vertical steel rods by means of a laser level.

After the contours were leveled into position and checked, the model was filled with pea gravel (carefully vibrated into place) and leveled flush with the rods. The final step was to consolidate the surface of the gravel with successive applications of liquid cement slurry. Special care was taken to respect as closely as possible a Manning number of 0.035 for a rock surface by lightly applying fine gravel on the final cement coat.

The spillway and sluiceway components were made of Plexiglas, which approximates the roughness of concrete at model scale. The rollways were constructed such that they could be quickly installed in the sluiceways when carrying out tests with the completed bays. Each bay was equipped with two gate slots and gates to allow simulation of the full range of possible flows.

Water was supplied to the model from a permanent pumping station and measured with a calibrated 90-degree V-notch weir (precision of ±1 percent). Entry to the model was via a perforated T-header at the upstream limit. At the downstream end of the model, the tailwater was controlled by the natural river bed contours.

Water levels in the model were determined by means of manually read point gages located above manometer pots. This arrangement allowed measurement of the water surface level to the nearest 0.1 millimeter on the model. Six gages were installed: three along the main river channel (gages NL-17, NL-15, and NL-14), and three in the spillway channels (gages A, B, and C). An additional gage, gage NL-16, was installed downstream from the main dam after completion of the spillway stage I cofferdam construction tests.

Point velocity values were measured at 0.2 and 0.8 of the depth of the model using a mini-current meter from Nixon equipped with a 12-millimeter propeller.

To obtain an overall evaluation of the flow conditions at any given location, researchers used time lapse photography to obtain surface velocities and trajectories. A digital camera was installed on a tripod above the zone to be covered, and surface floats were introduced into the flow upstream. Successive photographs were taken at known time intervals as the floats moved downstream. Velocity was calculated using the distance traveled by each float and the time interval between photos.

Modeling the proposed design

The first model test consisted of verifying stage-discharge curves at the gage locations where measurements were available. Initial calibrations showed water levels were too high in the downstream area of the model and too low in the upstream area. As a result, researchers lowered bed contours by 1 to 2 meters at the downstream end of the model and placed small clusters of rocks to increase the upstream riverbed roughness. These modifications resulted in obtaining model rating curves that were very close to the prototype.

Figure 2 on page 4 shows the comparison of CFD model and physical model rating curves and the prototype measurements for four different discharges between 1,600 cms and 6,100 cms at gage NL-17. The curve at this gage and the other two gages (NL-15 and NL-14) show good agreement.

Three batches of model material were prepared by blending sand, screenings, and gravel to obtain the average diameter and gradation range specified in the study requirements.

Modeling the spillway

Physical model tests also were carried out to reproduce construction of the cofferdam needed to build the spillway during a constant flow of 3,930 cms (1:2 year mean daily discharge). The most

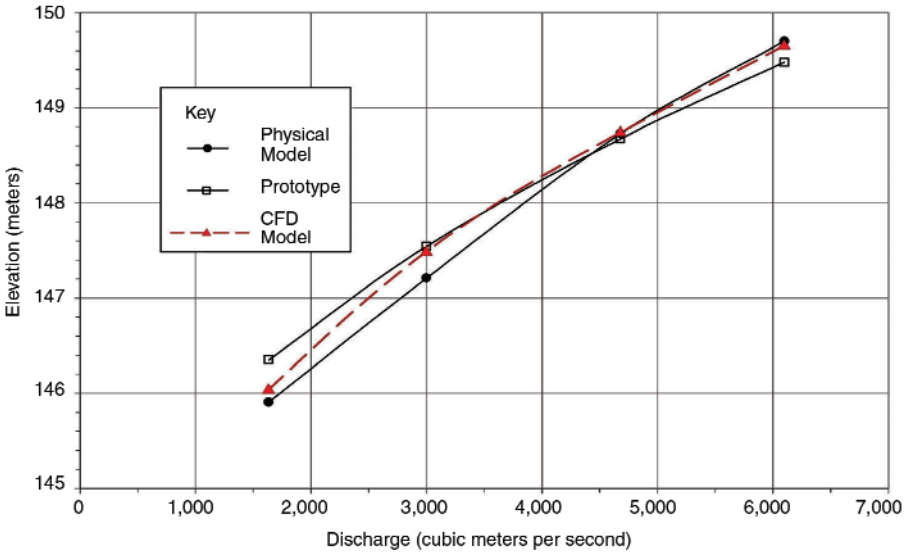

Figure 2: Comparison of the rating curves at one gage in the physical model for the 675-MW Keeyask station shows good agreement between the physical model, CFD model, and prototype.

critical zone during construction of this cofferdam was found to be at the upstream corner. This is where maximum velocities (about 5 meters per second) were measured. Once past this corner, the main flow followed the south channel of the river. This flow concentration, combined with the local bed form, resulted in the formation of back eddies along the river leg of the cofferdam. Velocities in that zone were low.

A similar flow pattern was observed at the downstream corner, and the final downstream leg of the cofferdam was also in a zone of very low velocities.

Water levels at gages NL-17 and NL-15 start to increase when about one-third of the cofferdam is built. Water levels at gage NL-14 remain unchanged.

With this cofferdam completed, the maximum rise in water level is about 0.55 meter at gages NL-17 and NL-15 and 0.2 meter at gage NL-14. The rise in water level will submerge one of the downstream islands.

After completion of this cofferdam, flow in the model was increased to 6,240 cms, the construction design flood. The cofferdam crest elevation was found to give the prescribed 1-meter freeboard predicted by the CFD model.

The spillway capacity tests with the initial configuration of the diversion structure showed that flow control was taking place at the entrance to the approach channel and not at the structure, due to high bed levels.

As a result, upstream water levels were higher than those anticipated from the original CFD studies. The original CFD model had assumed a bed elevation of 144 meters upstream of the spillway approach channel, as a result of limited bathymetric data in this area at the time of model construction. Newer bathymetric data used in the physical model upstream of the spillway approach channel resulted in the increased water levels. Excavation of the river bed to reduce the entrance of the approach channel to elevation 144 meters was carried out to lower upstream water levels to target values. With the entry of the channel excavated to elevation 144 meters, excavation along the left bank was limited to a small zone near the entrance to the approach channel. The spillway structure invert was raised by 0.6 meter, to an elevation of 138.9 meters, without affecting the upstream water levels. Subsequently, incorporating these changes into the CFD model confirmed that raising the spillway invert would not affect the upstream water levels during diversion.

Before construction of the cofferdam needed for construction of the south dam, the upstream and downstream legs of the spillway cofferdam were removed. Tests on the model showed that leaving the river leg cofferdam remnants in place had no effect on the spillway capacity and that the upstream leg of the cofferdam should be left in the upstream corner. This creates a zone of back eddies and low velocities in the first 250 meters of the south dam cofferdam upstream axis, thus maximizing the length along which it is possible to advance the south dam cofferdam with quarry run material.

Closure of the Nelson River with the south dam cofferdam was identified in the planning stage as one of the most difficult construction activities. To guard against all eventualities, three construction schemes were devised: single-, double-, and three-leg advancement approach. The cofferdam tests using the physical model showed that the single-leg method was feasible. Therefore, the other approaches were not tested.

The physical model test results also showed that the critical zone for advancement (i.e., where an increase in material from quarry run to average stone size of 0.6 meter is required) was located between 120 and 100 meters from the south bank. This zone applied to all flow rates tested. For all flow cases tested, the use of an average stone size of 1.2 meters was not required.

The natural profile of the south shoreline opposite the south dam cofferdam axis contributes in diverting the river flow away from the shore. As a result, velocities along the south shore both upstream and downstream of the south dam cofferdam are low and bank protection was not required.

Tests were done on the model to optimize the discharge channel configuration. Ideally, flow should be spread out as evenly as possible across the spillway discharge channel to maximize energy dissipation. However, it was observed that the flow in the discharge channel was skewed toward the right bank, due to the high bed levels at the discharge channel outlet along the left bank.

To reduce this skewness, the width of the discharge channel was modified. The right and left wall alignments were gradually adjusted until flow in the discharge channel was uniformly distributed. The final configuration resulted in a downstream width of 133 meters, instead of the original width of 215 meters. With the final rollways installed, the spillway capacity was tested for a wide range of flow conditions, from 3,000 cms to the spillway capacity design flow of 11,300 cms. Comparison of the model water levels registered at gage NL-15 and the theoretical levels showed good agreement.

Developing a partial model

A partial model consisting of two full bays and two half bays of the spillway, together with 200 meters of approach and 175 meters of tailrace channel, was constructed at an undistorted scale of 1/50.

Tests were carried out to confirm theoretically derived rating curves for full and partial gate openings for both diver-

sion and final rollway conditions. Tests were also made to verify pressures and gate behavior. These latter tests were dynamic and carried out with an aluminum gate equipped with a hydro-elastic hoist system.

The physical model test results for diversion and final rollway conditions with the gates fully open were compared to Flow 3-D simulations carried out by McGill University.[2] The result of the CFD and physical modeling for this aspect agreed very closely. With the diversion sluiceway fully open, the difference in results was 0.85 percent or less. With the final rollways fully open, the difference in results was 5.25 percent or less.

Lessons learned

There appears to be promising agreement between the results of the two modeling techniques. From using CFD analysis over the past ten years, our level of trust has increased to the point that we are now confident in using CFD modeling for general study purposes and some final design applications. Physical modeling is still recommended for final confirmation on major hydraulic applications where a significant element of risk is involved either for construction and/or operation.

CFD modeling has been found to be suitable in determining the final conceptual configuration for the hydraulic design of the facility in a relatively short time frame. It provides flexibility in being able to quickly change the conceptual details and configuration and allows the design to advance in stages, first using a coarse grid/mesh to scope the design and then refinining the grid/mesh as the design advances. It must be noted that care is required in setting up the CFD model (which comes from experience), which has a direct bearing on the modeling results. Another consideration is that the cost to buy or rent the CFD model is high, and this cost needs to be weighed against the cost of a physical model study.

On major design projects, it is recommended that both modeling techniques be integrated during design to provide the optimum hydraulic engineering solution. During the early stages of design, CFD modeling is quite adequate for purposes of feasibility and pre-engineering design. Before advancing to final design, the element of risk should be evaluated to determine whether confirmatory physical modeling would add value engineering to the final design. ■

Mr. Teklemariam may be reached at Manitoba Hydro, 444 St. Mary Avenue, 3rd Floor, Winnipeg, Manitoba R3C 3T7 Canada; (1) 204-480-5254; E-mail: eteklemariam@hydro.mb.ca. Mr. Shumilak may be reached at Manitoba Hydro, P.O. Box 815, 820 Taylor Avenue, Winnipeg, Manitoba R3C 2P4 Canada; (1) 204-474-3174; E-mail: beshumilak@hydro.mb.ca. Mr. Murray may be reached at Hatch Acres Corporation, 700 - 1066 W. Hastings Street, Vancouver, British Columbia V6E 3X2 Canada; (1) 604-683-9141; E-mail: dmurray@hatchacres.com. Mr. Holder may be reached at The LaSalle Consulting Group, 9620 St. Patrick Street, La Salle, Quebec H8R 1R8 Canada; (1) 514-366-2970, extension 14; E-mail: gkholder@gcl.qc.ca.

Notes

[1]"FLOW-3D User's Manual," Flow Science, Pasadena, Calif., 2002.

[2]Gacek, J., and S. Gaskin, "Thesis for Master of Engineering: Numerical Simulation of Flow through a Spillway/Diversion Structure," Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada.

Acknowledgments

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An Integral Approach Using Both Physical and Computational Modeling can be Beneficial in Addressing the Full Range of Hydraulic Design Issues

An Integral Approach Using Both Physical and Computational Modeling can be Beneficial in Addressing the Full Range of Hydraulic Design Issues

Efrem Teklemariam, P Eng, Bernie Shumilak, P Eng, Kevin Sydor, P Eng, Manitoba Hydro, Canada
Don Murray, P Eng, David Fuchs, P Eng, ACRES Manitoba Ltd, Canada
Graham Holder, P Eng, LaSalle Consulting Group Inc, Montreal, Canada

ABSTRACT

This paper discusses an integral approach of using both physical modelling and computational fluid dynamics (CFD) modelling that was used in the hydraulic design (spillway, powerhouse intake, river management) of the Keeyask Generating Station (GS) project. It is argued that although the potential is undoubtedly great and CFD is thus increasingly being used in the water resources industry for the assessment of design, screening and optimization of hydraulic structures, there are still significant areas in fluid mechanics which are poorly understood and need to be addressed through physical modeling. The hydraulic design of a water resources project can sometimes present the designer with unique and difficult challenges. In the past, the designer has usually resorted to the use of physical models to provide guidance on critical design issues that arise. However, with recent advancements in numerical modelling techniques and computing power, designers now have additional tools at their disposal to provide this guidance.

For many decades, the standard hydraulic laboratory tool, the physical model, was used to study flow phenomena. Empirical design rules have largely been formulated on the basis of the theoretical understanding aided by experimental data obtained in the field or the laboratory. However, computing power and the simultaneous development of sophisticated numerical techniques have reached the point where it is often claimed (or at least thought) that such 'old-fashioned' and 'surely expensive?' tools like physical models are redundant. Cannot everything now be predicted by the computer?

Whether numerical modelling will ever completely replace physical modelling depends on the complexity of the situation under consideration. However, it is important to recognize that CFD is not just an alternative method; in fact it brings a wide range of sophistication that can be used to enhance physical modelling, through increased interpretation and understanding. This paper addresses how an integral approach using both physical modelling and computational modelling can be beneficial in addressing the full range of design issues. Comparative results are provided where available.

1. INTRODUCTION

The proposed Keeyask Generating Station (GS) will be located 730 km north of Winnipeg Canada, on the Nelson River, at the upstream end of Stephens Lake (see Figure 1). The development will have a head of 18 meters and a plant outflow of 4000 m^3/s. The powerhouse will contain seven generating units with a total capacity of 675 MW.

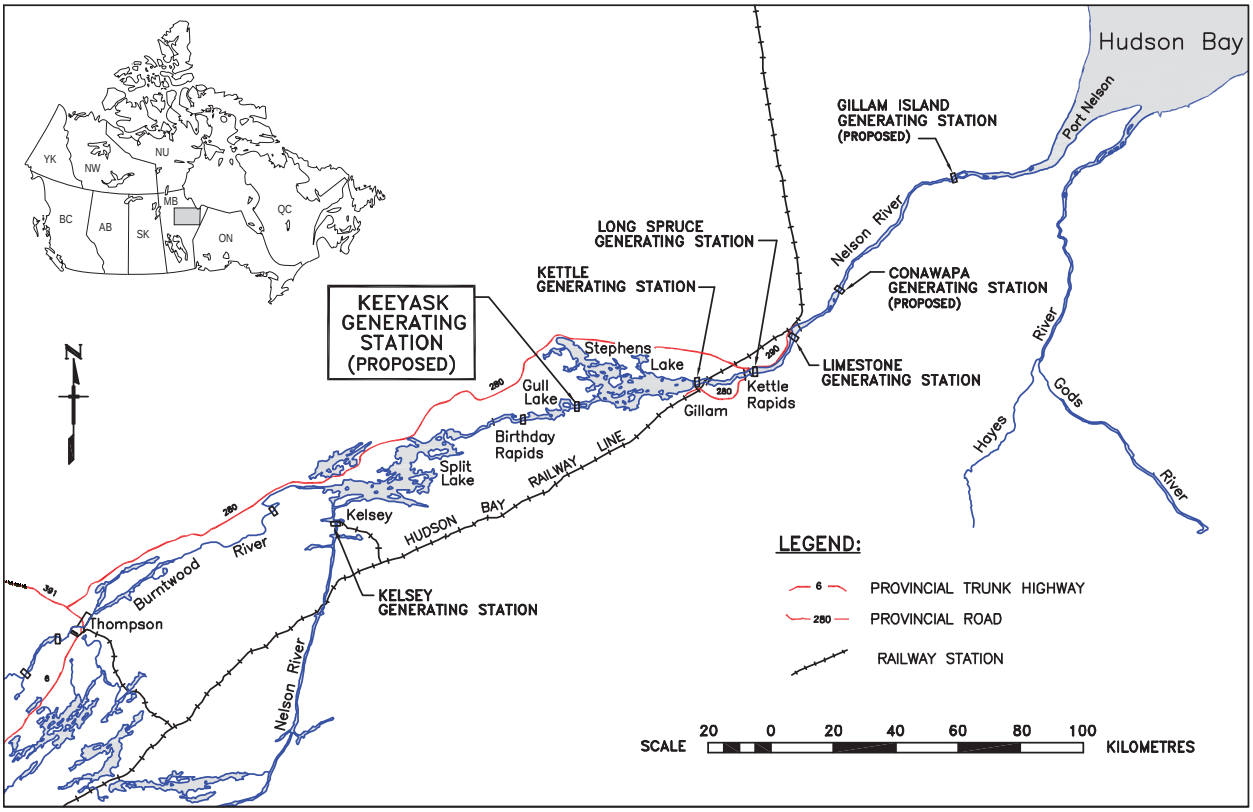


Figure 1. General Location Plan

Extensive Computational Fluid Dynamics (CFD) modelling has been used to assist in the design and optimization of the main hydraulic aspects of the project. The use of a three-dimensional numerical model allowed design engineers to quickly visualize resultant flow fields, and evaluate alternative designs early in the design process. Two separate physical model studies have been carried out: a comprehensive model study of the river closure, river diversion works and spillway; and a sectional model study of the spillway. Based on previous model and prototype experience on similar plants on the Lower Nelson River, the tests undertaken for the Keeyask GS were therefore considered confirmatory in nature and were not required to be as extensive. However, physical modeling was considered to be of primary importance due to a lack of confidence in some of the CFD results. Also, the physical model tests would reduce construction risk and provide the contractor with a higher level of confidence.

The comprehensive model study was to confirm the river management scheme during construction of the Keeyask GS and to confirm the overall performance and discharge carrying capacity of the proposed Spillway facility.

The other proposed sectional spillway model was to:

- Determine uplift and downpull forces on the gate and to detect any tendency for gate vibration.
- Determine the pressure along adjacent piers at selected locations, for structural design purposes, and on the rollway surface to confirm that excessive negative pressures do not occur for a variety of gate openings.
- Determine water surface profiles for the selection of pier height geometry
- Verify the estimated discharge capacity of the diversion sluiceway and final rollway

Over the past 10 years, Manitoba Hydro (MH) and ACRES Manitoba Ltd (AML) have applied the CFD model FLOW-3D® and physical modelling techniques as part of a range of engineering projects, with the goal of demonstrating the viability of using CFD as an operational/(re-)design tool within the company. Over this period, some problems identified as potentially benefiting most from CFD modelling have been addressed.

Projects where CFD modeling has been applied are: cavitation on stilling basin, vortices in the forebay of the powerhouse and design/optimization of new hydraulic structures for future generating plants. The modelling techniques used, the success of the projects, and some comparisons of the CFD outputs with field measurements are discussed in this paper. The use of CFD modelling to confirm suspected operational shortcomings, validate field measurements, and investigate 'what-if' scenarios is also discussed.

CFD modelling is emerging as a useful process evaluation and decision making tool for hydraulic design. CFD modelling can be used to simulate steady or unsteady flow scenarios (i.e. static or time-dependent) in one, two, or three spatial dimensions (i.e. through a three-dimensional physical volume, or an approximation thereof). CFD modelling is increasingly being used to simulate a range of processes to improve design or evaluate optimization options prior to construction.

2. PROJECT BACKGROUND

The site of the Keeyask Generating Station (GS) is at the head of Stephens Lake on the Nelson River as previously shown on Figure 1. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is Gull Rapids. The proposed axis for the principal structures crosses the Nelson River at the foot of the rapids. The preferred development of the site will have a full supply level (FSL) of el 159.0 m. The proposed facility will develop 18 m of the available 27 m of head between the tailwater of Kelsey GS and the reservoir of the Kettle GS on the Nelson River. Seven generating units are proposed to be installed, with a total capacity of 675 MW at a plant discharge of 4000 m^3/s.

Over a period of several years, various studies have examined a number of project arrangements, alternative structure axes, operating levels and plant operating modes for the development of the Keeyask GS. The selected arrangement consists of two long dykes, one on the north and one on the south river bank, linked to the principal structures which span the Nelson River at Gull Rapids as shown on Figure 2. The principal structures will consist of a powerhouse complex constructed on the north side of the river, which will be connected to the north dyke, via the north dam and the powerhouse north transition (a concrete gravity section). The spillway structure will be located on an existing island, approximately 1.5 km south of the powerhouse complex. The powerhouse south transition, the central dam and the spillway north transition will connect the spillway and the powerhouse. On the south riverbank, the south dyke will be connected to the spillway by a south dam and the spillway south transition.

In the stage I diversion phase of the project, the construction of a series of cofferdams will allow construction of the principal structures to take place in the dry. The north channel rock groin will be constructed first, to slow the flow of water through the north channel. This will enable construction of the north channel stage I cofferdam in tranquil flow conditions. Next, with the exception of the stage I central cam cofferdam, the construction of the remaining stage I cofferdams must be completed. These include the powerhouse stage I cofferdam, the stage I and II island cofferdams, and the spillway stage I cofferdam. With the stage I cofferdams constructed and dewatered, the construction of the powerhouse and spillway will proceed in the dry with the Nelson River being diverted through the south channel of the river.

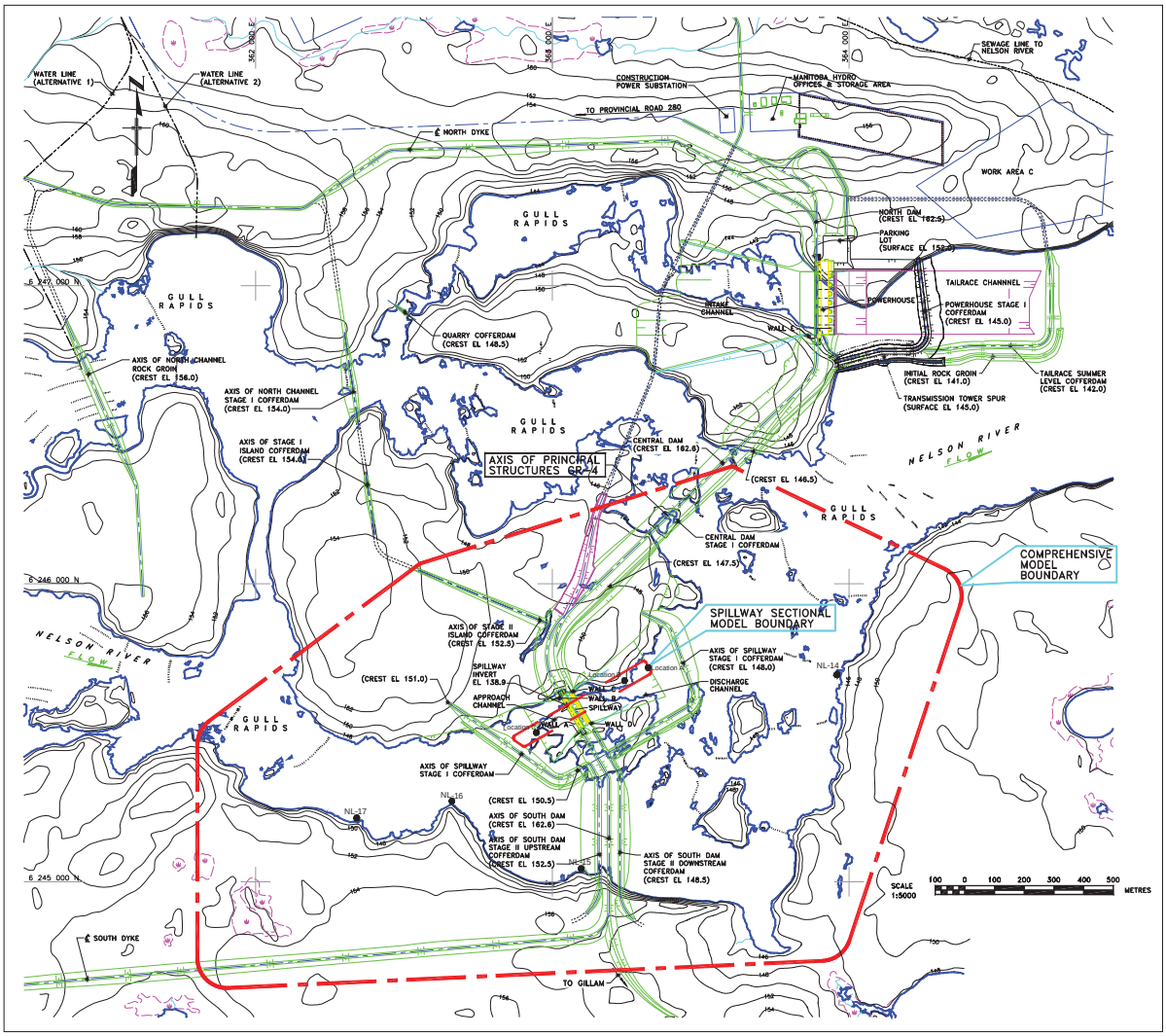


Figure 2. Keeyask GS General Arrangement

The stage II diversion phase of the project will be initiated by closing off the south channel of the Nelson River. Two parallel cofferdams, the south dam stage II upstream and downstream cofferdams, will be constructed across the south channel effectively diverting the river flow through the open sluices of the seven-bay spillway. During stage II diversion, the south dam will be constructed in the dry, between the south dam stage II upstream and downstream cofferdams.

A temporary ice boom will be constructed to ensure that an ice sheet will develop on the upstream side of the boom during river diversion. The ice boom will remain in service until the reservoir is impounded to el 159.0 m.

The spillway will be within a channel cut through the bedrock of an existing island located approximately 1.5 km south of the powerhouse. A central dam and two short concrete transition structures will connect the spillway and the powerhouse. During construction of the south dam, in the stage II diversion period, the partially completed spillway will be utilized to pass the river flow through the site.

To pass the selected 1:1000 year return period spillway design flood at the reservoir FSL, the spillway will be equipped with seven fixed roller vertical lift gates. The gates, four of which will be fully winterized, will be operated by double-drum wire rope hoists mounted on fully clad bridges and towers. River flows in excess of the spillway design flood, up to the probable maximum flood (PMF), will be discharged by the spillway, through temporary surcharging of the reservoir, in conjunction with a flow of 1400 m^3/s through the powerhouse, with the units operating at speed-no-load. The reservoir will be surcharged 1.1 m above the FSL for the PMF case.

Initially, the spillway will be only partially completed, in order to utilize it to pass the river's flow during the stage II diversion construction phase of the project. All seven bays will be constructed as simple sluices for this diversion stage.

Unlike some past construction projects on the Nelson River regulation of water levels is not required during stage II diversion to promote formation of a stable ice cover upstream of the construction area. The spillway gates will remain fully open during stage II diversion up to impounding of the reservoir. Therefore, they will not be subjected to impact by any ice floes during that period. However, during spring break in subsequent years ice impact is possible and is considered in the design. Nearing the end of stage II diversion, the spillway gates will be used to close the sluices, which will then be dewatered, one or two bays at a time, following placement of stoplogs at the upstream and downstream extremities of the piers. Rollways will then be constructed in each bay, in a sequenced schedule, using the downstream bridge deck for access.

The initiation of the stage II of the diversion works will see removal of the stage I cofferdams to the upstream and downstream of the spillway, followed by the closing of the south channel of the river with a cofferdam. This will divert the entire flow of the river through the open sluices of the 7-bay spillway.

3. CFD MODELLING

AML and MH staff have been using CFD analysis to advance the hydraulic design of many projects over the past number of years. These projects have varied in nature and complexity, and a number of previous papers by the authors have presented a cross section of specific design examples. In order to continue to share the experiences in using this powerful tool, additional design examples are presented herein

3.1 FLOW-3D®

The model selected for use in these assessments is the state-of-the-art CFD numerical model, FLOW-3D®. The FLOW-3D® model was developed and is distributed by Flow Science Incorporated of Sante Fe, New Mexico. This advanced CFD model is capable of simulating the dynamic and steady state behaviour of liquids and gases, in one, two, or three-dimensions through a solution of the complete Navier Stokes equations of fluid dynamics. The model is capable of simulating free surface flows, and can handle transitions between subcritical and supercritical flow within a single model setup. These capabilities make it well suited for simulating the varied and complex flow conditions associated with most water resource projects.

The FLOW-3D® modelling has been devised to address the principal hydraulic concerns. After gaining confidence in the FLOW-3D® model's capabilities, the model was used to assist in the pre-commitment level design studies for the proposed Keeyask Generating Station. The model was setup and used to provide guidance on a number of technical design issues, including but not limited to:

- Spillway design, including confirmation of spillway discharge estimates
- Simulation of three dimensional flow patterns in the project's forebay and tailrace area
- Refinement of cofferdam layouts
- Refinement of powerhouse intake design
- Channel design
- Refinement of river management strategies, including river closure.

Although it would not be possible to describe in detail each of the applications described above within the limitation of this paper, a number of representative examples have been presented below as they relate to the design of the Keeyask project.

3.2 Spillway Modelling

Three-dimensional numerical simulations were also undertaken to assist in the hydraulic design of the Keeyask GS spillway structure and to confirm the hydraulic performance of both the Spillway and the Stage II Diversion Structure. The model was setup and utilized to confirm the anticipated hydraulic performance of the proposed arrangement. The results indicated that pier and abutment losses would be very low with the proposed design, and as a result an approximate 5% increase in spillway capacity at the design head could be realized (Figure 3). This in turn allowed designers to raise the spillway invert, thereby reducing overall construction costs.

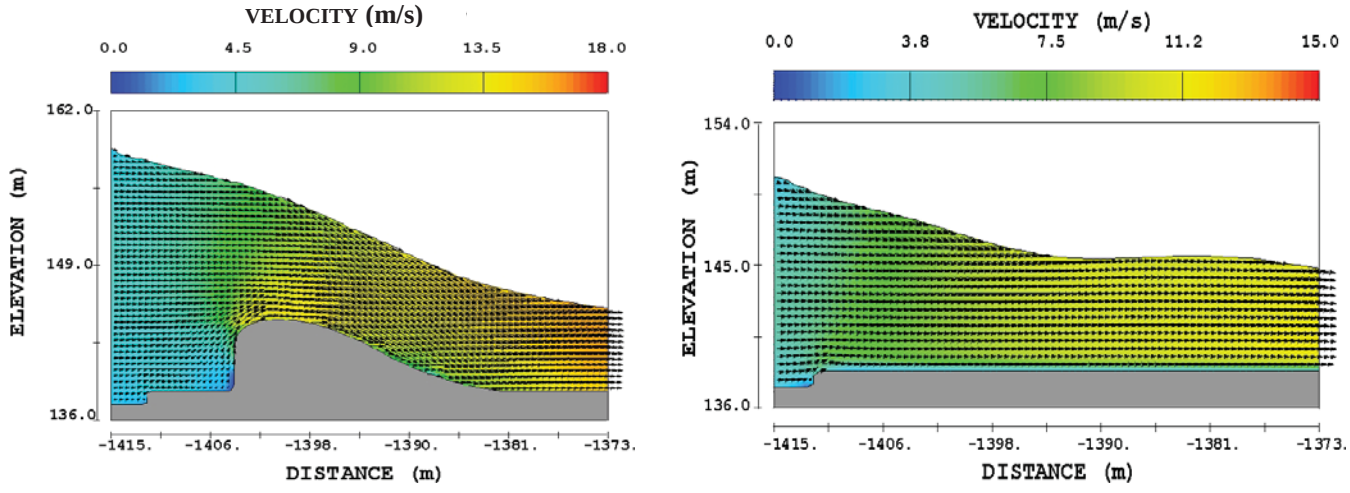


Figure 3. 3-D velocity magnitude plots of Keeyask spillway with final rollway for FSL (left), and spillway during the construction design flood (right)

3.3 River Management During Construction

A two stage program is proposed to divert the Nelson River and allow the construction of the project along the axis. The first stage involves blocking off the north and central channel to facilitate

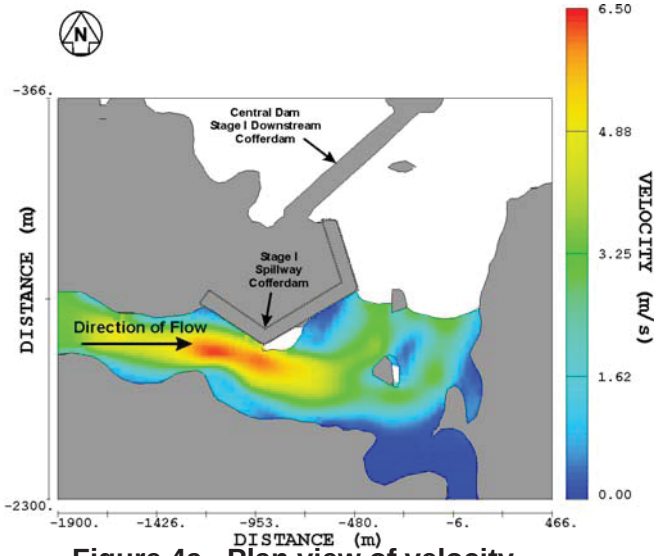


Figure 4a. Plan view of velocity magnitude around the Stage I Cofferdam (FLOW-3D)

Figure 4b. The completed Stage I Cofferdam (Comprehensive Physical Model)

construction of the Central Dam and Power House Cofferdam, as seen in Figure 4a. The second stage of diversion will involve removal of the spillway cofferdam, to allow the south dam cofferdam across the southern portion of the river, as shown in Figure 5a. The three dimensional, Flow-3D, and a one dimensional backwater model were used to estimate water levels and velocities under open water conditions at various locations during stage I & II cofferdam construction. The velocity

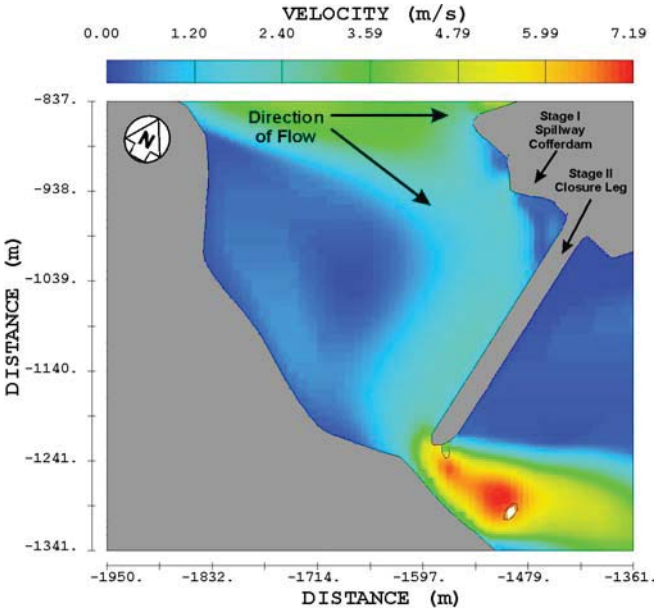


Figure 5a. Plan view of velocity magnitudes of Stage II river closure (FLOW-3D)

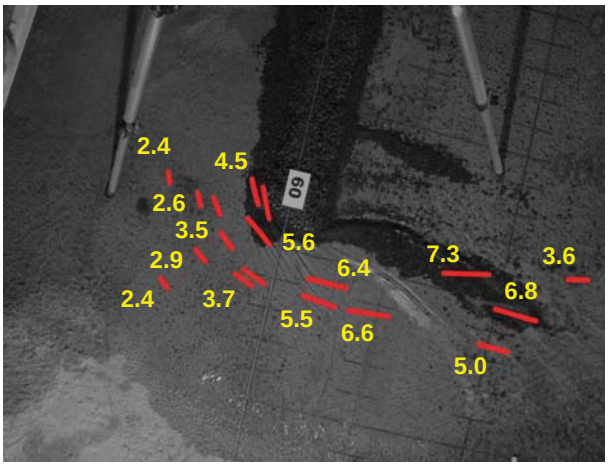


Figure 5b. Plan view of velocity magnitudes of Stage II river closure (Comprehensive Physical Model)

magnitudes were used to determine the stone sizes required to protect the cofferdam for selected river discharges.

In planning for the river closure operation, it is important to have a good estimate of the velocities and tractive forces acting on the dumped rockfill as the cofferdam is advanced. Appropriate rock sizes must be selected to resist drag forces, which will act to move individual boulders downstream. These design rock sizes will vary depending on the degree of advancement of the cofferdam, and the local bathymetry along the closure line.

Figure 5 illustrates sample simulation and physical modeling results for a case in which the cofferdam is nearly complete near the right bank closure site. Velocities appear to be similar.

4. PHYSICAL HYDRAULIC MODELS DESCRIPTIONS AND SETUP

Two physical hydraulic models, a comprehensive model and a partial model, were constructed in the LaSalle Consulting Group laboratory in order to confirm and refine the spillway structure design and address potential problems during the construction of Spillway Stage I and South Dam Stage II diversion cofferdams.

4.1 The Comprehensive Model

The comprehensive model, built to an undistorted scale of 1/120, encompassed a 3 km stretch of the Nelson River, centered on the projected spillway zone (as previously shown in Figure 2).

Model boundaries were defined by concrete block perimeter walls, waterproofed on the inside and laid directly on the laboratory floor. Selected riverbed contours were then drawn on the floor within the walls using projection slides taken directly off the bathymetric and topographic charts. Once all the contours were traced, aluminum rods were bent to take the individual contour line shape, and were then carefully adjusted to the correct level on top of vertical steel rods by means of a laser level (see Figure 6).

After the contours over the whole model area were leveled into position and checked, the model was filled with pea gravel, which was carefully vibrated into place, and screeded flush with the rods. The final step in completing the model topography was to consolidate the surface of the gravel with successive applications of liquid cement slurry. Special care was taken to respect as closely as possible a Manning number of 0.035 for a rock surface by lightly applying fine gravel on the final cement coat.

Figure 6. Model construction

The spillway and sluiceway components were built out of Plexiglas, a material which gives at model scale a roughness equivalent to that of concrete on the prototype. The rollways were constructed such that they could be quickly installed in the sluiceways when carrying out tests with the completed bays. Each bay was equipped with two gate slots and gates so as to allow the full range of possible flows through the spillway to be simulated.

Model Instrumentation

Water was supplied to the model from one of the permanent pumping stations in the laboratory and measured with a calibrated 90° V-notch weir (precision of ± 1%). Entry to the model was via a perforated T header located on the upstream limit.

At the downstream end of the model the tailwater was controlled by the natural river bed contours.

Water levels were measured at selected gauge locations by means of manually read point gauges located above manometer pots. This arrangement allowed accurate measurement of the water surface level, to the nearest 0.1 mm on the model, or 12 mm prototype.

A total of six gauges were initially installed: three along the main river channel (gauges NL-17, NL-15 and NL-14), and three in the spillway channels (gauges Location A, B and C). An additional gauge, gauge NL-16, was installed following completion of the Spillway Stage I Cofferdam construction tests. Gauge locations are presented in the previous Figure 2.

Point velocity values were measured at 0.2 and 0.8 of the depth using a Nixon mini-current meter with a 12 mm propeller.

Time lapse photography was used to obtain surface velocities and trajectories with a digital camera and surface floats. The camera was installed on a tripod above the zone to be covered. Floats were introduced into the flow upstream and successive photographs at known time intervals taken as they moved downstream. The velocity was calculated from the distance travelled by each individual float, divided by the time interval between photographs.

Model Calibration

Following the construction of the model, the first test consisted of verifying stage-discharge curves at the gauge locations where prototype measurements were available. Initial calibrations showed water levels were too high in the downstream area of the model and too low in the upstream area. Bed contours were therefore lowered by one to two metres at the downstream end of the model and small clusters of rocks placed to increase the upstream riverbed roughness. These two modifications resulted in obtaining model rating curves that were very close to the prototype.

Figures 7a and 7b show the comparison of numerical model and final model rating curves and the prototype measurements for four different discharges between 1600 m³/s and 6100 m³/s at gauges NL-17, NL-15, and NL-14. Curves at the three gauges show good agreement, particularly for the 1:2 year flow of 3930 m³/s.

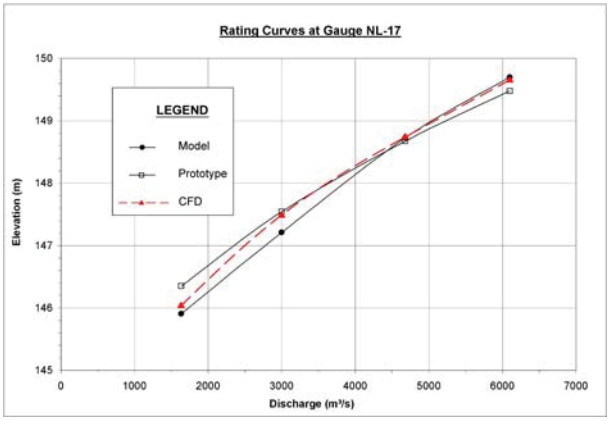
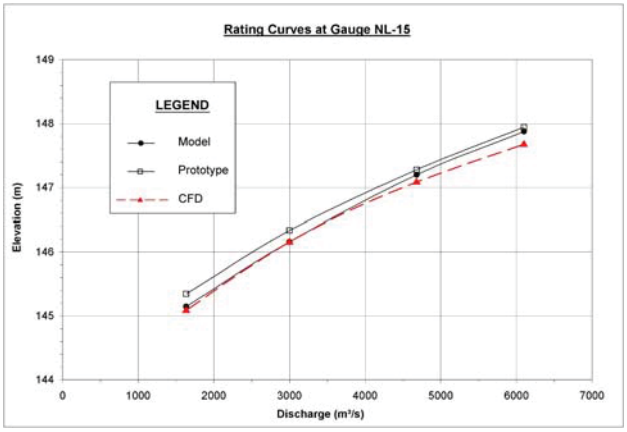


Figure 7a. Comparison of CFD, final model rating curves and prototype measurements at gauge NL-17



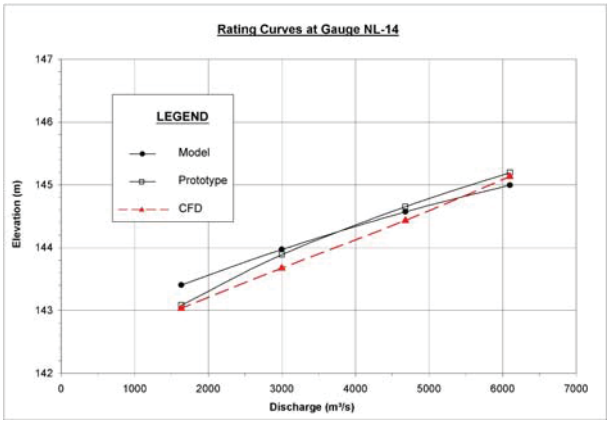
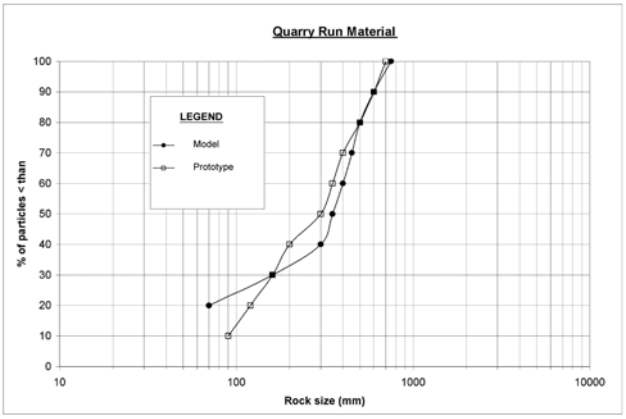
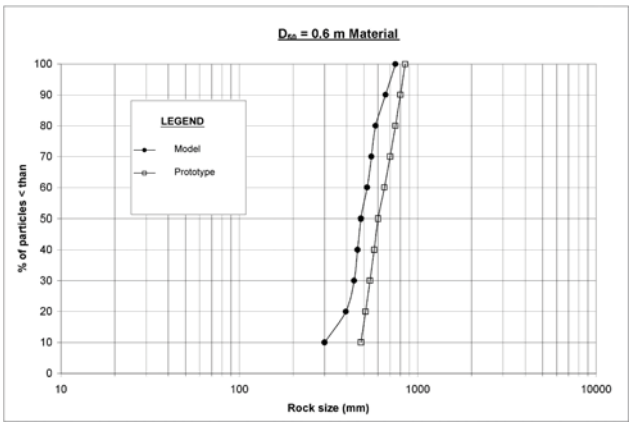


Figure 7b. Comparison of CFD, final model rating curves and prototype measurements at gauges NL-15 and NL-14

Three batches of model material were prepared by blending commercially available sand, screenings and gravel to obtain the D_{50} and gradation range specified in the study requirements.

Following several trials, the final model/prototype grain size distributions obtained for the quarry run, D_{50} = 0.6 m and D_{50} = 1.2 m cofferdam material are shown in Figure 8.





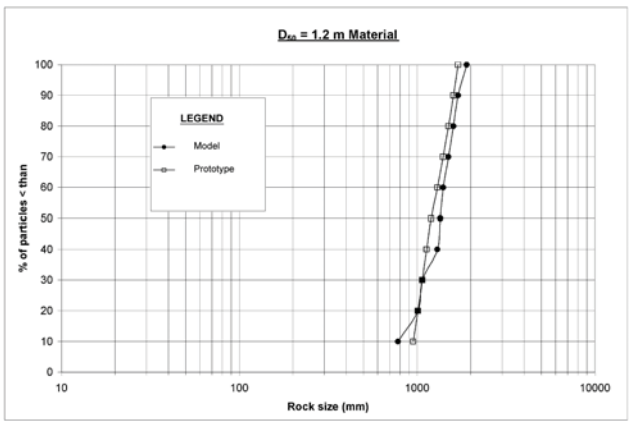


Figure 8. Comparison of cofferdam material model/prototype grain size distributions

Tests

Following model calibration, tests were carried out to reproduce the construction of the Spillway Stage I cofferdam for a constant discharge of 3930 m³/s (1:2 year mean daily discharge). During the test, only the portion along the most upstream axis of the cofferdam and at the highest crest elevation was built. The impervious fill was not reproduced. The results showed that cofferdam construction can be carried out using quarry run material without any significant losses.

The most critical zone during advancement of the Spillway Stage I cofferdam was found to be at the upstream corner. This is where maximum velocities in the order of 5.0 m/s were measured. Once past this corner, the main flow followed the south channel. This flow concentration, combined with the local bed form, resulted in the formation of back eddies along the river leg of the cofferdam. Velocities in that zone were low.

A similar flow pattern was observed at the downstream corner and the final downstream leg of the cofferdam was also in a zone of very low velocities.

Water levels at gauges NL-17 and NL-15 start to increase when approximately 1/3 of the cofferdam is built. Water levels at gauge NL-14 remain unchanged.

With the Spillway Stage I cofferdam completed, the maximum rise in water level is about 0.55 m at gauges NL-17 and NL-15, and 0.2 m at gauge NL-14 (in previous Figure 4b). The rise in water level is sufficient to submerge one of the downstream islands.

Following completion of the Spillway Stage I cofferdam, the flow was increased to 6240 m³/s, the construction design flood. The cofferdam crest elevation was found to give the prescribed 1 m freeboard predicted by the numerical model.

The Spillway Capacity Tests with the initial configuration of the diversion structure showed that control of the flow was taking place at the entrance to the approach channel and not at the structure, due to high bed levels (Figure 9).

Figure 9. Diversion structure - Initial approach flow conditions

As a result, upstream water levels were higher than those anticipated from the numerical modelling studies. Excavation of the river bed to reduce the entrance of the approach channel to elevation 144 m was carried out in order to lower upstream levels to target values. With the entry of the channel excavated to elevation 144 m, excavation along the left bank was limited to a small zone near the entrance to the approach channel. The spillway structure invert was raised by 0.6 m, to an elevation of 138.9 m, without affecting the upstream water levels.

Prior to construction of the South Dam Stage II cofferdam, the upstream and downstream legs of the Spillway Stage I cofferdam were removed. Tests on the model showed that leaving the river leg cofferdam remnants in place had no effect on the spillway capacity and that the upstream leg of the cofferdam should be left in to the upstream corner. This creates a zone of back eddies and low

velocities in the first 250 m of the South Dam Stage II cofferdam upstream axis, thus maximizing the length along which it is possible to advance the South Dam Stage II cofferdam with quarry run material (see Figure 10).

Closure of the Nelson River with the South Dam Stage II cofferdam was identified in the planning stage as one of the most difficult construction activities. To guard against all eventualities, three construction schemes were devised: single-leg, double-leg and three-leg advancement approach. The Stage II Cofferdam Tests showed that the single-leg method was feasible and the other approaches were therefore not tested (see previous Figure 5b).

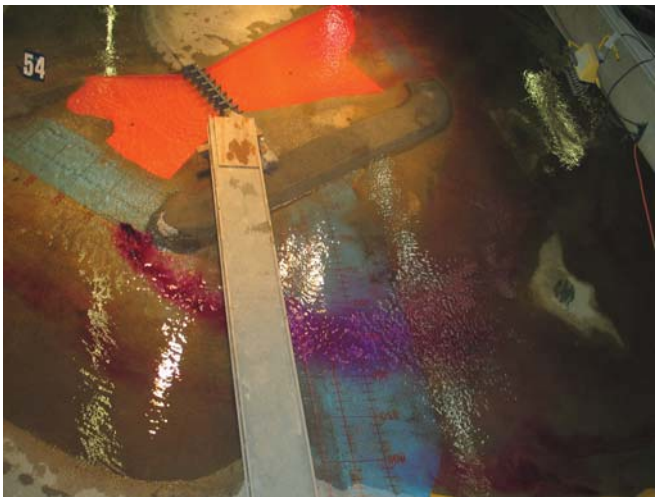


Figure 10. Flow patterns with Stage I cofferdam remnants

The model test results also showed that the critical zone for advancement, i.e. where an increase in material from quarry run to D_{50} = 0.6 m is required, was located between 120 m and 100 m from the south bank. This zone applied to all flow rates tested. For all flow cases tested, the use of D_{50} = 1.2 m material was not required.

The natural profile of the south shoreline opposite the South Dam Stage II Cofferdam axis contributes in diverting the river flow away from the shore. As a result, velocities along the south shore both upstream and downstream of the South Dam Stage II cofferdam are low and bank protection was not required.

Tests were done on the model to optimize the discharge channel configuration. Ideally, flow should be spread out as evenly as possible across the spillway discharge channel in order to maximize energy dissipation. However, it was observed that the flow in the discharge channel was skewed towards the right bank, due to the high bed levels at the discharge channel outlet along the left bank.

To reduce this skewness, the width of the discharge channel was modified. The right and left wall alignments were gradually adjusted until the flow in the discharge channel was uniformly distributed. The final configuration resulted in a downstream width of 133 m, instead of the original width of 215 m. With the final rollways installed the spillway capacity was tested for a wide range of flow conditions, from 3000 m³/s to the spillway capacity design flow of 11,300 m³/s. Comparison of the model water levels registered at gauge NL-15 and the theoretical levels showed good agreement (Figure 11).

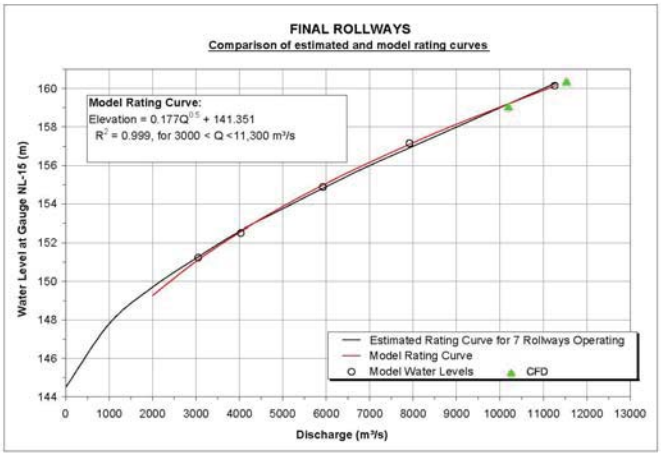


Figure 11. Spillway final rollways – comparing estimated, model and CFD rating curves

4.2 The Partial Model

A partial model consisting of two full bays (4 and 5) and two half bays (3 and 6) of the spillway together with 200 m of approach and 175 m of tailrace channel was constructed at an undistorted scale of 1/50 (Figure 12).

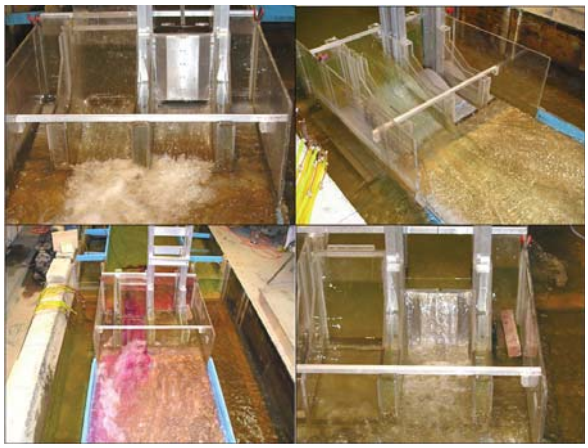


Figure 12. The Partial Model

Tests

Tests were carried out on the model to confirm theoretically derived rating curves for fully open and partial gate openings for both diversion and final rollway conditions. Tests were also made to verify pressures and gate behavior. These latter tests were dynamic and carried out with an aluminum gate equipped with a hydro-elastic hoist system.

The physical model test results for diversion and final rollway conditions with the gates fully open were compared to Flow 3-D simulations carried out by McGill University [(2)]. The results are presented in Tables 1 and 2, together with the percentage difference between the two discharge values.

Table 1 Diversion Sluiceway Fully Open

Fully Open	NL-15 (m)	HWL-C (m)	Q_{F3D} (m^3/s/bay)	Q_L (m^3/s/bay)	Difference (%)
	159	158.47	1817.28*	1830.927	0.75
	154	153.47	1200.00*	1177.975	-1.87
	151	150.25	829.63*	834.027	0.53

Table 2 Final Rollways Fully Open

Fully Open	NL-15 (m)	HWL-C (m)	Q_{F3D} (m^3/s/bay)	Q_L (m^3/s/bay)	Difference (%)
	159	158.47	1817.28*	1830.927	0.75
	151	150.25	829.63*	834.027	0.53

5. CONCLUSIONS

There appears to be promising agreement between the results of the two modelling techniques. From our experience in using CFD analysis over the past 10 years our level of trust has increased significantly to the point that we are now confident in using CFD modelling for general study purposes and some final design applications. Physical modelling is still recommended for final confirmation on major hydraulic applications where a significant element of risk is involved either for purposes of construction and/or operation.

CFD modelling has been found to be suitable in determining the final conceptual configuration for the hydraulic design of the facility in a relatively short time frame. It provides the flexibility in being able to quickly change the conceptual details and configuration and allows the design to advance in stages, first using a coarse grid/mesh to scope the design and then refining the grid/mesh as the design advances. It must be noted that care is required in setting up the CFD model (which comes from experience) which has a direct bearing on whether the modelling results are erroneous or accurate. Another consideration is that the cost to buy or rent the CFD model is high and this cost needs to be weighed against the cost of a physical model study.

On major design projects it is recommended that both modelling techniques be integrated during design to provide the optimum hydraulic engineering solution. During the early stages of design, CFD modelling is quite adequate for purposes of feasibility and pre-engineering design. Before advancing to final design, the element of risk should be evaluated to determine whether confirmatory physical modelling would add value engineering to the final design.

6. ACKNOWLEDGEMENTS

The authors would like to thank J. Gacek, M.Eng. and Prof. S. Gaskin, Department of Civil Engineering, McGill University, Montreal for the FLOW-3D modelling study of the Spillway Diversion and Rollway used in this paper

References

1 Flow Science, “FLOW-3D® User’s Manual”, New Mexico, 2002, 301 pages.

2 Preliminary report: Gull Rapids Hydro Power Project, Flow 3D modeling of Spillway Diversion and Rollway; J. Gacek, M.Eng, Student and S. Gaskin, Assoc. Prof., Dept. of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada

Authors

Efrem Teklemariam, M.Sc., P. Eng., is a a Senior Hydrotechnical Studies Engineer at Manitoba Hydro. He conducted the hydrologic studies and Flow3D modelling for the preparation of the Keeyask Generating Station EIS.

Don Murray, M.Sc, P.Eng., is a Senior Hydraulic Engineer at Hatch Acres. He was responsible for the supervision and coordination of the hydraulic studies for the Keeyask GS Stage IV Studies including the FLOW-3D® numerical modelling and physical hydraulic model studies.

David M. Fuchs, M.Sc., P. Eng, is an Intermediate Hydraulic Engineer at Acres Manitoba Ltd. He was responsible for carrying out hydraulic studies and design of the proposed hydroelectric project presented in this paper, and was involved in coordinating and applying the FLOW-3D model

Bernie Shumilak, P. Eng., is a Special Studies Hydrotechnical Engineer at Manitoba Hydro. He coordinated the physical and project description of the Keeyask EIS

Graham K. Holder, B.Sc., P.Eng. is a Senior Engineer and President of the LaSalle Consulting Group. He was responsible for the physical hydraulic model studies of the Keeyask GS.

Kevin Sydor, M.Sc., P. Eng., is a Senior Hydrotechnical Studies Engineer at Manitoba Hydro. He conducted the Flow3D modelling for the preparation of the Keeyask Generating Station EIS

MAY 13 2009

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.3.22
Rev. 0, May 8, 2009

River Closure Schedule for Sequence of Final Concreting of Spillway Rollways

Manitoba Hydro File 00195-11400-0036_01

Prepared by _____ (JOHN ROGERS) May 8, 2009
KGS Acres Ltd.

Checked by _____ (D.G. Murray) May 8, 2009
KGS Acres Ltd.

Reviewed by _____ (I.R. Dewar) May 8, 2009
KGS Acres Ltd.

Approved by _____ (N.J. Smith) May 8, 2009
KGS Acres Ltd.

Accepted by _____ (D.S. MAGNUSSON) June 8, 2009
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I.R. Dewar	**Date**	May 8, 2009
		File No.	P10008(16).13.01& .36
From	J.B. Rogers, K.J. Freeman	**cc**	D.M. Fuchs D.G. Murray
Subject	**Keeyask Generating Station Stage IV Studies, Axis GR-4 River Closure Schedule for Sequence of Final Concreting of Spillway Rollways Memorandum GN-4.3.22, Rev 0 Manitoba Hydro 00195-11400-0036_01**		

Note of Caution: *This memorandum presents data that have been revised in subsequent studies and associated memoranda.*

Initially issued on October 15, 2004 as Memorandum GN-4.3.22 Draft 1, this memorandum was prepared using the estimated 1:20 year monthly flows that were current at that time and the resulting upstream water levels that are estimated to occur utilizing the suggested closure sequence.

With the exception of several minor edits, this memorandum has not been revised since its initial issue as "Draft 1". This revised document presents the design considerations that led to setting the crest elevation of the South Dam Stage II Upstream Cofferdam at 154.2m; which has subsequently been referenced in numerous other Stage IV Design Memoranda.

Since the original issue of this memorandum, the estimated 1:20 year monthly flow rates have been revised (See GN-2.3 "Hydrologic Review – Construction Design Flood" Draft 3 20080410). This results in a change to the estimated upstream water levels (See Table 5 of Memorandum GN-2.8 "River Management During Construction" Rev 0).

As the water levels reported in Table 5 of GN-2.8 are slightly lower than the water levels used in GN-4.2.2.2, "Design of North and South Dykes", the alternative recommended in this memorandum remains valid (i.e. raising the elevation of the South Dam Stage II Upstream Cofferdam is still the preferred option). However, the elevation recommended in GN-4.2.2.2 may no longer be suitable and should be re-evaluated during the Stage V Studies using the elevations recommended in GN-2.8.

In addition, it should be noted that the costs and values of electricity provided in the original Memorandum were current as of 2004. Although the costs have not been updated to 2009 values, this will not impact the final recommendation.

1 Introduction

This memorandum summarizes the study of river closure alternatives for the final concreting of the Spillway rollways during the construction of Keeyask Generating Station (Keeyask GS). The purpose of the investigation was to identify and evaluate viable river closure alternatives with respect to schedule and cost and to select a preferred river closure sequence for final concreting of the Spillway rollways for Keeyask GS.

2 Background

During Stage II River Diversion, the entire flow of the Nelson River diverts through the partially completed seven-bay Spillway. In this stage of construction, the Spillway operates as a sluiceway and the Spillway gates are used to regulate flow of water through the Spillway. The temporary closure of bays with stoplogs will allow for the construction of the Spillway Rollways. Figure 1 shows a cross-section of the Spillway prior to the construction of the Rollway with upstream and downstream stoplogs removed and the Spillway gate in the open position. Figure 2 shows a cross-section of the final configuration of the Spillway. During construction of a Spillway Rollway, the Spillway Bay will be maintained in a dry and dewatered state by using the upstream and downstream stoplogs. An increase in water level accompanies the closure of each Spillway Bay. Due to the rise in water levels, the number of closed bays at any given time is limited by:

- the height of Stage II Cofferdams,
- the height of Spillway gates and stoplogs,
- the progress of concurrent construction (South Dam and South Dyke),
- variations in average monthly 1:20 maximum flow.

Prior to impounding the forebay to el 159.0 m for First Power, the Spillway must be capable of regulating flows up to and including the passing of the Construction Design Flood (CDF). Based on Spillway rating curves, shown in Figure 4 (copied from Memorandum GN-4.3.6 “Hydraulic Design of Spillway” [Ref 1]), a minimum of four completed rollways are required to impound the forebay while at the same time maintain the capability of passing the Construction Design Flood. Alternatively, two completed rollways and two sluiceways (operating with high head gates) will also pass the Construction Design Flood with the forebay impounded.

The crest elevation of the South Dam Stage II Upstream Cofferdam has been set 1 m higher than the maximum estimated forebay level for summer months. This will permit passage of the CDF during the period leading up to the completion of the South Dam. The raising of the forebay level above the crest of the South Dam Stage II Upstream Cofferdam is not anticipated to be possible until late August of Year Y[1], due to the rate of placement of the South Dam's fill.

Figure 3 shows the current general arrangement of the generating station structures. The numbering convention for Spillway Bays followed in this memorandum sets the bays in sequential order with S1 being the northern-most bay and S7 the southern-most bay.

3 Alternatives

The three river closure alternatives studied are as follows:

Alternative 1 – Winter Concreting

This alternative would see the construction of the rollways begin in August of Year Y (the year of first power). Four rollways (S1, S3, S5, and S7) would be complete prior to impounding the forebay, with the last of these rollways scheduled for completion on November 30, Year Y. While construction of two of these rollways (S1 and S3) would occur in "summer months" (assumed to be from May 15 to October 15), construction of the other two rollways (S5 and S7) would occur in "winter months" (assumed to be from October 15 to May 15) and would likely require substantial heating and hoarding of concreting operations. The three remaining bays (S2, S4 and S6) would be closed via stoplogs until such time that their rollways were constructed the following summer (Year Y+1).

(new paragraph)To permit a comparison of the alternatives, an estimate of the extra over cost for winter concreting was prepared. The results are presented in Table 1. The extra over cost for Alternative 1 – Winter Concreting would be approximately [redacted] (2004 dollars). Table 2 shows the sequence of construction of the rollways, the design flows and the corresponding water levels during the final concreting of the Spillway rollways for this alternative. The water levels, shown in Table 2, are based on Spillway discharge rating curves shown in Figure 4. Figure 5 shows the assumed construction schedule for this alternative.

[1] "Year Y" is a designation given to the calendar year in which first power is produced. "Year Y + 1" refers to the calendar year following the year in which final power is produced.

Alternative 2 – High Head Gates

This alternative would see the construction of the rollways also begin in August of Year Y. However, only two rollways (S1 and S3) would be completed prior to impounding the forebay, with the latter of these two rollways scheduled for completion on October 31, Year Y. Concreting operations are expected to be substantially complete prior to October 15, Year Y and thus only negligible heating and hoarding would be required. Two high head gates, installed in two of the remaining bays (S5 and S7), would operate in conjunction with the two completed bays (S1 and S3) to permit passage of flows up to the construction design flood value. Five rollways (S5, S7, S2, S4 and S6) would be completed the following summer. Table 1 shows the estimated extra cost to provide the high head gates, modify the Spillway Hoist Tower and increase the capacity of two hoists to accommodate the larger size gates. The extra over cost for Alternative 2 – High Head Gates is approximately [redacted] (2004 dollars). Table 3 shows the sequence of construction of the rollways, the design flows and the corresponding water levels during the final concreting of the Spillway Rollways for this alternative. The water levels, shown in Table 3, are based on Spillway discharge rating curves shown in Figure 4. Figure 6 shows the assumed construction schedule for this alternative.

Alternative 3 – Raise Crest of South Dam Stage II Upstream Cofferdam

Raising the crest elevation of the South Dam Stage II Upstream Cofferdam from el 152.5 m to el 154.2 m would allow construction of the rollways to begin one month earlier than in the above-mentioned alternatives, i.e., in July Year Y, a period with higher 1:20 year flows. In this alternative, due to the early start to construction, completion of four rollways (S1, S3, S5 and S7) would occur by October 31, Year Y. Concreting operations are expected to be substantially complete prior to October 15, Year Y and therefore only negligible heating and hoarding would be required. The remaining three rollways (S2, S4, and S6) would be constructed during the following summer. This alternative does not require the use high head Spillway Gates. Table 1 shows the estimated cost to raise the crest elevation of the South Dam Stage II Upstream Cofferdam. The extra over cost for Alternative 3 – Raise Crest of South Dam Stage II Upstream Cofferdam is approximately [redacted] (2004 dollars). Table 4 shows the sequence of construction of the rollways, the design flows and the corresponding water levels during the final concreting of the Spillway Rollways for this alternative. The water levels, shown in Table 4, are based on Spillway discharge rating curves shown in Figure 4. Figure 7 shows the assumed construction schedule for this alternative.

4 Schedule

The construction schedules for the three alternatives, shown in Figures 5, 6 and 7, assume that the duration for construction of each rollway is approximately 8 weeks. The assumed scheduled duration for the construction of a rollway includes the following activities.

- Install and seal upstream and downstream stoplogs,
- Dewater,
- Remove precast diversion slabs,
- Prepare existing concrete surfaces,
- Provide enclosure for heating and hoarding (Alternative 1 only),
- Install concrete conveyor,
- Install supports and guide rails for slip form,
- Install forms, rebar and pour concrete (assumes three lifts are required),
- Install and grout (and cure) sill beam,
- Curing concrete (the duration assumes that a minimum of 2 weeks for curing of concrete is required between the last pour of concrete and the removal of stoplogs),
- Remove upstream and downstream stoplogs.

Due to the possibility of inclement weather (mainly an early start to winter and below normal temperatures), there are a number of schedule related risks involved with each alternative. The average temperatures at the site during October (-0.4°C) and November (-12.1°C) are below freezing. The extreme lows during October (-26.9°C) and November (-39.4°C) are well below freezing. In extreme cold, work typically proceeds at production rates that are slower than those achieved in midsummer. Water spray or ice fog from adjacent open bays and seepage through stoplogs may also become problematic in extreme cold.

The cost of not completing the construction of the rollways in time for first power is significant. Assuming the current value of electricity, approximately [redacted] (2004 dollars), a one month delay in construction resulting in a corresponding one month delay in first power would translate into approximately [redacted] in lost revenue for each unit delayed (= 620/7 MW * 31 days/month * 24 hours/day * [redacted]). A 1-day delay in the delivery of first power translates into lost revenue of [redacted] (2004 dollars).

Alternative 1 – Winter Concreting is assessed to be the alternative with the greatest risk of negatively impacting the date for first power. With construction of four rollways from the beginning of August through the end of November of Year Y, the construction of the rollways are scheduled for completion just prior to the current December 1, Year Y date for First Power. With no float in the schedule between the end of construction of the rollways and first power, delays in construction of any of the rollways would result in postponement of impounding the forebay to its full supply level. Such delay would result in the first unit(s) being operated with a reduced head until the rollways are complete and thus a loss in revenue. In addition, the schedule for Alternative 1 does not include an allowance for full head wet testing of the first unit prior to First Power.

Alternative 2 – High Head Gates is assessed to be the lowest risk alternative with only two rollways being required prior to impounding the forebay for first power. Alternative 2 would see the construction of the rollways required for impounding completed 1 month before the December 1, Year Y date for first power. This schedule thus includes a 1 month allowance for full head wet testing of the first unit prior to First Power.

Alternative 3 – Raise Crest of South Dam Stage II Upstream Cofferdam with construction of four rollways by October 31, Year Y, has been assessed to have only slightly more risk than Alternative 2. Alternative 3 would see the construction of the rollways required for impounding completed 1 month before the December 1, Year Y date for first power. This schedule thus also includes a 1-month allowance for full head wet testing of the first unit prior to First Power.

In the case of Alternatives 2 and 3, winter concreting remains an option if construction of the rollways lags behind schedule.

5 Other Considerations

Due to the small size of the Keeyask GS reservoir in relation to the inflow to the reservoir, it would be possible to achieve full impoundment of the reservoir in a very short period of time. However, restrictions on the rate of rise in water levels of the forebay are typically put in place to ensure embankment stability and allowance for performance monitoring. A restriction on the average rate of rise of the forebay to 1 m per day is currently envisaged.

As mentioned earlier, the raising of the forebay level above the crest of the South Dam Stage II Upstream Cofferdam is anticipated to be possible in late August of Year Y, due to the advancement of the South Dam's fill elevation. The maximum forebay level during construction of the rollways that will allow closure of the Spillway gates and installation of stoplogs in the open sluices is approximately el 153.3 m. It is anticipated that just prior to the commencement of activities leading to full impoundment of the reservoir the water level will be at or near el 153.3 m. With the above noted restriction in place for rate of rise of the forebay it will take approximately 6 days, from completion of the rollways, to fill the reservoir to FSL (el 159.0 m). The duration required to fill the reservoir would be approximately the same in all three alternatives.

6 Conclusion and Recommendation

Although Alternative 1 – Winter Concreting is the lowest capital cost alternative, it is not recommended as the preferred alternative. With no float in the schedule between the end of construction of the rollways and the December 1, Year Y date for First Power, there is increased risk of delays to the first unit ISD and increased potential of operating initially at partial head.

While Alternative 2 – High Head Gates offers a slightly more conservative schedule than Alternative 3 – Raising the Crest of the South Dam Stage II Upstream Cofferdam, it is believed that the increase in cost (approximately [redacted] 2004 dollars) is not justified.

Alternative 3 – Raising the Crest of the South Dam Stage II Upstream Cofferdam is recommended as the preferred alternative. The estimated capital cost of raising the South Dam Stage II Upstream Cofferdam is approximately [redacted] (2004 dollars) more than the estimated cost of Alternative 1 - Winter Concreting. However, due to the increased likelihood of achieving First Power by December 1, Year Y, Alternative 3 – Raising the Crest of the South Dam Stage II Upstream Cofferdam is considered to be the preferred alternative.

Prepared By

J.B. Rogers

per

K.J. Freeman

Attach
References
Tables 1 to 4
Appendix A
Figure 1 to 7
DMF:DGM:spa

References

1. KGS Acres Manitoba Limited, Memorandum GN-4.3.6, “Keeyask Generating Station, Stage IV Studies, Hydraulic Design of Spillway”, Manitoba Hydro File 00195-23100-0008.

2. Acres Manitoba Limited, Deliverable C-7.00.1, “Conawapa Generating Station, Recommitment Studies Project Status Update, Cost Estimate Summary”, Manitoba Hydro File 00192-04220-0054.

Tables

Tables 1 to 4

Table 1
Keeyask Generating Station - GR4
Comparison of River Closure Alternatives for Concreting Rollways
(Minister Announces Project Oct 2007, First Power Dec 2012)

Table 2
Keeyask GS - Stage IV Studies, Axis GR-4
Summary of Flows and Estimated Water Levels
During Final Concreting of Spillway Bays
Alternative 1 - Winter Concreting

Month	1:20 Year Flow (m^3/s)	Spillway Configuration	Upstream Water Level (m)	Spillway Bay Closure Sequence
May Year Y	4590	All 7 sluices open	148.6	
Jun Year Y	5780	All 7 sluices open	150.3	
Jul Year Y	5900	All 7 sluices open	150.5	
Aug Year Y	5720	6 sluices open, constructing 1 rollway	151.5	Sluice 1 (S1)
Sep Year Y	4820	5 sluices open, constructing 2 rollways	151.6	S1, S3
Oct Year Y	4360	3 sluices open, 1 bay complete, constructing 3 rollways	153.8	S3, S5, S7
Nov Year Y	4360	3 sluices open, 2 bays complete, constructing 2 rollways	152.6	S5, S7
Dec Year Y	4230	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jan Year Y+1	4270	3 sluices closed (stoplogs), 4 bays complete	FSL	
Feb Year Y+1	4170	3 sluices closed (stoplogs), 4 bays complete	FSL	
Mar Year Y+1	3920	3 sluices closed (stoplogs), 4 bays complete	FSL	
Apr Year Y+1	3780	3 sluices closed (stoplogs), 4 bays complete	FSL	
May Year Y+1	4590	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jun Year Y+1	5780	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jul Year Y+1	5900	4 bays complete, 2 sluices closed (stoplogs), constructing 1 rollway	FSL	S2
Aug Year	5720	4 bays complete, 1 sluice closed (stoplogs),	FSL	S2, S4

Month	1:20 Year Flow (m^3/s)	Spillway Configuration	Upstream Water Level (m)	Spillway Bay Closure Sequence
Y+1		constructing 2 rollways		
Sep Year Y+1	4820	5 bays complete, constructing 2 rollways	FSL	S4, S6
Oct Year Y+1	4360	6 bays complete, constructing 1 rollway	FSL	S6
Nov Year Y+1	4360	Spillway complete	FSL	
Dec Year Y+1	4230	Spillway complete	FSL	

Notes:

1 South Dam (upstream) and Island Stage II Cofferdam crest elevations are 152.5 m and 152.9 m, respectively.

2 River closure with Stage II Cofferdam scheduled for August Year Y-1.

3 South Dam construction scheduled for spring of Year Y.

4 South Dam construction complete by September 1, Year Y.

5 First unit commissioned December 1, Year Y.

6 Forebay impoundment to FSL (el 159.0 m) is scheduled in December Year Y. Temporary stoplogs imported from Limestone will be sufficient to close three bays during the first winter of operation.

7 Rollway construction, assumes 2-month durations to place and cure all secondary concrete.

8 South Dam (inside Stage I Spillway Cofferdam) constructed to el 162.7 m by Year Y-1.

Table 3
Summary of Flows and Estimated Water Levels
During Final Concreting of Spillway Bays
Alternative 2 - Requires Two High Head Gates

Month	1:20 Year Flow (m^3/s)	Spillway Configuration	Upstream Water Level (m)	Spillway Bay Closure Sequence
May Year Y	4590	All 7 sluices open	148.6	
Jun Year Y	5780	All 7 sluices open	150.3	
Jul Year Y	5900	All 7 sluices open	150.5	
Aug Year Y	5720	6 sluices open, constructing 1 rollway	151.5	Sluice 1 (S1)
Sep Year Y	4820	5 sluices open, constructing 2 rollways	151.6	S1, S3
Oct Year Y	4360	5 sluices open, 1 bay open, constructing 1 rollway	150.0	S3
Nov Year Y	4360	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	156.4	
Dec Year Y	4230	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	FSL	
Jan Year Y+1	4270	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	FSL	
Feb Year Y+1	4170	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	FSL	
Mar Year Y+1	3920	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	FSL	
Apr Year Y+1	3780	3 sluices closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates	FSL	
May Year Y+1	4590	1 sluice closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates, constructing 2 rollways	FSL	S5, S7
Jun Year Y+1	5780	1 sluice closed (stoplogs), 2 bays complete, 2 sluices operating with high head gates, constructing 2 rollways	FSL	S5, S7
Jul Year Y+1	5900	3 bays complete, 1 sluice complete, constructing 3 rollways	FSL	S7, S2, S4
Aug Year Y+1	5720	4 bays complete, 1 sluice complete, constructing 2 rollways	FSL	S2, S4
Sep Year Y+1	4820	5 bays complete, constructing 2 rollways	FSL	S4, S6
Oct Year Y+1	4360	6 bays complete, constructing 1 rollway	FSL	S6
Nov Year Y+1	4360	Spillway complete	FSL	
Dec Year Y+1	4230	Spillway complete	FSL	

Table 3
Summary of Flows and Estimated Water Levels
During Final Concreting of Spillway Bays
Alternative 2 - Requires Two High Head Gates

Notes:

1 South Dam (upstream) and Island Stage II Cofferdam crest el 152.5 m.

2 River closure with Stage II Cofferdam scheduled for August Year Y-1.

3 South Dam construction scheduled for spring of Year Y.

4 South Dam construction complete by September 1, Year Y.

5 First unit commissioned December 1, Year Y.

6 Forebay impoundment to FSL (el 159.0 m) is scheduled in December Year Y. Temporary stoplogs imported from Limestone will be sufficient to close three bays during the first winter of operation.

7 Rollway construction, assumes 2-month durations to place and cure all secondary concrete.

8 South Dam (inside Stage I Spillway Cofferdam) constructed to el 162.7 m by Year Y-1.

Table 4
Summary of Flows and Estimated Water Levels
During Final Concreting of Spillway Bays
Alternative 3 - Raising South Dam Stage II Upstream Cofferdam Crest

Month	1:20 Year Flow (m^3/s)	Spillway Configuration	Upstream Water Level (m)	Spillway Bay Closure Sequence
May Year Y	4590	All 7 sluices open	148.6	
Jun Year Y	5780	All 7 sluices open	150.3	
Jul Year Y	5900	6 sluices open, constructing 1 rollway	151.8	Sluice 1 (S1)
Aug Year Y	5720	5 sluices open, constructing 2 rollways	153.2	S1, S3
Sep Year Y	4820	3 sluices open, 1 bay open, constructing 3 rollways	154.7	S3, S5, S7
Oct Year Y	4360	3 sluices open, 2 bays open, constructing 2 rollways	152.4	S5, S7
Nov Year Y	4360	3 sluices closed (stoplogs), 4 bays complete	156.4	
Dec Year Y	4230	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jan Year Y+1	4270	3 sluices closed (stoplogs), 4 bays complete	FSL	
Feb Year Y+1	4170	3 sluices closed (stoplogs), 4 bays complete	FSL	
Mar Year Y+1	3920	3 sluices closed (stoplogs), 4 bays complete	FSL	
Apr Year Y+1	3780	3 sluices closed (stoplogs), 4 bays complete	FSL	
May Year Y+1	4590	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jun Year Y+1	5780	3 sluices closed (stoplogs), 4 bays complete	FSL	
Jul Year Y+1	5900	4 bays complete, 2 sluices closed (stoplogs), constructing 1 rollway	FSL	S2
Aug Year Y+1	5720	4 bays complete, 1 sluice closed (stoplogs), constructing 2 rollways	FSL	S2, S4
Sep Year Y+1	4820	5 bays complete, constructing 2 rollways	FSL	S4, S6
Oct Year Y+1	4360	6 bays complete, constructing 1 rollway	FSL	S6
Nov Year Y+1	4360	Spillway complete	FSL	
Dec Year Y+1	4230	Spillway complete	FSL	

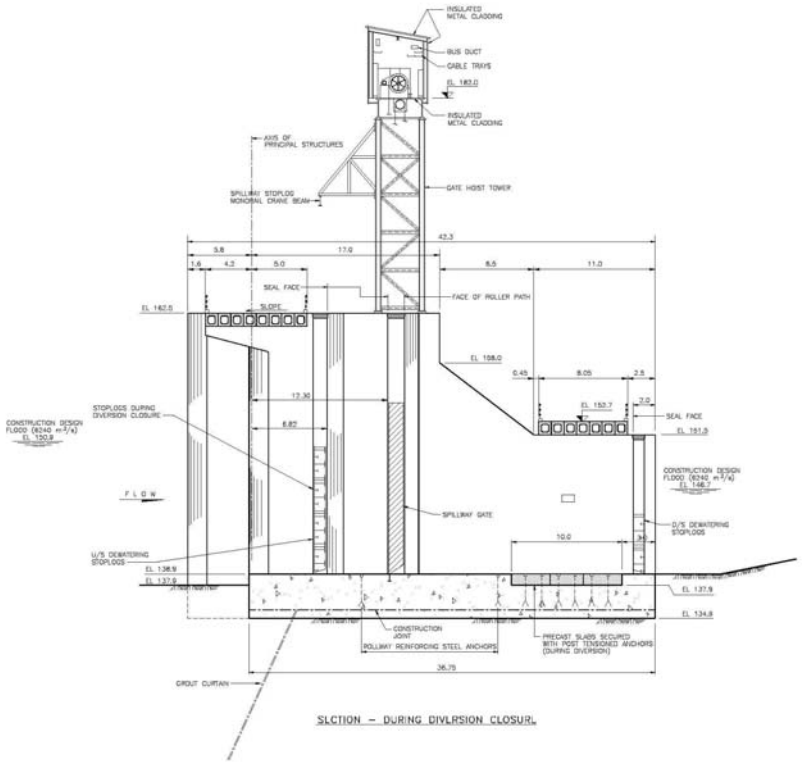
Notes:

1 South Dam (upstream) crest el 154.2 m and Island Stage II Cofferdam crest el 152.5 m.
2 River closure with Stage II Cofferdam scheduled for August Year Y-1.
3 South Dam construction scheduled for spring of Year Y.
4 South Dam construction complete by September 1, Year Y.
5 First unit commissioned December 1, Year Y.
6 Forebay impoundment to FSL (el 159.0 m) is scheduled in December Year Y. Temporary stoplogs imported from Limestone will be sufficient to close three bays during the first winter of operation.
7 Rollway construction, assumes 2-month durations to place and cure all secondary concrete.
8 South Dam (inside Stage I Spillway Cofferdam) constructed to el 162.7 m by Year Y-1.

Figures

Figures 1 to 7

FIGURE 1 0001



SECTION – DURING DIVERSION CLOSURE

SCALE 1:125 7.5 0 7.5 5 7.5 10 12.5 METRES

2006-04-03	0.0	ISSUED AS A FIGURE FOR MEMO REFERENCE GR-4.3.03, REV. 0	BJH

KGS ACRES LTD

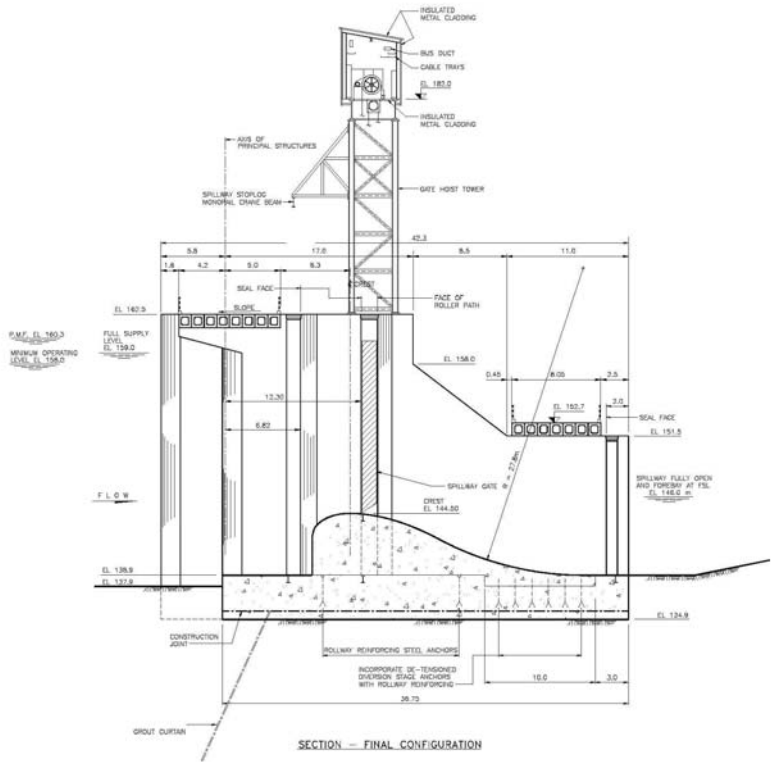
MANITOBA HYDRO

KEYYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
HYDRAULIC DESIGN OF SPILLWAY
7-BAY SPILLWAY-DIVERSION STAGE
CROSS SECTION

FIGURE 1 0001

FIGURE 2 0001



SECTION – FINAL CONFIGURATION

SCALE 1:125 7.5 0 2.5 5 7.5 10 12.5 METRES

REV	DATE	BY	CHK	APP	REMARKS
0000	04				ISSUED AS A FIGURE FOR MEMO REFERENCE SH-4.3.12, REV. 0

KGS ACRES

KGS ACRES LTD

MANITOBA HYDRO

KEYYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
HYDRAULIC DESIGN OF SPILLWAY
7-BAY SPILLWAY-FINAL CONFIGURATION
CROSS SECTION

FIGURE 2 0001

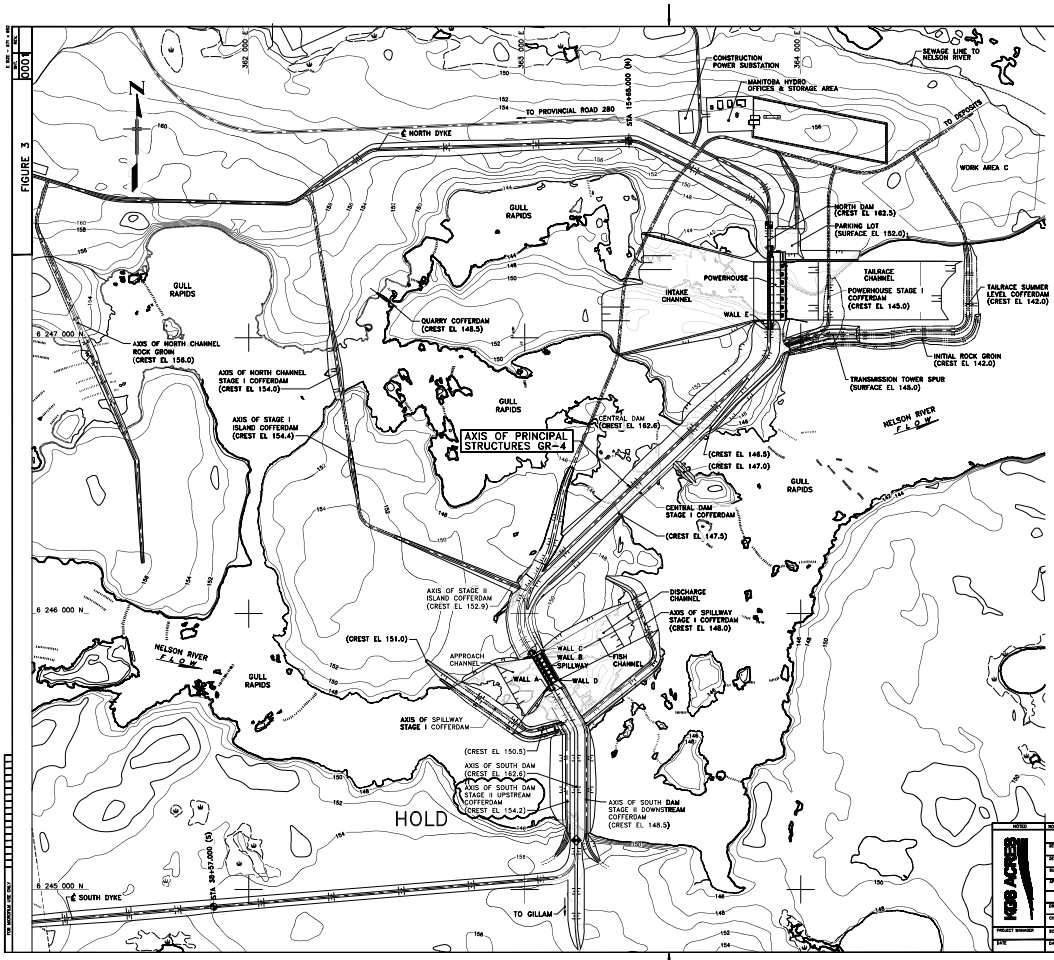


FIGURE 3
0001
LEGEND:
SHORELINE (SEE NOTE 6)
GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
REFERENCE POINT (SEE NOTE 12)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. WATER LEVELS SHOWN ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 158.0. FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR-3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.
SCALE 1:5000
100 0 100 200 300 400 500 METRES
AXIS OF PRINCIPAL STRUCTURES GR-4
HOLD
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO OFFICES & STORAGE AREA
SEWAGE LINE TO NELSON RIVER
TO PROVINCIAL ROAD 280
TO DEPOSITS
WORK AREA C
NORTH DAM (CREST EL 162.5)
PARKING LOT (SURFACE EL 152.0)
TAILRACE CHANNEL
POWERHOUSE
INTAKE CHANNEL
WALL E
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.0)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 142.0)
INITIAL ROCK GROIN (CREST EL 142.0)
TRANSMISSION TOWER SPUR (SURFACE EL 148.0)
QUARRY COFFERDAM (CREST EL 148.5)
AXIS OF NORTH CHANNEL ROCK GROIN (CREST EL 156.0)
AXIS OF NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
AXIS OF STAGE I ISLAND COFFERDAM (CREST EL 154.4)
CENTRAL DAM (CREST EL 152.4)
(CREST EL 146.5)
(CREST EL 147.0)
CENTRAL DAM STAGE I COFFERDAM
(CREST EL 147.5)
AXIS OF STAGE II ISLAND COFFERDAM (CREST EL 152.5)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM (CREST EL 148.0)
(CREST EL 151.0)
APPROACH CHANNEL
WALL C
WALL B SPILLWAY
WALL A
WALL D
FISH CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
AXIS OF SOUTH DAM (CREST EL 152.6)
AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 154.2)
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
TO GILLAM
GULL RAPIDS
NELSON RIVER FLOW
6 247 000 N
6 246 000 N
6 245 000 N
℄ NORTH DYKE
℄ SOUTH DYKE
STA 15+65.000 (N)
STA 38+57.000 (S)
362 000 E
363 000 E
364 000 E
N
2009 - 04 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-4.3.22, REV. 0
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT
AS SHOWN
FIGURE 3
0001

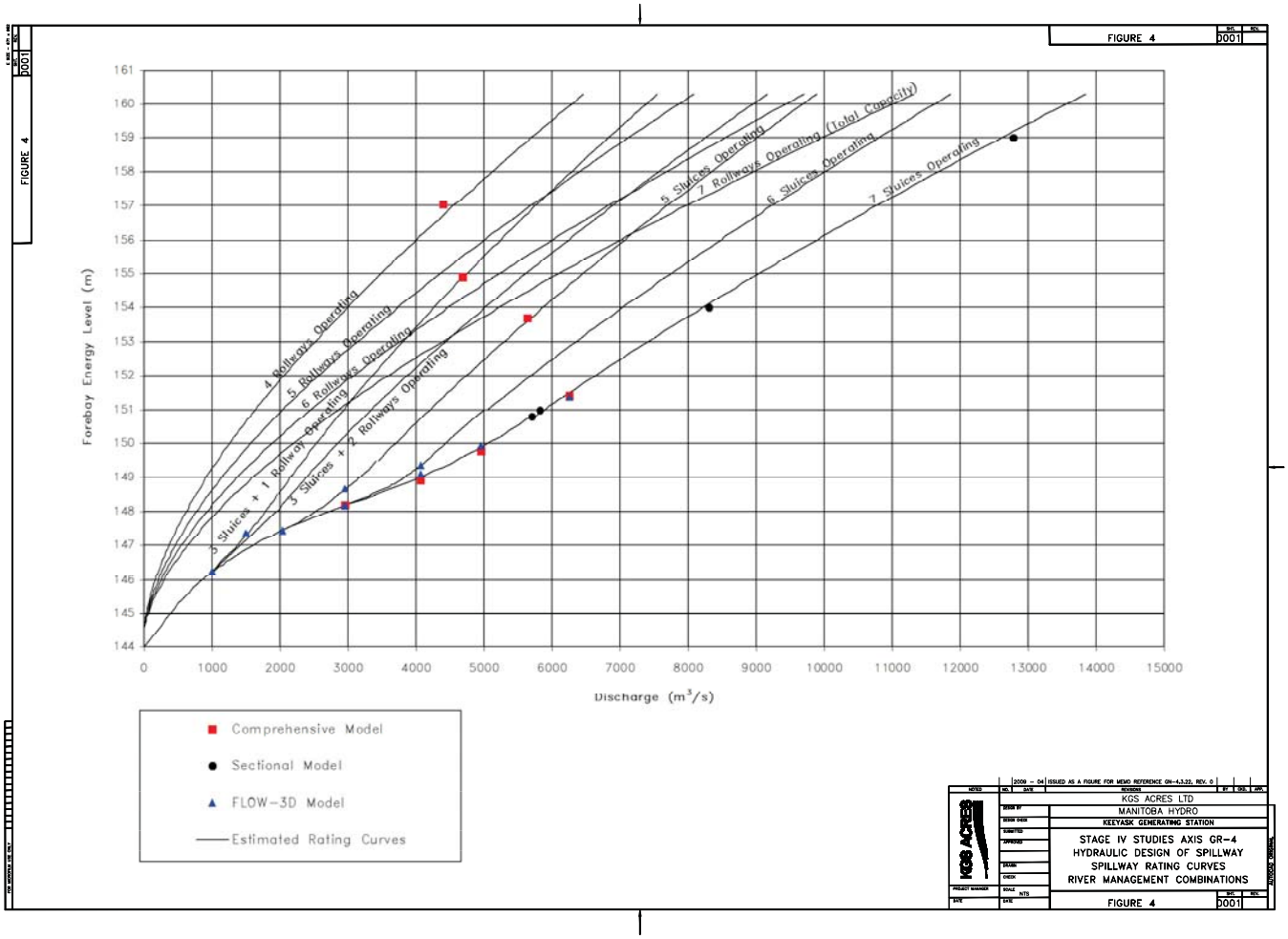
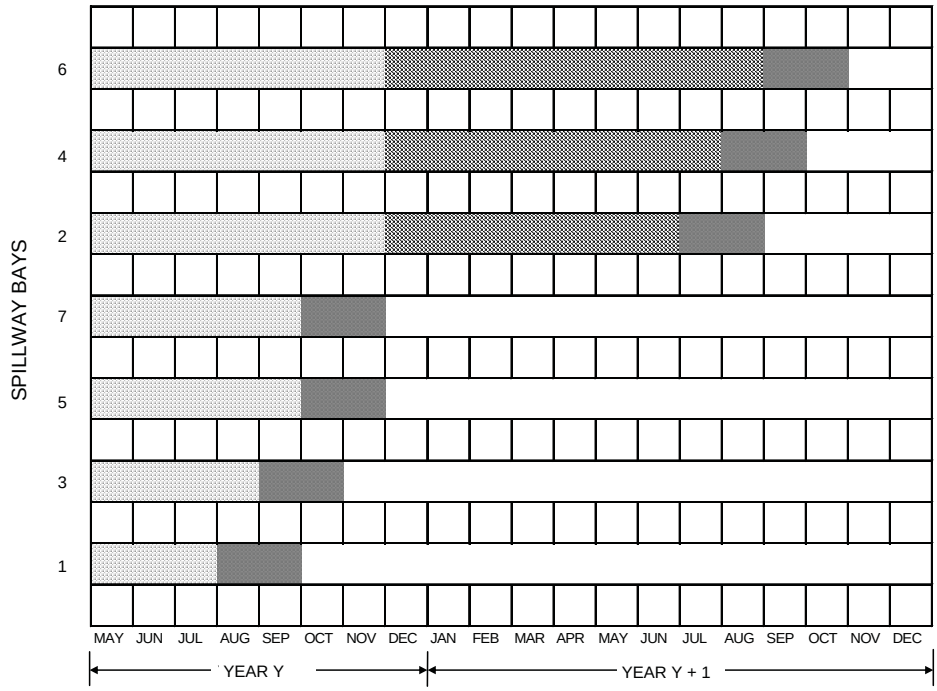


FIGURE 4
0001
Forebay Energy Level (m)
Discharge (m³/s)
4 Rollways Operating
5 Rollways Operating
6 Rollways Operating
7 Rollways Operating (Total Capacity)
3 Sluices + 1 Rollway Operating
3 Sluices + 2 Rollways Operating
5 Sluices Operating
6 Sluices Operating
7 Sluices Operating
Comprehensive Model
Sectional Model
FLOW-3D Model
Estimated Rating Curves
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
HYDRAULIC DESIGN OF SPILLWAY
SPILLWAY RATING CURVES
RIVER MANAGEMENT COMBINATIONS
FIGURE 4
0001



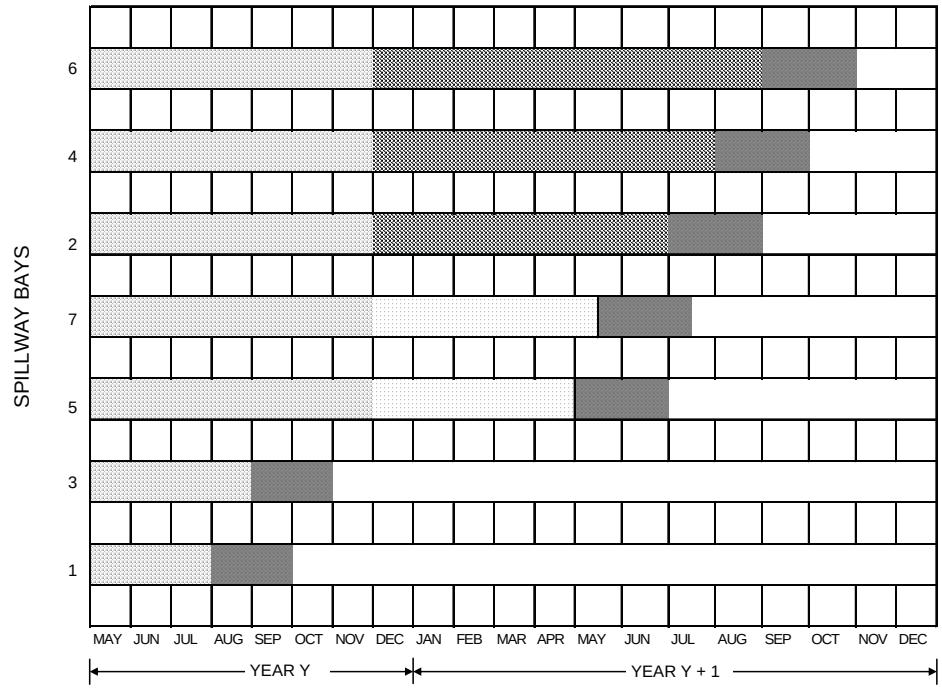
LEGEND:

- SLUICEWAY OPEN
- BAY CLOSED, FINAL ROLLWAY UNDER CONSTRUCTION
- SLUICEWAY CLOSED (STOPLOGS)
- SPILLWAY BAY COMPLETE

NOTES:

- YEAR Y REFERS TO YEAR OF COMMISSIONING

KGS ACRES	KGS ACRES MANITOBA LTD.
	MANITOBA HYDRO
	KEEYASK GENERATING STATION
PROJECT P10008.16	SCHEDULE FOR ROLLWAY CONSTRUCTION ALTERNATIVE 1 - WINTER CONCRETING
SCALE NTS	
DATE April 2009	REFERENCE TO MEMO NO. GN-4.3.22 — FIGURE 5



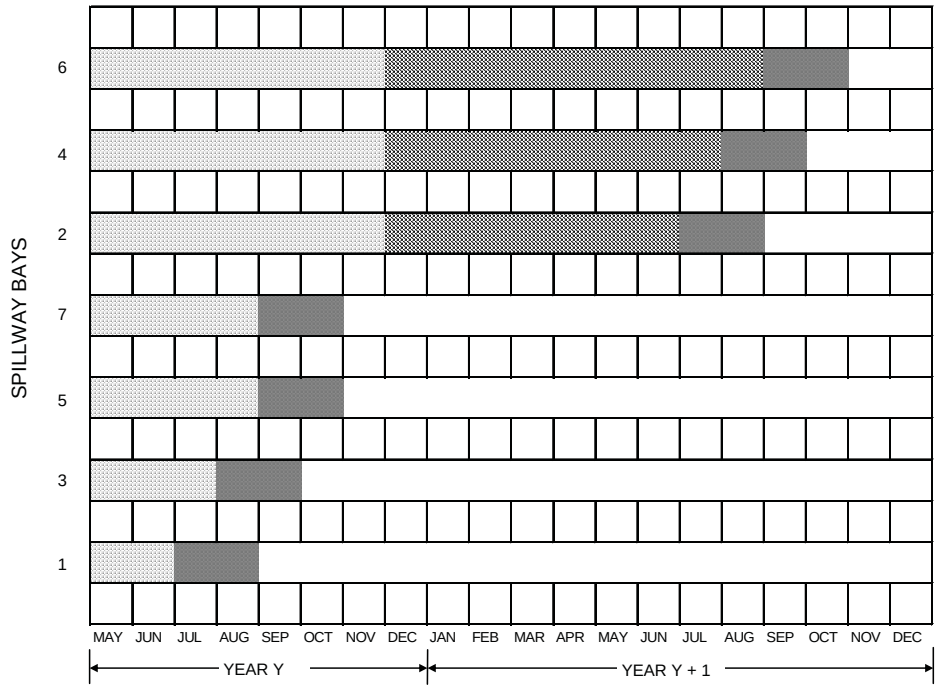
LEGEND:

- SLUICEWAY OPEN
- BAY CLOSED, FINAL ROLLWAY UNDER
- SLUICEWAY CLOSED (STOPLOGS)
- HIGH HEAD GATES OPERATING
- SPILLWAY BAY COMPLETE AND OPERATING

NOTES:

- YEAR Y REFERS TO YEAR OF COMMISSIONING

KGS ACRES	KGS ACRES MANITOBA LTD.
	MANITOBA HYDRO
	KEEYASK GENERATING STATION
PROJECT P10008.16	SCHEDULE FOR ROLLWAY CONSTRUCTION ALTERNATIVE 2 - HIGH HEAD GATES
SCALE NTS	
DATE April 2009	REFERENCE TO MEMO NO. GN-4.3.22 — FIGURE 6



LEGEND:

- SLUICEWAY OPEN
- BAY CLOSED, FINAL ROLLWAY UNDER CONSTRUCTION
- SLUICEWAY CLOSED (STOPLOGS)
- SPILLWAY BAY COMPLETE

NOTES:

- YEAR Y REFERS TO YEAR OF COMMISSIONING

KGS ACRES	KGS ACRES MANITOBA LTD.
	MANITOBA HYDRO
	KEEYASK GENERATING STATION
PROJECT P10008.16	SCHEDULE FOR ROLLWAY CONSTRUCTION ALTERNATIVE 3 - RAISE CREST OF SOUTH DAM STAGE II UPSTREAM COFFERDAM
SCALE NTS	
DATE April 2009	REFERENCE TO MEMO NO. GN-4.3.22 — FIGURE 7

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

RECEIVED
MAY 20 2011
HYDRO POWER
PLANNING DEPARTMENT

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-4.3.23

Revision 2, May 20, 2011

Post Impoundment Operating Procedure for Spillway Bays

Manitoba Hydro File 00195-06330-0008_03

Prepared by David Fuchs (David M.R. Fuchs) May 20, 2011
KGS Acres Ltd.

Checked by Joe Graeneveld (Joe Graeneveld) May 20, 2011
KGS Acres Ltd.

Reviewed by I.R. Dewar (I.R. Dewar) May 20, 2011
KGS Acres Ltd.

Approved by N.J. Smith (N.J. Smith) May 20, 2011
KGS Acres Ltd.

Accepted by M. St. Laurent (M. St. Laurent) Nov 8, 2011
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KGS ACRES **Office Memorandum**

To	I. R. Dewar	**Date**	May 20, 2011
		File No.	P10008(16).27.10
From	D.M. Fuchs/D.G. Murray	**cc**	J.L. Groeneveld

Subject **Keeyask Generating Station**
Stage IV Studies – Axis GR-4
Post-Impoundment Operating Procedure for Spillway Bays
Memorandum GN-4.3.23, Rev 2
Manitoba Hydro File 00195-06330-0008_03

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1 Introduction

This memorandum describes the recommended operating procedure for the Spillway for the proposed Keeyask Generating Station (Keeyask GS) located at the GR-4 Axis. The gated overflow Spillway will provide a means to convey excess water from the reservoir during flood events and discharge relief in the event of temporary plant outages.

The hydraulic design of the Spillway was undertaken in accordance with the procedures outlined in Memorandum, GN-4.1.2, "Hydraulic Design Criteria" [Ref 1]; the hydraulic design of the Spillway is described in Memorandum, GN-4.3.6, "Hydraulic Design of Spillway" [Ref 2].

2 Spillway Design Parameters

The list of parameters used for the hydraulic design of the Spillway is presented in Table 1.

The 13 m bay width was selected based on an optimization study, and offers the added advantage in that it conforms to bay widths of existing plants along the Nelson River. The rationale for the selection of the 7-Bay Spillway arrangement is described in Memorandum, GN-4.3.12, "Spillway Comparison 7, 8 and 9, Bay Alternatives" [Ref 3]. The Spillway crest elevation and invert elevations were designed to satisfy the following discharge capacity requirements:

- The Spillway and Powerhouse are capable of passing the PMF of 12 700 m^3/s without surcharging the reservoir above the allowable surcharge level of el 160.3 m. This will ensure that sufficient freeboard remains on the earth structures to contain wind and wave run-up associated with a coincident 1:2 year wind event. During this flood, the Powerhouse is assumed to be operating under speed-no-load conditions and capable of passing only 1 400 m^3/s.

- The Spillway is capable of passing in excess of the 1:1000 year SDF of 8 705 m^3/s without surcharging the reservoir above FSL (el 159.0 m) and with no reliance on Powerhouse discharges.

The winter spillway discharge capacity requirement (used to select the number of gate bodies that must be heated) was based on satisfying the greater of the following two conditions:

- Sufficient number of heated gate bodies to allow the spillway to pass a full load rejection and have one additional bay available as a spare to account for potential operational issues which may preclude the opening of one of the bays with heated gates.

- Sufficient number of bays to safely pass the 1:1000 year daily winter flood of 5240 m^3/s, with reliance on speed-no-load discharges through the Powerhouse, and with one additional bay available as a spare as a contingency for operational problems.

In order to meet these criteria, a total of four gate bodies will be heated.

3 Spillway Capacity

The Spillway overflow section is designed with an elliptical crest rollway with the crest apex at el 144.5 m and a design head of 14.5 m to provide the required Spillway discharge capacity.

The estimated Spillway capacity at FSL is 9 960 m^3/s, with the gates fully open. During discharge of the PMF, with the reservoir surcharged to el 160.3 m, the ratio of actual head to design head will equal approximately 1:1 (i.e., 15.8 m/14.5 m), resulting in an increased Spillway capacity of 11 315 m^3/s.

The Spillway capacity was confirmed using both a three-dimensional numerical flow model and a physical hydraulic model of the Spillway.

4 Description of Spillway

In the final configuration, the Spillway will be a 7-bay overflow type structure with individual vertical lift gates in each bay. Stoplogs, that can be placed in gate guides to the upstream of the vertical lift gates, will be installed under balanced head conditions prior to undertaking

any maintenance on a gate. The flow passed by the Spillway will discharge directly onto the bedrock surface downstream of the structure. A road bridge, supported on the upstream end of the piers, will form a part of the continuous road access across all the structures. A second bridge will be supported on the downstream end of the piers and will be utilized during construction and for installation of the downstream stoplogs, should maintenance be required on the structure's concrete surfaces.

The Spillway overflow section is designed with an elliptical crest rollway as noted above. The downstream section of the Spillway at the toe is a vertical curve that will rotate the flow to discharge horizontally into the Spillway Discharge Channel. The radius of this curve is equivalent to four times the estimated depth at the Spillway toe under the discharge capacity of the Spillway at FSL, which is greater than the SDF. This is a standard criteria selected to minimize the dynamic forces on the concrete due to the rotation of the flow.

The Spillway profile was checked for negative pressures that could occur at higher reservoir levels than the design level at FSL. The maximum operating level is the PMF level of el 160.3 m, which is 1.3 m above the design reservoir level. The negative pressures were determined to be less than the limiting minimum negative pressure of -2 m, thus showing that negative pressures are not a concern. Hence, operation of the Spillway at either FSL or PMF levels will not cause significant cavitation damage to the rest of the structure. Ultimately, this was confirmed on the physical hydraulic model study of the final rollway.

Vertical lift gates, four of which will have heated gate bodies to permit operation in the winter, will seal onto steel sill beams embedded in the crests of the rollways and, along their sides, to steel guides embedded in the piers (all seven sets of gate guides will be heated). The gates will be controlled by individual hoists supported on structural steel bridges, which, in turn will be supported by structural steel towers founded on the piers. The hoist bridges will be enclosed by an insulated metal clad housing which will extend across the seven bays and will also enclose the two end towers. A monorail will be used to place and remove the stoplogs. The stoplogs will be stored in the stoplog slots, at deck level, when not in use.

5 Frequency of Spillway Operation

The anticipated frequency of Spillway operation for the proposed Keeyask GS was evaluated as discussed in Memorandum, GN-4.3.15, "Frequency of Spillway Operation" [Ref 4]. The study concluded the following:

- There is no distinct pattern to Spillway releases.

- The majority of Spillway releases occur during the summer months. The probability of a particular summer month requiring release is approximately one out of every four years.

- It could be expected that any period of Spillway release would be longer than one month in duration.

6 Physical Hydraulic Model Study Results

Tests were carried out on a 1/120 comprehensive scale model [Ref 5] of the Keeyask GS to refine and confirm the hydraulic design of the Spillway structures and channels, as well as on a 1/50 sectional scale model [Ref 6] to refine and confirm the hydraulic design of the Diversion and Spillway structures and gates.

Steady state tests confirmed that the piers and bridge deck elevations are adequate.

A mean pressure of -0.72 m was measured on the rollway during the final rollway tests. This pressure gave a cavitation index value (σ) of 0.62. Pressure fluctuations taken with a pressure transducer at the critical locations showed that the lowest pressure measured was -2.73 m at the downstream face of the gate slot with a corresponding cavitation index value of 0.54. As a σ value of 0.4 to 0.5 should be acceptable as a lower limit for incipient cavitation, it can be concluded from these results that the concrete surfaces will be free of damage from cavitation.

The steady state tests also showed that for all gate openings (limited to a maximum of 6 m opening during the tests), no vortices developed upstream of the gates and that there was no ice drawdown.

Dynamic tests showed that there are no uplift/downpull forces or hydraulically induced vibrations on the spillway gates and that the load on the gate hoist never exceeds the weight of the spillway gate (plus or minus the friction).

Calibration tests carried out on Bay 4 established rating curves of the Spillway structure for full and partial gate openings. The fully open rating curve obtained from the physical model was very close to the theoretical rating curve developed during the preliminary design.

Prior to testing gate combinations, tests were done on the physical model to optimize the discharge channel configuration. Ideally, flow should be spread out as evenly as possible across the Spillway Discharge Channel in order to maximize energy dissipation.

Release of flow through the Spillway can be carried out using many combinations of gate openings. A number of these combinations, however, may cause unacceptable hydraulic conditions. Tests were therefore conducted on the physical model to explore preferred opening sequences.

A series of sequential gate openings were tested to document the hydraulic conditions in the tailrace channel for the operation of one bay at a time and for all bays at equal gate openings. Single bay operation produced asymmetrical flow in the discharge channel with return eddies

on both sides of the issuing jet. With equal gate openings in all bays, the discharge across the channel was more uniform. The flow velocity along each side wall was less than the flow velocity in the central part of the channel, however, there were no return currents.

All gates operating at partial/full gate openings led to effective energy dissipation in the discharge channel. Gate openings less than 1 m, however, should be avoided due to the presence of rooster tails which will produce spray and icing in winter.

With Bays 2, 4 and 6 open 4 m each and the reservoir at el 159.0 m, it was noted that the discharge would hug the south face of the channel. A re-circulation eddy was evident on the north side. The most southerly bay was briefly opened and then closed which resulted in the discharge shifting to the north side wall and a re-circulation eddy forming on the south side. This anomaly was evident whenever Bays 2, 4 and 6 were operated uniformly. The tailrace discharge would hug either side wall, depending on the initial flow pattern. This type of flow condition is typical of diverging transitions. This flow pattern did not occur with equal discharges through Spillway Bays 1, 3, 5 and 7, as the outside bays prevented the formation of the re-circulation eddy along either sidewall.

7 Preferential Spillway Bay Operation

Criteria for establishing the recommended post-impoundment operating procedure for the Spillway gates include the following:

- Minimum gate opening of approximately 1.0 m in order to minimize the potential for gate vibration, and for cavitation and spray of jet issuing from under the gate, which is more prevalent at smaller gate openings.

- Maximum partial gate opening of approximately 6.0 m in order to minimize risk of damage to the bottom of the gate due to drawdown of debris and/or ice.

- Gates shall be operated in a pattern as symmetrical as possible.

- Only gate bodies 1, 3, 5 and 7 are winterized (heated) for winter operation. All seven bays shall have heated gate guides.

- Spillway gate opening versus discharge capacity are based on the results of the physical hydraulic model study [Ref 6].

- The recommended sequence of gate operation commences with Gates 3, 5, 1, then 7, etc. (or Gates 5, 3, 7, then 1, etc.) in order to avoid the anomaly of the discharge flows hugging either side of the channel as observed during the physical hydraulic model study.

Table 2 and Figures 1 and 2 provide the recommended operating procedure for the Spillway gates during open water conditions. Table 3 and Figures 3 and 4 provide the recommended operating procedure for the Spillway gates during winter conditions

Prepared By

D.M. Fuchs

Attach
References
Tables 1 to 3
Figures 1 to 4

DMF:DGM:spa

for D.G. Murray

References

1. KGS Acres Ltd., Memorandum GN-4.1.2, “Keeyask Generating Station – Stage IV Studies – Axis GR-4 – Hydraulic Design Criteria”, Manitoba Hydro File 00195-06330-0004.

2. KGS Acres Ltd., Memorandum GN-4.3.6, “Keeyask Generating Station – Stage IV Studies – Axis GR-4 – Hydraulic Design of Spillway”, Manitoba Hydro File 00195-23100-0008.

3. Acres Manitoba Limited, Memorandum GN-4.3.12, “Keeyask Generating Station – Stage IV Studies – Axis GR-4 – Spillway Comparison – 7, 8 & 9-Bay Alternatives”, Manitoba Hydro File 00195-23100-0007.

4. KGS Acres Ltd., Memorandum GN-4.3.15, “Keeyask Generating Station – Stage IV Studies – Axis GR-4 – Frequency of Spillway Operation”, Manitoba Hydro File 00195-06330-0006.

5. LaSalle Consulting Group Inc., “Keeyask Generating Station Stage IV – Comprehensive Model Study R.1572”, August 2005.

6. LaSalle Consulting Group Inc., “Keeyask Generating Station Stage IV – Sectional Model Study R.1584”, November 2005.

Tables

Table 1
Keeyask GS - Stage IV Studies, Axis GR-4
Keeyask Spillway Hydraulic Design Parameters

Parameter	Value
Full Supply Level (FSL)	159.0 m
Probable Maximum Flood (PMF) Level	160.3 m
Minimum Operating Level (MOL)	158.0 m
Spillway Design Flood (SDF)	8 705 m^3/s
Probable Maximum Flood (PMF)	12 700 m^3/s
Spillway Crest Elevation	144.5 m
Spillway Approach and Discharge Invert	138.9 m
Number of Gates	7
Width of Gates	13 m
1:1000 Year Daily Winter Flood	5240 m^3/s

Table 2
Keeyask Generating Station
Operating Procedure for Spillway Gates - Open Water Conditions

Discharge (m^3/s)	Reservoir Elevation (m)	Gate Discharge (m^3/s)	Gate Opening (m)						
			Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6	Bay 7
150	159.0	150	-	-	1.0	-	-	-	-
300	159.0	300	-	-	2.3	-	-	-	-
300	159.0	150	-	-	1.0	-	1.0	-	-
600	159.0	300	-	-	2.3	-	2.3	-	-
600	159.0	200	1.4	-	1.4	-	1.4	-	-
900	159.0	300	2.3	-	2.3	-	2.3	-	-
900	159.0	225	1.7	-	1.7	-	1.7	-	1.7
1,200	159.0	200	2.3	-	2.3	-	2.3	-	2.3
1,200	159.0	240	1.8	1.8	1.8	-	1.8	-	1.8
1,500	159.0	300	2.3	2.3	2.3	-	2.3	-	2.3
1,500	159.0	250	1.9	1.9	1.9	-	1.9	1.9	1.9
1,800	159.0	300	2.3	2.3	2.3	-	2.3	2.3	2.3
1,800	159.0	257	2.0	2.0	2.0	2.0	2.0	2.0	2.0
5,700	159.0	814	6.0	6.0	6.0	6.0	6.0	6.0	6.0
5,700	159.0	714/1414	5.2	5.2	Fully	5.2	5.2	5.2	5.2
6,300	159.0	814/1414	6.0	6.0	Fully	6.0	6.0	6.0	6.0
6,300	159.0	695/1414	5.0	5.0	Fully	5.0	Fully	5.0	5.0
6,900	159.0	814/1414	6.0	6.0	Fully	6.0	Fully	6.0	6.0
6,900	159.0	665/1414	Fully	4.8	Fully	4.8	Fully	4.8	4.8
7,500	159.0	814/1414	Fully	6.0	Fully	6.0	Fully	6.0	6.0
7,500	159.0	615/1414	Fully	4.4	Fully	4.4	Fully	4.4	Fully
8,100	159.0	814/1414	Fully	6.0	Fully	6.0	Fully	6.0	Fully
8,100	159.0	515/1414	Fully	Fully	Fully	3.8	Fully	3.8	Fully
8,700	159.0	814/1414	Fully	Fully	Fully	6.0	Fully	6.0	Fully
8,700	159.0	216	Fully	Fully	Fully	1.6	Fully	Fully	Fully
9,300	159.0	814/1414	Fully	Fully	Fully	6.0	Fully	Fully	Fully
9,900	159.0	1414	Fully	Fully	Fully	Fully	Fully	Fully	Fully
10,000	159.1	1,429	Fully	Fully	Fully	Fully	Fully	Fully	Fully
11,000	160.1	1,571	Fully	Fully	Fully	Fully	Fully	Fully	Fully
11,300	160.3	1,614	Fully	Fully	Fully	Fully	Fully	Fully	Fully

Notes:

1. Recommend minimum gate opening of approximately 1.0 m proposed in order to minimize gate vibration, potential for cavitation, and spray of jet issuing from under gate.
2. Gates shall be operated in a pattern as symmetrical as possible.
3. Recommend maximum partial gate opening of approximaely 6.0 m in order to minimize risk of damage to the bottom of the gate due to drawdown of debris and/or ice.
4. For all discharges equal to or greater than 9,900 m^3/s, all gates shall be fully open.
5. Discharge of 11,300 m^3/s represents the Probable Maximum Flood (PMF) discharge. An additional discharge of 1,400 m^3/s (speed-no-load) would be released through the Powerhouse.

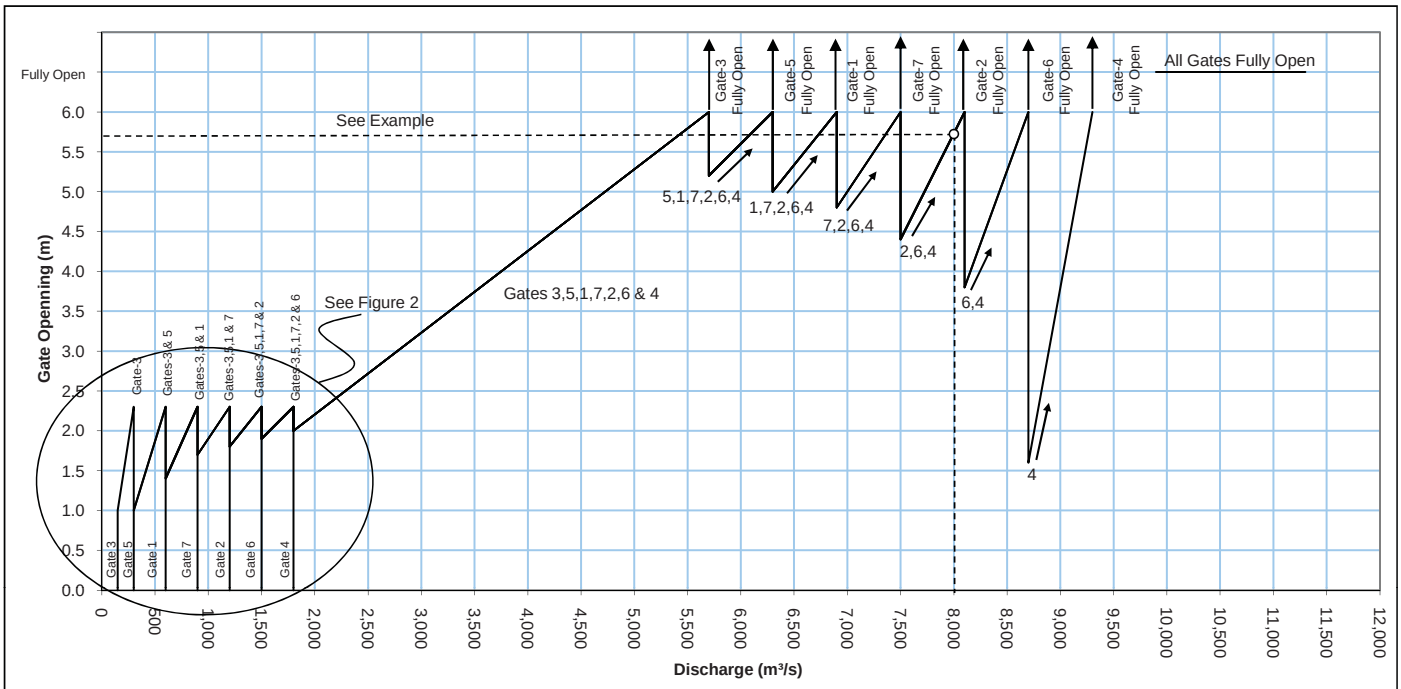
Table 3
Keeyask GS - Stage IV Studies, Axis GR-4
Operating Procedure for Spillway Gates - Winter Conditions

Discharge (m^3/s)	Reservoir Elevation (m)	Gate Discharge (m^3/s)	Gate Opening (m)						
			Bay 1	Bay 2	Bay 3	Bay 4	Bay 5	Bay 6	Bay 7
150	159.0	150	-	-	1.0	-	-	-	-
300	159.0	300	-	-	2.3	-	-	-	-
300	159.0	150	-	-	1.0	-	1.0	-	-
600	159.0	300	-	-	2.3	-	2.3	-	-
600	159.0	200	1.4	-	1.4	-	1.4	-	-
900	159.0	300	2.3	-	2.3	-	2.3	-	-
900	159.0	225	1.7	-	1.7	-	1.7	-	1.7
3,250	159.0	814	6.0	-	6.0	-	6.0	-	6.0
3,250	159.0	615/1414	4.4	-	Fully	-	4.4	-	4.4
3,850	159.0	814/1414	6.0	-	Fully	-	6.0	-	6.0
3,850	159.0	515/1414	3.8	-	Fully	-	Fully	-	3.8
4,450	159.0	814/1414	6.0	-	Fully	-	Fully	-	6.0
4,450	159.0	216/1414	Fully	-	Fully	-	Fully	-	1.6
5,050	159.0	814/1414	Fully	-	Fully	-	Fully	-	6.0
5,650	159.0	1414	Fully	-	Fully	-	Fully	-	Fully

Notes:

1. Gates 1, 3, 5 and 7 are winterized (heated) for winter operation.
2. Recommend minimum gate opening of approximately 1.0 m proposed in order to minimize gate vibration, potential for cavitation, and spray of jet issuing from under gate.
3. Recommend maximum partial gate opening of approximately 6.0 m in order to minimize risk of damage to the bottom of the gate due to drawdown of debris and/or ice.
4. Gates shall be operated in a pattern as symmetrical as possible.

Figures



Notes: 1) Once a gate is fully open, it will remain fully open as discharge increases.
2) Discharges apply only for a FSL of 159.0 m.

Example: In order to pass 8000 m³/s, Gates 3,5,1 and 7 are fully open and Gates 2,6 and 4 are open to 5.7 m.

KGS ACRES	KGS ACRES MANITOBA LIMITED
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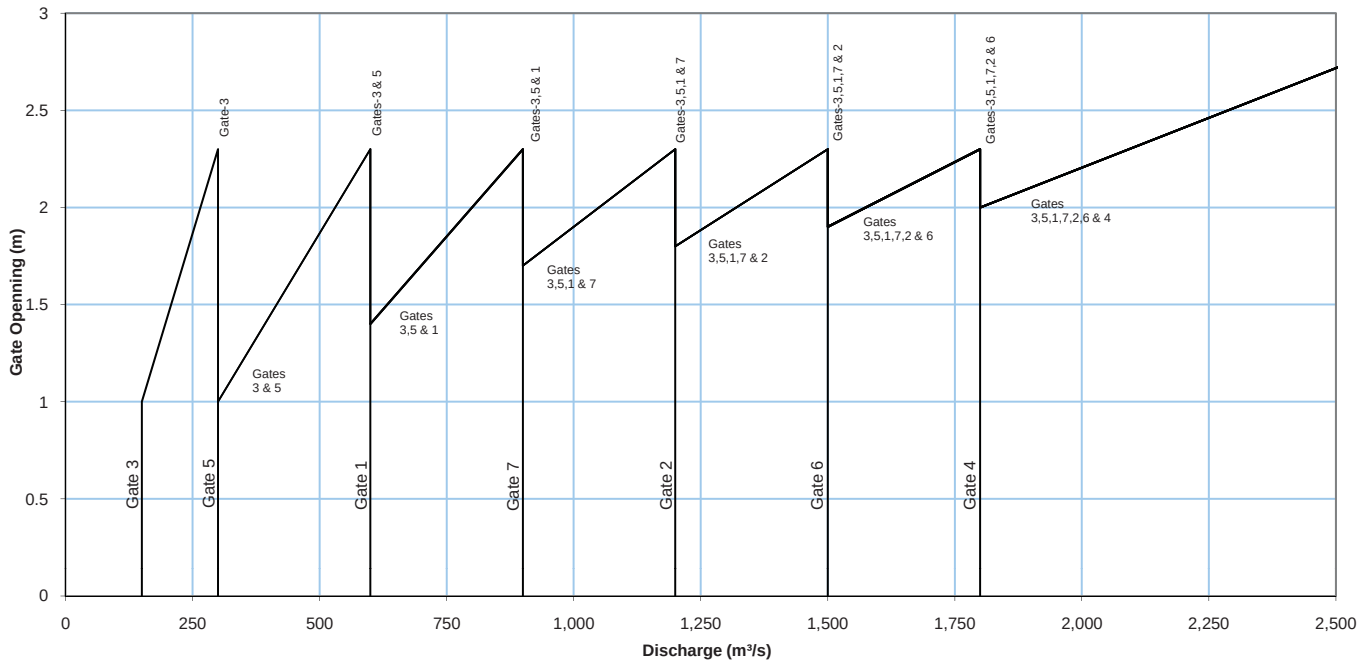
	KEEYASK GENERATING STATION
PROJECT P10008(16)	Operating Procedure for Spillway Gates - Open Water Conditions
SCALE NTS	
DATE 04/05/2011	REFERENCE TO MEMO NO. GN-4.3.23 Rev 2
	FIGURE 1



Note: 1) Discharges apply only for a FSL of 159.0 m.

KGS ACRES	KGS ACRES MANITOBA LIMITED
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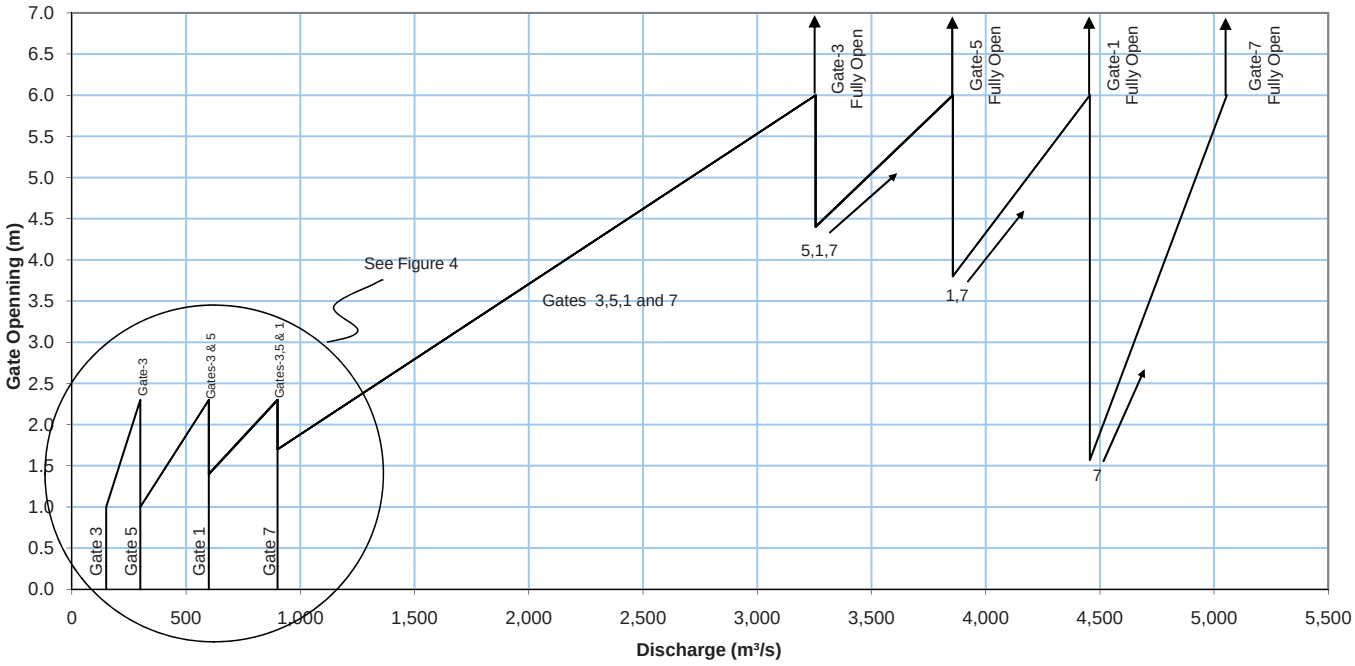
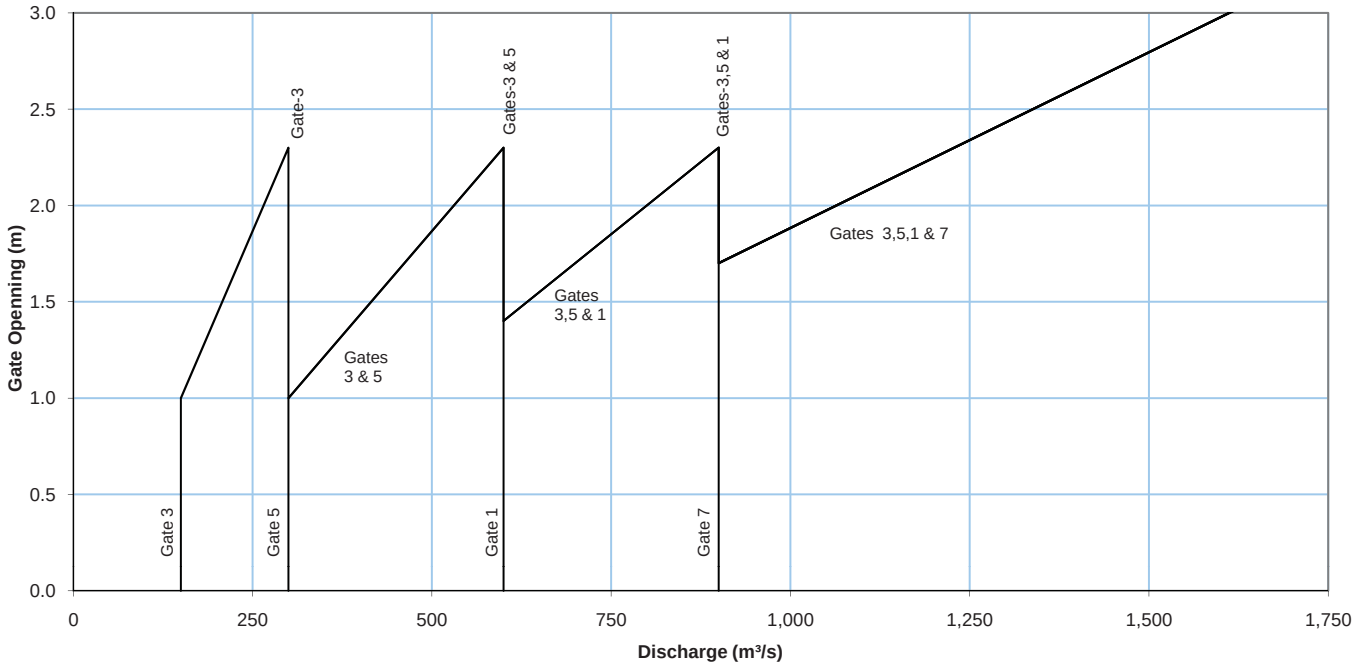
	KEEYASK GENERATING STATION
PROJECT P10008(16)	Operating Procedure for Spillway Gates - Open Water Conditions
SCALE NTS	
DATE 04/05/2011	REFERENCE TO MEMO NO. GN-4.3.23 Rev 2
	FIGURE 2



Notes: 1) Once a gate is fully open, it will remain fully open as discharge increases.
2) Discharges apply only for a FSL of 159.0 m.

KGS ACRES	KGS ACRES MANITOBA LIMITED	
	MANITOBA HYDRO	
	KEEYASK GENERATING STATION	
PROJECT P10008(16)	Operating Procedure for Spillway Gates - Winter Conditions	
SCALE NTS		
DATE 04/05/2011	REFERENCE TO MEMO NO. GN-4.3.23 Rev 2	FIGURE 3



Note: 1) Discharges apply only for a FSL of 159.0 m.

KGS ACRES	KGS ACRES MANITOBA LIMITED
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	KEEYASK GENERATING STATION
PROJECT P10008(16)	Operating Procedure for Spillway Gates - Winter Conditions
SCALE NTS	
DATE 04/05/2011	REFERENCE TO MEMO NO. GN-4.3.23 Rev 2 — FIGURE 4

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-9.1.6

Rev. 0, March 24, 2011

Existing Environment Ice Processes

Manitoba Hydro File 00195-11100-0141_02

Prepared by (Rajib Ahsan) March 24, 2011
for J.L. Groeneveld
KGS Acres Ltd.

Checked by Andrew J. Barylka (Andrew Barylka) March 24, 2011
KGS Acres Ltd.

Reviewed by (I.R. Dewar) March 24, 2011
KGS Acres Ltd.

Approved by (N.J. Smith) March 24, 2011
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - PHYSICAL ENVIRONMENT

EXISTING ENVIRONMENT ICE PROCESSES

REV 0

DELIVERABLE GN 9.1.6

MANITOBA HYDRO FILE: 00195-11100-00141_02

2011/03/24

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

KGS ACRES

Hydropower Limited Partnership

EXECUTIVE SUMMARY

This memorandum provides an overview of the ice processes typically encountered on northern rivers and provides a detailed description of the existing environment ice regime along the Nelson River between Split Lake and Stephens Lake. Also discussed are the long term winter field observation program and the development and calibration of numerical ice models which were used to simulate winter conditions along the study reach. A characterization of the ice regime in terms of water levels at key locations along the study reach is provided, as derived through numerical modeling efforts, for both existing environment and future environment without the Project conditions.

Within the study area, two distinct ice covers typically develop over the course of a winter season; a large **hanging ice dam** forms downstream of Gull Rapids and a mechanically thickened cover forms upstream of these rapids. A thermal cover usually develops first on Split Lake and Stephens Lake. The steeper gradient, and therefore higher velocities in the main river channel between these two lakes precludes formation of a similar thermal cover, and this reach remains open initially. As temperatures drop, frazil ice is produced in these open water reaches and this ice passes downstream until it reaches the solid ice cover developing on Stephens Lake. Ice begins to collect in this area over the course of the winter, leading to the formation of a large **hanging ice dam** and considerable staging at the foot of Gull Rapids.

In the reach above Gull Rapids, border ice begins to grow from each bank (where velocities are low enough) gradually reducing the open water width of the river. As the open water width narrows and generated ice pans become larger and larger, these pans eventually jam at a narrow section of the river, creating an ice bridge. This bridge typically forms within the vicinity of Gull Lake and permits the advancement of an upstream ice cover through juxtaposition. The cover frequently shoves and thickens as it progresses upstream in response to the hydraulic forces applied to it.

The cover stalls at the foot of Birthday Rapids owing to the high velocities at that location, leading to the formation of another **hanging ice dam** downstream of these rapids. This leads to staging of the river in this area and in most winters, once stages have increased sufficiently to drown out Birthday Rapids, the cover is able to continue advancing upstream. The upstream progression of the ice cover eventually stops downstream of Clark Lake.

Anchor ice typically forms on the invert of the channel at the outlet of Clark Lake and Split Lake each winter. The build-up of this ice results in an increase in winter water levels on Split Lake that can range from 0.3 m to 1.2 m above equivalent open water levels with the average increase being approximately 0.6 m.

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1 INTRODUCTION

Developing a good understanding of ice processes is an important part of any hydro power planning study for a development located within a northern climate. Ice processes along the Nelson River were studied to:

- estimate environmental impact associated with the Project,
- assist in setting winter plant operation policies;
- assist in developing construction management strategies; and
- predict winter power production.

Ice processes have been studied in detail along the lower Nelson River since the mid 1970's. These studies have included the development of:

i) a comprehensive field observation program, and

ii) detailed numerical models, which have been successfully used to simulate the complex ice conditions along the river.

This memorandum discusses the current ice regime of the Nelson River between Split Lake and Stephens Lake as well as the ice regime expected for the future environment without the Project. Findings are based on the observations gathered and results of the numerical modeling efforts. The sections for this memorandum are organized in the following manner:

- Section 2 provides a general overview of the **ice processes** typically encountered on a northern river like the Nelson River.
- Section 3 provides a brief overview of the approach adopted in these ice studies, and summarizes the observed data available for use in this study.
- Section 4 provides a detailed description of existing **ice processes** on the river, on a reach by reach basis.
- Section 5 describes the numerical models which have been developed to simulate winter conditions on this reach of the Nelson River.
- Sections 6 and 7 provide a summary of expected water levels along the reach, as derived through the numerical modeling efforts for the existing environment and

future environment without the Project respectively. Water levels presented relate to key locations along the river reach.

2 GENERAL OVERVIEW OF ICE PROCESSES

In a typical northern river, an ice cover begins to form with the onset of cool winter temperatures. The nature of the cover varies with location and water velocity, but generally can be described as either smooth "lake ice" or rougher more dynamic "river ice".

Lake ice usually forms in areas of very low velocity, such as lakes, or deep, slow-moving river sections. It forms when cold air temperatures cool the water surface to freezing at the beginning of the winter. This type of ice cover forms very quickly, often within the span of a single night, and grows steadily in thickness with time. The thickness of lake ice is primarily governed by air temperature and the depth of snow cover on the ice. If the snow cover becomes excessively deep, it can weigh the ice cover down causing it to sink below the water surface. This can cause cracks to form in the ice, allowing water to flood over the ice surface creating "slush" on the lake.

In more swiftly moving sections of a river, the nature of the ice cover is significantly different than that in the lake portions. In these areas, the cover evolves based on five basic processes, namely:

- **ice generation**,
- **ice front** progression and formation of large **hanging ice dams**,
- ice cover consolidation/**shoving**,
- **border ice** formation, and
- **anchor ice** formation.

Figure 1 illustrates these typical river **ice processes**.

Ice generation takes place in open water sections of a river reach. With the onset of winter, water temperatures within the river begin to fall, and eventually drop to near freezing. When the temperature drops below freezing, small ice crystals begin to form in the river. These small crystals, known as **frazil ice**, resemble fine snow crystals and are highly attracted to solid objects and each other. They gather together (or agglomerate), and eventually rise to the surface to form **ice pans**. These pans drift along the water surface, and in turn join together forming larger ice sheets.

Where **ice pans** and ice sheets encounter an existing ice cover, such as at a lake, they accumulate, and the cover advances upstream. The upstream end of an advancing ice cover is called the **ice front**. If flow velocities at the **ice front** are low enough, the ice cover continues to advance upstream through the accumulation of these sheets and pans, a process known as **juxtaposition**. However, if the advancing cover reaches a section of high velocity, the cover "stalls", and the **ice pans** begin to be drawn down under the cover and accumulate there. This formation is referred to as a **hanging ice dam**, and can result in a substantial rise in water level as the ice dam grows and thickens. Figure 2 illustrates a typical **hanging ice dam** formation.

In particularly steep or high velocity reaches, the advancing ice cover may frequently adjust and thicken as it grows. This "shoving" mechanism is a response to the internal pressures which will gradually mount within the cover due to the collection of ice on the leading edge, the weight of the growing cover, and the hydrodynamic drag forces applied to the underside of the cover by the moving water. Each time this occurs, the ice front retreats and the cover thickens. The thickening of the cover strengthens it, and provides it with a greater ability to resist these applied forces. Figure 3 shows the typical profile generated by such a mechanically thickened ice cover. As shown on the diagram, the toe (downstream limit) of the mechanically thickened portion of the cover is generally located at a section of a river with a stronger thermally grown ice cover (i.e. at a lake or reservoir), or at an ice bridging point in the river. The toe of the cover is generally the thickest region, and upstream of this toe, the ice cover exhibits a relatively constant thickness i.e. the minimum thickness required to generate sufficient strength to resist externally applied forces.

Border ice forms along the shoreline of a river, where velocities are low. The overall process by which **border ice** forms is similar to that described for lake ice. Lateral growth rates are sometimes augmented as drifting **ice pans** attach to the **shorefast ice**. Throughout the winter, the **border ice** continues to grow by these processes, gradually reducing the area of open water. In particularly low velocity locations, the **border ice** forming along each shore may eventually grow together, creating an **ice bridge** and hence an **ice front** against which drifting **ice floes** can begin to accumulate. The extent of **border ice** formation is governed by the flow velocity, river geometry, and winter temperatures. Figure 4 illustrates a typical **border ice** growth formation.

Anchor ice typically forms on the river bed at locations that are shallow and flowing rapidly, such as at the brink of a set of rapids or a waterfall. At these locations, the turbulent, high velocity flow causes mixing of the newly formed **frazil ice**. The **frazil ice** comes into contact with the river bed and attaches to the material on the river bottom. As this ice mass slowly grows, it begins to constrict or block the river channel,

and can result in a substantial rise in upstream water levels. Figure 5 illustrates a typical **anchor ice** accumulation.

3 APPROACH AND METHODS

To understand and document ice formation processes on the Nelson River, a comprehensive hydrometric monitoring program was established, with data collected during the winter months from November through to April. The winter program was initiated in the mid 1970's. Data collected each winter as part of the hydrometric monitoring program includes:

- photographic/video records of ice cover development and advancement. These videos and photos are taken periodically during the winter during helicopter reconnaissance flights,
- satellite imagery,
- periodic observations of ice cover progression and **anchor ice** formation along the reach,
- periodic survey measurements of water surface profiles,
- ice thickness measurements,
- collection of daily air temperatures (recorded at the Gillam Airport MSC meteorological station [Station No. 5061001]), and
- Split Lake outflow data (back-calculated from flows and levels recorded at the downstream Kettle Generating Station [G.S.]).

Based on this information, detailed numerical ice models were developed and calibrated to simulate the formation of the ice cover and its growth throughout the winter [Ref 1, Ref 2 and Ref 3]. Using actual daily air temperature and flow data, the models were calibrated to match observed field conditions. Once calibrated, the models were used to assess how the nature of the cover may vary depending on the severity of the winter air temperatures, and winter flow scenarios.

4 EXISTING ENVIRONMENT

The lower Nelson River within the Keeyask G.S. study area, which for the purposes of this memorandum is defined to start at the outlet of Split Lake and end at the inlet to Stephens Lake, is a fast flowing river with reaches separated by rock controls, rapids, and

lakes. This reach of the river contains some lakes which are shallow and narrow (simple widenings of the river) such as Clark Lake and Gull Lake. The river drops approximately 26 m along its 56 km course between Split Lake and Stephens Lake.

Ice formation on the lower Nelson River within the study area is a relatively complex process, and has been studied for many years by Manitoba Hydro. The following sections summarize major observations that have been gathered on the existing ice regime for the lower Nelson River in the study area.

4.1 ICE FORMATION ON THE NELSON RIVER

Under the present winter regime, a thermal cover usually develops first on the reach's bounding lakes (Split Lake and Stephens Lake), while other faster sections of the river remain open, generating large volumes of **frazil ice**. As a result, two distinct ice covers typically develop over the course of a winter season in this reach:

i) a large **hanging ice dam** forms immediately downstream of Gull Rapids, and

ii) a mechanically thickened cover forms in the reach between Gull Rapids and Split Lake. In addition, **anchor ice** growth at the outlets to both Clark Lake and Split Lake results in winter staging on these lakes.

Major **ice processes** observed along the river, from Split Lake to the Inlet of Stephens Lake, are briefly described below for each of these reaches.

4.1.1 SPLIT LAKE TO GULL RAPIDS

In this reach, the Nelson River drops 13 m, from an elevation of approximately 166 m on Split Lake, down to an elevation of approximately 153 m on Gull Lake. The majority of this head drop occurs over a relatively steep section of the river located between the outlet of Clark Lake down to a point which is approximately 15 km upstream of Gull Rapids. The higher velocities in this reach have a significant impact on overall ice formation processes.

Figure 6 provides an overview of the **ice processes** observed along this section of the lower Nelson River. The following bullets provide additional explanation:

- Each year, a competent ice cover forms on Split Lake relatively quickly, usually beginning sometime between mid-October and mid-November. This cover then gradually thickens over the winter period, depending on the air temperature, and the

snow cover. The thickness of ice on the lake can range from 0.8 m to 1.2 m depending on the meteorological conditions.

- Downstream of Split Lake, ice initially forms as a thin strip of **border ice** along each bank. Where velocities are relatively low, such as in Clark Lake, **border ice** growth is significant, and can cover a large portion of the lake. In other areas, like the relatively steep reach between the outlet of Clark Lake and Birthday Rapids, velocities are considerably higher. These higher velocities typically limit the growth of **border ice** to thin strips along the shoreline that are generally 20 m in width or less, as shown in Figure 7.

- At the same time, **frazil ice** is generated in the open water sections of the river, and these generated flocs agglomerate into **ice floes** and eventually, into larger **ice pans** and sheets. These pans gradually grow in size with time of exposure, and distance travelled downstream. Figure 8 shows a reach of the river near Gull Rapids, and gives an indication of the density and size of some of these pans.

- As the generated **ice pans** become larger and larger, they normally begin to jam at a narrow section of the river, creating an **ice bridge**. This bridge typically forms within the vicinity of Gull Lake, and thus permits the progression or advancement of an upstream ice cover. As shown on Figure 6, the **ice bridge** typically occurs at one of three locations in the vicinity of Gull Lake. Figure 9 shows the ice cover at a bridging point located near Gull Lake. The date at which this **ice bridge** may form is quite variable. Typically, bridging occurs by mid-December, but it has been known to occur as early as mid-November, and in other years, has not been observed to occur at all.

- Once initiated, this cover advances upstream through a **juxtaposition** process, as shown in Figure 10. The typical cover in the downstream reach of the lake (i.e. up to 10 km upstream of Gull Rapids) is relatively thin, and smooth, as the cover is able to advance fairly quickly and easily against the lower velocities in this area. However, the cover in the upstream reach of the lake is considerably thicker and rougher, as it must periodically **shove** and thicken. Each time this occurs, the ice cover collapses and consolidates, and ice may move downstream along the shore of each bank. This can expose sections of the shoreline to possible abrasion if they are in direct contact with this **pack ice**. Figure 11, taken at a section near the upper end of Gull Lake in mid-January, illustrates the roughness of this **pack ice**. The cover typically grows to be between 5 m and 8 m thick in this area of the river.

- If sufficient **border ice** exists in a river reach, the **border ice** acts as a buffer between the **pack ice** and the shore, and the interaction of the **pack ice** with the shorezone is

reduced. However, the hydraulic forces exerted on the river ice cover in the streamwise direction also create stresses in the **pack ice** which are partially spread laterally towards the riverbanks. Therefore it is also possible for **pack ice** in the river reach to be pushed laterally into the banks in response to this lateral pressure, or to push the **border ice** sections into the bank. The thicker the accumulation, the greater the developed lateral pressures will be. This can sometimes cause portions of the ice cover to buckle against the bank, or even be pushed up over the bank. Figure 12, taken in mid-January shows this bulldozing action at a location near the upstream end of Gull Lake. This action may also strip the shoreline of vegetation over large reaches.

- The advancing ice cover typically stalls either temporarily or for the season at the foot of Birthday Rapids, owing to the higher velocities present at this location. These high velocities causes **ice pans** to submerge and be carried under the leading edge, leading to the formation of a **hanging ice dam** downstream of the rapids. The formation of the **hanging ice dam** can result in a considerable accumulation of ice in a very local area. This congestion restricts the conveyance capacity of the channel below Birthday Rapids, and can lead to significant local staging. As the cover grows over the winter, significant internal stresses/pressures develop, and the cover can shift often as the matrix of ice fragments/floes grows. A portion of these loads can be transferred to the banks, due to lateral pressure exerted by the accumulation. If the accumulation of ice in the **hanging ice dam** is large enough, it can also result in some redirection of flow along the river banks as the main channel conveyance capacity drops. Figure 13 shows an ice dam which formed downstream of Birthday Rapids during one winter.

- As the **hanging ice dam** grows downstream of Birthday Rapids, it initially leads to increases in water levels at the foot of Birthday Rapids. Eventually, water levels may rise to a point that is high enough to "drown out" the rapids, lowering flow velocities, and allows the cover to begin advancing upstream again. This does not occur every year, but if it does, the cover eventually stalls at a location which is approximately 5 km upstream of Birthday Rapids. The cover advancement stalls at this point due in part to the steepness of the reach, in part due to the warming of air temperatures and increased solar radiation in late winter, and in part due to a reduction in the upstream open water area (in which **frazil ice** is generated) as the cover advances.

- The ice cover in the reach upstream of Birthday Rapids is mechanically thickened in order to provide sufficient strength to resist forces created by the flowing water and the weight of the upstream ice pack. Figure 14 shows the leading edge of this cover after it has progressed upstream of Birthday Rapids. The typical end of winter thickness of the cover is 2 m to 3 m in this area.

- The **hanging ice dams** and the mechanically thickened potions of the ice cover are hydraulically very rough when they are first formed. However, over the course of the winter, the rough underside of the ice will slowly become smoother due to the erosion of ice protrusions by the flowing water, and the infilling of gaps and holes within the cover by smaller **frazil ice** pieces. This smoothing effect can lead to a drop in water levels later in the winter.

- **Anchor ice** also typically forms just downstream of the outlet of Clark Lake, and also at the immediate outlet of Split Lake, as shown in Figure 6. These accumulations slowly restrict the conveyance of the channel in this area, leading to staging upstream along both Clark Lake and Split Lake. Historical records on Split Lake have shown that this increase in stage may range from as little as 0.3 m to as much as 1.2 m over the course of a winter. The average winter increase in level on Spilt Lake is approximately 0.6 m. Figure 15 shows **anchor ice** growth observed immediately downstream of Clark Lake. Figure 16 illustrates historical staging patterns observed on Split Lake due to this **anchor ice** growth. Also shown on this figure is an average winter staging pattern for the lake. As shown, on average, water levels begin to exceed open water stages at the beginning of November, when air temperatures begin to fall. These stages typically reach a maximum in late January/early February, and begin to fall again to open water levels later in the winter as these **anchor ice** accumulations begin to detach and release from the streambed. Over the course of the winter, the **anchor ice** may release due to thermal gain from the sun, and then subsequently reform later at night resulting in fluctuations in upstream water levels.

4.1.2 GULL RAPIDS TO STEPHENS LAKE

In this reach, the Nelson River drops 13 m, from an elevation of approximately 153 m on Gull Lake to an elevation of 140 m (typical) on Stephens Lake. The majority of this head drop occurs within Gull Rapids over a distance of approximately 4 km. Although the rapids contain three separate channels (north, centre, and south) the majority of flow remains in the south channel of the river. Velocities in this branch are high, as flows cascade downstream over a series of rock controlled shelves. These high velocities have a significant impact on ice formation processes. Ice formation processes within this reach are described below:

- In this reach of the river, an ice cover initially forms on Stephens Lake in the early fall, typically by November 1st, although these formation dates may vary somewhat depending on the fall air temperatures. Historical observations have shown ice formation dates on Stephens Lake falls within a window between mid-October and mid-November.

- Once Stephens Lake freezes over, and before the upstream cover can bridge at one of the three locations on Gull Lake shown in Figure 6, all ice generated in the upstream reach pass through Gull Rapids, collect on the leading edge of the cover, and cause the cover to begin to advance upstream. However, the opportunity for upstream progression is limited and the **ice front** typically stalls at the site of the proposed Keeyask G.S. due to the high velocities. Any incoming ice is submerged and deposited under the ice cover resulting in the formation of a large **hanging ice dam** downstream of Gull Rapids. The growth of this ice dam is initially very rapid, but slows considerably when and if an **ice bridge** forms upstream in Gull Lake.

- The **hanging ice dam** continues to grow throughout the winter. However, the ice cover does not progress through Gull Rapids even under an extremely cold winter. The formation of the **hanging ice dam** can result in a considerable accumulation of ice in a very local area, as shown in Figure 17, which was taken just downstream of Gull Rapids one winter. This congestion restricts the conveyance capacity of the channel below the rapids, and can lead to significant local staging (7 m to 8 m above open water levels have been observed). As the cover grows over the winter, significant internal stresses/pressures develop, and the cover can shift often as the matrix of ice fragments/floes grows. A portion of these loads can be transferred to the banks, due to lateral pressure exerted by the accumulation. In this environment, the banks become susceptible to erosion when ice is pushed up against the bank, or moves directly along the shoreline, abrading the river bank. Figure 18 illustrates a typical example of this at a section immediately downstream of Gull Rapids.

- If the accumulation of ice in the **hanging ice dam** is large enough, it can also result in some re-direction of flow along the river banks as the main channel conveyance capacity drops. This has been observed to occur on a number of occasions in the reach within and downstream of Gull Rapids. Figure 19 illustrates flow through a new channel that was formed during the 2000/2001 winter, a year in which ice dam formation was particularly severe in this area because an **ice bridge**, and thus an ice cover, did not form upstream of Gull Rapids. It should be noted that there have been at least three winters over the past 15 years in which formation of an ice cover in the upstream reach was delayed, leading to the formation of a massive **hanging ice dam** downstream of Gull Rapids.

- The **hanging ice dam** typically extends approximately 5 km into Stephens Lake, and can lead to significant **shoving** of ice onto downstream islands within this area.

- As noted previously, typically at some point in the winter, the ice cover bridges in the vicinity of Gull Lake. This greatly reduces the amount of ice being passed through Gull Rapids and deposited in the **hanging ice dam**.

4.2 SPRING BREAKUP ON THE NELSON RIVER

In the spring, breakup of the river ice in this area is preceded by the release of **anchor ice** at the outlet of Split Lake and Clark Lake. This usually begins to occur in late February, and as a result, water levels on Clark Lake and Split Lake begin to drop in these latter winter months. The river ice then begins to deteriorate in late March and throughout April, as the sun's stronger solar radiation begins to weaken the ice, and snowmelt runoff begins. Open water leads begin to form throughout the main cover, as shown in Figure 20. In tandem with this, rising flows cause stages along the river to increase, and with this rise in water level, the cover eventually loses its bank resistance against the **shorefast ice**. The leading edge of the cover then begins to retreat down river as the cover progressively breaks, and reforms, at times possibly resulting in an **ice jam**. In areas where the **pack ice** is contained by wider **border ice** reaches, the **border ice** tends to remain in place slightly longer, and the **pack ice** retreats in the center of the river. Eventually, the leading edge will retreat to the location of the stronger lake ice, leaving open water in upstream areas. Figure 21 illustrates a section of the existing river (located just downstream of Birthday Rapids) at a point late in the breakup process. The de-staging of water levels in the reach typically begins in March, and continues through until mid-May, at which time levels return to open water levels throughout most of the reach.

Ice remnants located along the shore zone downstream of Birthday Rapids continue to melt and deteriorate, typically into June. Figures 22 and 23 illustrate typical remnants of **shorefast ice** that have become grounded along the river reach, and are melting in situ. In Figure 22, the ice remnants shown are the remains of a thinner **border ice** cover which has collapsed onto the existing shoreline following retreat of the main ice cover. Figure 23 illustrates ice remnants along the shore in an area of heavy **pack ice**.

Downstream of Gull Rapids, the large **hanging ice dam** also begins to deteriorate, leading to the development of open leads within the cover. The cover begins to melt, and with the onset of higher flows associated with the spring freshet, flush out into Stephens Lake.

5 MODELING

Sophisticated numerical models have been developed over the years to help better understand and predict ice conditions on river reaches. Prior to this study, a robust and comprehensive model, ICESIM, had been developed and calibrated to represent the steady growth of the ice cover over a typical winter period [Ref 3]. This model was used to predict ice conditions in the vicinity of the proposed Keeyask G.S. for use in the design and construction of the Project. For this study, this past work has been augmented with the development of a hydrodynamic ice model, ICEDYN, which is capable of

dynamically routing projected daily discharge and/or forebay level variations through the study reach.

5.1 ICEDYN MODEL

The ICEDYN model is a powerful ice simulation program derived from the earlier ICESIM model developed for use on the Nelson River. Like its predecessor, the ICEDYN model is fully capable of simulating typical ice formation processes, including **ice generation**, deposition, advancement, **shoving** and thickening. In addition, the program is also capable of dynamically routing river flows and/or forebay water level variations through the study reach. It does so through a solution of the St. Venant equations of unsteady fluid flow. The model also has the ability to represent staging due to **anchor ice** formation along a river reach by way of a time dependent staging factor.

One of the weaknesses of both the ICESIM and the ICEDYN models are that they tend to over estimate water levels for winter dates beyond when peak staging occurs (after the **ice front** has stalled). **Ice processes** are difficult to simulate when this occurs due to the longer days (increased exposure to sunlight) and smoothing of the ice surface (reduction in ice roughness). These factors tend to result in an **ice front** recession and a reduction in water levels which these models cannot predict. As a result, the ICEDYN model cannot simulate the de-staging of water levels and the subsequent return to open water levels in the spring. For these reasons, the use of the ICEDYN model to predict winter water levels throughout the entire winter period must not be viewed as an absolute, but rather as an indicator of the trend.

5.2 MODEL SETUP

Due to the two separate ice covers which occur over the study area, modeling of the entire river reach with one model was not possible. To overcome this complication, two separate ICEDYN models were set up. One model was set up to simulate the reach upstream of Gull Rapids (between Split Lake and Gull Rapids) which will be referred to as the upstream model reach, and the other to simulate the reach downstream of Gull Rapids (between Gull Rapids and Stephens Lake) which will be referred to as the downstream model reach. A map showing these modelled reaches along with the location of the key sites along the reach where water levels have been characterized is provided on Figure 24.

Cross sections for the simulation were derived directly from existing backwater datasets of the reach. In total 58 cross sections were utilized in the reach from Split Lake to Gull Rapids, and 11 cross sections were utilized to model the reach from Gull Rapids to Stephens Lake. These cross sections were surveyed as a part of earlier hydrometric

surveys, and are consistent with those sections utilized in concurrent open water studies. Additional interpolated cross sections were added to the two models to ensure stability of the dynamic model.

When user specified air temperatures fall below 0^{o}C, the simulation of **ice processes** on the river reach begins including:

- the generation of **frazil ice**,
- the initiation of **border ice** growth, and
- the advancement of the ice cover by **juxtaposition**, and where velocities are high, the deposition of ice in a **hanging ice dam** downstream of the higher velocity area.

5.3 CALIBRATION

Following its initial setup, the models were calibrated to match open water rating curves previously derived at a number of specific locations along the river reach using an open water backwater model. After obtaining a suitable match under open water conditions, the models were then used to simulate the development of an ice cover on the two study reaches for particular winters in which ice observation data was available. Ice parameters for the models were initially selected based on the parameter sets identified in earlier studies undertaken with the ICESIM model. These parameters were then adjusted as necessary to obtain a good match between the ICEDYN modelled levels and those measured in the field for a number of past winters. The variation in the ice parameters used in the models are attributed to the differences in the two model algorithms, as well as the collection of additional low flow water level data since the ICESIM model was developed.

Anchor ice staging was incorporated at two locations, at the outlet of Clark Lake, and at the outlet of Split Lake. This staging is incorporated into the model using a user defined **anchor ice** pattern applied to specific cross sections. The timing and magnitude of the **anchor ice** adjustment was derived based on past experience and measured water level data. For all simulations undertaken, these patterns were assumed to be constant. While it is recognized that the timing and amount of **anchor ice** formation is variable from year to year, there is no adequate way to simulate its development and growth. Based on the measured data available, the staging factors incorporated are expected, on average, to adequately represent the degree of staging.

The upstream boundary condition of the models consisted of a user defined flow hydrograph, while the downstream boundary condition consisted of a user defined stage

hydrograph. Air temperature sequences utilized in the models were based on meteorological data collected at the Gillam airport.

Under open-water conditions, the models were calibrated to within 0.25 m of the open-water rating curves derived at the key location in the study area. Under winter conditions, a good overall match was achieved between measured and modelled water level data. Figure 25 illustrates a sample water surface profile for the upstream reach of river between Spilt Lake and Gull Rapids. As shown in Figure 25, the first 10 km (i.e. within Gull Lake), and the ice profile consists of a relatively thin, **juxtaposed** cover. Upstream of this, the cover is considerably thicker, and consists of a much rougher, mechanically thickened profile.

The upstream model was able to reproduce winter water levels at key locations upstream of Gull Rapids to within 0.5 m, on average, of those observed during the freeze-up period. Downstream of Gull Rapids, the downstream model was able to reproduce observed freeze-up water levels to within 0.75 m on average. Differences between measured and modelled water levels of up to 2 m did however exist at certain locations in some years (Birthday Rapids and downstream of the outlet of Clark Lake). Such deviations are to be expected given the lack of available data for some years on the timing and location of the **ice bridge** which initiates the upstream winter cover. This lack of data made it necessary to assume bridging locations and dates for many years based on general trends observed in other years. An error in the selection of the timing or location of the bridging points could lead to differences in the modelled arrival of the **ice front**, which at locations more susceptible to channel blockages due to ice, can lead to these larger differences.

Figure 26 shows the results of a simulation for the 1991/1992 flow year for the reach of river downstream of the Gull Rapids. Also shown for comparison on this figure are a number of surveyed water levels that were taken for this period. As discussed in Section 4, a large **hanging ice dam** typically forms in this area, and the significant size of this **hanging ice dam** is well demonstrated in this profile plot. Again, the reasonable match obtained between observed and computed water levels demonstrates the ability of the numerical model to represent ice conditions along the reach.

5.4 SPRING DE-STAGING

As indicated earlier, the de-staging of water levels during the spring cannot be modelled with ICEDYN. Based on observed data, the de-staging of water levels generally begins at the beginning of March. To accommodate this, ICEDYN modelled water levels after March 1st had a time varying de-staging factor applied to them such that as spring

progressed, the modelled water levels returned to their open water equivalents. A representative de-staging factor and the date of the return to open water levels were estimated based on historical observations and were applied to all simulations undertaken. Under the existing environment, water level staging was linearly reduced by 20% over the month of March, with the remaining 80% of the total winter staging being eliminated linearly over the months of April and May. Predicted water levels were thus returned to their open water equivalents by June 1st.

6 CHARACTERIZATION OF EXISTING WATER LEVELS

The observed winter data collected in this reach, although excellent for calibration of the numerical models, is not gathered frequently enough to be able to provide a continuous characterization of water levels over the winter period. Measurements are only gathered at specified intervals. To augment this data, and thereby provide a more continuous record of levels, numerical models were setup to simulate each winter season from 1977 through to 2007.

The ICEDYN model was given actual flow and air temperature data for each winter, and simulations were then run for each winter season. Modelled winter water levels were then extracted at key locations throughout the study reach and processed into duration curves to provide a more complete picture of the range of water levels experience along this reach in the winter. Figures 27 to 37 show these duration curves of winter water levels for the key sites located along this river reach. The duration curves for Stephens Lake and Split Lake, Figures 27 and 37 respectively, reflect measured water level data rather than modelled water levels. All of these charts represent the estimated frequency with which various winter stages are experienced at each key site between November 1st and May 1st over the period from 1977 to 2007.

7 FUTURE CONDITIONS WITHOUT THE PROJECT

It is expected that without the development of the Project, and assuming that climatic and watershed conditions remain as they currently are, that the winter regime for this reach of the Nelson River would continue to be the same as that described in this memorandum for the existing environment. The severity of **ice processes** will vary from year to year depending on specific meteorological conditions, but in general the major **ice processes** and thus the ice regime will be unchanged.

With this in mind, Figures 38 to 41 illustrate representative hydrographs for the key sites on this river reach under the 5th, 50th, and 95th percentile average seasonal winter flows (November to March). Note that hydrographs for the Keeyask Tailrace site (Figure 41)

reflect typical ice bridging dates on Gull Lake. Thus, water levels will be higher than illustrated at this site in years when the bridging of Gull Lake is delayed or does not occur. The 5th, 50th, and 95th percentile Stephens Lake levels are assumed to occur with the 5th, 50th, and 95th percentile average seasonal winter flows.

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REFERENCES

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FIGURES

Figure 1: Typical river ice processes (after Ashton 1986) [Ref 4]

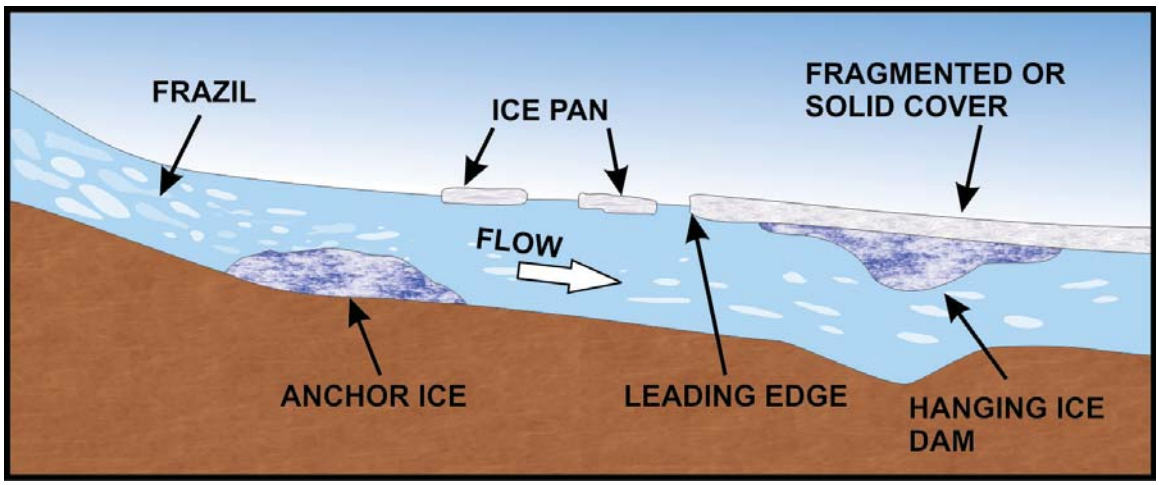


Figure 2: Typical hanging ice dam (after Ashton 1986) [Ref 4]

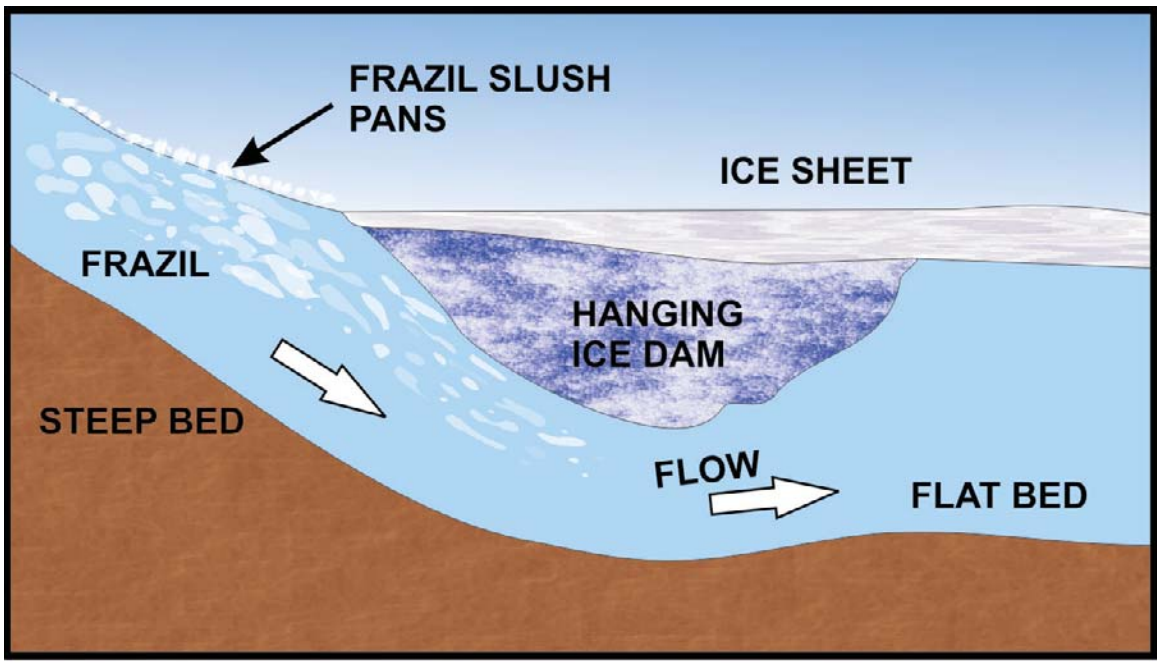


Figure 3: Typical mechanically thickened ice profile (after Ashton 1986) [Ref 4]

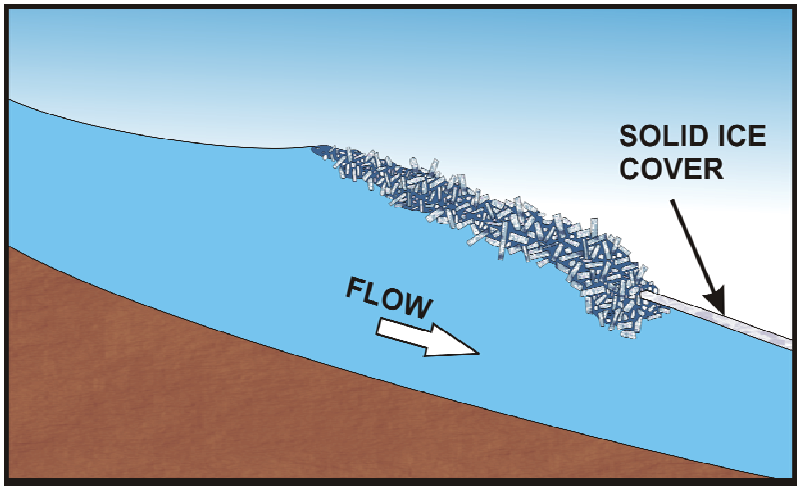


Figure 4: Typical border ice growth (after Ashton 1986) [Ref 4]

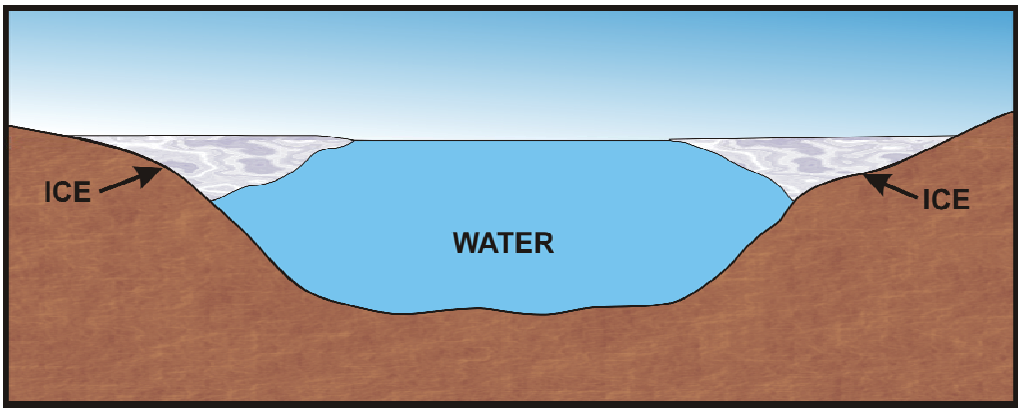
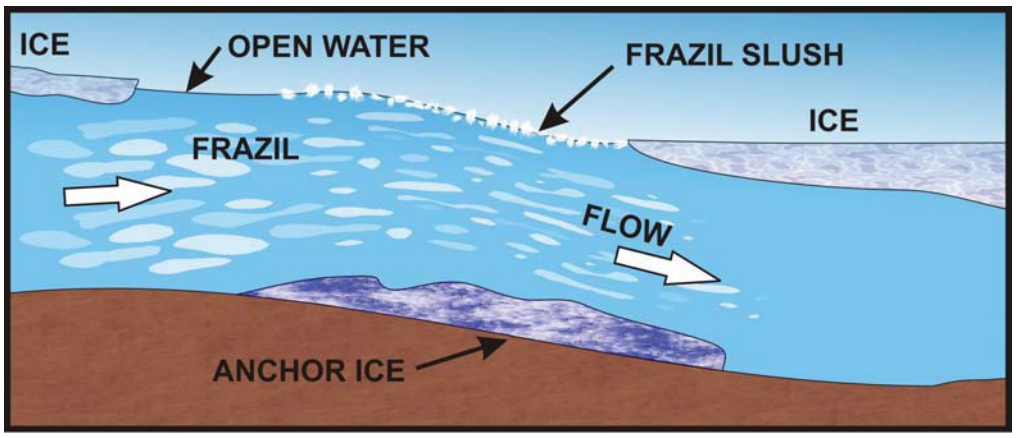


Figure 5: Typical anchor ice accumulation (after Ashton 1986) [Ref 4]



KEEYASK

0 1,500 3,000 6,000 Meters

0 7,500 15,000 30,000 Feet

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Baseline lakes, rivers, roads, railway and transmission line, taken from NTDB maps 64A and 54D, 2005. Her Majesty the Queen in right of Canada. All rights reserved.

Mapping grade, for general reference only.

Ice Processes from Split Lake to Stephens Lake

Figure 6

Created By:
KGS ACRES

Date Created:
March 13, 2009

Figure 7: Outlet of Clark Lake (looking downstream)

Figure 8: Typical ice pan density, just upstream of Gull Rapids (looking downstream)

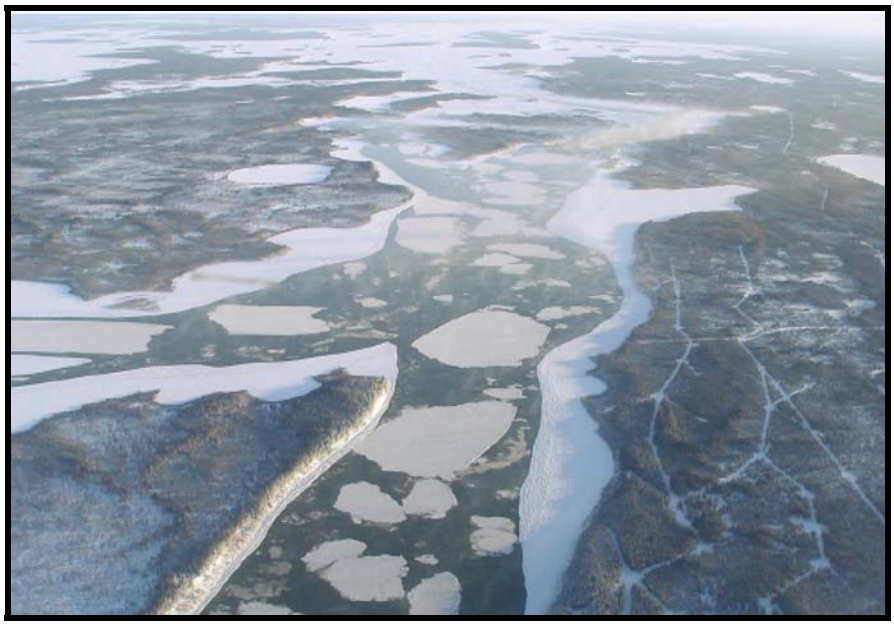


Figure 9: Typical bridging point near Gull Lake
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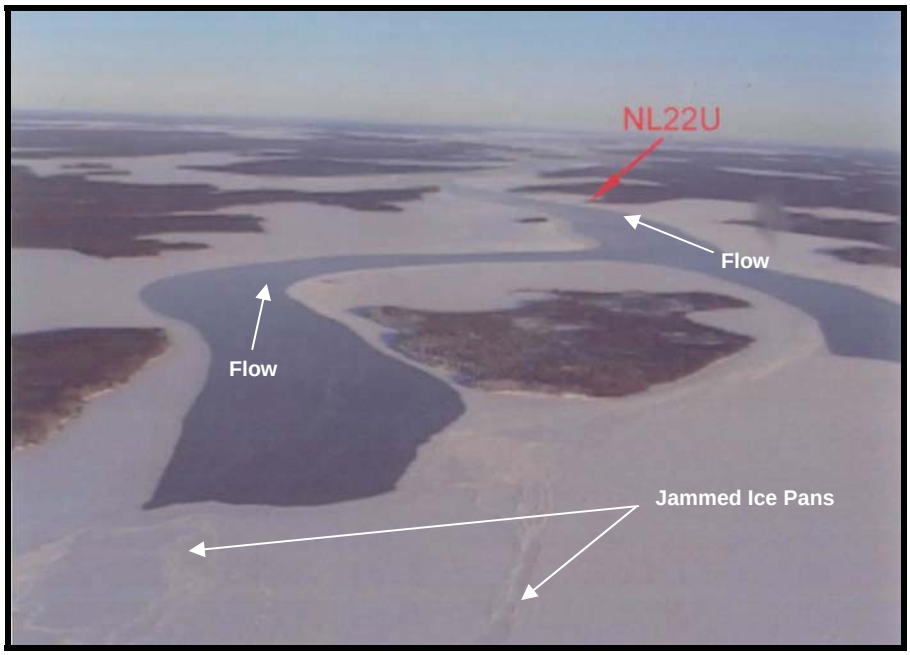

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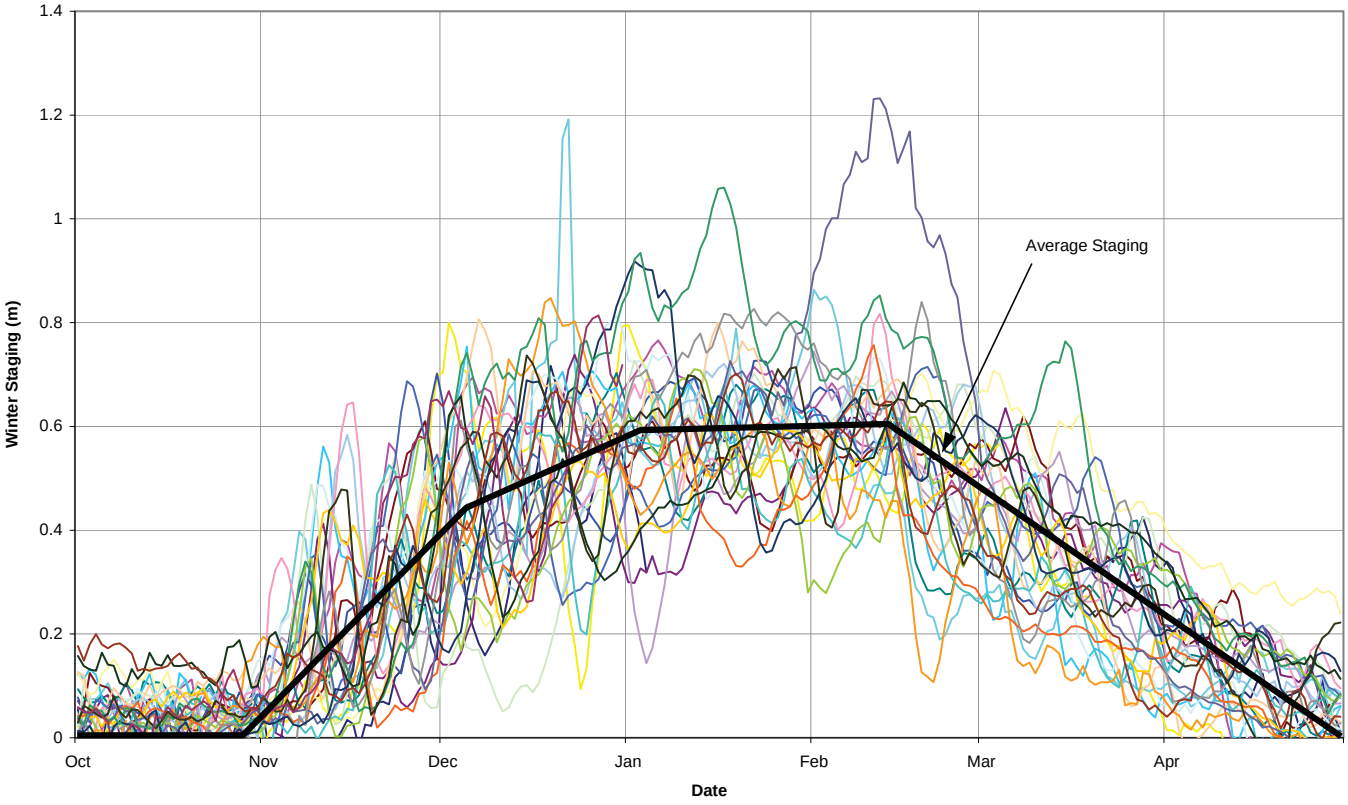

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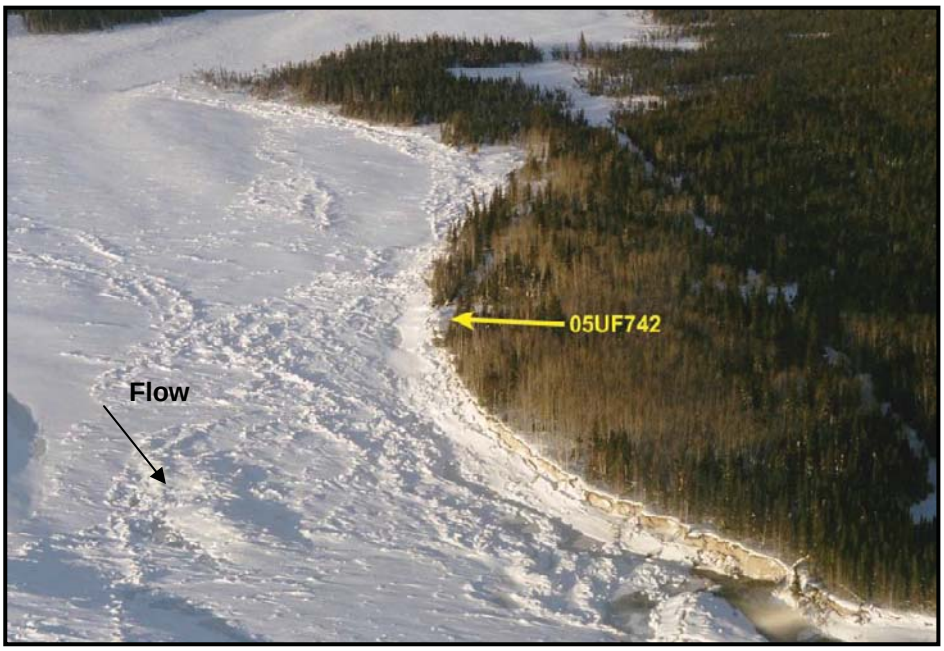

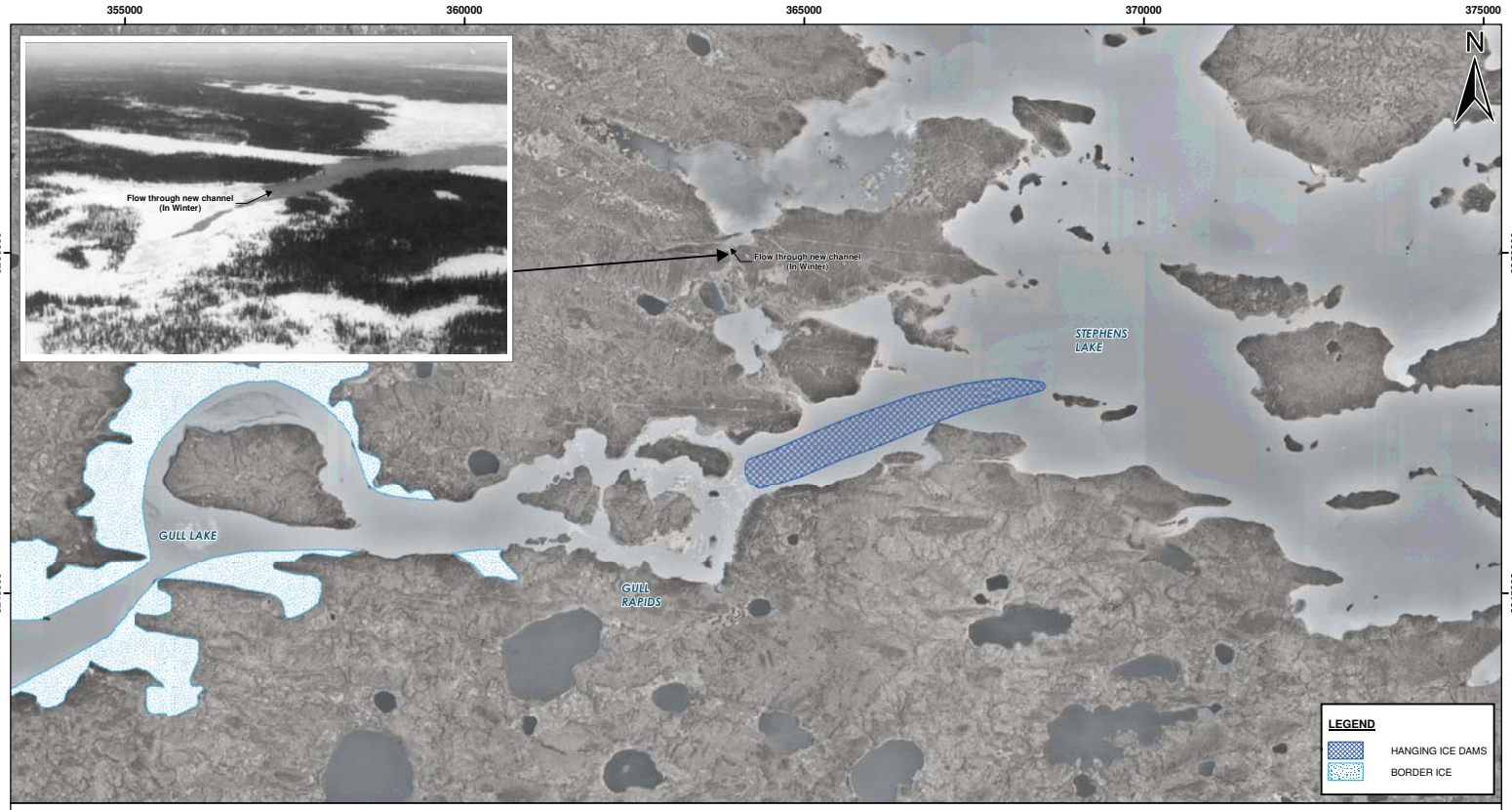

KEEYASK

0 500 1,000 2,000 Meters

0 2,500 5,000 10,000 Feet

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Air Photos Provided by Manitoba Hydro 2006

Mapping grade, for general reference only.

Hanging Ice Dam Downstream of Gull Rapids Leads to Erosion of New Channel

Figure 19

Created By:
KGS ACRES

Date Created:
April 04, 2009

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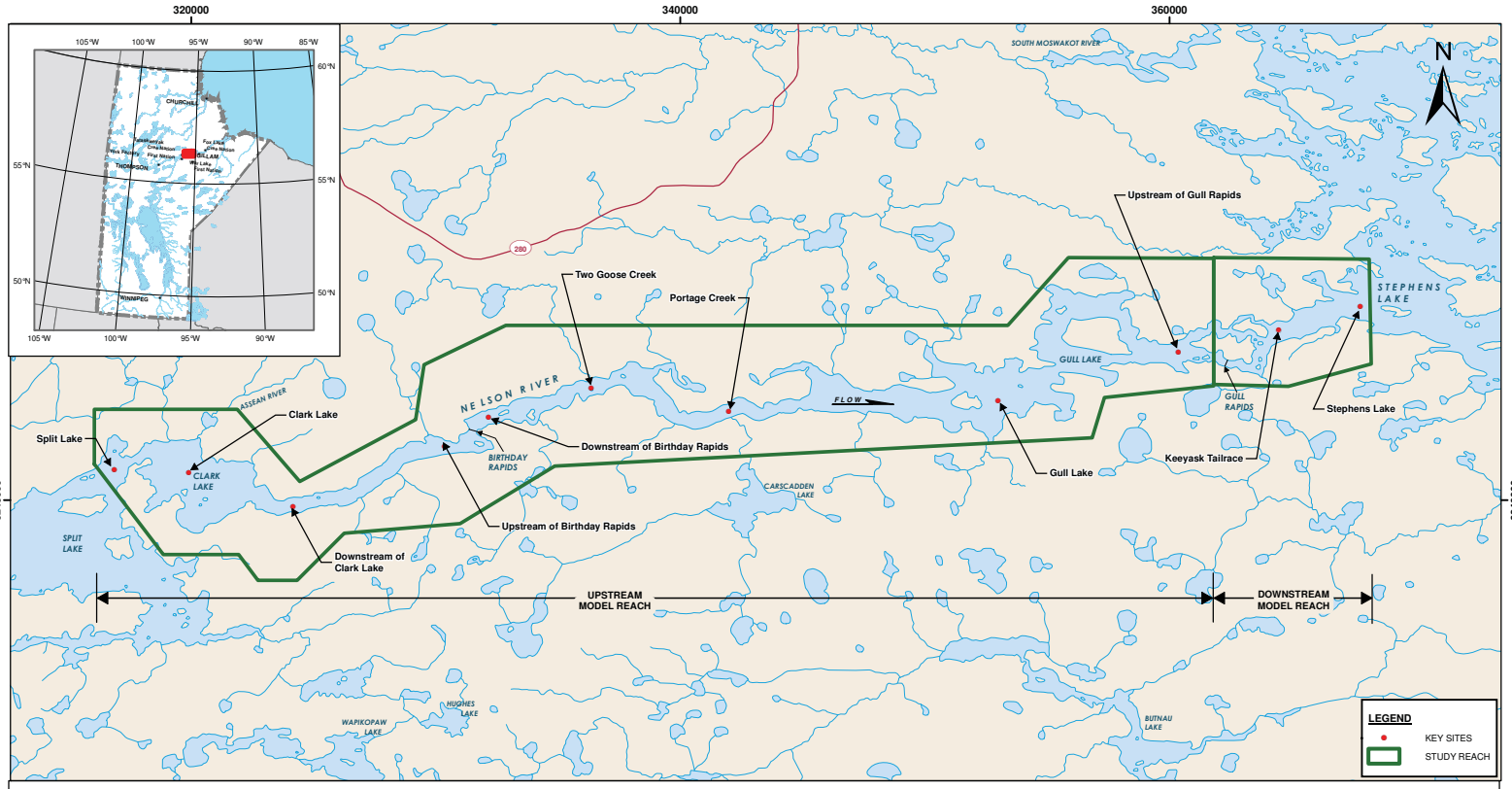

KEEYASK

0 1,500 3,000 6,000 Meters

0 7,500 15,000 30,000 Feet

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Baseline lakes, rivers, roads, railway and transmission line, taken from NTDB maps 64A and 54D, 2005. Her Majesty the Queen in right of Canada. All rights reserved.

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Key Sites Along Study Reach

Figure 24

Created By:
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Date Created:
March 13, 2008

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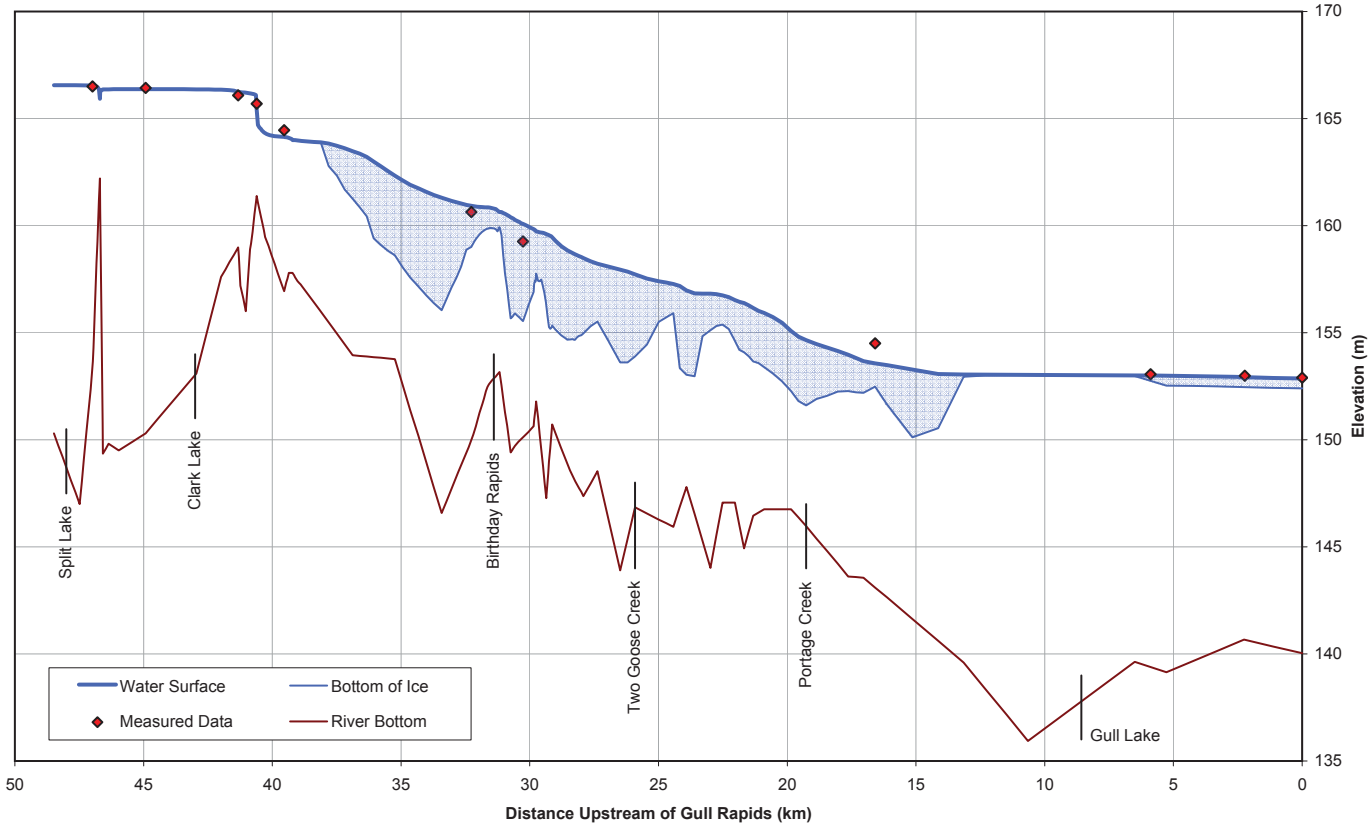

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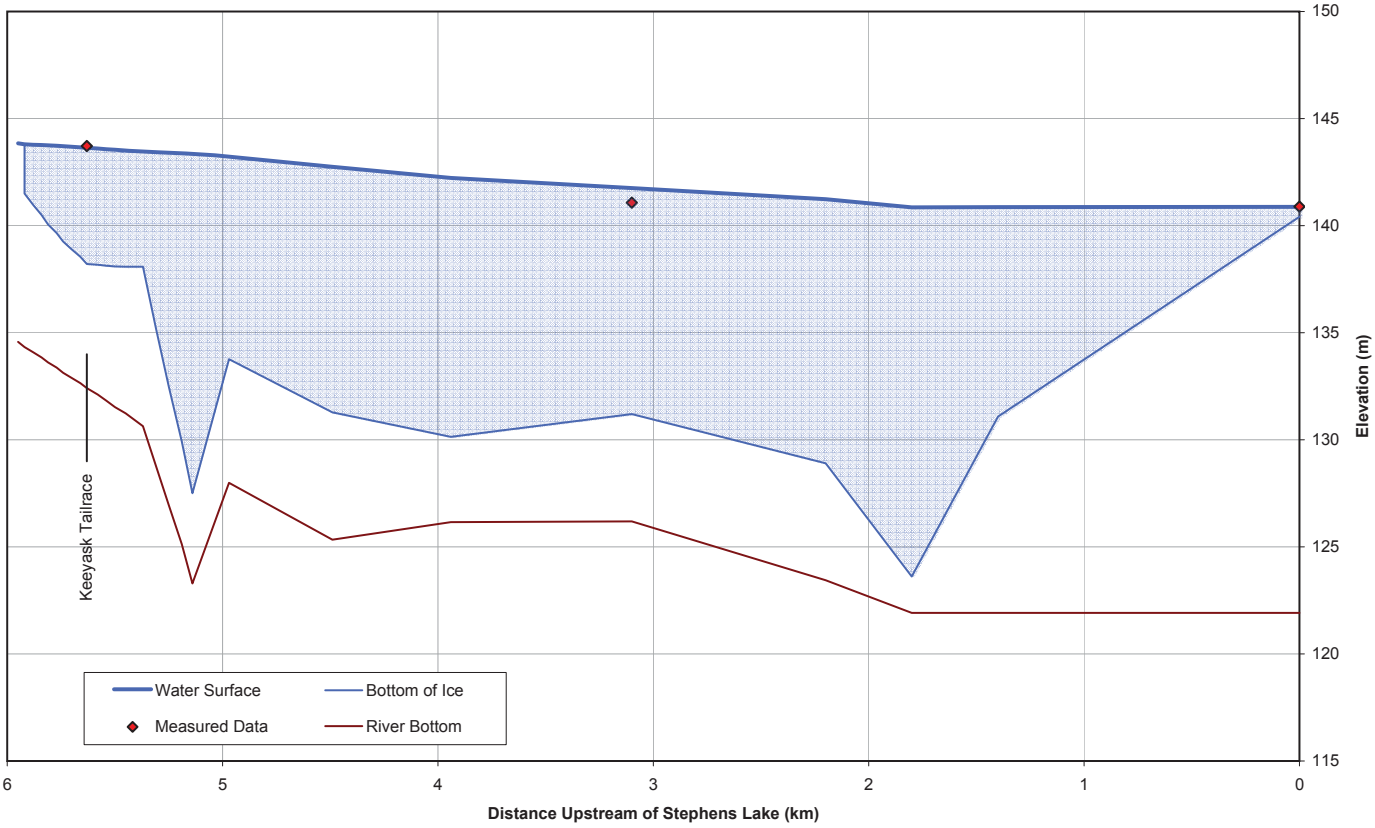
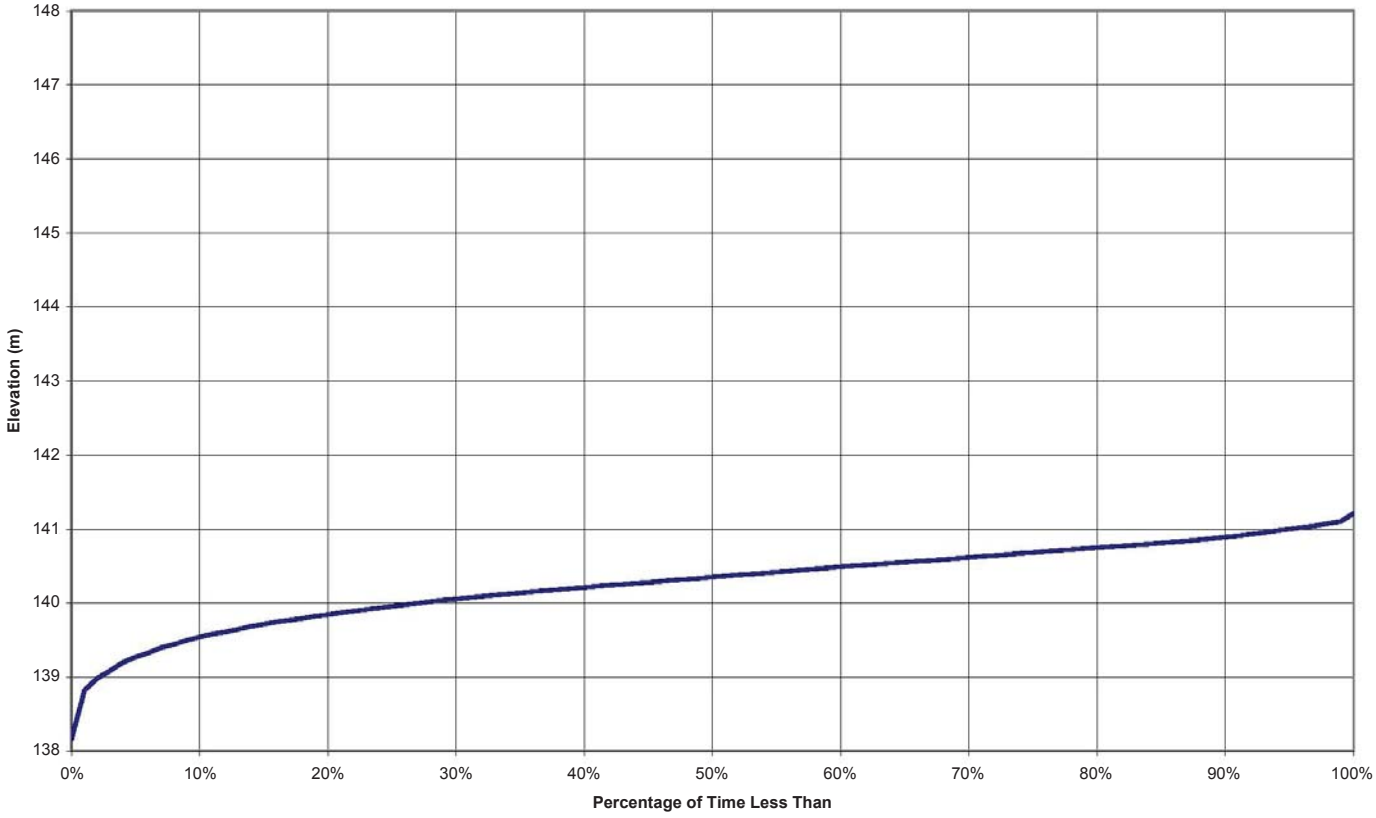


Figure 27
Nelson River at Stephens Lake - Existing Environment Winter Stage Duration Curve



Note: Curve based on measured winter water levels between November 1 and May 1

Figure 28
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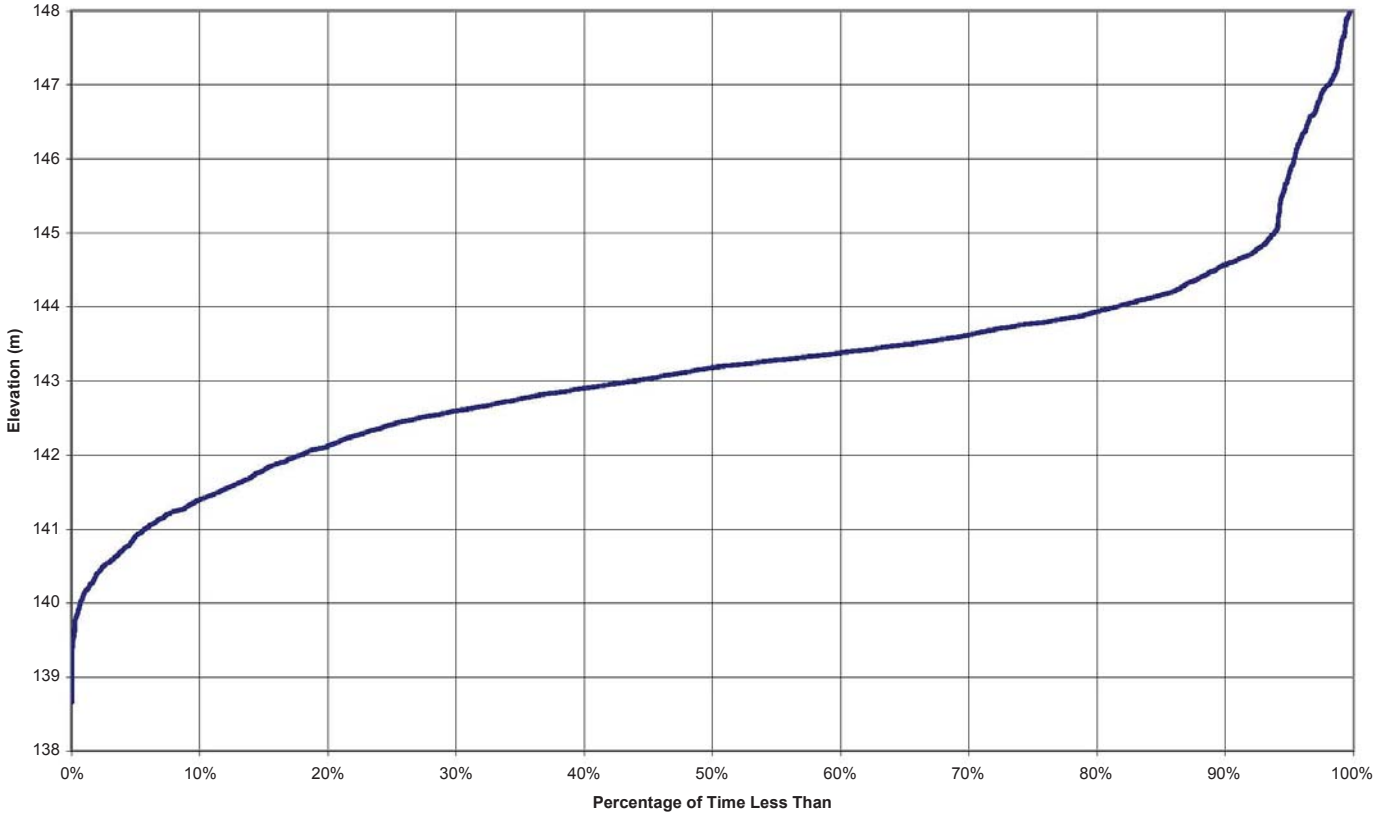
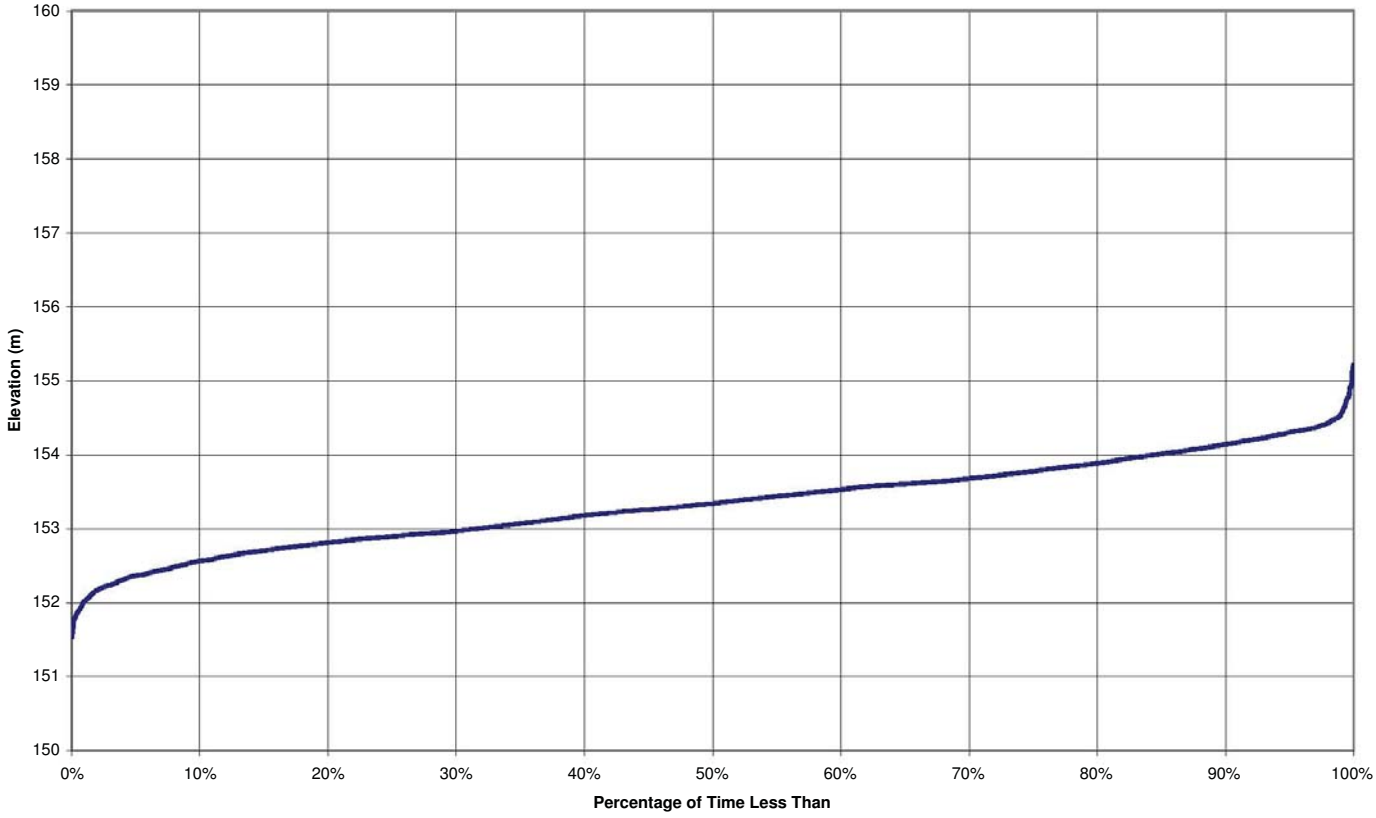


Note: Curve based on winter water level modeling between November 1 and May 1

Figure 29
Nelson River above Gull Rapids - Existing Environment Winter Stage Duration Curve



Note: Curve based on winter water level modeling between November 1 and May 1

Figure 30
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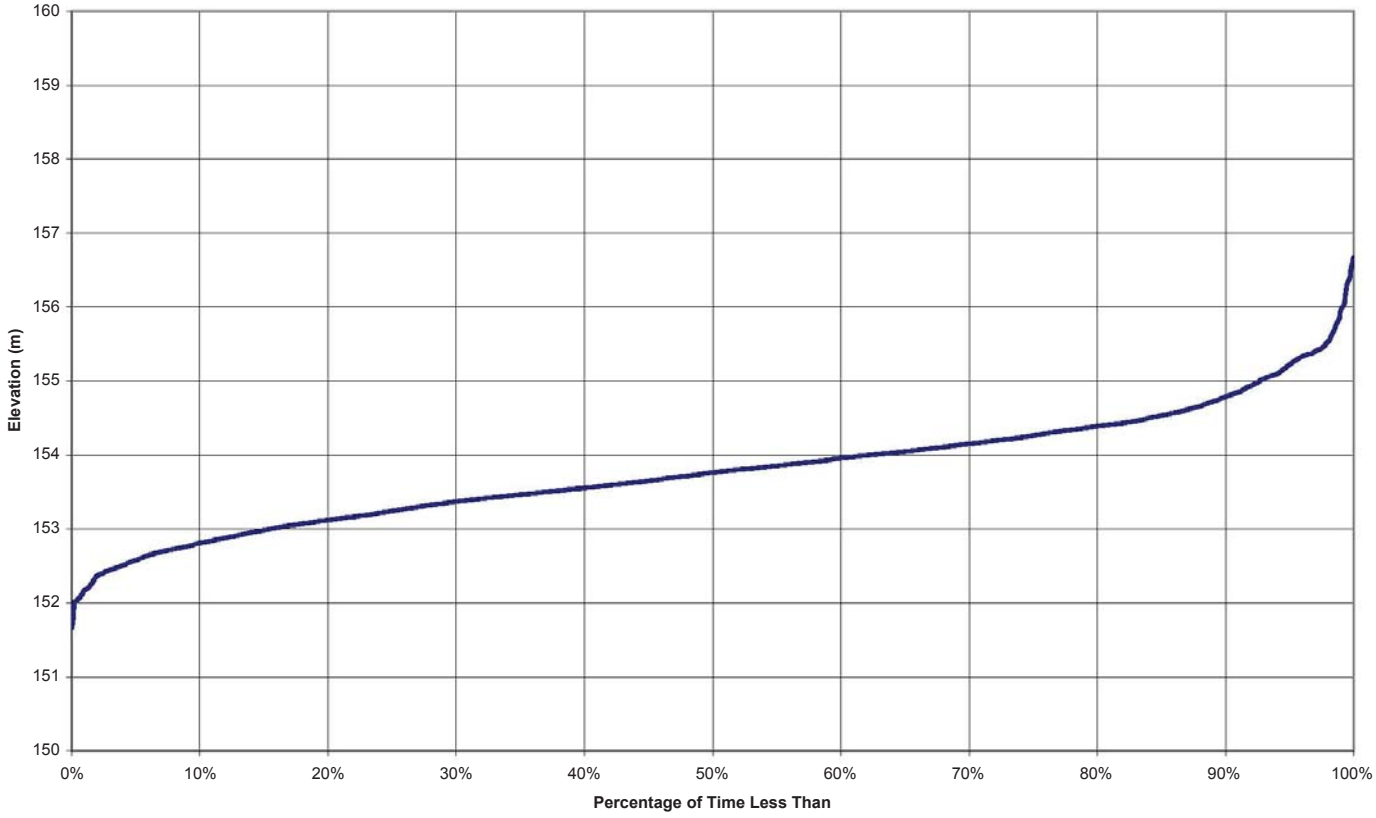

Note: Curve based on winter water level modeling between November 1 and May 1

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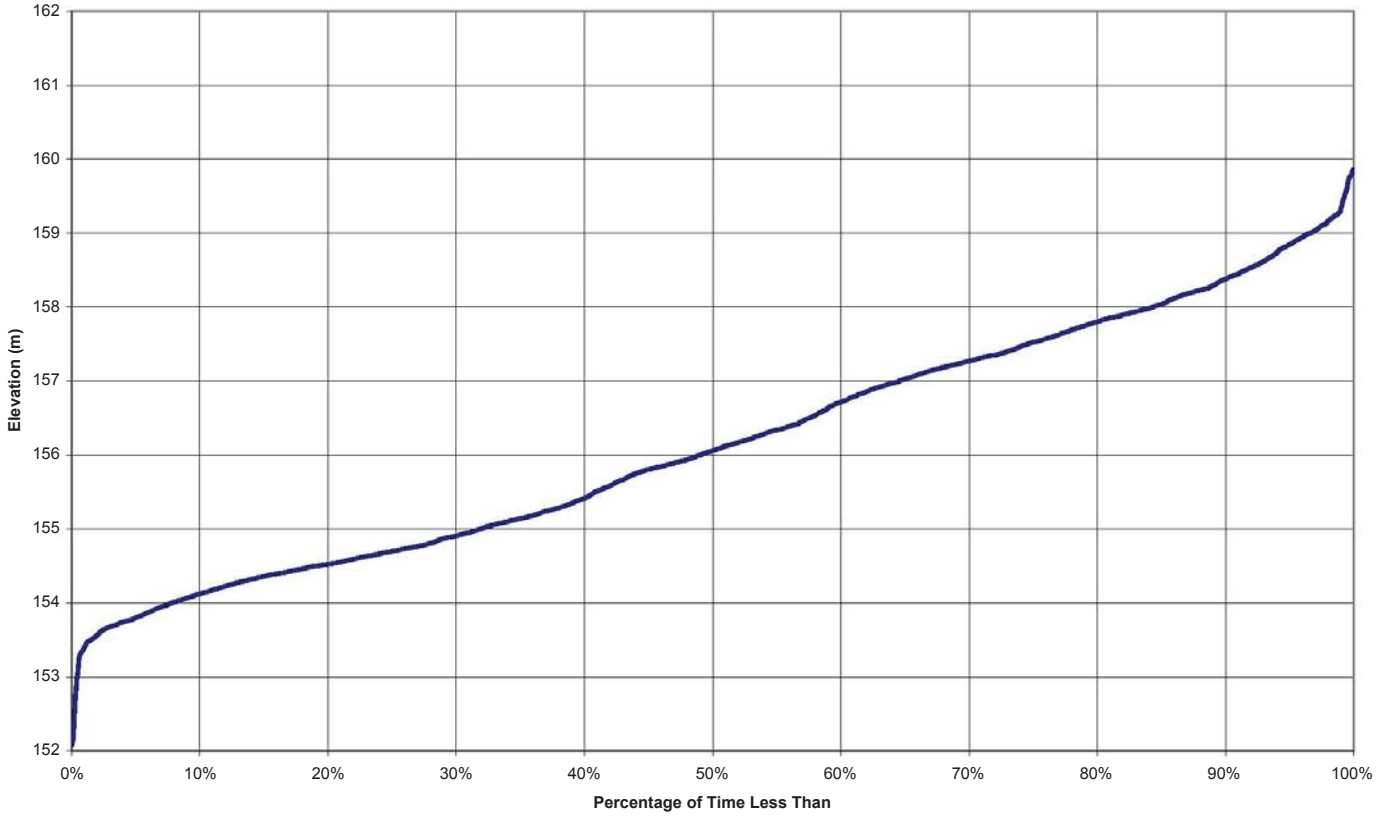

Note: Curve based on winter water level modeling between November 1 and May 1

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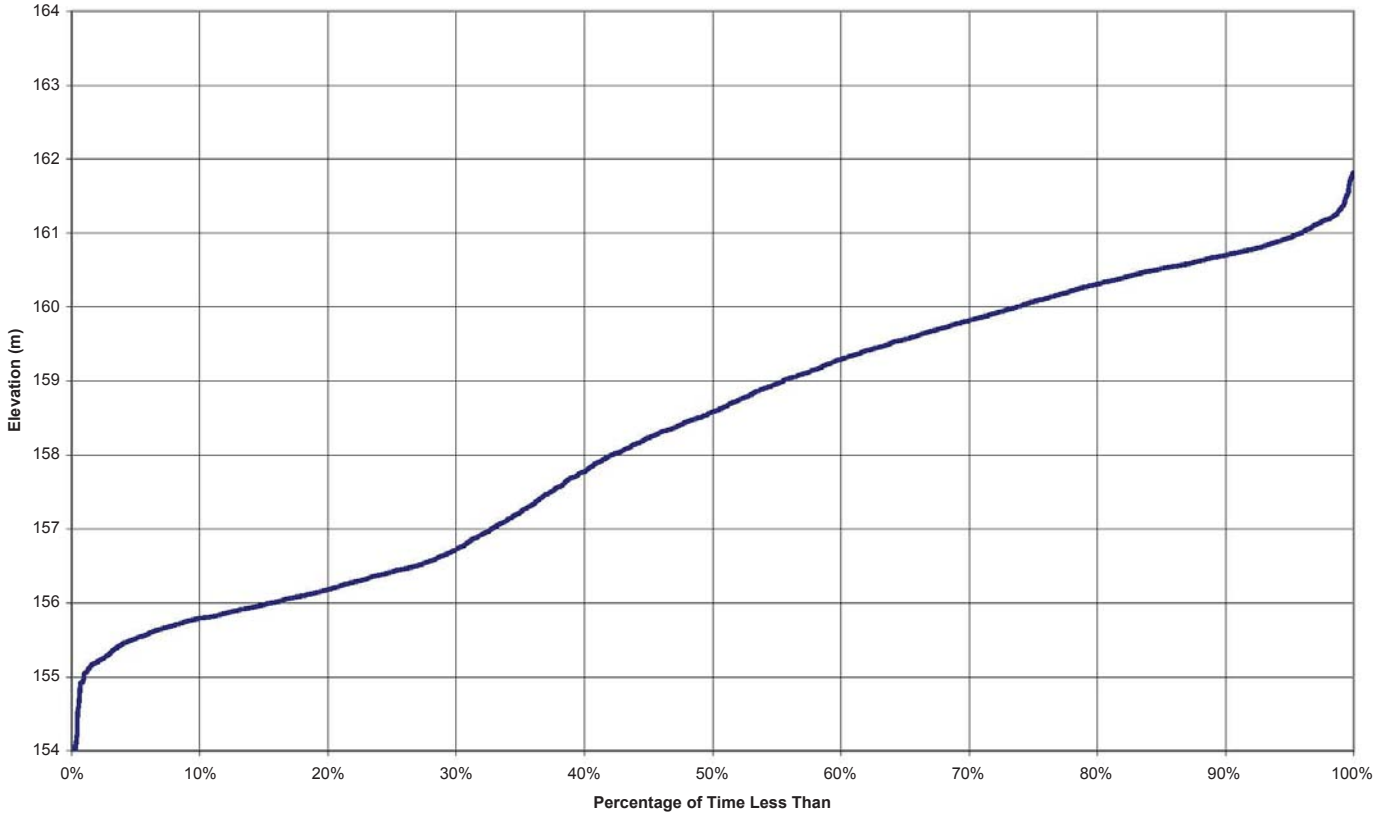

Note: Curve based on winter water level modeling between November 1 and May 1

Figure 33
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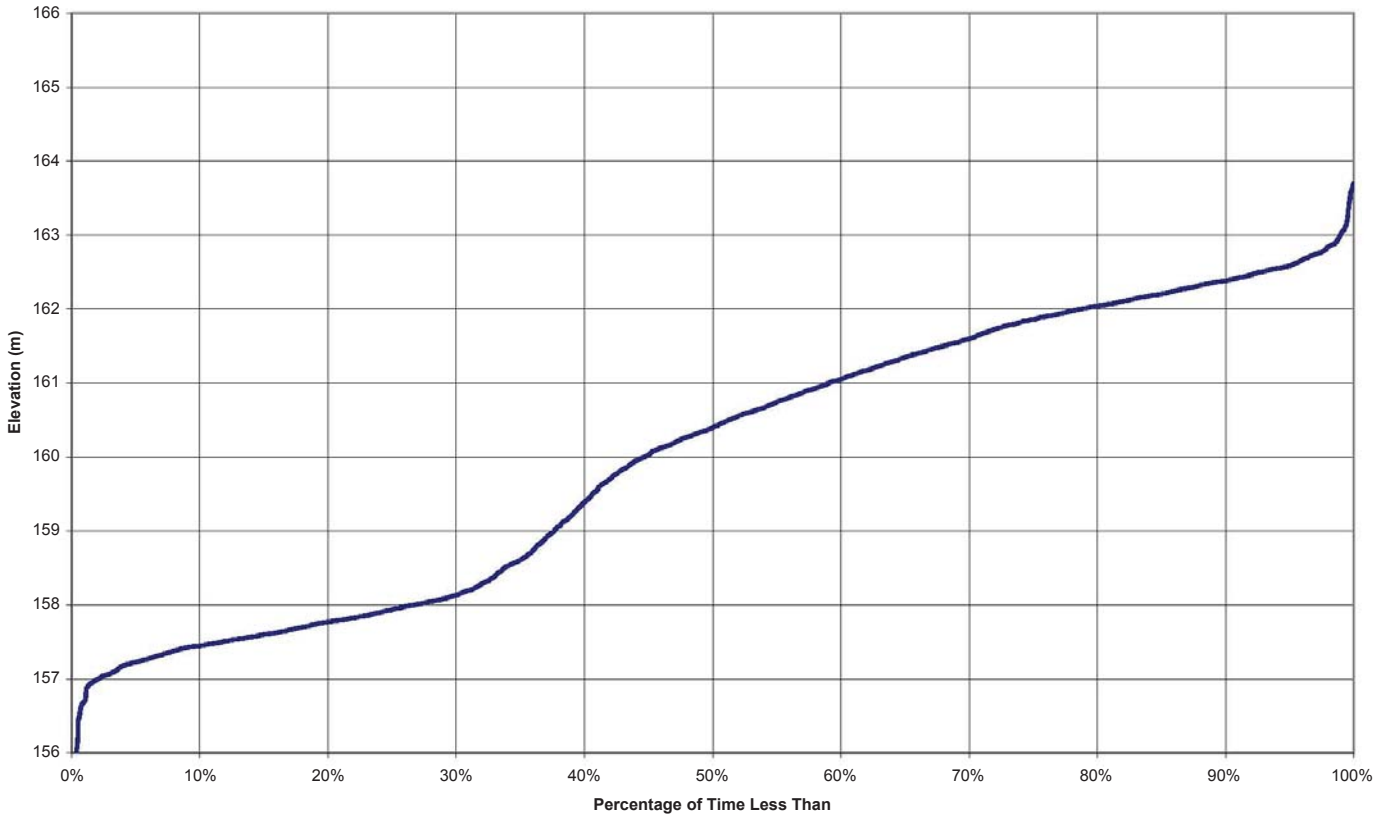

Note: Curve based on winter water level modeling between November 1 and May 1

Figure 34
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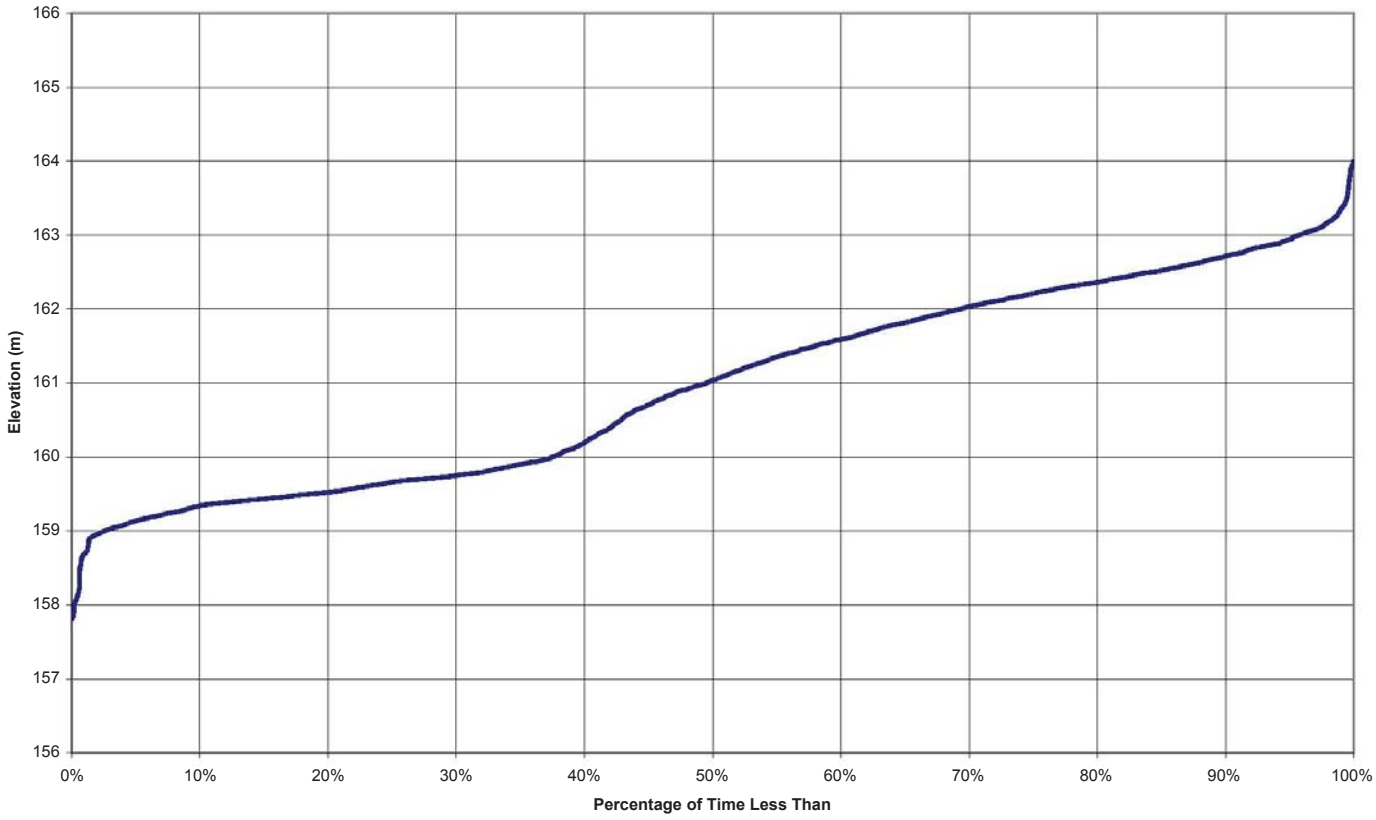

Note: Curve based on winter water level modeling between November 1 and May 1

Figure 35
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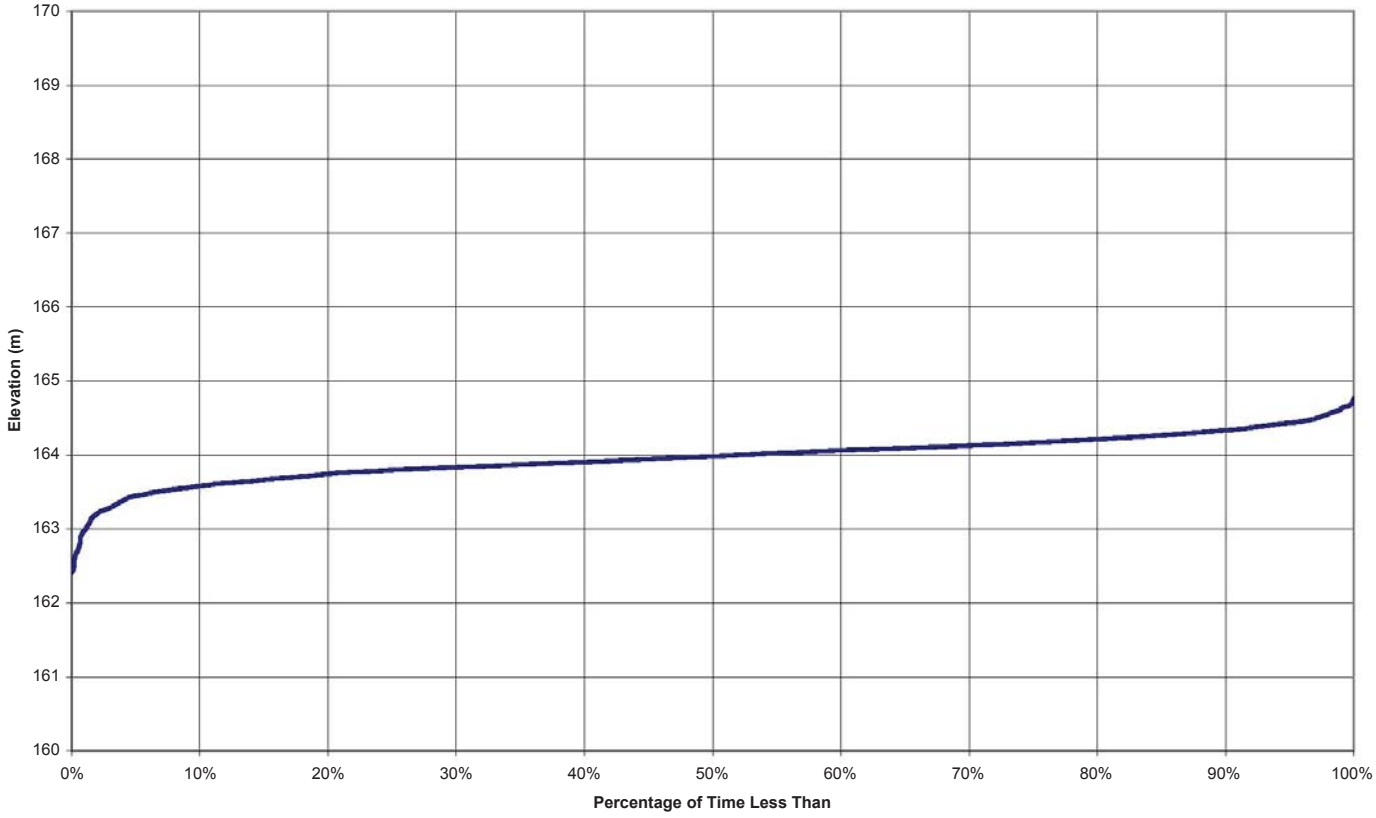

Note: Curve based on winter water level modeling between November 1 and May 1

Figure 36
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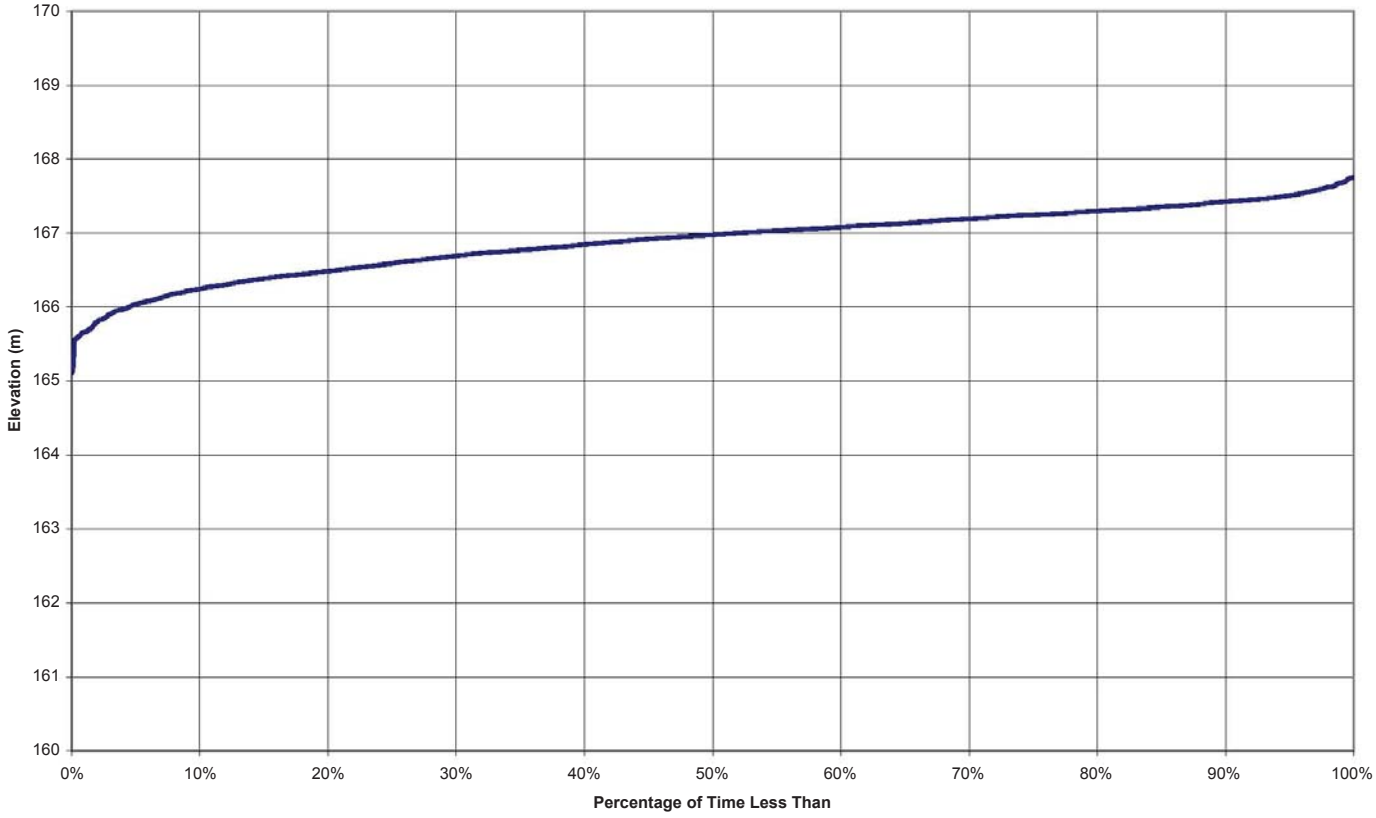

Note: Curve based on winter water level modeling between November 1 and May 1

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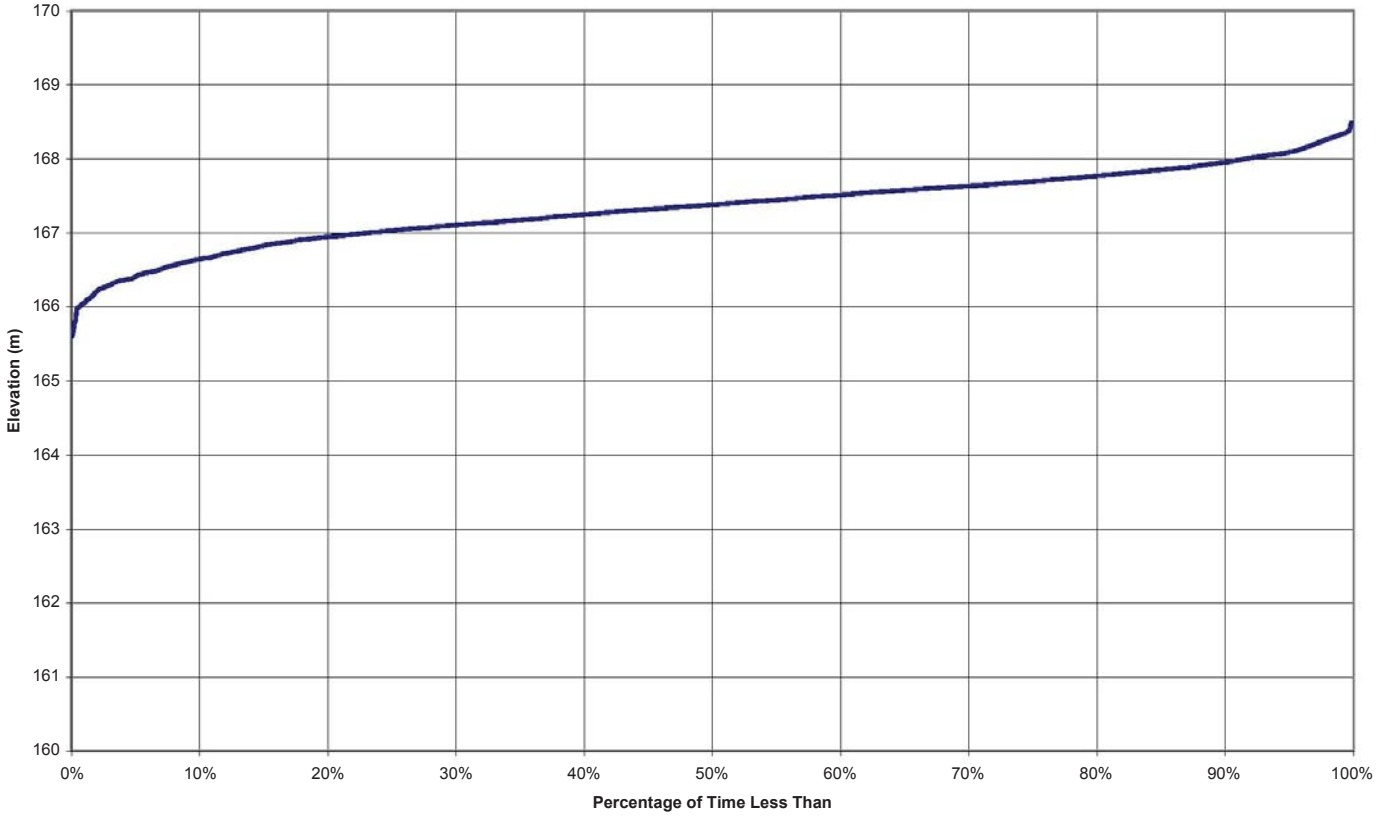

Note: Curve based on measured winter water levels between November 1 and May 1

Figure 38
Nelson River above Gull Rapids - Modeled Winter Stage Hydrographs
Future Environment without Keeyask G.S. - 5%tile Flow

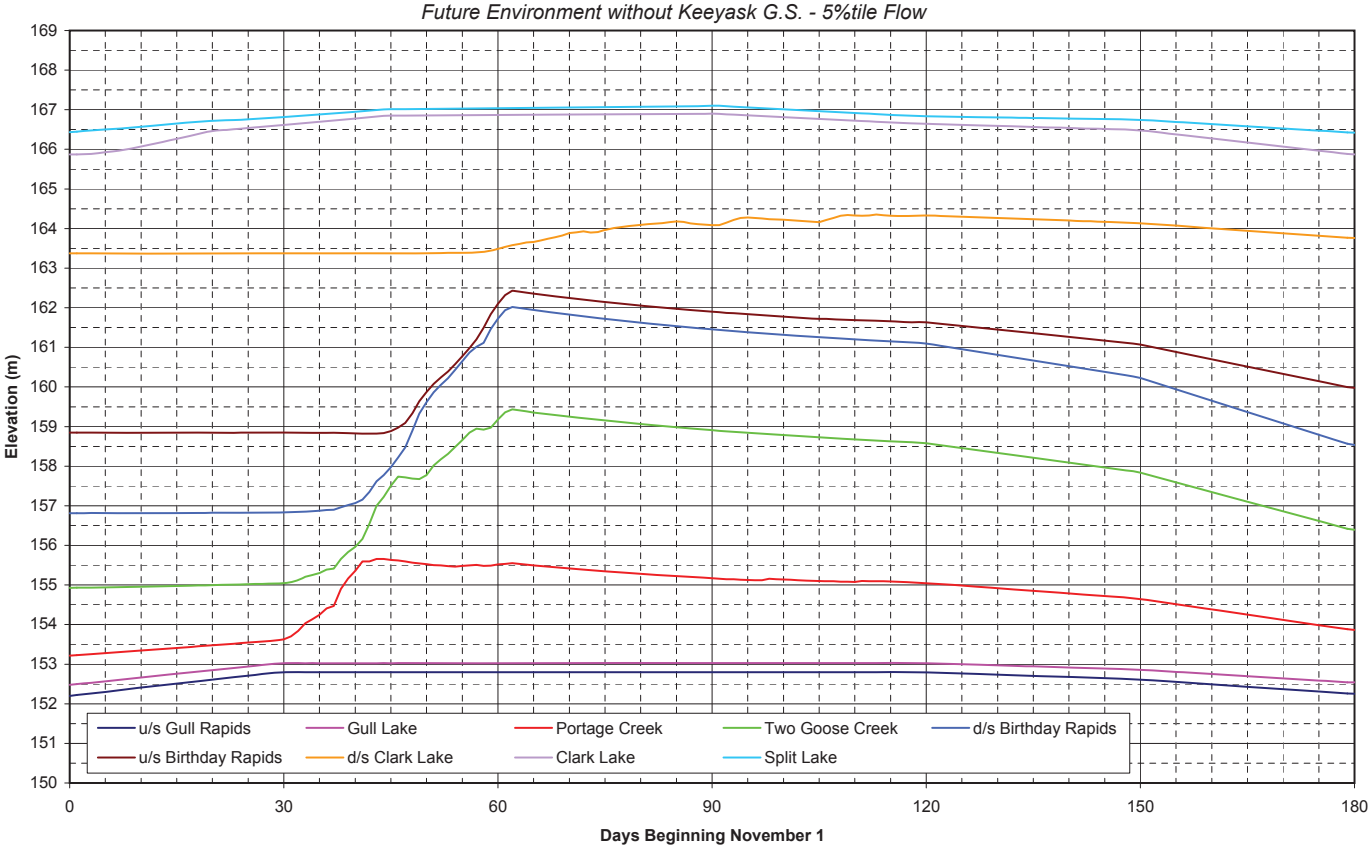

Figure 39
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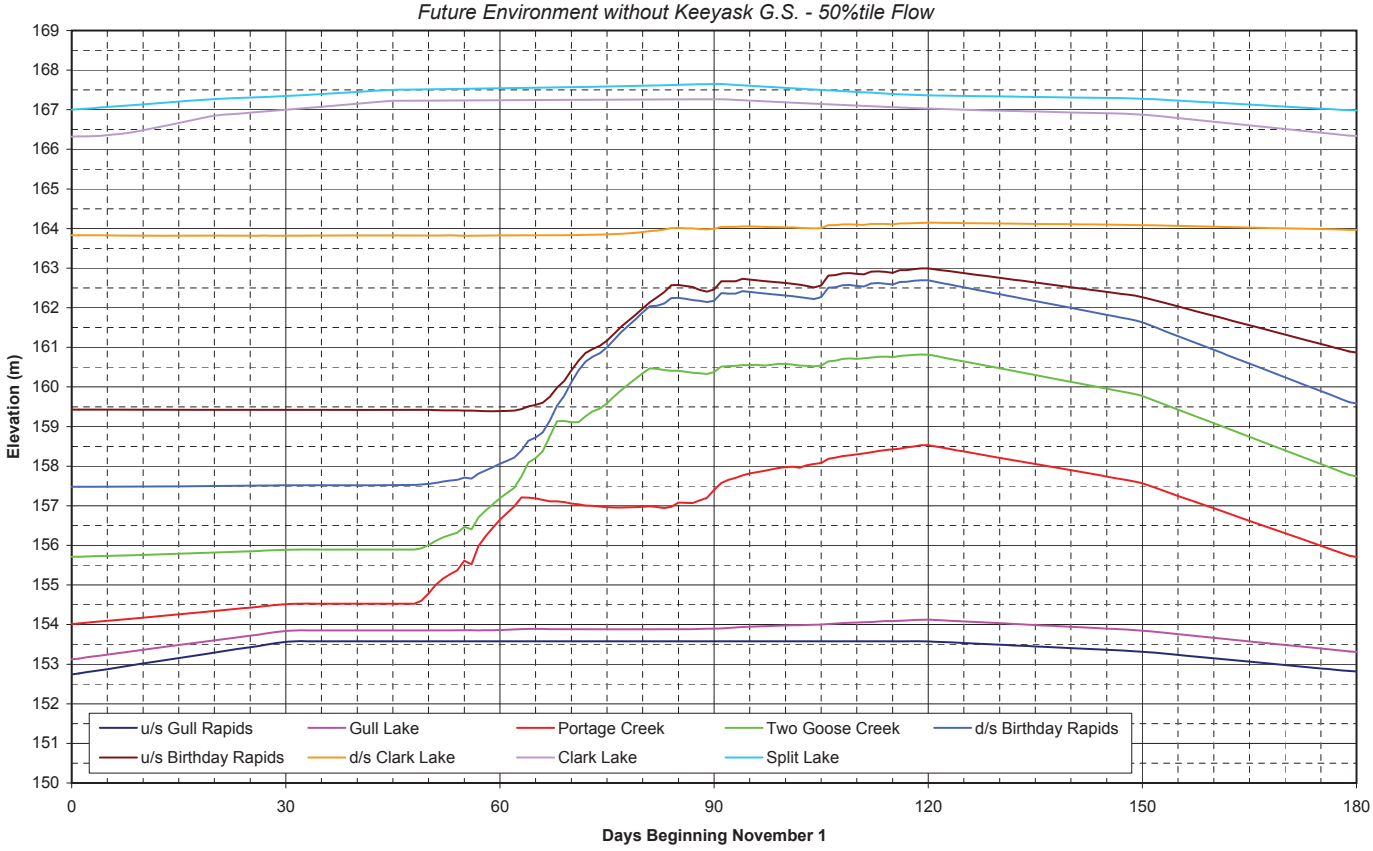

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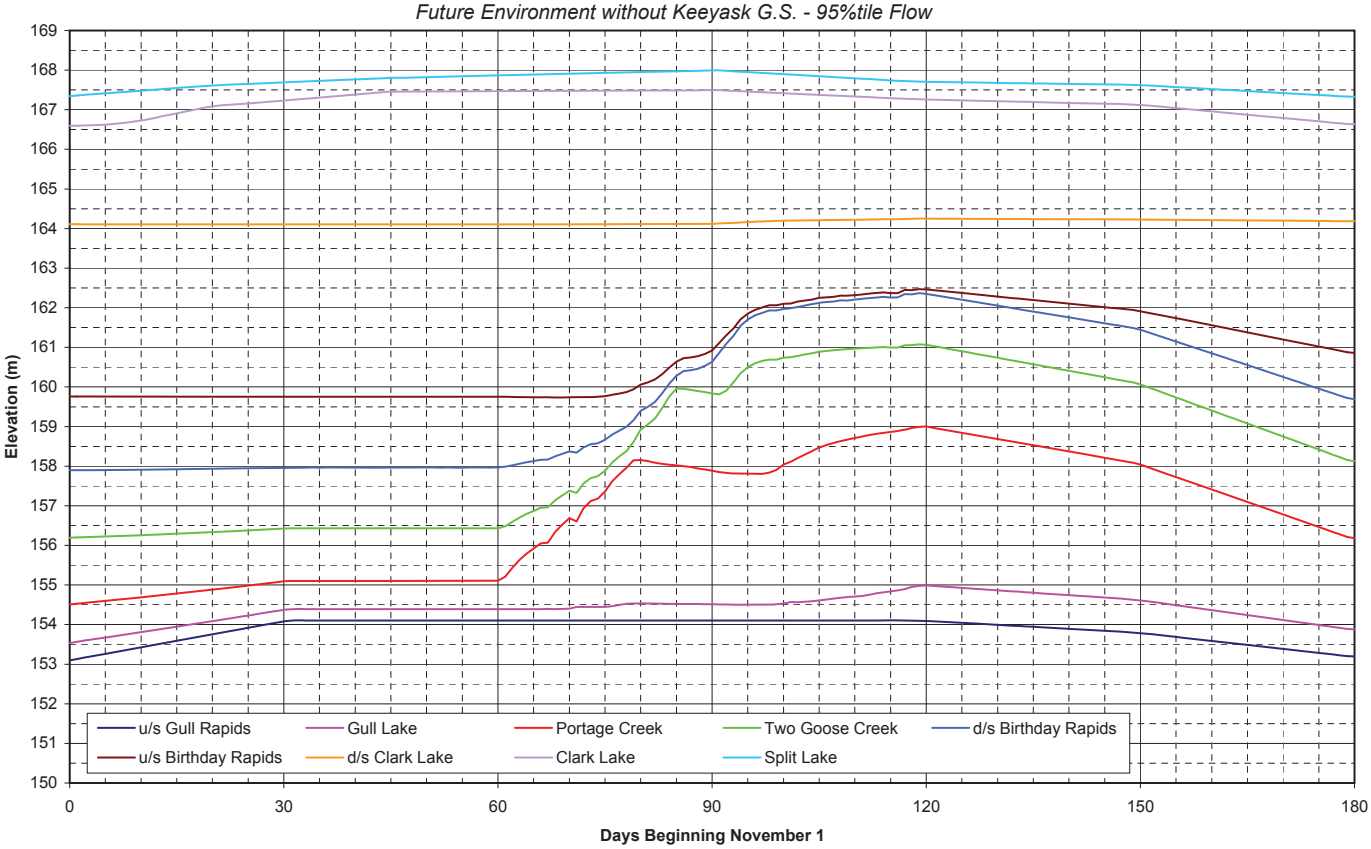

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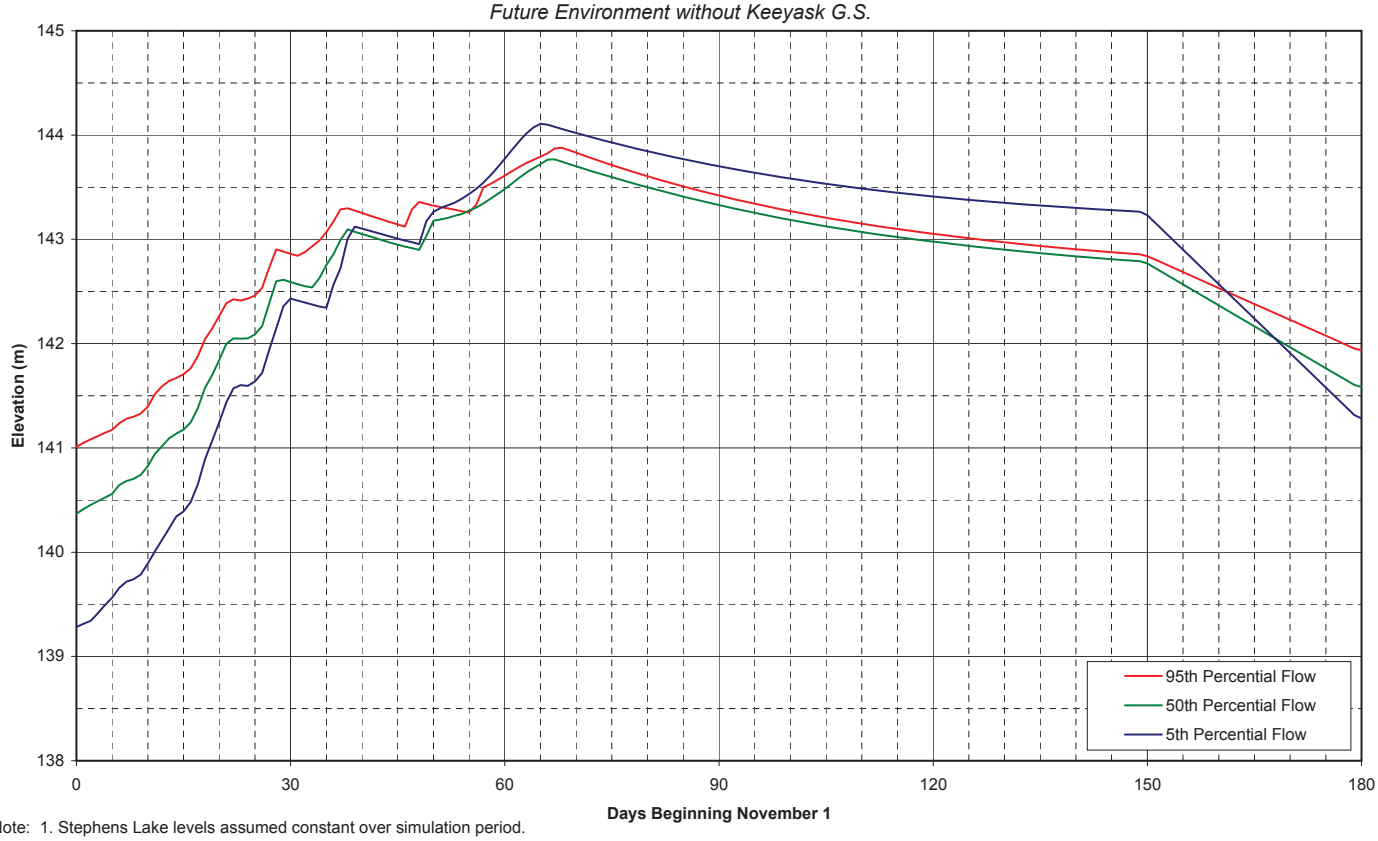

Note: 1. Stephens Lake levels assumed constant over simulation period.
2. Percentile level of Stephens Lake assumed to correspond to flow percentile under consideration (ie. 50th percentile level occurs with 50th percentile flow)

APPENDIX A - ABBREVIATIONS

ABBREVIATIONS

˚C .....degrees Celsius

D/S .....Downstream

FSL.....full supply level

ft .....feet

ft^3/s .....cubic feet per second

G.S. .....generating station

km .....kilometre

m .....metre

m^3.....cubic metre

m^3/s or cms.....cubic metres per second

U/S .....Upstream

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - PHYSICAL ENVIRONMENT

ESTIMATE OF SHORELINE EROSION DURING CONSTRUCTION

DRAFT 2

DELIVERABLE GN 9.2.10

MANITOBA HYDRO FILE: 00195-11100-0161_01

2009/07/10

DRAFT

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

KGS ACRES

KEEYASK
GENERATION PROJECT

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1 INTRODUCTION

The site of the proposed Keeyask Generating Station (Keeyask GS) is approximately 730 km north of Winnipeg at the head of Stephens Lake on the Nelson River, approximately 50 km downstream of Split Lake and 30 km west of Gillam (Figure 1).

The location of the permanent works (Powerhouse, Spillway, earthworks, and roadways), as well as the locations of the temporary works necessary for construction of the permanent works, are shown in Figure 2. The layout corresponds to that proposed for Axis GR-4, with a Full Supply Level (FSL) at el 159.0 m. The selected arrangement consists of two long river bank dykes, on the north and on the south, linked to the Principal Structures which span the Nelson River at Gull Rapids.

As a consequence of the construction activities involved in river management, water levels will increase in the vicinity of the construction area as well as in the forebay area, causing shoreline materials to be wetted that would otherwise not be. This may result in erosion along the shorelines where water velocities and riverbed shear stresses are significant.

This memorandum presents the extent of the study area, the data sources used, overview of sediment transport equations, results of studies aimed at predicting the locations where shoreline erosion is likely to occur, and provides an estimate of the quantity of mineral soils that may erode during the course of construction. If deemed necessary, these areas will be protected to reduce the erosion.

2 STUDY AREA

The shoreline erosion during construction study area consists of the riverbed, shore zones, and overland flow areas within the hydraulic zone of influence of the proposed Keeyask Generating Station from the upstream end of Gull Lake to the upstream end of Stephens Lake (Figure 1).

3 DATA SOURCES

The following data sources have been used in the present study:

- Multi-season field observations and photographs from ECOSTEM Ltd., [Ref 1]
- Shore zone classification data [Ref 2],
- Existing environment mineral erosion [Ref 3]
- Existing environment sediment data [Ref 4],

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- Existing and project environment flows [Ref 5],
- River management during construction [Ref 6],
- Cofferdam arrangement and design [Ref 7].

4 APPROACH AND METHODOLOGY

The approach taken in this study was to estimate erosion during construction based on selected sediment transport equations with hydraulic conditions in the river based on one-dimensional backwater models. The details of the hydraulic models and utilization of the sediment transport equations are presented in Sections 8 and 9, with the results of the riverine erosion model presented and discussed in Section 10. Background information provided in Sections 5, 6, and 7 is as follows:

- Section 5: the existing riverbed and shoreline terrestrial environment and existing flow conditions in the study area are briefly summarized
- Section 6: the method of river management and resulting changes in water levels during construction are presented
- Section 7: erosion mechanisms and scour criteria for both cohesionless and cohesive materials are presented.

5 EXISTING ENVIRONMENT

5.1 SITE CONDITIONS

5.1.1 GENERAL

The site for the Keeyask GS is contained within the Canadian Shield and is underlain by variable thicknesses of up to 30 m of overburden over competent Precambrian bedrock. In general, the overburden stratigraphy consists of a thin organic cover on postglacial lacustrine clay which overlies deposits of glacial outwash, till or the bedrock directly. Preglacial deposits of sand and silty sand are also occasionally found in bedrock lows. All or some of these deposits are exposed on the riverbanks/riverbed at various locations in the study area.

5.1.1.1 Postglacial Deposits

Two types of postglacial deposits have been identified: (i) Lake Agassiz silts and clays and (ii) alluvium.

Lake Agassiz silts and clays are encountered over most of the study area.

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The silts and clays form a veneer of up to several metres in thickness over the glacial deposits. These fine-grained deposits are commonly varved and tend to be of greater thickness in the topographic lows.

Alluvium generally consists of cobbles and boulders overlying sands and gravels and is locally present in the base of present-day stream and river channels.

5.1.1.2 Glacial Deposits

The glacial deposits are widespread and consist of materials deposited by several glacial ice sheets which advanced over the Gull Rapids area and deposited till and stratified water lain deposits. The tills contain discontinuous occurrences of permafrost are generally well graded, compact, have a relatively low moisture content, and generally have a low ice content when frozen.

Three separate till or till-like horizons have been identified at the Keeyask site. The upper silty sand/sandy silt till unit (Till 1), whose presence is the most widespread over the Keeyask area, generally consists of a light brown horizon (Till 1a) overlying a grey horizon (Till 1b) with essentially identical soil gradations. Beneath the silty sand/sandy silt till units, Till 2 and Till 3 consist of grey, low plasticity clays. Not all of the area at Keeyask GS site encountered these till units. The till units may be separated by discontinuous intertill units, especially in areas of bedrock lows or in drumlin features.

5.1.2 RIVERBED AND RIVER BANK MATERIALS

The riverbed and river bank materials of the Nelson River in the reach under investigation include material ranging from clay size to cobble size, and includes both cohesive and granular cohesionless material, as well as peat (in the upland zones of the study area).

Information on the river bank and upland area materials is available from the various surface mapping and geotechnical investigations that have been conducted for the Keeyask GS. Information on the material in the riverbed is limited to a few locations. The grain size distribution curves for the materials in the main stratigraphic units are presented in Figure 3 and are based on available information.

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5.2 NELSON RIVER FLOWS AND WATER LEVELS AT GULL RAPIDS

5.2.1 OBSERVED FLOWS

At Gull Rapids, the Nelson River is divided into four channels:

(i) the south channel, (ii) the middle channel, (iii) the north channel; and (iv) as small cross-flow channel between the middle and south channels. Approximately 76% of the total river discharge presently flows through the south channel.

Flows on the Nelson River downstream of Split Lake show a large seasonal variation, generally with high flows occurring during the summer months and low flows during the winter months. Average daily flows of the Nelson River at the outlet of Split Lake recorded since 1977 vary from a minimum of approximately 1330 m^3/s in October, 2003 up to a maximum of approximately 6,600 m^3/s in August and September, 2005 (Figure 4). The water elevation of Gull Lake (at Box Bay Creek and Broken Boat Creek) was between el 154.2 and 154.9 during the high flow period that occurred during the summer of 2005.

5.2.2 STUDY DESIGN FLOW

A design flow of 4855 m^3/s has been adopted for use in this study. This is the 95^{th} percentile all-season daily flow.

5.2.3 COMPARISON OF OBSERVED AND DESIGN FLOWS

Over the past 30 years, the Nelson River at Gull Rapids has experienced flows greater than the 95^{th} study design flow condition during six years (Figure 4). These high flows have resulted in the erosion of the finer materials from the riverbed and shorelines over large areas of Gull Rapids, which has resulted in a self-armouring of portions of the shorelines.

6 RIVER MANAGEMENT DURING CONSTRUCTION

6.1 STAGE I RIVER DIVERSION

6.1.1 GENERAL

Stage I Diversion involves blocking off the north and middle channels of the Nelson River. The arrangement of the Stage I Diversion Cofferdams will direct the entire flow of the Nelson River to the southern channel of the river (Figure 5). This will permit the undertaking of excavation works for the Principal Structures, as well as construction of the Spillway, Powerhouse and associated channels. The cofferdam designs and construction methods will incorporate measures to minimize the erosion of materials during their construction.

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The Stage I earthwork structures consist of six cofferdams and one rockfill groin located at the head of the river's northern channel. During the first year of construction, a U-shaped cofferdam will enclose the construction area for the Spillway and its associated Approach and Discharge Channels, which are located on the island adjacent to the north bank of the south channel. An L-shaped Powerhouse Cofferdam, in combination with the Central Dam and North Channel Cofferdams, when constructed, will allow the construction of the Central Dam, Powerhouse and Intake Channel and the upstream portion of the Tailrace Channel. A Tailrace summer level cofferdam will be constructed downstream of the Powerhouse Stage I Cofferdam to permit excavation of the downstream portion of the tailrace channel. Details of the cofferdam construction are provided in Memorandum GN-4.2.2.11 [Ref 7].

6.1.2 WATER LEVELS DURING STAGE I DIVERSION

Stage I Diversion will last approximately 3 1/2 years, during which the Nelson River will be diverted such that all flow passes through the South Channel at Gull Rapids. As a result of this diversion of the river, the water levels upstream of the location of the Spillway will increase above the levels that would occur under existing conditions, as shown in Table 2.

Table 2
Keeyask GS - Stage IV Studies, Axis GR-4
Stage I Diversion - Water Level Increases

Location	H01 Model Section Name	Water Elevation (m)[(1)]	
		Existing Conditions	Stage I Diversion
Immediately Upstream of the Spillway Stage I Cofferdam	GR362752	146.2	147.3
Downstream end of Gull Lake	GL359974	153.5	154.4
Upstream end of Gull Lake	GL345223	154.0	154.7

Note:
(1) Water elevations are given for the 95th percentile flow of 4,855 m^3/s.

6.2 STAGE II RIVER DIVERSION AND RESERVOIR IMPOUNDMENT

6.2.1 GENERAL

Stage II of the diversion works will close off the south river channel of the Nelson River and direct the flow into the open sluiceways within the Spillway (Figure 6). Minimal riverbed scour is expected in the gap between the closure leg and the river's south bank since high velocities and ice have been assessed to have already removed the majority of the overburden in this area and left the bed as being bedrock controlled. In addition, bedrock outcrop is present on the south shore to elevations above that at which closure will take place.

Stage II Diversion will allow the construction of the South Dam and access to the south shore. The Stage II Diversion works will close the south river channel and direct flow into the Spillway, which will consist of seven sluiceways during the initial portion of the Stage II Diversion, in order to facilitate closure of the south channel of the river and to minimize upstream water surface elevations. During the latter portion of Stage II Diversion, the Spillway rollways will be constructed in a sequential manner resulting in progressive increases in the forebay water levels.

The sequence of concreting of Spillway rollways is discussed in detail in Memorandum GN-4.3.22 [Ref 8]. The construction of the rollways is scheduled to take place after substantial construction of the South Dam, during Year Y, the year in which power is first generated. Rising forebay water levels will be monitored so that they do not place the South Dam at risk of overtopping.

6.2.2 WATER LEVELS DURING STAGE II DIVERSION AND ROLLWAY CONSTRUCTION

Stage II Diversion and reservoir impoundment will occur over a period of approximately one year, during which the water levels upstream of the Spillway will be increased in stages. Each stage in increasing water level will generally last for the period of one month. The allowable rate of water level rise on the forebay will be limited by embankment stability and performance monitoring considerations. The estimated water levels during Stage II Diversion and rollway construction, including reservoir impoundment, are presented in Memorandum GN-4.3.2.2 [Ref 8].

Water levels upstream of Gull Rapids will not be significantly affected by impoundment until September 1, Year Y, when the level will rise to over 1 m above the water level that would occur under the existing environment for a flow of 4,855 m^3/s.

7 EROSION MECHANISMS AND SCOUR CRITERIA

The riverbed and riverbank materials of the Nelson River in the reach under investigation include material ranging from clay size to cobble size, and include both cohesive and granular cohesionless materials. The erosion mechanisms of cohesionless and cohesive materials are very different. The erosion mechanism of cohesionless granular material is better understood and documented than that of cohesive material. The following sub-sections discuss the erosion potential of granular and cohesive materials.

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7.1 EROSION OF COHESIONLESS GRANULAR MATERIAL

Cohesionless granular material gets displaced when the applied shear stress is larger than the critical shear stress of the material. Initiation of motion of granular material occurs for each individual particle that is subjected to the shear stress excess. The critical shear stress for granular material has been studied by many investigators, with a wide range of values, as shown in Figure 7. The critical shear stress tends to decrease as particle size decreases, however, for very small particles with diameters less than 0.1 mm, the critical shear stress tends to increase with decreasing particle size.

7.2 EROSION OF COHESIVE MATERIAL

Cohesive material is finer grained than granular material. Shear stress over cohesive material may generate three modes of erosion. They are Surface Erosion, Mass Erosion and Entrainment of Fluid Mud. Hydrodynamic lift and drag forces break up inter particle bonds of cohesive material and cause 'Surface Erosion' at lower bed shear stress. This mode of erosion includes removal of individual particles, flocs, or aggregates from the surface of the bed. Sediments erode in blocks and are brought in suspension in 'Mass Erosion'. The third mode of erosion is entrainment of fluid mud in the order of decreasing shear stress. The complex erosion behaviour of cohesive materials depend upon a wide range of factors, including sediment composition, microscopic and macroscopic sediment properties, moisture content, pore pressure, consolidation pressure, pH and temperature.

The particle erosion rate of cohesive soils can be estimated using either Parthenaide's equation, which relates erosion rate to the excess stress ratio:

$E = dm/dt = M_p * (\tau_c - \tau) / \tau_c$

Where:

E = mass erosion flux ($kg/m^2/s$)
m = mass of sediment on the bed (kg/m^3)
t = time (seconds)
τ = bed shear stress (Pa)
τ_c = critical shear stress for erosion (Pa)
M_p = 'Parthenaides Coefficient', erosion rate at a bed shear stress of $2*\tau_c$ ($kg/m^2/s$)

Or an equation of the form:

$Q_u = M_p (\tau - \tau_c)^{\alpha}$

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Which relates erosion rate to the applied excess stress. If it is assumed that the particle erosion rate varies linearly with shear stress, then $\alpha = 1$ and the equation becomes the equation used by Hanson [Ref 14], namely:

$E = k_d (\tau - \tau_c)$

Where

k_d = soil erodibility (cm^3/N-s)
τ = bed shear stress (Pa)
τ_c = critical shear stress for erosion (Pa)

The critical shear stress, 'Parthenaides Coefficient' or the soil erodibility (k_d) have to be determined experimentally, using the combination of sediment and water under consideration. Based on the literature, values of 'Parthenaides Coefficient' generally range from 10^{-3} to 10^{-5} kg/m^2/s (i.e. by a factor of 100) (Winterwerp, 1989 as reported in [Ref 29]), and values of critical shear stress can vary widely, from less than 1 Pa to over 40 Pa.

The erosion properties for various materials as determined by various investigators are summarized in Table 3.

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Table 3 Erosion Properties Determined by Various Investigators using Various Methods and Soil Types [Ref 24]

Table 4.6. Erosion properties determined by several investigators using various methods and soil types

Investigator	Soil type	Flume type	Varied parameter	τ^{c}_{se} [Pa]	τ [Pa]	Q_{se} [kg/m^{2}s]	M_{se} [kg/m^{2}s]	$M_{se,ex}$ [kg/m^{2}s]	a [-]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Arulanandan et al. (1975)[a]	Yolo loam @ 0.1N salt concentration	Rotating cylinder	Sodium Adsorption Ratio SAR = (12.4)	0.5	0.7 - 1.9	1.8×10^{-3} - 14×10^{-3}	0.0054	--------	----- --
			SAR = (9.4)	1.3	1.6 - 3.6	0.7×10^{-3} - 3.8×10^{-3}	0.0020	-------	-----
			SAR = (1.4)	3.8	5.6 - 6.7	0.9×10^{-3} - 2.5×10^{-3}	0.0055	-------	-----
		Rotating cylinder	SAR = (10.7)	0.05	0.4 - 1.2	5.0×10^{-3} - 15×10^{-3}	0.0009	-------	-----
			SAR = (3.0)	0.2	0.5 - 1.3	3.3×10^{-3} - 8.3×10^{-3}	0.0010	-------	-----
			SAR = (1.6)	0.5	1.0 - 2.1	1.2×10^{-3} - 3.5×10^{-3}	0.0015	-------	-----
Ariathurai and Arulanandan (1978)[a]	30% Illite (remaining material not provided in text)	Rotating cylinder	Temperature = (42°C)	1.2	1.3 - 2.4	0.4×10^{-3} - 4.0×10^{-3}	0.0040	-------	-----
			(23°C)	2.2	2.6 - 4.8	0.5×10^{-3} - 2.8×10^{-3}	0.0023	-------	-----
			(18°C)	2.4	2.6 - 6.0	0.2×10^{-3} - 2.8×10^{-3}	0.0019	-------	-----
			(9.5°C)	2.6	2.6 - 6.0	0.03×10^{-3} - 1.6×10^{-3}	0.0012	-------	-----
Partheniades (1965)[b]	S. F. Bay mud in water at ocean salinity	Straight flume	Placed bed	0.057	.011 - 0.93	0.15×10^{-6} - 5.0×10^{-6}	1.3×10^{-7}	-------	-----
			Flocculated bed	0.057	0.15 - 0.44	0.12×10^{-6} - 1.5×10^{-6}	2.7×10^{-7}	-------	-----
Nachtergale and Poesen (2002)[a,c]	Loess - undisturbed cylinder samples from crop land tested in fresh water	Straight flume	A Horizon	0.57	1.65 - 3.96	1.9×10^{-2} - 15×10^{-2}	0.0299	-------	-----
			B Horizon	0.66	1.65 - 3.96	3×10^{-2} - 4×10^{-2}	f	-------	-----
			C Horizon	1.65	1.65 - 3.96	1×10^{-2} - 41×10^{-2}	0.2827	-------	-----
Dennet et al. (1998)[d]	Kaolinite flocculated bed in fresh water	Straight flume	Natural Organic Matter (NOM) = 0.00%	0.77	0.96 - 1.82	3.7×10^{-3} - 22×10^{-3}	0.0155	-------	----- --
			NOM = 0.002%	0.54	0.97 - 1.67	9.4×10^{-3} - 26×10^{-3}	0.0127	-------	-----
			NOM = 0.006%	0.39	0.97 - 1.67	18×10^{-3} - 40×10^{-3}	0.0121	-------	-----
			NOM = 0.012%	0.36	0.97 - 1.67	19×10^{-3} - 37×10^{-3}	0.0107	-------	-----

Table 3 Erosion Properties Determined by Various Investigators using Various Methods and Soil Types

Table 4.6. Erosion properties determined by several investigators using various methods and soil types

Investigator	Soil type	Flume type	Varied parameter	τ^c_{se} [Pa]	τ [Pa]	Q_{se} [kg/m^{2}s]	M_{se} [kg/m^{2}s]	$M_{se,ex}$ [kg/m^{2}s]	a [-]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Ravisangar et al. (2001)[a]	Kaolinite flocculated bed in fresh water	Straight flume	pH of pore and eroding fluid = 3.5	0.91	0.97 - 1.67	1.3×10^{-3} - 12×10^{-3}	0.0151	-------	-------
			pH = 4.0	0.91	0.97 - 1.67	1.9×10^{-3} - 16×10^{-3}	0.0186	-------	-------
			pH = 5.0	0.88	0.97 - 1.67	5.5×10^{-3} - 30×10^{-3}	0.0279	-------	-------
			pH = 6.0	0.12	0.97 - 1.67	14×10^{-3} - 25×10^{-3}	0.0021	-------	-------
			pH = 7.0	-----	0.97 - 1.5	6.0×10^{-3} - 9.2×10^{-3}	f	-------	-------
			pH = 8.0	-----	0.97 - 1.5	2.8×10^{-3} - 3.9×10^{-3}	f	-------	-------
Westrich et al. (1997)	Undisturbed reservoir mud in fresh water	Straight flume	Upper-most horizon	0.74	0.77 - 1.76	0.16×10^{-4} - 4.0×10^{-4}	0.00031	-------	-------
			Second horizon	0.66	0.69 - 1.68	0.77×10^{-4} - 14.4×10^{-4}	0.00087	-------	-------
Johansen et al. (1997)[d,e]	Reconstructed bay (Wadden Sea) mud in water at ocean salinity	Annular flume	Consolidation time = 12 hrs, depth = 0-0.75 cm	0.055-0.20	0.088 - 0.25	7.9×10^{-6} - 64×10^{-6}	-------	0.6776[g]	3.1891[g]
			Consolidation time = 24 hrs, depth = 0-0.81 cm	0.055 - 0.25	0.088 - 0.31	4.6×10^{-6} - 53×10^{-6}	-------	0.3753[g]	3.1415[g]
			Consolidation time = 48 hrs, depth = 0-0.29 cm	0.084 - 0.37	0.11 - 0.44	1.6×10^{-6} - 44×10^{-6}	-------	2.8612[g]	4.1504[g]
Ravens and Gschwend (1999)[a,g]	Boston Harbor mud, in-situ test, 12% clay, 51% silt, saline environment	In-situ flume	Applied shear stress	0.12	0.15-0.49	0.13×10^{-3} - 1.3×10^{-3}	-------	0.0032	1

a. τ^c_{se} was determined with x-intercept of shear stress vs. E curve.

b. Both initial and final erosion rates were reported in the literature however only final erosion rates are reported in this table.

c. Five soil horizons were tested using soil sampled with cylinders from agricultural lands, only results from the top horizon are reported in this table.

d. Applied shear stresses are not reported and are calculated from a best-fit equation and averaged over the three runs.

e. Critical shear stress calculated for several depths.

f. Erosion rate constant, *M* is indeterminate because the slope is near or less than 0.

g. A dimensionless form was not possible, shear stress values must be in Pa (N/m^2).

7.3 ESTIMATION OF CRITICAL SHEAR STRESS FOR COHESIVE MATERIALS

The cohesive materials that could erode due to changes in hydraulic conditions during Stage I and Stage II Diversion at Keeyask are the surficial Lake Agassiz silts and clays. The methods used to estimate the critical shear stress for the clays at Keeyask, as used in the analyses reported herein, were based on published experience and equations. The methodologies employed to estimate the possible range of critical shear stresses are discussed below, and the anticipated range in critical shear stress is shown in Table 4. The critical shear stress for the lacustrine clays has been estimated to be in the range of 0.9 Pa to 16 Pa, and has been assumed to be 3 Pa for the Base Case calculations in this study, which is within the lower range of critical shear stress that other studies have found to cause erosion of cohesive material.

Table 4
Keeyask GS - Stage IV Studies, Axis GR-4
Post-Glacial Clays - Estimated Critical Shear Stress

Description	Estimated Critical Shear Stress (Pa)
Published values	0.05 to 27
Julian & Torres	>16
Based on Voids Ratio	>5
USDA	3.2
Smerdon & Beasley	>1.8
Based on Bulk Density	0.9

7.3.1 CRITICAL SHEAR STRESS BASED ON PUBLISHED VALUES

The range of critical shear stress for erosion of cohesive material can be very wide. Winterwerp and van Kesteren (2004) suggested typical range of critical shear stress, according to previous studies, from 0.1 to 5 Pa. Winterwerp and van Kesteren (2004) also suggested that critical shear stress vary with depth, consolidation and physiochemical effects. Danish Hydraulic Institute (2004) used the range of critical shear stress from 0.05 to 2 Pa for cohesive material. Gaskin et. al. [Ref 26] found critical shear stresses for undisturbed samples from the banks of St. Lawrence River in Quebec to be in the range of 6 to 20 Pa. Laboratory tests conducted on riverbank clay of the Red River in Winnipeg by Fernardo [Ref 9] yielded critical shear stresses between 17 Pa and 27 Pa, depending on how the laboratory data was interpreted.

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7.3.2 CRITICAL SHEAR STRESS BASED ON VOIDS RATIO

Permissible bed shear stresses versus void ratio for material of varying plasticity (based on experience in the USSR), has been plotted by Chow [Ref 10] (see Figure 8).

7.3.3 CRITICAL SHEAR BASED ON BULK DENSITY

A relationship between the critical shear stress and bulk density of the bed material proposed by Hwang and Mehta [Ref 11] is as follows:

$\tau_c = a\ (\rho_b - r_l)^b + c$

Where

τ_c = critical shear stress (Pa)
a = 0.883
b = 0.2
c = 0.05
ρ_l = bulk density of the fluid = 1.065 g/cm^3
ρ_b = bulk density of the bed material (g/cm^3)

7.3.4 CRITICAL SHEAR STRESS - USDA

The United States Department of Agriculture (USDA) has developed the following formula for estimating the permissible shear on cohesive soils [Ref 12]:

$\tau_{p\ soil} = (c_1 P1^2 + c_2 P1 + c_3)\ (c_4 + c_5 e) 2c_6$

Where,

$\tau_{p\ soil}$ = soil permissible shear stress, N/m^2(lb/ft^2)
P1 = plasticity index
e = void ration
$c_1, c_2, c_3, c_4, c_5, c_6$ = coefficients (Table 5)

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Table 5
Keeyask GS - Stage IV Studies, Axis GR-4
Coefficients for Permissible Soil Shear Stress [Ref 12]

ASTM Soil Classification[(1)]	Applicable Range	c1	c2	c3	c4	c5	c6 (SI)	c6 (CU)
GM	10 < PI < 20 20 < PI	1.07	14.3	47.7 0.076	1.42 1.42	-0.6 -0.61	4.8x10-3 48	10-4 1.0
GC	10 < PI < 20 20 < P	0.0477	2.86	42.9 0.119	1.42 1.42	-0.61 -0.61	4.8x10-2 48	10-3 1.0
SM	10 < PI < 20 20 < PI	1.07	7.15	11.9 0.058	1.42 1.42	-0.61 -0.61	4.8x10-3 48	10-4 1.0
SC	10 < PI < 20 20 < PI	1.07	14.3	47.7 0.076	1.42 1.42	-0.61 -0.61	4.8x10-3 48	10-4 1.0
ML	10 < PI < 20 20 < PI	1.07	7.15	11.9 0.058	1.48 1.48	-0.57 -0.57	4.8x10-3 48	10-4 1.0
CL	10 < PI < 20 20 < PI	1.07	14.3	47.7 0.076	1.48 1.48	-0.57 -0.57	4.8x10-3 48	10-4 1.0
MH	10 < PI < 20 20 < PI	0.0477	1.43	10.7 0.058	1.38 1.38	-0.373 -0.373	4.8x10-2 48	10-3 1.0
CH	20 < PI			0.097	1.38	-0.373	48	1.0

[(1)] **Note:** **Typical names**
GM Silty gravels, gravel-sand silt mixtures
GC Clayey gravels, gravel-sand-clay mixtures
SM Silty sands, sand-silt mixtures
SC Clayey sands, sand-clay mixtures
ML Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
MH Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
CH Inorganic clays of high plasticity, fat clays
(SI)= Coefficient used to yield critical shear in metric (Pa)
(CU)= Coefficient used to yield critical shear in imperial units (psf)

The estimated permissible shear stress for the post-glacial clays based on this approach is 3.2 Pa, using the following average properties:

Moist Unit Weight = 18.5 kN/m^3
Plasticity Index = 18%
Moisture Content = 26%
Specific Gravity = 2.72

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7.3.5 JULIAN AND TORRES (2006)

Based on erosion measurements taken of the Sand River in South Carolina and estimation of applied bank shear stress, Julian and Torres [Ref 27] arrived at the following relationship between critical shear stress and silt-clay content:

$\tau_c = 0.1 + 0.1779\ (SC\%) + 0.0028\ (SC\%)^2$

$-2.34E-5\ (SC\%)^3$

Where:

τ_c = critical shear stress (Pa)
SC% = percent clay size material, with clay size = 0.063 mm

This relationship as well as the 95% confidence limits are shown in Figure 9. The post-glacial clays at the Keeyask site generally contain at least 80% clay size particles (Figure 3), which corresponds to a critical shear stress of at least 16 Pa based on the lower-bound 95% confidence limit shown in Figure 9.

7.3.6 SMERDON AND BEASLEY (1961)

Based on flume tests conducted on 11 samples of Mississippi cohesive soils, Smerdon and Beasley [Ref 29] arrived at the following relationship of critical shear stress with plasticity index:

$\tau_c = 0.16\ (PI)^{0.84}$

Where:

τ_c = critical shear stress (Pa)
PI = plasticity index (%)

Based on the above relationship, the estimated critical shear stress for an average PI=18% is 1.8 Pa

It should be noted that the material used in these tests was simply leveled in the flume and not compacted. Consequently it represents critical shear stresses that are below those for intact clay materials, which was confirmed by Clark and Wynn [Ref 13].

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7.4 ESTIMATION OF EROSION RATES FOR CLAYS

The methods used to estimate the erosion rate of the clays at Keeyask, as reported herein, were based on published experience and equations. The methodologies employed to estimate the erosion rates are discussed below, and the anticipated range in erosion rates is given in Table 6. The erosion rates for the clays at Keeyask have been estimated to be in the range of 8×10^{-6} to $< 2 \times 10^{-4}$ kg/m^{2}s, with a value of 3×10^{-5} kg/m^{2}s being assumed for the base case erosion estimates.

Table 6
Keeyask GS - Stage IV Studies, Axis GR-4
Post-Glacial Clays - Estimated Erosion Rates

Description	Soil Erodibility Factor (k_d) (Hanson and Simon 2001) (cm^3/Ns)	Erosion Rate Coefficient (Arulanandan, 1980)	
		(g/cm^2/min)	(kg/m^2/s)
Published values	0.9 to 0.01	<0.01	$<3 \times 10^{-5}$
Based on Voids Ratio	0.09	0.002	6×10^{-6}
USDA	0.11	0.012	3×10^{-5}
Smerdon & Beasley	0.15	<0.04	$<1 \times 10^{-4}$
Based on Bulk Density	0.21	0.07	$<2 \times 10^{-4}$

7.4.1 ESTIMATION OF PARTHENAIDE'S COEFFICIENT

7.4.1.1 Hwang and Mehta

Hwang and Mehta [Ref 11] investigated the erosion properties of sediments in Lake Okeechobee, Florida, using a rotating annular flume. The tests were conducted on placed beds of sediment that had been allowed to consolidate and were subsequently subjected to a known shear stress. This allowed the erosion rate to be determined directly for a given bed density and shear stress. They derived the following relationship of the surface erosion rate to the dry bulk density of the sediment:

$$\log_{10} M_u = 0.23 \exp \frac{(0.198)}{P_{wb} - 100.23}$$

Where:

P_{wb} = the dry bulk density of the deposit in g/l
M_u = the surface erosion in kg/m^2/s

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The clays at Keeyask have an average dry bulk density of 14.7 kN/m^3 or 1.5 g/cm^3, based on a moist unit weight of 18.5 kN/m^3 with a moisture content of 26%, and a specific gravity of 2.72, which yields an average erosion rate of 4 x 10 $^{-6}$ kg/m^2/s. This bulk density is slightly above the densities used by Hwang and Mehta in their studies, consequently, it may be an underestimate of the "true" erosion rate. However, a review of the plotted data from Hwang and Mehta indicates that the "true" erosion rate for the Keeyask clays should be less than 8 x 10 $^{-6}$ kg/m^2/s.

7.4.1.2 Teisson-Latteux and Cormault

$$M_u = 0.55 \frac{(P_{wb})^3}{1000}$$

Where:

P_{wb} = the dry bulk density of the deposit in g/l
M_u = the surface erosion in kg/m^2/s

7.4.2 ESTIMATION OF HANSON'S SOIL ERODIBILITY FACTOR (K_D)

7.4.2.1 Arulanandan et al.:

Arulanandan et al [Ref 15] developed a method to calculate the rate of erosion of cohesive bank material based on laboratory work conducted at the Waterways Experiment Station at Vicksburg, Mississippi. They found that the erosion of cohesive soil largely depends on the physical and chemical make-up of the soil, as well as the types and amounts of salts in the pore and eroding waters.

$$k_d = (1/\gamma)\ 0.0223 \exp(-0.13\tau_{c)}$$

Where:

k_d = soil erodibility (cm^3/N-s)
γ = specific weight of water
τ_c = critical shear stress (Pa)

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8 HYDRAULIC MODELING

8.1 BACKWATER PROGRAM H01F

The erosion rate and amount of material eroded from the riverbanks is dependent on the shear stresses imposed on the banks and the material forming the banks. The shear stresses due to flowing water, i.e. current action, were estimated from the results of a one-dimensional backwater model using the program H01F. The H01F model was used to estimate energy and water levels along the study reach, from which flow velocities and flow induced shear stresses were calculated.

The H01F Program is a steady state one-dimensional hydraulic backwater computer program H01F, developed by Acres Manitoba Limited. The capabilities of the H01F are virtually identical to and produce similar results as the US Army Corps of Engineers Model HEC-RAS.

8.1.1 MODEL DESCRIPTION

The reach of the Nelson River between the upstream end of Gull Lake and the upstream end of Stephen's Lake was modeled using the H01F backwater program. This reach covers the reach of the river that could be impacted during the construction phase, other than for the relatively short period during reservoir impoundment. In the order of 250 hydraulic cross sections were used to model the approximate 24 km of river in this reach of the Nelson River.

The hydraulic cross-sections for the H01 models were based on topography contained in the document entitled "2005 KEEYASK TOPGRAPHIC & BATHYMETRIC COMPOSITE NAD83 Z15 REV. 7", which is included as Figure 10 for documentation purposes.

Hydraulic backwater models were set-up to model the following conditions:

- existing environment
- conditions during Stage I Diversion
- conditions during Stage II Diversion, including reservoir impoundment

The locations of the hydraulic sections are shown in Figures 11 to 14 for the existing environment. In the Gull Rapids area, only flow through the Middle and South Channels was modeled. Flow through the North Channel was not included as it represents a small fraction of the total river flow, will not effect calibration results in the areas of concern, and there will be no flow in this channel after Stage I Diversion is initiated.

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The hydraulic sections upstream of Gull Rapids were used for all cases modeled. The hydraulic sections within Gull Rapids for the existing environment were also used for Stage I Diversion. Changes to the hydraulic sections within the Gull Rapids area were required to model Stage II Diversion/impoundment due to changes in flow patterns. The revised hydraulic sections at Gull Rapids for Stage II Diversion/impoundment are shown in Figure 15.

The hydraulic cross-sections for the H01 models were set-up such that the flow in the river channel was divided into a maximum of eight n-value subtypes, as follows:

- main river channel: defined by the average water level, was divided into four subtypes
- overbank flow areas: defined as the area between the average water level and the historic shoreline adjacent to the main channel on both sides river
- overland flow areas: the area beyond the overbank flow areas on both sides of the river

Each n-value subtype was capable of having a different Manning's coefficient of roughness (i.e. n-value). The shoreline and overbank flow area are defined in Figure 16 and the eight subtypes are described in Figure 17.

8.1.2 CALIBRATION

The results of the H01F model of the existing environment were compared to observed water levels at gauge locations. The results from the model used in this study compare favorably with the above mentioned results, as shown in Figure 18.

8.2 ESTIMATED SHEAR STRESSES

Shear stresses for the existing environment, Stage I Diversion, and Stage II Diversion/Impoundment were calculated from the hydraulic conditions output from the H01F model in order to do a preliminary assessment of the potential for erosion of the river channel.

8.2.1 STAGE I DIVERSION

During Stage I Diversion, the flow of the Nelson River will be restricted to the south channel at Gull Rapids. As a result, the water levels upstream of Gull Rapids will be higher than under the existing environment for any particular flow rate. Due to the higher water levels during Stage I Diversion, the shear stresses along the shorelines for Stage I Diversion will be generally less than or equal to those for the existing environment for the flows described below.

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The shear stresses for the construction design floods for Stage I and II Diversion were compared to the shear stresses for a flow of 5,270 m^3/s for the existing environment. The flow of 5,270 m^3/s is the August 1:20 year mean monthly discharge, which was used as the design flow for river closure during Stage II Diversion, and is similar to the flow of 5,320 m^3/s which occurred during the sediment monitoring program conducted in the summer of 2006 in which no significant change in TSS levels were observed along the reach of the river investigated.

Since the shear stresses in the main river channel will be less than those experienced in 2006 when no change in TSS levels were observed, it can be assumed that erosion of the main river channel will not occur during Stage I Diversion and can be ignored in the riverine erosion modeling.

8.2.2 STAGE II DIVERSION

To be added, after initial review of this memorandum.

9 RIVERINE EROSION MODEL

9.1 GENERAL

The riverine erosion model developed for this study is a one-dimensional sediment concentration model which can calculate the sediment concentration for a maximum of 8 sub-regions in a section taken across a river channel. Calculations proceed in an upstream to downstream direction along the river.

Sediment concentrations are calculated at each "Hydraulic Section", i.e. at each section that was defined in the H01F backwater model, as well as at "Sediment Sections" which are inserted between the Hydraulic Sections at each change in riverbed material. The hydraulic properties for these Sediment Sections are linearly interpolated from the properties of the adjacent Hydraulic Sections.

The mode of sediment transport (i.e. whether sediment is transported as bedload, suspended load, or wash load) has been based on the Rouse Number, and was obtained only for the most downstream Hydraulic Section.

At the present time, the erosion model can accommodate the following:

- a maximum of 11 sub-channels can be used to define the reach of river being investigated
- a maximum of eight n-value sub-regions at each section

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- a maximum of 30 riverbed materials
- non-uniform material gradations
- equilibrium concentrations based on:
    - Ackers-White (1993) [Ref 23]
    - Laursen (1958) [Ref 17]
    - Wu et al [Ref 16]
    - Engelund-Hansen [Ref 25]
    - Yang [Ref 25]

The following assumptions have been made in the development of the erosion model:

- riverbed material properties are constant between adjacent sections (note that these sections may be Hydraulic Sections or Sediment Sections)
- hydraulic properties between adjacent Hydraulic Sections are assumed to vary linearly
- the river channel is a fixed bed, i.e. no changes occur to the channel geometry due to erosion or deposition of sediment
- riverbed material gradations do not change due to erosion
- mixing of sediment across n-value sub-regions occurs only at Hydraulic Sections, no mixing occurs at Sediment Sections
- sediment is fully mixed at each Hydraulic Section
- the methods used to estimate the equilibrium concentrations can be applied to an individual n-value subtype in a section, as well as to the entire river section

The general flow chart of the riverine erosion model calculations is shown in Figure 19.

9.2 RIVERBED MATERIALS USED IN THE RIVERINE EROSION MODEL

The riverbank and shoreline materials used in the sediment model were based on the materials described in the shoreline mapping completed by ECOSTEM Ltd. The verbal descriptions of the materials that were provided by ECOSTEM combined with the grain size distribution of samples obtained from the field investigation programs, were used as a basis for estimating the material properties given in Table 7 and in developing the assumed grain size distributions (Figure 20).

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Table 7
Keeyask GS - Stage IV Studies, Axis GR-4
Riverbed Material Properties

Description	Cohesive	Grain Size Distribution Ref. No.(1)	Unit Weight (kN/m3)	Friction Angle for Cohesionless Materials (degrees)	Cohesive Materials	
					Critical Shear (Pa)	Mass Erosion Flux (kg/m2/s)
Bedrock	x		26		1000000	0
Boulders		4	19	30		
Cobbles		4	19	30		
Gravel		3	19	30		
Boulder till	x		20		1000	
Sand w Rock		2	19.4	30		
Sand with Cobbles		2	19.4	30		
Sand		2	19.4	30		
Sand with Till		1	19.4	30		
Till		1	18.1	30		
Clay with Rock	x		18.5		2	0.0001
Clay with Boulders	x		18.5		2	0.0001
Clay with Cobbles	x		18.5		2	0.0001
Clay with Gravel	x		18.5		2	0.0001
Clay with Till	x		18.5		2	0.0001
Clay	x		18.5		2	0.0001
Peat with Cobbles with Boulders	x		12		20	
Peat with Cobbles	x		12		20	
Peat with Rock	x		12		20	
Peat	x		12		20	
To be classified	x		12		20	

Note:
(1) Grain Size Distribution Curves are shown in Figure 20.

9.3 EQUILIBRIUM SEDIMENT CONCENTRATIONS

The selection of the appropriate sediment transport equation for the calculation of equilibrium sediment concentrations may be considered to be more art than science. As stated in the HEC-RAS manual, "Sediment transport modeling is notoriously difficult. The data utilized to predict bed change is fundamentally uncertain and the theory employed is empirical and highly sensitive to a wide array of physical variables." The uncertainty in estimating sediment transport is illustrated by the large variation in the computed versus measured sediment transport shown in Figure 21.

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9.3.1 SEDIMENT TRANSPORT EQUATIONS

In order to select the appropriate sediment transport equation for use, items such as particle sizes, general assumptions, theory, and similarity between the experiment/field measurements that the equation is based on, must be compared to the case under consideration.

The approximate parameter ranges for the applicability of selected sediment transport equations are presented in Table 8. The applicable particle size range for various sediment transport equations is shown in Figure 20.

Table 8
Keeyask GS - Stage IV Studies, Axis GR-4
Sediment Transport Formulae - Parameter Ranges
[Ref 22]

Function	d	d_m	s	V	D	S	W	T
Ackers-White (*flume*)	0.04 - 7.0	NA	1.0 - 2.7	0.07 - 7.1	0.01 - 1.4	0.00006 - 0.037	0.23 - 4.0	46 - 89
Englund-Hansen (*flume*)	NA	0.19 - 0.93	NA	0.65 – 6.34	0.19 – 1.33	0.000055 – 0.019	NA	45 - 93
Laursen (*field*)	NA	0.08– 0.7	NA	0.068 – 7.8	0.67 – 54	0.0000021 – 0.0018	63 – 3640	32 - 93
Laursen (*flume*)	NA	0.011 - 29	NA	0.7 - 9.4	0.03 – 3.6	0.00025 – 0.025	0.25 – 6.6	46 - 83
Meyer-Peter Muller (*flume*)	0.4 – 29	NA	1.25 – 4.0	1.2 – 9.4	0.03 – 3.9	0.0004 – 0.02	0.5 – 6.6	NA
Tofaletti (*field*)	0.062 – 4.0	0.095 – 0.76	NA	0.7 - 7.8	0.07 – 56.7 (R)	0.000002 – 0.0011	63 - 3640	32 – 93
Tofaletti (*flume*)	0.062 – 4.0	0.45 – 0.91	NA	0.7 - 6.3	0.07 – 1.1 (R)	0.00014 – 0.019	0.8 – 8	40 - 93
Yang (*field-sand*)	0.15 – 1.7	NA	NA	0.8 - 6.4	0.04 – 50	0.000043 – 0.028	0.44 – 1750	32 - 94
Yang (*field-gravel*)	2.5 – 7.0	NA	NA	1.4 - 5.1	0.08 – 0.72	0.0012 – 0.029	0.44 – 1750	32 - 94

Where: d = Overall particle diameter, mm
d_m = Median particle diameter, mm
s = Sediment specific gravity
V = Average channel velocity, fps
D = Channel depth, ft
S = Energy gradient
W = Channel width, ft
T = Water temperature, °F
(R) = Hydraulic Radius, ft
NA = Data not available

9.3.2 SEDIMENT TRANSPORT EQUATIONS USED IN THE RIVERINE EROSION MODEL

The sediment presently carried by the Nelson River between the upstream end of Gull Lake and the upstream end of Stephen's Lake consists primarily of fine grained material with particle diameters less than 0.1mm. The gradations of the riverbed, riverbank, and materials on the abutments of the river indicate that the potential material that can be eroded is generally smaller than 1mm in diameter, with average diameters of less than 0.5mm. These particle sizes are at the lower end of the particle size ranges for most of the available sediment transport equations (Figure 20).

Based on grain size diameter, hydraulic conditions, and a review of the literature, the following methods have been used to calculate equilibrium sediment concentrations:

- Ackers-White (1993)
- Laursen (1958)
- Wu et al
- Engelund – Hansen
- Yang

10 SEDIMENTATION MODEL INTERIM RESULTS

10.1 STAGE I DIVERSION

The reach of the Nelson River that was modeled extends from the upstream end of Gull Lake to the upstream end of Stephen's Lake (Figure 1). This reach of river has been divided into 11 channels for modeling purposes.

Erosion of cohesionless material along the riverbanks was estimated for the study design flow of 4,855 m^3/s using the five sediment transport equations presented in Section 9.3.2. Only one area of potentially significant erosion was identified. This area is located along the south shore of the South Channel at Gull Rapids in the vicinity of the South Dam (Figure 22). The area extends approximately 1,200 m upstream and 700 m downstream of the Axis of Principal Structures. The riverbank material consists of sand or sand with cobbles.

The estimated sediment yield (bedload component) in this area was found to vary by two orders of magnitude depending on which equilibrium transport equation was used, as shown in Table 9.

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Table 9
Keeyask GS - Stage IV Studies, Axis GR-4
Shoreline Erosion During Stage I Diversion
Estimated Increase in Sediment Load

Equilibrium Transport Equation	Total Increase in Sediment Load at Stephen's Lake, Fully Mixed (mg/L)	
	Bedload	Total Suspended Solids (TSS)
Ackers-White (D50)	9	1
Laursen (1958)	1	1
Wu	101	1
Engelund – Hansen	11	1
Yang	6	1

These estimated increases in sediment load include cohesive materials and cohesionless materials with particle diameters ranging from clay size up to the 32-64 mm sediment size range.

The results of the sediment model indicate that potential erosion of the riverbanks for Stage I Diversion will be limited to a local area of the South Channel at Gull Rapids (Figure 22). It is estimated that the increase in sediment load above background levels for fully mixed conditions at the upstream end of Stephens Lake, will be in the order of 1 mg/L (TSS) and 10 mg/L (bedload).

10.2 STAGE II DIVERSION AND IMPOUNDMENT

To follow after review of Draft 2.

10.3 SHORELINE EROSION IN GULL LAKE

10.3.1 EROSION DURING STAGE I DIVERSION

Stage I Diversion will result in a small increase (approximately 0.9 m for the study design flow) in water levels in Gull Lake (see Section 6.1.2). However, because the increased water levels are within the current water level fluctuation range, no increase in mineral shore erosion rates are expected to occur. As a result, erosion rates during Stage I Diversion are expected to be within the range predicted for existing environment mineral erosion rates reported in Memorandum GN-9.2.2 [Ref 3].

10.4 EROSION DURING STAGE II DIVERSION

To be completed after initial review of this memorandum.

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10.5 EROSION DURING FINAL RESERVOIR FILLING

Final reservoir filling will follow completion of installation of the gates in the Spillway. Depending on flow conditions, final reservoir filling to full supply level will be completed in approximately 5 to 20 days. During this time, the reservoir will submerge vegetated slopes below elevation of 159 m and a new shoreline will form at 159 m when the full supply level has been reached. Little additional mineral erosion is expected to occur below the full supply elevation during reservoir filling because of the rapid rate at which water levels will rise and because the vegetation and root systems will effectively stabilize mineral soil slopes during this brief period. After water levels reach full supply levels, the project erosion estimates described in Memorandum GN-9.2.8 [Ref 18] will apply.

11 SUMMARY AND CONCLUSION

A sediment model was developed to simulate shoreline erosion due to current action of the reach of the Nelson River from the upstream end of Gull Lake to the upstream end of Stephens Lake. The results of the model for Stage I Diversion indicated that erosion would occur over an isolated area on the south shore immediately upstream of the Axis of Principal Structures at Gull Rapids.

For the study design flow of 4,855 m^3/s, under fully mixed conditions, the estimated increase of sediment load above background levels at the upstream end of Stephens Lake, are as follows:

- Stage I Diversion: total increase in bedload of 10 mg/L and an increase in TSS of 1 mg/L
- Stage II Diversion: to be determined after review of Draft 2 of this memorandum.

For Stage I Diversion, sediment load due to ongoing erosion of the shorelines of Gull Lake (due to wave action) will remain within the range predicted for the existing environment.

The sediment load is anticipated to show little or no increase during the final filling of the reservoir to FSL 159.0

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APPENDIX A

APPENDIX A
SHORELINE MAPPING BY ECOSTEM

This appendix has to be updated to include only the most current version of the attached data.

Keeyask Shore Classification 2003-R4 070404- Dataset Metadata

General

Name: Keeyask Shore Classification Mainland 2003 – R4 070404

Type: Digital, spatial, line.

- To provide better simultaneous visual representation of multiple attributes on a single map, a polygon version (i.e., segmented bands following both sides of the shoreline) will also be provided at a future date.

Description: Classifies mainland shoreline of the Nelson River between Clark Lake and Stephens Lake.

- Shoreline defined as the visible extent of historic water and ice regime effects on upland and peatland habitat adjacent to the Nelson River. For upland habitat, the shoreline generally coincides with the top of bank.
- Key attributes are beach material type, bank material type, bank height, beach slope and bank slope.

Purposes:

- Classify 2003 shore zone into types suitable for various specialists involved in the terrestrial environment effects assessment.
- Provide basis for predicting effects of proposed development on shore zone habitat.
- Provide basis for developing qualitative relationships between habitat and selected mammals and birds.
- Provide data for some of the variables needed to predict Project effects on erosion and peatland disintegration.

Limitations:

- No field data upstream of Birthday Rapids due to extension of the hydraulic zone of influence and for some smaller islands.
- Beach and bank slopes to be added.
- Final ground truthing to occur in summer 2007.

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Sources:

- Shoreline location provided by photo-interpretation of 1:15,000 aerial photography acquired by Manitoba Hydro contractor in summer 2003. Manitoba Hydro 1:60,000 1998 digital ortho imagery (DOIs) used as the base map for heads-up digitization.
- Shoreline segmentation occurs wherever there is a change in one of the key shoreline attributes.
- Initial classification of the Gull Lake area was completed by aerial surveys from a Bell Jet Ranger conducted in summer 2002.
- Validation and further classification was completed during aerial surveys from a Bell Jet Ranger and limited boat surveys in 2003 to 2005 and from aerial photographs acquired from 2005 and 2006.

Reliability: Expected to be moderate to high for most attributes and areas.

Providers: DOIs, 1:15,000 aerial photography and helicopter support by Manitoba Hydro; aerial and boat surveys by James Ehnes of ECOSTEM Ltd.; polygon geometry and addition of attributes to spatial dataset by ECOSTEM Ltd.; initial shore material attribute categories from J. D. Mollard & Associates Wuskwatim generating station EIS contribution.

Date Provided: April 4, 2007.

Status:

- Ready for internal distribution to EIS study team for review.
- Suitable for preliminary analyses.
- Classification of segments with "unknown" for attribute values to occur in summer 2007.
- Final ground truthing to occur in summer 2007.

Spatial

Approximate Bounding Rectangle For Data Extents:

Min X	330,100
Min Y	6,241,400
Max X	366,200
Max Y	6,249,000

Coordinate System: **UTM Zone 15** Datum: **NAD 1983**

Attributes

Attribute: **Shore_Type**

Data Type: **Integer**

Attribute Description: Concatenation of beach and bank material types.

Legend: See descriptions for Beach_TypeT and Bank_TypeT attributes.

Attribute: **Beach_Type**

Data Type: **Integer**

Attribute Description: Numeric code for dominant type of beach material.

Legend: See "Code" column in Table 1.

Status: Complete except for a small proportion of data gaps to be filled in summer 2007.

Attribute: **Beach_TypeT**

Data Type: **Character**

Attribute Description: Descriptive name for numeric codes used in the Beach_Type attribute.

Legend: See "Name" and "Description" columns in Table 1.

Status: Complete except for a small proportion of data gaps to be filled in summer 2007.

Attribute: **Bank_Type**

Data Type: **Integer**

Attribute Description: Dominant type of shore material. Same categories as developed by J. D. Mollard and Associates with additions to capture Keeyask conditions.

Legend: See "Code" column in Table 1.

Status: Complete except for a small proportion of data gaps to be filled in summer 2007.

Attribute: **Bank_TypeT**

Data Type: **Character**

Attribute Description: Descriptive name for numeric codes used in the Beach_Type attribute.

Legend: See "Name" and "Description" columns in Table 1.

Status: Complete except for a small proportion of data gaps to be filled in summer 2007.

Table 1. Numeric and descriptive codes for beach and bank material types.

Code	Name	Description
1	Bedrock	[to be added]
3	Boulders	
5	Cobbles	
7	Gravel	
8	Boulder till	
9	Sand w Rock	
11	Sand w Cobbles	
13	Sand	
15	Sand w Till	
19	Till	
21	Clay w Rock	
23	Clay w Boulders	
25	Clay w Cobbles	
27	Clay w Gravel	
29	Clay w Till	
31	Clay	
35	Peat w Cobbles w Boulders	
37	Peat w Cobbles	
39	Peat w Rock	
41	Peat	
99	To be classified	

Attribute: **Beach_Slope**

Data Type: **Integer**

Attribute Description: Numeric class for beach slope.

Legend: See Table 2.

Status: To be completed.

Table 2. Beach slope classes.

Code	Description
1	Low (~< 10%)
2	Medium (~10 - 100%)
3	High (~> 100%)
99	To be classified

Attribute: **Bank_Height**

Data Type: **Decimal (F2.1)**

Attribute Description: Height class for elevation difference from top of bank to toe of bank. Intention is to convert this to an estimated height using LIDAR data.

Limitations beyond those described for the dataset: Reliability is moderate.

Legend: See Table 3.

Status: Complete except for a small proportion of data gaps to be filled in summer 2007.

Table 3. Bank height classes.

Category	Description
0	No bank
1	Low: < ~1m
1.5	Mixture of low and medium
2	Medium: ~1-3 m
3	High: >~ 3 m
99	To be classified

Attribute: **Bank_Slope**

Data Type: **Integer**

Attribute Description: Numeric class for bank slope.

Legend: [to be determined].

Status: To be completed.

Attribute: **Bank_Aspect**

Data Type: **Integer**

Attribute Description: Azimuth of direction the bank faces.

Limitations beyond those described for the dataset: ??

Legend: [to be determined].

Status: To be completed.

Attribute: **Length**

Data Type: **Integer**

Attribute Description: Spherical length of segment in meters.

Legend: Not applicable.

Attribute: **ID**

Data Type: **Integer**

Attribute Description: Unique identifier for each line segment. No further edits to this field permitted for existing segments. Any additional segments resulting from either additions or subdivisions must be assigned the next highest available integer value.

Legend: Not applicable.

Attribute: **Mainland**

Data Type: **Character**

Attribute Description: Whether segment is on mainland or island shoreline.

Legend: Y = yes; N = no.

APPENDIX B

APPENDIX B

SEDIMENT MODEL DOCUMENTATION

B1 ACKERS-WHITE METHOD [REF 23]
B2 LAURSEN (1958) METHOD [REF 17]
B3 WU ETAL (2000) METHOD [REF 16]

APPENDIX B1
ACKERS-WHITE (1993)

The Ackers-White equation used in the sediment model is based on equations presented in the HEC-RAS manual [Ref 23], and incorporates the correction by Ackers (1993), and the use of the hiding factor from Profitt and Sutherland.

Dimensionless grain diameter,

$$d_{gr} = d_{si}\left[\frac{g\cdot(s-1)}{\nu^2}\right]^{\frac{1}{3}}$$

Shear velocity u

$$u_{star} = \sqrt{g \cdot D \cdot S}$$

Sediment size-related transition exponent n,

0.56

$$n = \begin{cases} 1 & \text{if } d_{gr} \le 1 \\ \left(1 - 056 \cdot \log\left(d_{gr}\right)\right) & \text{if } 1 < d_{gr} \le 60 \\ 0 & \text{if } d_{gr} > 60 \end{cases}$$

Initial motion parameter A,

$$A = \begin{cases} \left(\frac{0.23}{\sqrt{d_{gr}}} + 0.14\right) & \text{if } d_{gr} \le 60 \\ 0.17 & \text{otherwise} \end{cases}$$

Sediment mobility number F_{gr},

$\alpha = 10$ (assumed value used in HEC6 and SAM)

$$F_{gr} = \frac{u_{star}^{\ n}}{\sqrt{g \cdot d_{si} \cdot (s-1)}} \cdot \left(\frac{V}{\sqrt{32} \cdot \log\left(\alpha \cdot \frac{D}{d_{si}}\right)} \right)^{1-n}$$

Hiding Factor HF,

Shield's Mobility Parameter □,

$$\theta = \frac{u_{star}^{\ 2}}{g \cdot (s-1) d_{50}}$$

$$dRatio = \begin{cases} 1.1 & \text{if } \theta \le 0.04 \\ \left(2.3 - 30 \cdot \theta\right) & \text{if } 0.04 < \theta \le 0.045 \\ \left(1.4 - 10 \cdot \theta\right) & \text{if } 0.045 < \theta \le 0.095 \\ 0.45 & \text{otherwise} \end{cases}$$

$$dAdjust = d_{50} \cdot dRatio$$

$$HFRatio = \frac{d_{si}}{dAdjust}$$

$$HF = \begin{cases} 1.30 & \text{if } HFRatio \geq 3.7 \\ (0.53 \cdot \log(HFRatio) + 1) & \text{if } 0.075 \leq HFRatio < 3.7 \\ 0.40 & \text{otherwise} \end{cases}$$

Adjust Sediment Mobility Number for Hiding Factor

$$F_{gr} = HF \cdot F_{gr}$$

Check for too fine sediment based on F_{gr} and A,

$$Check = \frac{F_{gr}}{A}$$

Sediment transport function exponent m,

$$m = \begin{cases} \left(\frac{6.83}{d_{gr}} + 1.67\right) & \text{if } d_{gr} \leq 60 \\ 1.78 & \text{otherwise} \end{cases}$$

Check for too fine sediment based on m,

$$Check = \begin{cases} 0 & \text{if } m > 6 \\ Check & \text{otherwise} \end{cases}$$

Sediment transport function coefficient C,

$$C = \begin{cases} 10^{2.79 \cdot \log(d_{gr}) - 0.98(\log(d_{gr}))^2 - 3.46} & \text{if } d_{gr} \leq 60 \\ 0.025 & \text{otherwise} \end{cases}$$

Transport parameter G_{gr},

$$G_{gr} = C \cdot \left(\frac{F_{gr}}{A} - 1 \right)^{m}$$

Sediment flux X, in parts per million by fluid weight,

$$X = \frac{G_{gr} sd_{si}}{D \left(\frac{u_{star}}{V} \right)^{n}}$$

Sediment Discharge, lb/s

$$G = \gamma_w Q X$$

Sediment Discharge, tons/day

$$G_s = \frac{86400}{2000} \cdot G$$

Check to make sure particle diameter and mobility functions are not too low,

$$G_s = \begin{cases} G_s & \text{if Check} > 1 \\ 0 & \text{otherwise} \end{cases}$$

APPENDIX B2
LAURSEN (1958)

The Laursen (1958) sediment transport equation is as reported in Ref 17. The Visual Basic code has been based on the program code of the USGS program SEDDISCH.

The equation developed by Laursen (1958) to compute the mean concentration of bed-material discharge is based on empirical relations:

$$\bar{C} = \sum_{i=1}^{n} i_b \left(\frac{D_{si}}{d}\right)^{7/6} \left(\frac{\tau'_o}{\tau_c} - 1\right) f\left(\frac{V_*}{\omega_i}\right) \qquad (32)$$

in which

$$\tau'_o = \frac{\rho V^2}{58} \left(\frac{D_{50}}{d}\right)^{1/3} \qquad (33)$$

$$\tau_c = Y_c \, \rho \, g \, (S_g - 1) \, D_{si} \qquad (34)$$

where $\bar{C}$ - the concentration of bed-material discharge, in percent by weight;

n - the number of size fractions in the bed material;

i_b - the fraction, by weight, of bed material in a given size fraction;

D_{si} - the mean grain diameter, in ft, of the sediment in size fraction, i;

d - the mean depth, in ft;

τ'_o - Laursen's bed shear stress due to grain resistence;

τ_c - critical shear stress for particles of a size fraction;

f denotes function of;

V_* - the shear velocity, in ft/s;

ω_i - the fall velocity, in ft/s, of sediment particles of diameter D_{si};

ρ - the density of water, in slugs per ft^3;

V - the mean velocity, in ft/s;

D_{50} - the particle size, in ft, at which 50 percent of the bed material by weight is finer;

Y_c - a coefficient relating critical tractive force to sediment size;

g - acceleration of gravity, in ft/s/s; and

S_g - the specific gravity of sediment.

The density, ρ, has been introduced into the original τ_o' equation presented by Laursen so that the equation is dimensionally homogeneous, and Laursen's coefficient has been changed accordingly.

Substituting for τ_o' and τ_c' in equation 32 and converting $\bar{C}$ to C gives:

$$C = 10^4 \sum_{i=1}^{n} i_b \left(\frac{D_{si}}{d}\right)^{7/6} \left[\frac{V^2}{58\, Y_c\, D_{si}\, (S_g - 1)\, g} \left(\frac{D_{50}}{d}\right)^{1/3} - 1\right] f\left(\frac{V_*}{\omega_i}\right) \quad (35)$$

where C - the concentration of bed material discharge, in parts per million by weight.

Values of $f(V_*/\omega_i)$ are shown in Laursen's (1958) figure 14, which expresses the relation between V_*/ω_i and $f(V_*/\omega_i)$. The computer program uses a series of equations to approximate Laursen's figure 14.

The value of Y_c was related by Laursen to D_{si}/δ as follows:

$$Y_c = 0.04 \quad \text{for} \quad \frac{D_{si}}{\delta} > 0.1 \quad (36)$$

$$Y_c = 0.08 \quad \text{for} \quad 0.1 > \frac{D_{si}}{\delta} > 0.03 \quad (37)$$

$$Y_c = 0.16 \quad \text{for} \quad 0.03 > \frac{D_{si}}{\delta} \quad (38)$$

in which δ = the thickness, in ft, of the laminar sublayer;

$$\delta = 11.6 \left(\frac{\nu}{V_*}\right) \quad (39)$$

where ν - kinematic viscosity, in square feet per second (ft^2/s).

Laursen's formula was developed using natural sediments with a specific gravity of 2.65, and medium diameters that ranged from 0.011 to 4.08 mm.

APPENDIX B3
WU ET AL (2000) [REF 16]

1. Determine the Settling Velocity for Each Sediment Size:

$$\omega_i = \sqrt{[(13.95*\nu/d_i)^2 + 1.09(\gamma_s/\gamma - 1)gd_i]} - 13.95\nu/d_i$$

Where

ω_i = settling velocity of the *i*th fraction of sediment

ν = kinematic viscosity of water

d_i = diameter of the *i*th fraction of sediment

γ_s = specific weight of sediment

γ = specific weight of water

g = acceleration of gravity

2. Calculate the Hidden and Exposed Probabilities:

$$p_{hi} = \sum_{j=1}^{N} p_{bj} \frac{d_j}{d_i + d_j}$$

$$p_{ei} = \sum_{j=1}^{N} p_{bj} \frac{d_i}{d_i + d_j}$$

Where

N = the total number of particle size fractions of non-uniform sediment mixtures

p_{hi} = total hidden probability of particles d_i

p_{ei} = total exposed probability of particles d_i

3. Determine the Critical Shear Stress for Each Size Fraction:

$$\frac{\tau_{ci}}{(\gamma_s - \gamma)d_i} = \theta_c \left(\frac{p_{ei}}{p_{hi}}\right)^m$$

Where

τ_{ci} = critical shear stress for d_i

θ_c = non-dimensional critical shear stress for uniform sediment
= 0.03 (Wu et al, 2000)

m = empirical parameter = -0.6 (Wu et al, 2000)

γ_s = specific weight of soil = taken as a constant = 26 kN/m^3

4. Calculate the Bed Shear Stress:

$\tau_b = \gamma RS$

Where

τ_b = average bed shear stress

γ = unit weight of water

R = hydraulic radius

S = friction slope

5. Calculate Manning's Roughness Coefficient Corresponding to the Grain Shear Stress:

$n' = 0.05 d_{50}^{(1/6)}$

Where

n' = Manning's roughness coefficient corresponding to the grain shear stress

d_{50} = 50% sieve diameter of the bed materials

6. Calculate the Non-Dimensional Fractional Transport Rate for Non-Uniform Bed Load:

$$\phi_{bi} = 0.0053\left[\left(\frac{n'}{n}\right)^{3/2}\frac{\tau_b}{\tau_{ci}} - 1\right]^{2.2}$$

Where

φ_{bi} = fractional transport rate for non-uniform bed load

7. Calculate the Non-Dimensional Fractional Transport Rate for Non-Uniform Suspended Load:

$$\phi_{si} = 0.0000262\left[\left(\frac{\tau}{\tau_{ci}} - 1\right)\frac{U}{\omega_i}\right]^{1.74}$$

Where

φ_{si} = fractional transport rate for non-uniform suspended load

τ = shear stress

τ_{ci} = critical shear stress for d_i

U = average flow velocity

8. Calculate The Total Fractional Sediment Transport Rate

$$q_{bi,} = \phi_{bi} / [p_{bi} \sqrt{(\gamma_s / \gamma - 1) g d_i^3}$$

$$q_{si} = \phi_{si} / [p_{bi} \sqrt{(\gamma_s / \gamma - 1) g d_i^3}$$

$$= q_{bi,} + q_{si} \ (m^2/s)$$

where

q_{si} = fractional sediment transport rate in m^2/s

9. Convert the Fractional Sediment Transport Rate for Suspended Load into mg/L

(i) Convert from m^2/s into kg/s:

$q_{mi} = q_{si} \cdot W * (\gamma_s - 1000)$ [units = kg/s]

(ii) Convert from kg/s into mg/L:

$TSS_{si} = 1000 * (q_{mi}) / Q_{avg}$ [units = mg/L]

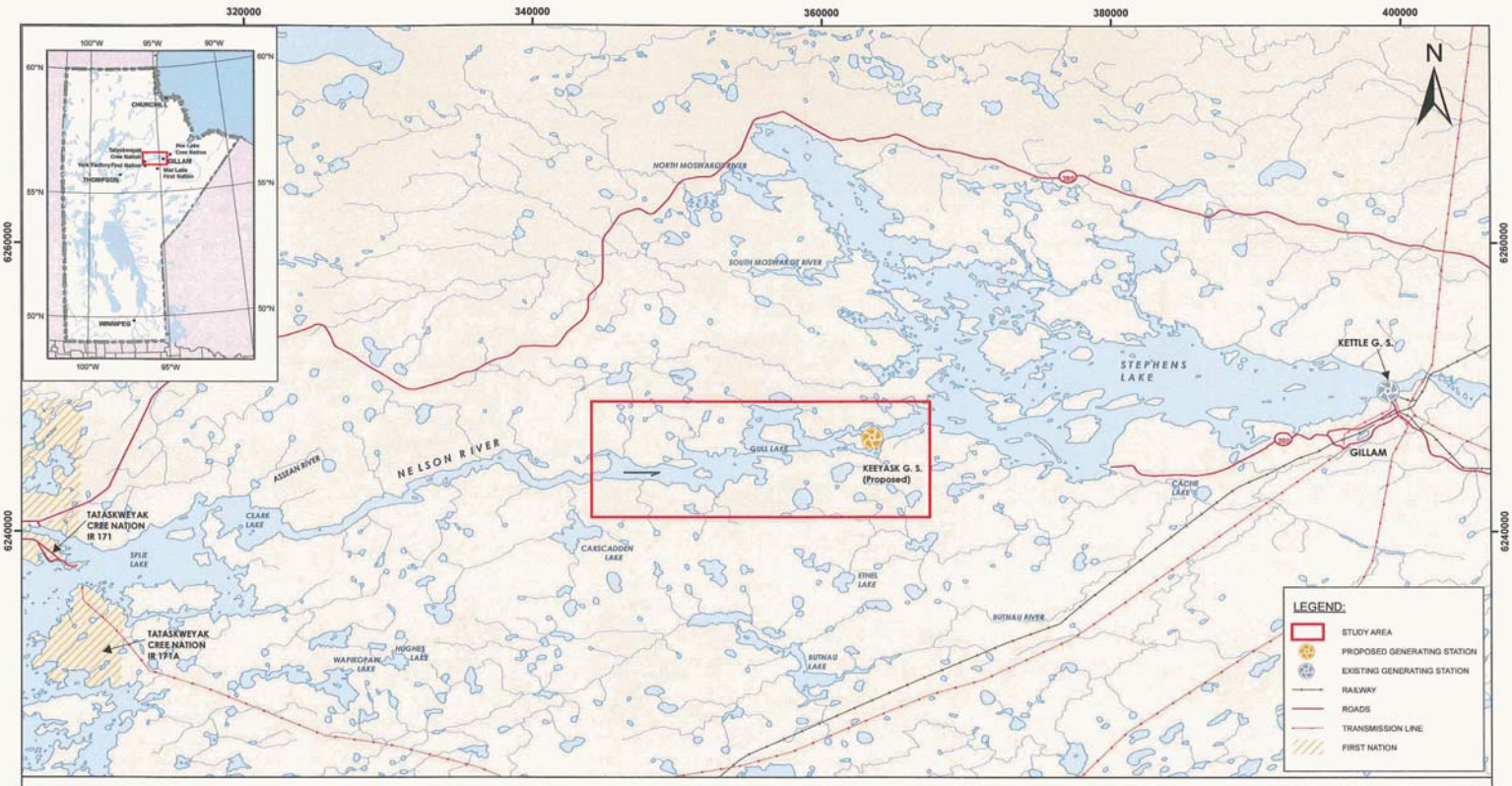
Where

q_{mi} = fractional sediment transport rate for an n-value subtype
q_{si} = fractional sediment transport rate

W = width of the n-value subtype
γ_s = unit weight of eroding soil
TSS_{si} = total suspended solids for particles of diameter d_i
Q_{avg} = average flow in the n-value subtype between adjacent sections

FIGURES

FIGURES 1 TO 22



KEEYASK

0 2,500 5,000 10,000 Meters

0 12,500 25,000 50,000 Feet

Scale: 1: 250 000

Projection: Universal Transverse Mercator (UTM)
Zone 15N, North American Datum 1983 (NAD 83)

Data Source: Baseline lakes, rivers, roads, railway and transmission line, taken from NTDB map 54Dand 64A, 2005. Her Majesty the Queen in right of Canada. All rights reserved.

Mapping grade for general reference only.

Source Map Scale: 1: 250 000

SHORELINE EROSION DURING CONSTRUCTION STUDY AREA

FIGURE 1

KGS ACRES

Date Created:
June 24, 2009

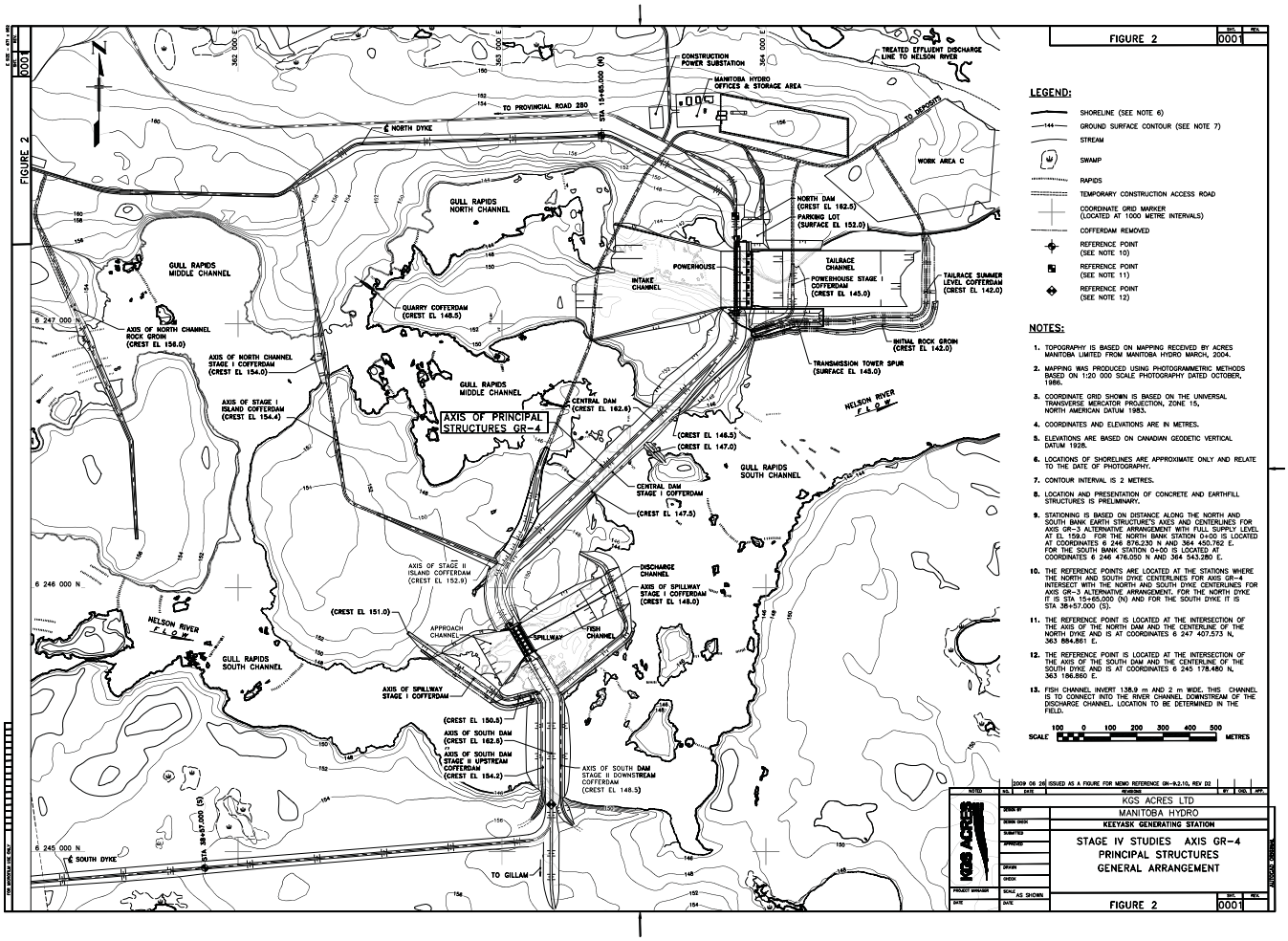


FIGURE 2
0001
LEGEND:
SHORELINE (SEE NOTE 6)
144 GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
TEMPORARY CONSTRUCTION ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
REFERENCE POINT (SEE NOTE 12)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL 159.0 FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.050 N AND 364 543.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+65.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.
SCALE 100 0 100 200 300 400 500 METRES
2009 06 29 ISSUED AS A FIGURE FOR MEMO REFERENCE GN–9.2.10, REV 02
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR–4
PRINCIPAL STRUCTURES
GENERAL ARRANGEMENT
FIGURE 2
0001
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO OFFICES & STORAGE AREA
TREATED EFFLUENT DISCHARGE LINE TO NELSON RIVER
TO PROVINCIAL ROAD 280
℄ NORTH DYKE
STA 15+65.000 (N)
TO DEPOSITS
WORK AREA C
GULL RAPIDS NORTH CHANNEL
NORTH DAM (CREST EL 162.5)
PARKING LOT (SURFACE EL 152.0)
GULL RAPIDS MIDDLE CHANNEL
POWERHOUSE
TAILRACE CHANNEL
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.0)
INTAKE CHANNEL
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 142.0)
QUARRY COFFERDAM (CREST EL 148.5)
6 247 000 N
AXIS OF NORTH CHANNEL ROCK GROIN (CREST EL 156.0)
INITIAL ROCK GROIN (CREST EL 142.0)
AXIS OF NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
TRANSMISSION TOWER SPUR (SURFACE EL 148.0)
GULL RAPIDS MIDDLE CHANNEL
CENTRAL DAM (CREST EL 162.6)
AXIS OF STAGE ISLAND COFFERDAM (CREST EL 154.4)
AXIS OF PRINCIPAL STRUCTURES GR–4
NELSON RIVER FLOW
(CREST EL 148.5)
(CREST EL 147.0)
GULL RAPIDS SOUTH CHANNEL
CENTRAL DAM STAGE I COFFERDAM
(CREST EL 147.5)
AXIS OF STAGE II ISLAND COFFERDAM (CREST EL 152.8)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM (CREST EL 148.0)
6 246 000 N
(CREST EL 151.0)
APPROACH CHANNEL
SPILLWAY
FISH CHANNEL
NELSON RIVER FLOW
GULL RAPIDS SOUTH CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
AXIS OF SOUTH DAM (CREST EL 162.5) AT AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 154.2)
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
6 245 000 N
℄ SOUTH DYKE
STA 38+57.000 (S)
TO GILLAM

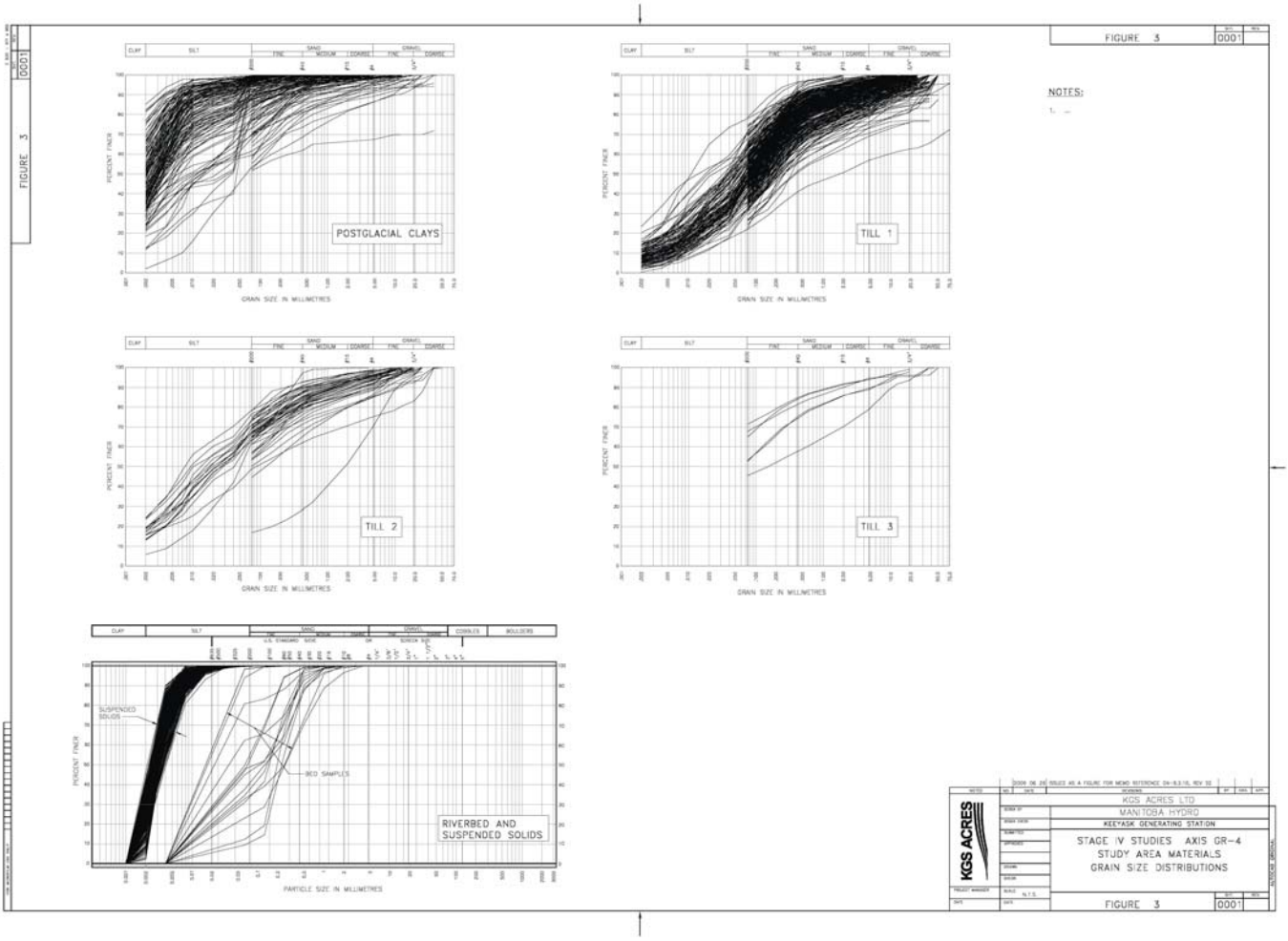
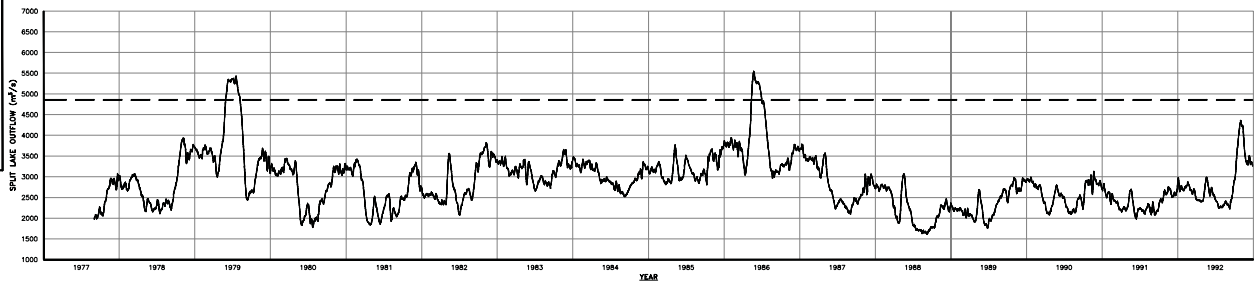


FIGURE 3
0001
CLAY
SILT
SAND
FINE
MEDIUM
COARSE
GRAVEL
FINE
COARSE
PERCENT FINER
GRAIN SIZE IN MILLIMETRES
POSTGLACIAL CLAYS
TILL 1
TILL 2
TILL 3
COBBLES
BOULDERS
U.S. STANDARD SIZE
OR
SCINCH SIZE
SUSPENDED SOLIDS
BED SAMPLES
RIVERBED AND SUSPENDED SOLIDS
PARTICLE SIZE IN MILLIMETRES
NOTES:
1. ...
FIGURE 3
0001
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
STUDY AREA MATERIALS
GRAIN SIZE DISTRIBUTIONS
FIGURE 3
0001

FIGURE 4 0001



DAILY FLOW > 4855 m³/s	
YEAR	DAYS (NO.)
1979	70
1979	70
1986	49
1996	53
1997	85
2001	35
2005	188
2006	54
2007	11
2008	51

2009 06 29 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-8.2.10, REV 02

KGS ACRES LTD

MANITOBA HYDRO

KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
NELSON RIVER
SPLIT LAKE OUTFLOWS
YEARS 1977 TO 2008

SCALE AS SHOWN

FIGURE 4 0001

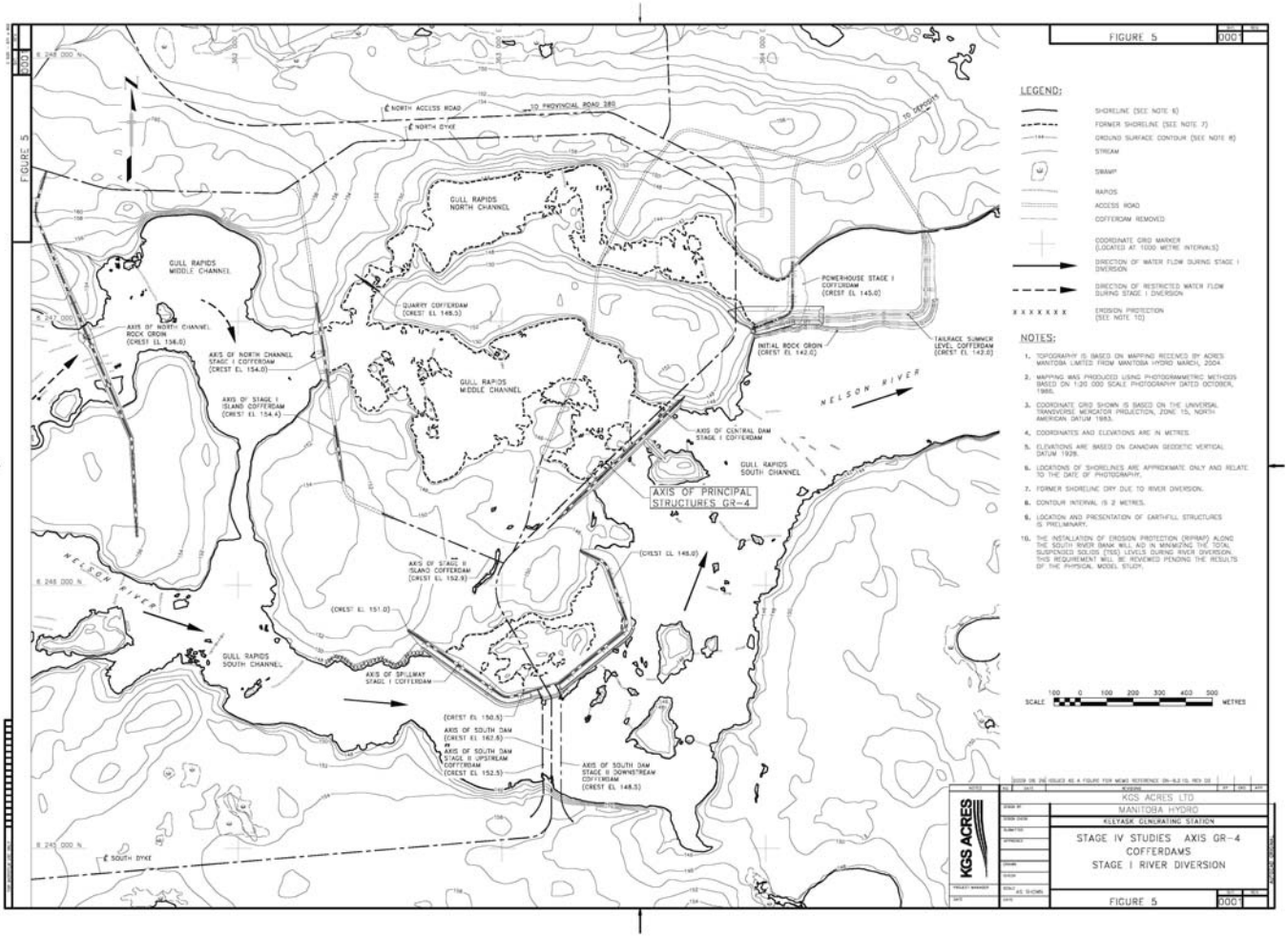


FIGURE 5
LEGEND:
SHORELINE (SEE NOTE 6)
FORMER SHORELINE (SEE NOTE 7)
GROUND SURFACE CONTOUR (SEE NOTE 8)
STREAM
SWAMP
RAPIDS
ACCESS ROAD
COFFERDAM REMOVED
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
DIRECTION OF WATER FLOW DURING STAGE I DIVERSION
DIRECTION OF RESTRICTED WATER FLOW DURING STAGE I DIVERSION
EROSION PROTECTION (SEE NOTE 10)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
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5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. FORMER SHORELINE DRY DUE TO RIVER DIVERSION.
8. CONTOUR INTERVAL IS 2 METRES.
9. LOCATION AND PRESENTATION OF EARTHFILL STRUCTURES IS PRELIMINARY.
10. THE INSTALLATION OF EROSION PROTECTION (RIPRAP) ALONG THE SOUTH RIVER BANK WILL AID IN MINIMIZING THE TOTAL SUSPENDED SOLIDS (TSS) LEVELS DURING RIVER DIVERSION. THIS REQUIREMENT WILL BE REVIEWED PENDING THE RESULTS OF THE PHYSICAL MODEL STUDY.
SCALE 100 0 100 200 300 400 500 METRES
GULL RAPIDS NORTH CHANNEL
GULL RAPIDS MIDDLE CHANNEL
GULL RAPIDS SOUTH CHANNEL
NELSON RIVER
AXIS OF PRINCIPAL STRUCTURES GR-4
℄ NORTH ACCESS ROAD
℄ NORTH DYKE
℄ SOUTH DYKE
TO PROVINCIAL ROAD 280
TO GILLAM
QUARRY COFFERDAM (CREST EL 148.5)
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.0)
INITIAL ROCK GROIN (CREST EL 142.0)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 142.0)
AXIS OF NORTH CHANNEL ROCK GROIN (CREST EL 136.0)
AXIS OF NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
AXIS OF STAGE I ISLAND COFFERDAM (CREST EL 154.4)
AXIS OF CENTRAL DAM STAGE I COFFERDAM
(CREST EL 146.0)
AXIS OF STAGE II ISLAND COFFERDAM (CREST EL 152.5)
(CREST EL 151.0)
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
AXIS OF SOUTH DAM (CREST EL 162.5) AND AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 152.5)
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
KGS ACRES LTD
MANITOBA HYDRO
KELSEY GENERATING STATION
STAGE IV STUDIES AXIS GR-4 COFFERDAMS STAGE I RIVER DIVERSION
FIGURE 5
000

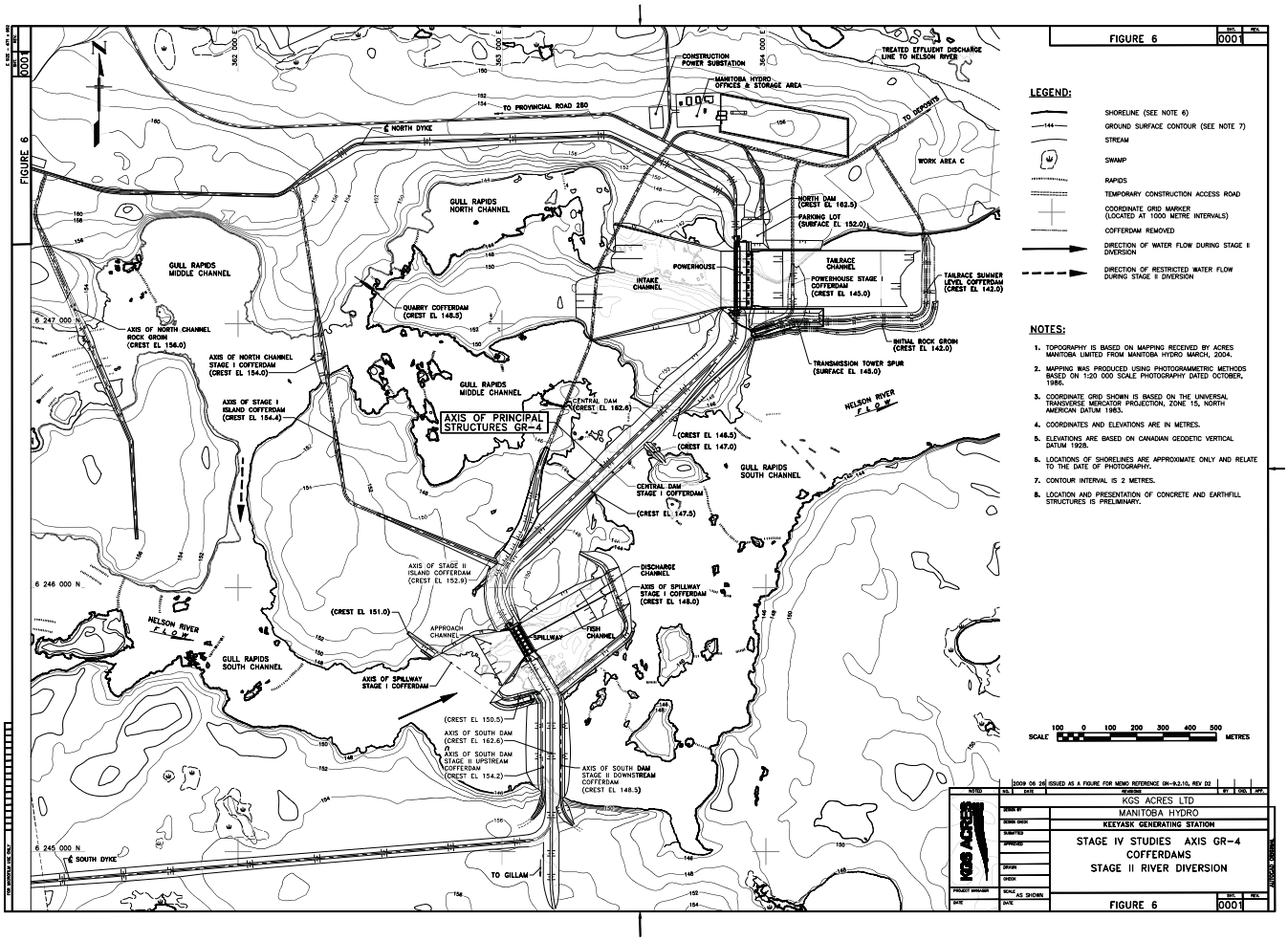
FIGURE 6
0001
LEGEND:
SHORELINE (SEE NOTE 6)
144 GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
TEMPORARY CONSTRUCTION ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
DIRECTION OF WATER FLOW DURING STAGE II DIVERSION
DIRECTION OF RESTRICTED WATER FLOW DURING STAGE II DIVERSION
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
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4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CANADIAN GEODETIC VERTICAL DATUM 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
SCALE 100 0 100 200 300 400 500 METRES
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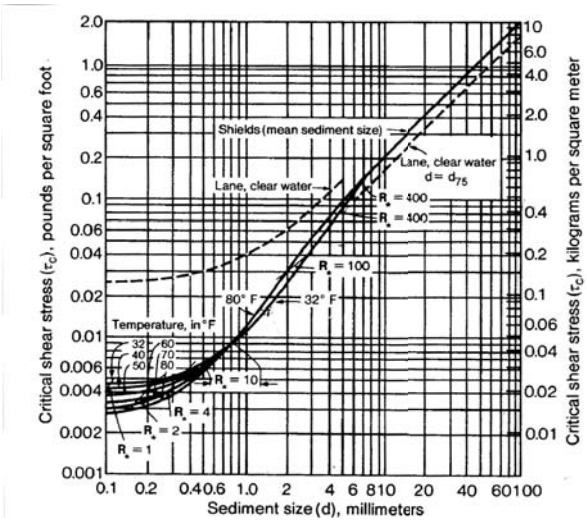
MANITOBA HYDRO
KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4 COFFERDAMS
STAGE II RIVER DIVERSION
FIGURE 6
0001
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO OFFICES & STORAGE AREA
TREATED EFFLUENT DISCHARGE LINE TO NELSON RIVER
TO PROVINCIAL ROAD 280
TO DEPOSITS
WORK AREA C
℄ NORTH DYKE
GULL RAPIDS NORTH CHANNEL
GULL RAPIDS MIDDLE CHANNEL
NORTH DAM (CREST EL 162.5)
PARKING LOT (SURFACE EL 152.0)
TAILRACE CHANNEL
POWERHOUSE
INTAKE CHANNEL
POWERHOUSE STAGE I COFFERDAM (CREST EL 145.0)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL 142.0)
QUARRY COFFERDAM (CREST EL 148.5)
INITIAL ROCK GROIN (CREST EL 142.0)
TRANSMISSION TOWER SPUR (SURFACE EL 148.0)
AXIS OF NORTH CHANNEL ROCK GROIN (CREST EL 156.0)
AXIS OF NORTH CHANNEL STAGE I COFFERDAM (CREST EL 154.0)
AXIS OF STAGE I ISLAND COFFERDAM (CREST EL 154.4)
GULL RAPIDS MIDDLE CHANNEL
AXIS OF PRINCIPAL STRUCTURES GR-4
CENTRAL DAM (CREST EL 162.6)
NELSON RIVER FLOW
(CREST EL 148.5)
(CREST EL 147.0)
GULL RAPIDS SOUTH CHANNEL
CENTRAL DAM STAGE I COFFERDAM
(CREST EL 147.5)
AXIS OF STAGE II ISLAND COFFERDAM (CREST EL 152.8)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM (CREST EL 148.0)
(CREST EL 151.0)
APPROACH CHANNEL
SPILLWAY
FISH CHANNEL
NELSON RIVER FLOW
GULL RAPIDS SOUTH CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL 150.5)
AXIS OF SOUTH DAM (CREST EL 162.6)
AXIS OF SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL 154.2)
AXIS OF SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL 148.5)
℄ SOUTH DYKE
TO GILLAM
6 247 000 N
6 246 000 N
6 245 000 N
302 000 E
303 000 E
304 000 E

FIGURE 7 0001



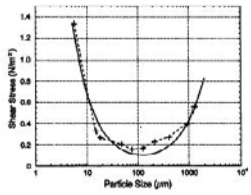


Fig. 4. Comparison of theory and experimental results for critical shear stress as function of particle size for bulk density of 1.83 kg/L; solid line is theory, while experimental results are shown as dashed line

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SUBMITTED	KEEYASK GENERATING STATION
APPROVED	STAGE IV STUDIES AXIS GR-4 COHESIONLESS MATERIALS CRITICAL SHEAR STRESS VS PARTICLE DIAMETER
DRAWN	
CHECK	
PROJECT MANAGER	SCALE N.T.S.
DATE	DATE

FIGURE 7 0001

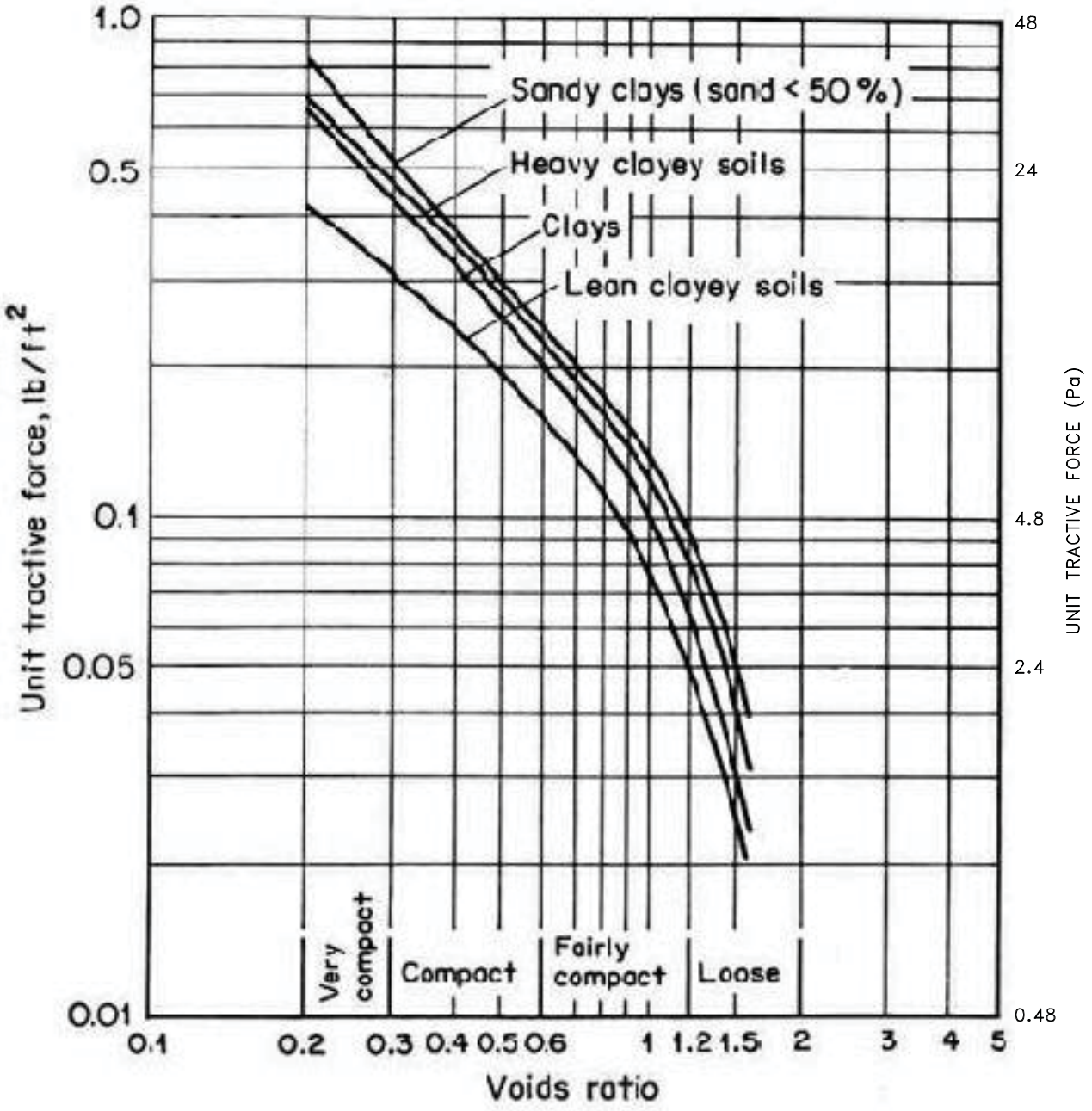

FIGURE 8 | SHT. 0001 | REV.

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STAGE IV STUDIES AXIS GR-4
PERMISSIBLE
TRACTIVE FORCES FOR CANALS
IN COHESIVE MATERIAL

DESIGN BY	
DESIGN CHECK	
SUBMITTED	
APPROVED	
DRAWN	
CHECK	
PROJECT MANAGER	SCALE
DATE	DATE

FIGURE 8 | SHT. 0001 | REV.

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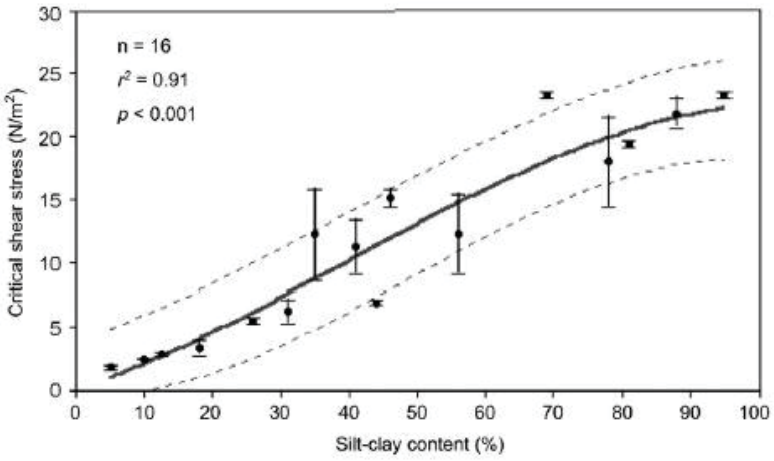

Fig. 4. Critical shear stress estimate from silt–clay percentage (data from Dunn, 1959). Each point represents the average critical shear stress for a soil sample with a specified silt–clay percentage. The average value was derived from 2 to 4 tests on each soil sample. Error bars represent the maximum and minimum values of the tests for each soil sample. The dashed lines are 95% confidence intervals for the rating curve.

NOTE:

1. CLAY SIZE IS DEFINED AS 0.063 mm.

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STAGE IV STUDIES AXIS GR-4
CRITICAL SHEAR STRESS VS
SILT-CLAY CONTENT

FIGURE 9

SHT. 0001 REV.

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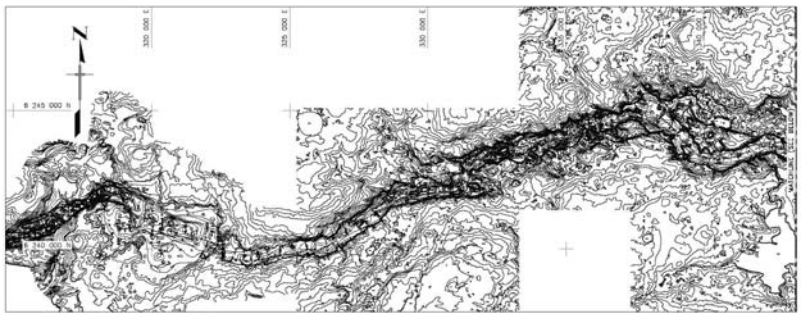
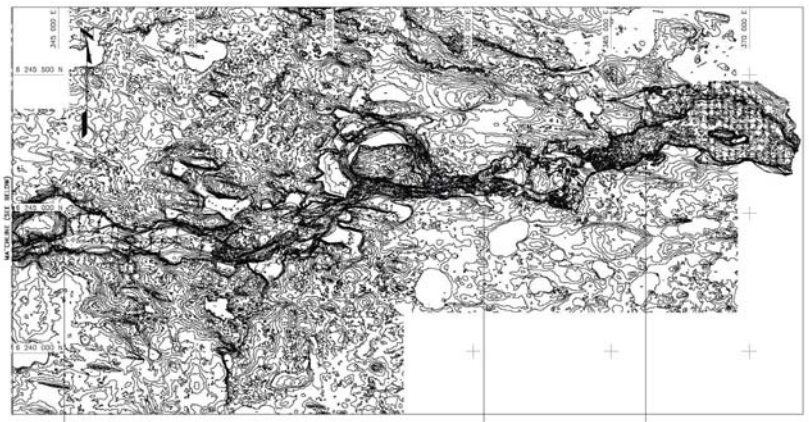
FIGURE 10 0001





GULL LAKE | GULL RAPIDS | STEPHENS LAKE

NOTE:

1. INFORMATION TAKEN FROM "2005 KEEYASK TOPOGRAPHIC & BATHYMETRIC COMPOSITE NAD83 Z15 REV. 7".

THIS FIGURE IS FOR RECORD PURPOSES. IT IS NOT INTENDED TO BE LEGIBLE IN HARD COPY.

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STAGE IV STUDIES AXIS GR-4
HQ1Γ HYDRAULIC MODEL
BASE TOPOGRAPHY

FIGURE 10 0001

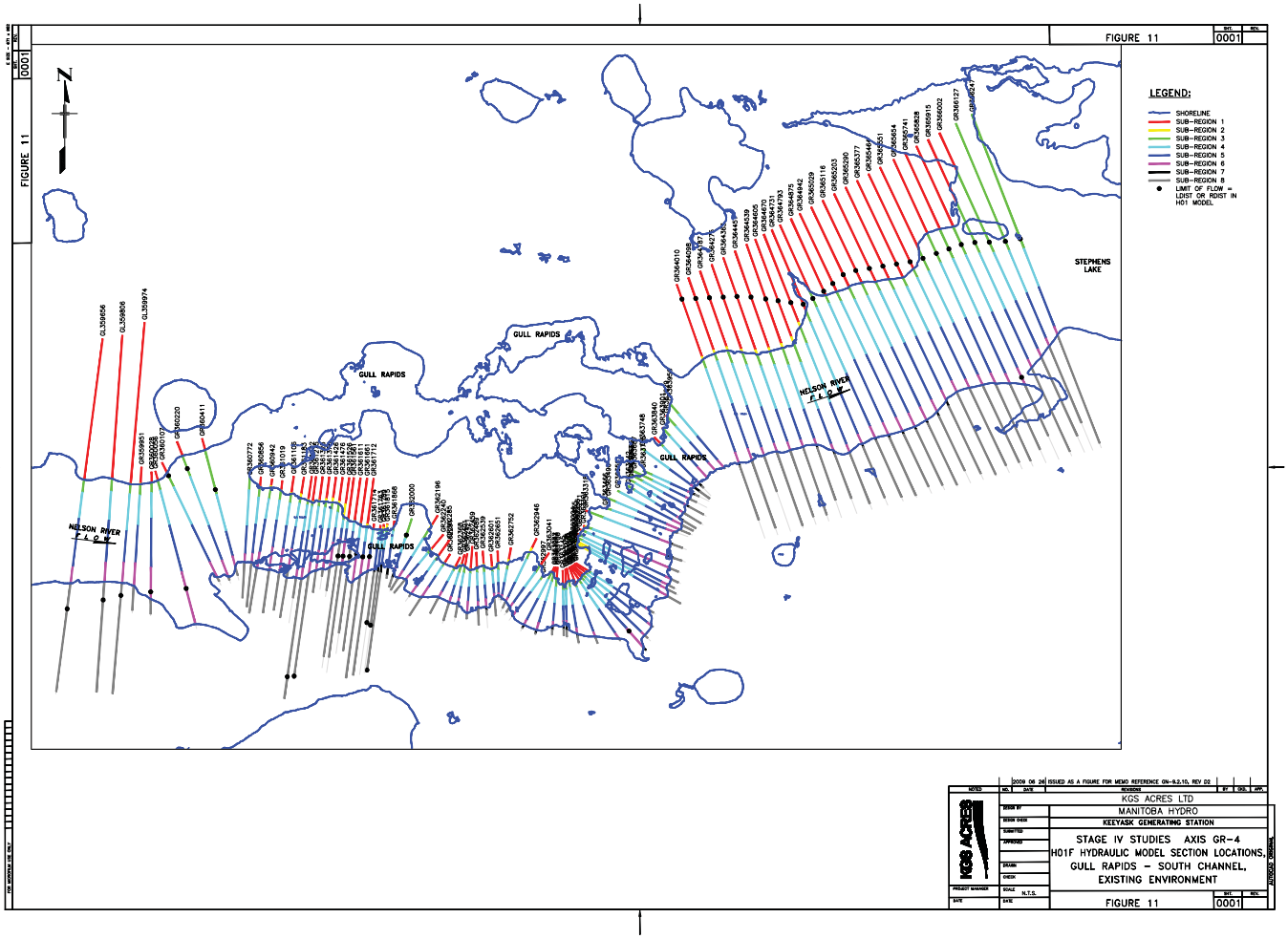
FIGURE 11
0001
N
LEGEND:
SHORELINE
SUB-REGION 1
SUB-REGION 2
SUB-REGION 3
SUB-REGION 4
SUB-REGION 5
SUB-REGION 6
SUB-REGION 7
SUB-REGION 8
LIMIT OF FLOW = LDIST OR RDIST IN H01 MODEL
GULL RAPIDS
STEPHENS LAKE
NELSON RIVER FLOW
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STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL SECTION LOCATIONS,
GULL RAPIDS - SOUTH CHANNEL,
EXISTING ENVIRONMENT
N.T.S.
FIGURE 11
0001

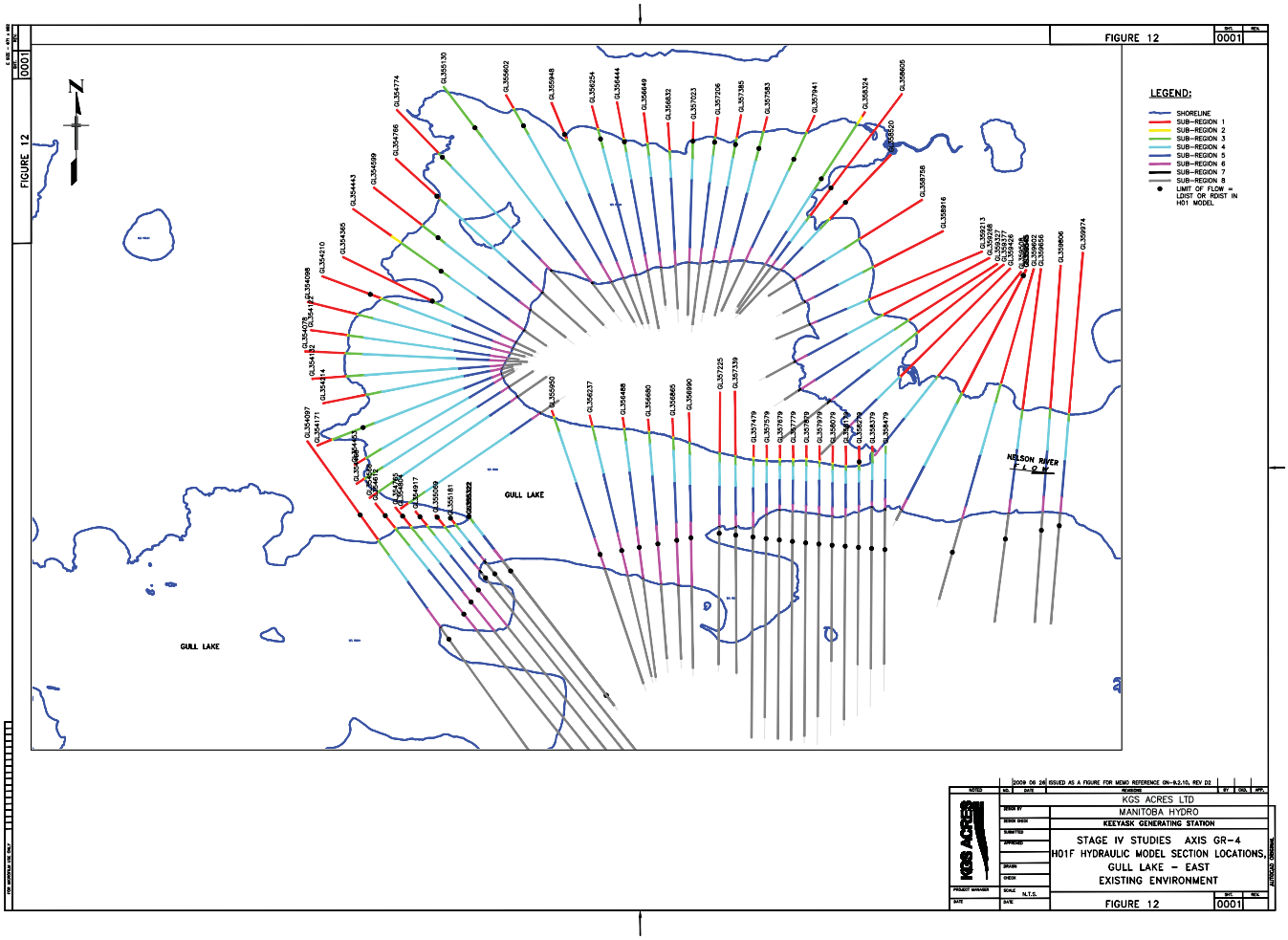
FIGURE 12
0001
N
LEGEND:
SHORELINE
SUB-REGION 1
SUB-REGION 2
SUB-REGION 3
SUB-REGION 4
SUB-REGION 5
SUB-REGION 6
SUB-REGION 7
SUB-REGION 8
LIMIT OF FLOW = LDIST OR RDIST IN H01 MODEL
GULL LAKE
GULL LAKE
NELSON RIVER
FLOW
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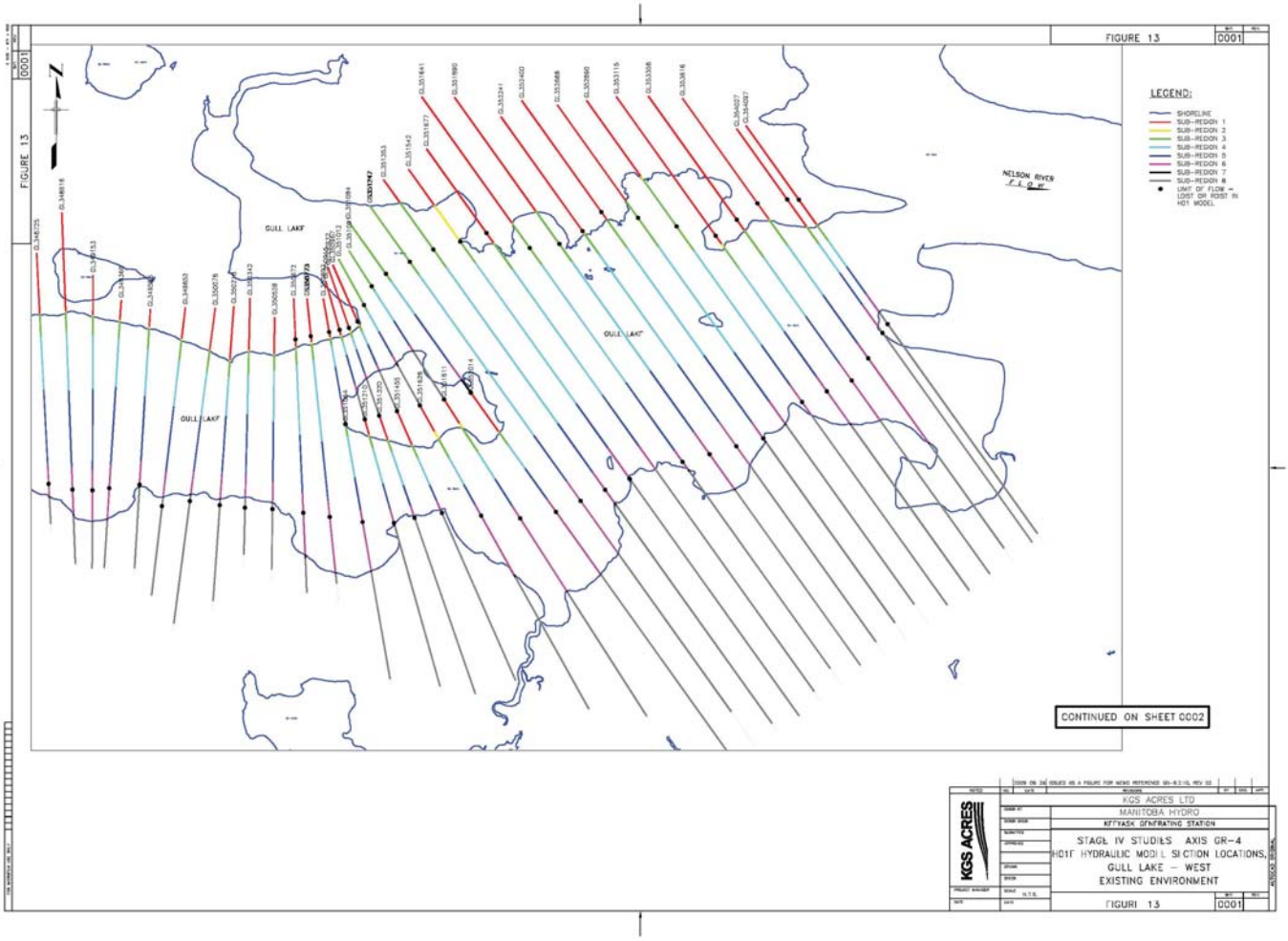
STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL SECTION LOCATIONS,
GULL LAKE - EAST
EXISTING ENVIRONMENT
FIGURE 12
0001
N.T.S.



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KETTASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
HD1F HYDRAULIC MODEL SECTION LOCATIONS,
GULL LAKE – WEST
EXISTING ENVIRONMENT

FIGURE 13 0001

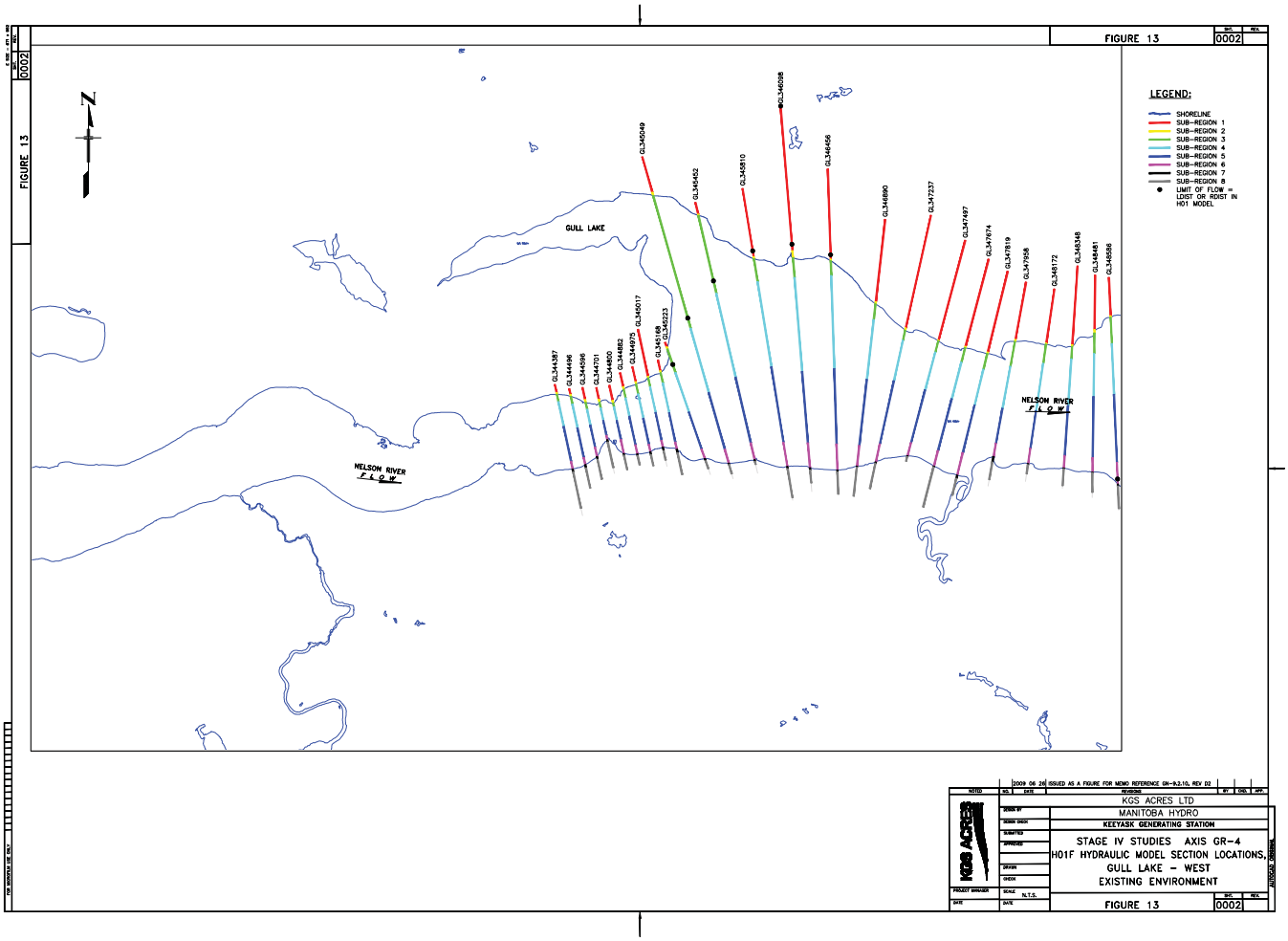
FIGURE 13
0002
LEGEND:
SHORELINE
SUB-REGION 1
SUB-REGION 2
SUB-REGION 3
SUB-REGION 4
SUB-REGION 5
SUB-REGION 6
SUB-REGION 7
SUB-REGION 8
LIMIT OF FLOW = LDIST OR RDIST IN H01 MODEL
GULL LAKE
NELSON RIVER
F L O W
NELSON RIVER
F L O W
GL345049
GL345452
GL345810
GL346098
GL346456
GL346890
GL347237
GL347497
GL347674
GL347819
GL347958
GL348172
GL348348
GL348461
GL348586
GL344387
GL344496
GL344596
GL344701
GL344800
GL344882
GL344975
GL345017
GL345168
GL345233
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STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL SECTION LOCATIONS,
GULL LAKE - WEST
EXISTING ENVIRONMENT
FIGURE 13
0002
N.T.S.

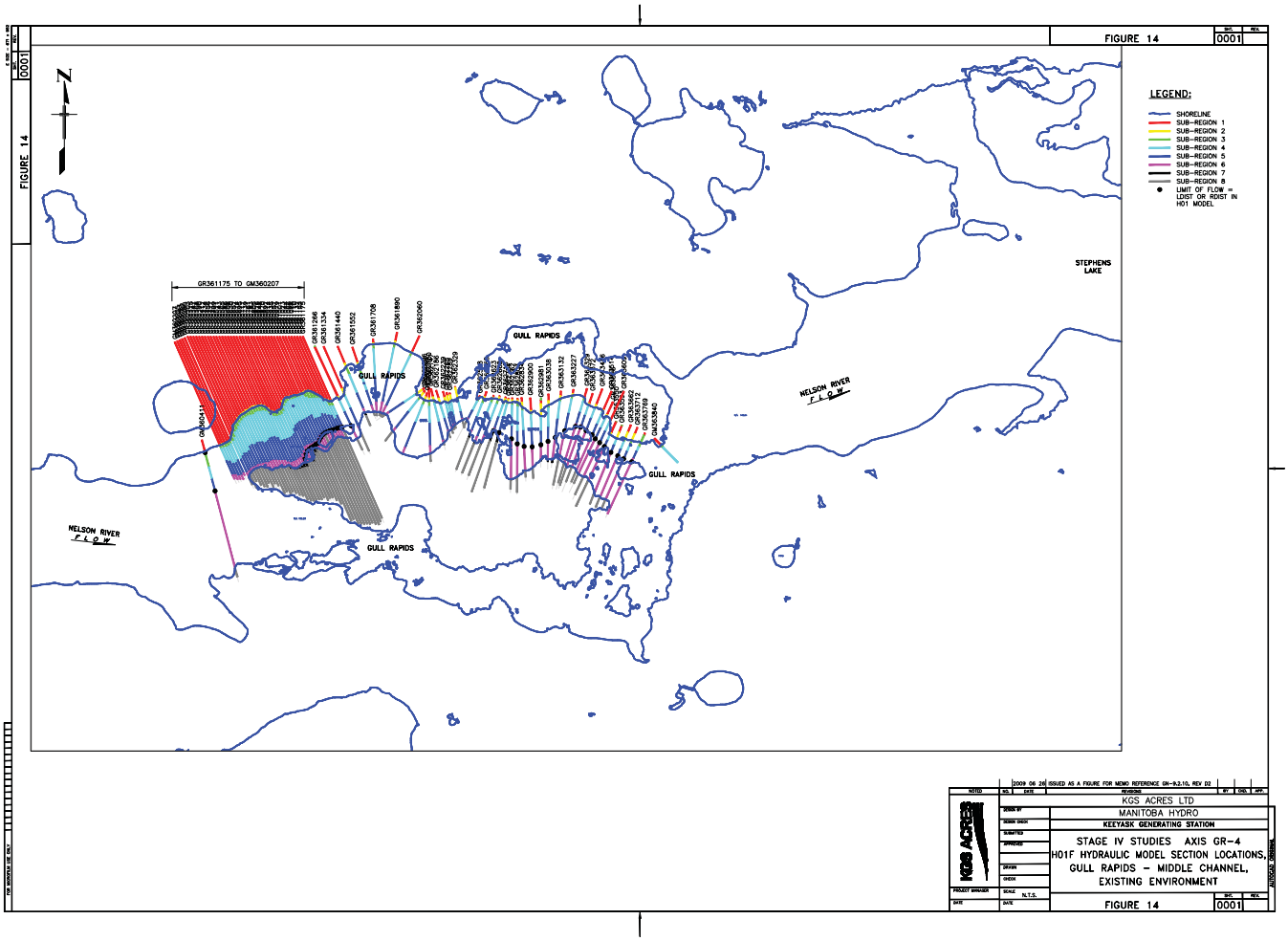
FIGURE 14
0001
N
LEGEND:
SHORELINE
SUB-REGION 1
SUB-REGION 2
SUB-REGION 3
SUB-REGION 4
SUB-REGION 5
SUB-REGION 6
SUB-REGION 7
SUB-REGION 8
LIMIT OF FLOW = LDIST OR RDIST IN H01 MODEL
GR361175 TO GM360207
GULL RAPIDS
GULL RAPIDS
GULL RAPIDS
GULL RAPIDS
STEPHENS LAKE
WELSON RIVER
flow
WELSON RIVER
flow
2009 06 26 ISSUED AS A FIGURE FOR MEMO REFERENCE GH-9.2.10, REV 02
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KEEYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL SECTION LOCATIONS,
GULL RAPIDS - MIDDLE CHANNEL,
EXISTING ENVIRONMENT
N.T.S.
FIGURE 14
0001

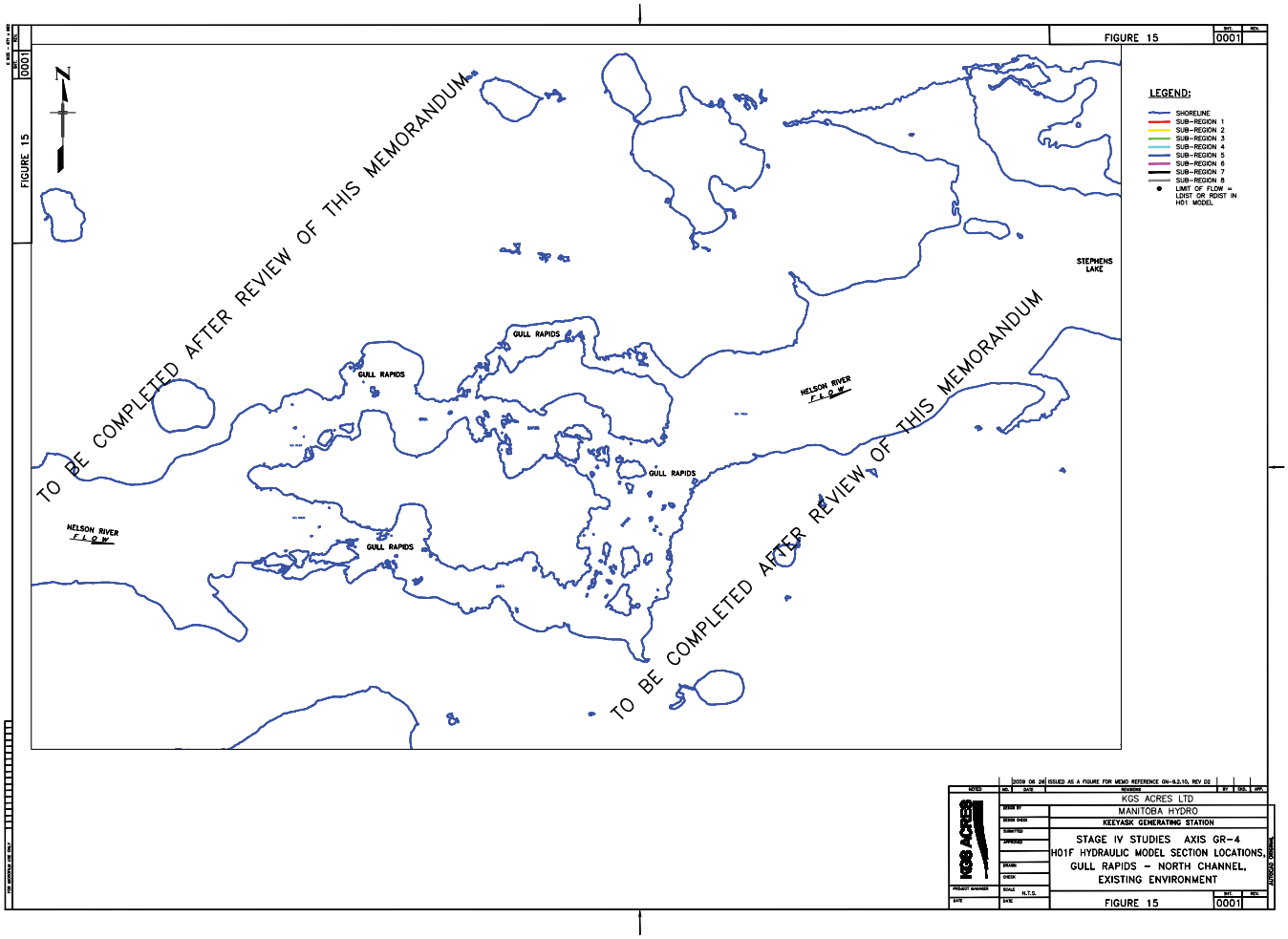
FIGURE 15
0001
N
LEGEND:
SHORELINE
SUB-REGION 1
SUB-REGION 2
SUB-REGION 3
SUB-REGION 4
SUB-REGION 5
SUB-REGION 6
SUB-REGION 7
SUB-REGION 8
LIMIT OF FLOW = LDIST OR RDIST IN H01 MODEL
TO BE COMPLETED AFTER REVIEW OF THIS MEMORANDUM
TO BE COMPLETED AFTER REVIEW OF THIS MEMORANDUM
STEPHENS LAKE
GULL RAPIDS
GULL RAPIDS
GULL RAPIDS
GULL RAPIDS
NELSON RIVER FLOW
NELSON RIVER FLOW
2009 08 28 ISSUED AS A FIGURE FOR MEMO REFERENCE GN-9.3.10, REV 02
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STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL SECTION LOCATIONS,
GULL RAPIDS - NORTH CHANNEL,
EXISTING ENVIRONMENT
N.T.S.
FIGURE 15
0001

FIGURE 16 0001

VIEW OF SHORELINE

FIGURE 16 0001

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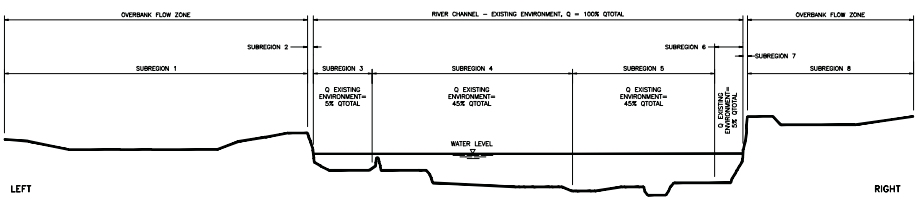
KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
SHORELINE & OVERBANK FLOW AREA DEFINITION

FIGURE 16 0001

N.T.S.

FIGURE 17 0001



TYPICAL HYDRAULIC SECTION – LOOKING DOWNSTREAM

NOTES:

1. MAXIMUM HISTORIC SHORELINE IS THE VISIBLE EXTENT OF HISTORIC WATER AND ICE REGIME EFFECTS ON UPLAND AND PEAT LAND HABITAT (ECOSTEM).
2. n–VALUES ASSUME CLEARING HAS BEEN COMPLETED.

HYDRAULIC MODEL n–VALUE SUBTYPES

	LOCATION	DESCRIPTION (SEE NOTE 1)	n–VALUE FOR EXISTING CONDITIONS	n–VALUE DURING CONSTRUCTION
1	OVERLAND FLOW – LEFT SIDE OF CHANNEL	AREA BEYOND THE MAXIMUM HISTORIC SHORELINE (IN THE UPLAND AREA)	0.09	0.06 (SEE NOTE 2)
2	OVERBANK FLOW – LEFT SIDE OF CHANNEL	AREA BETWEEN AVERAGE SHORELINE AND THE MAXIMUM HISTORIC SHORELINE	0.032	0.032
3	MAIN CHANNEL – ADJACENT TO AVERAGE SHORELINE	AREA BETWEEN THE AVERAGE SHORELINE FOR EXISTING CONDITIONS AND SUB–REGION 4	0.03	0.03
4	MAIN CHANNEL – LEFT CENTRAL PORTION	AREA CARRIES APPROXIMATELY 45% OF THE TOTAL FLOW FOR EXISTING CONDITIONS	0.03	0.03
5	MAIN CHANNEL – RIGHT CENTRAL PORTION	AREA CARRIES APPROXIMATELY 45% OF THE TOTAL FLOW FOR EXISTING CONDITIONS	0.03	0.03
6	MAIN CHANNEL – ADJACENT TO AVERAGE SHORELINE	AREA BETWEEN SUB–REGION 5 AND THE AVERAGE SHORELINE FOR EXISTING CONDITIONS	0.03	0.03
7	OVERBANK FLOW – RIGHT SIDE OF CHANNEL	AREA BETWEEN AVERAGE SHORELINE AND THE MAXIMUM HISTORIC SHORELINE	0.032	0.032
8	OVERLAND FLOW – RIGHT SIDE OF CHANNEL	AREA BEYOND THE SHORELINE (IN THE UPLAND AREA)	0.09	0.06 (SEE NOTE 2)

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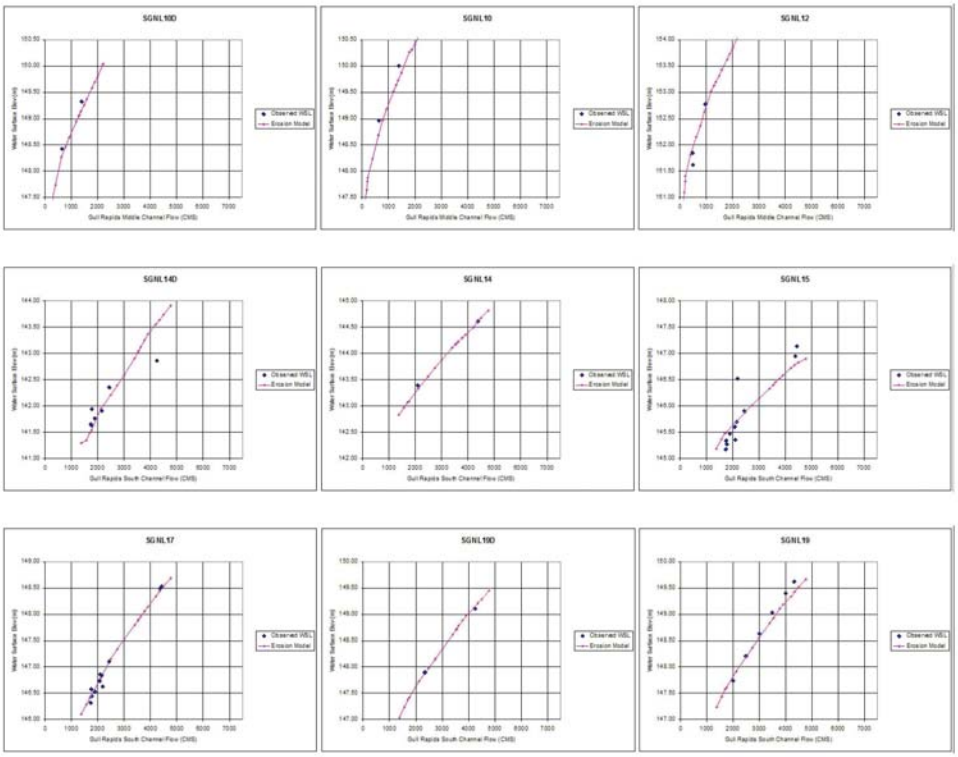
STAGE IV STUDIES AXIS GR–4
H01F HYDRAULIC MODEL
n–VALUE SUBTYPES

SCALE N.T.S.

FIGURE 17 0001

FIGURE 18 0001



CONTINUED ON SHEET 0002

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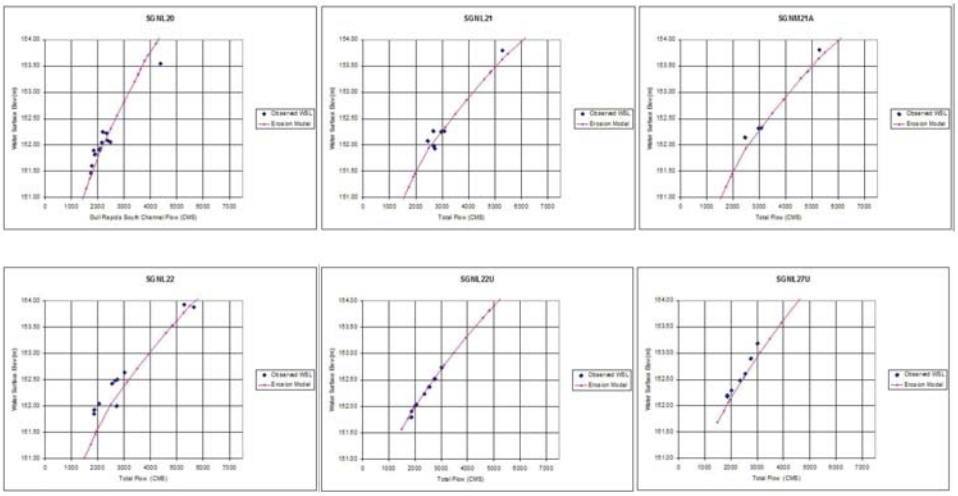
KELYASK GENERATING STATION
STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL
EXISTING ENVIRONMENT
CALIBRATION RESULTS
FIGURE 18 0001

FIGURE 18 0002



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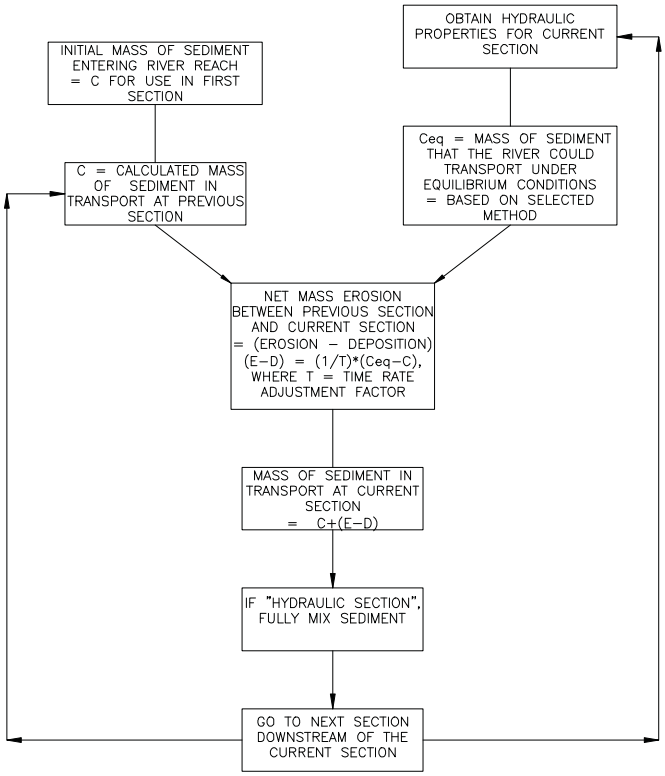
KELYASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
H01F HYDRAULIC MODEL
EXISTING ENVIRONMENT
CALIBRATION RESULTS

FIGURE 18 0002



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KEEYASK GENERATING STATION

STAGE IV STUDIES AXIS GR–4
SEDIMENT MODEL
PROGRAM FLOW CHART

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FIGURE 19 | SHT. 0001 | REV.

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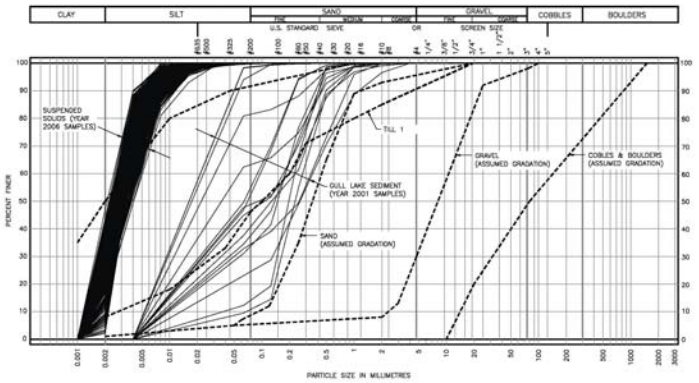
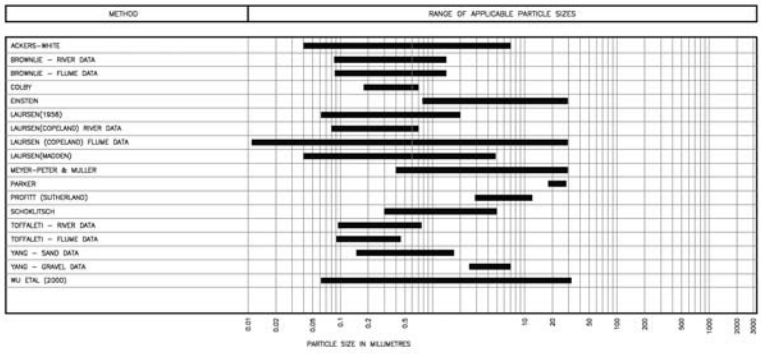
FIGURE 20 0001





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		MANITOBA HYDRO	
		KEEYASK GENERATING STATION	
		STAGE IV STUDIES AXIS GR-4 SEDIMENT TRANSPORT EQUATIONS RANGE OF APPLICABLE PARTICLE DIAMETERS	
		FIGURE 20	0001

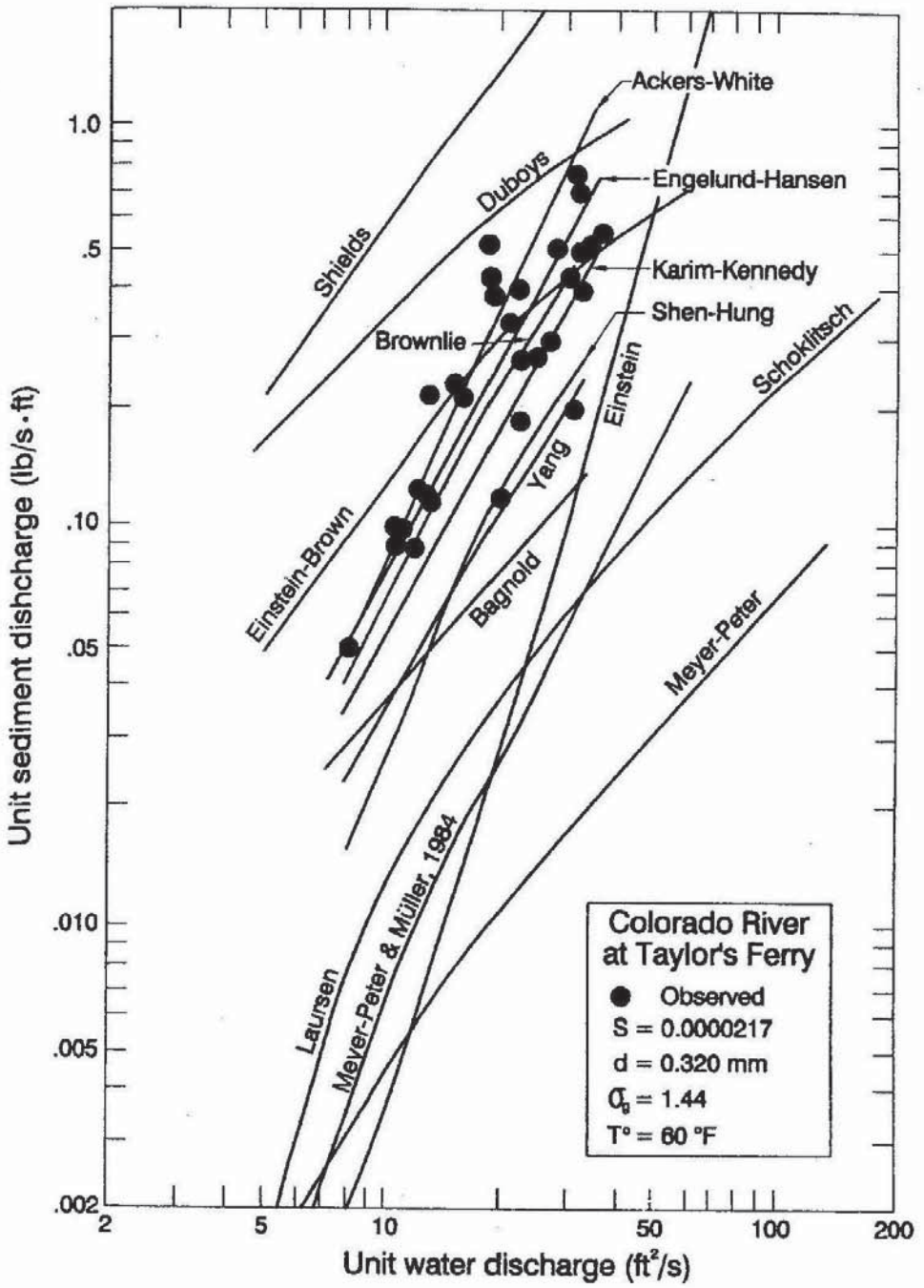
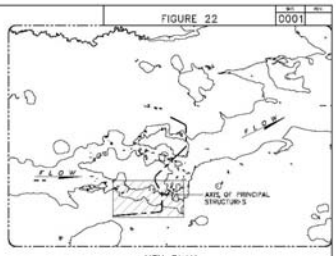


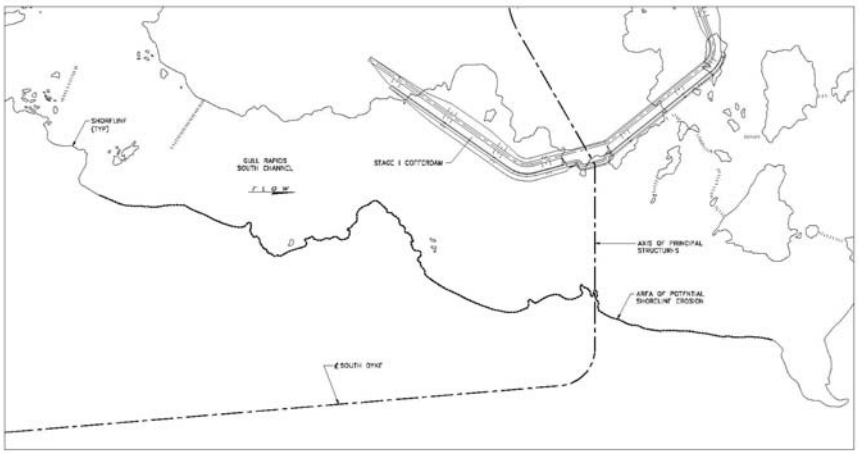
FIGURE 9.15 Measured total sediment discharge in Colorado River at Taylor's Ferry and comparison with several transport equations (*after Vanoni et al., 1960*).

FIGURE 22 0001

FIGURE 22 0001



KEY PLAN



PLAN

SCALE 50 0 50 100 150 200 250 300 METRES

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KGS ACRES LTD

MANITOBA HYDRO

KETTASK GENERATING STATION

STAGE IV STUDIES AXIS GR-4
SEDIMENT MODEL
STAGE I DIVERSION
AREA OF POTENTIAL EROSION

FIGURE 22 0001

AUTOCAD DRAWING

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-9.2.15

Rev. 0, May 31, 2013

Preliminary Consideration of Methods to Manage Sediment During Construction

Manitoba Hydro File 00195-11100-0188_03

Prepared by John Rogers (J. ROGERS) MAY 31, 2013
Hatch

Checked by P. Pantel (P. PANTEL) May 31, 2013
Hatch

Reviewed by I.R. Dewar (I. R. Dewar) May 31, 2013
Hatch

Approved by I.R. Dewar (I. R. Dewar) May 31, 2013
Hatch

Accepted by _____
Manitoba Hydro

Hatch
Winnipeg, Manitoba

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - PHYSICAL ENVIRONMENT

PRELIMINARY CONSIDERATION OF METHODS TO MANAGE SEDIMENT DURING CONSTRUCTION

REV 0

DELIVERABLE GN 9.2.15

MANITOBA HYDRO FILE: 00195-11100-0188_03

2013/05/31

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

HATCH LTD.

KEEYASK
GENERATION PROJECT

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GLOSSARY OF ABBREVIATIONS

DFO	Department of Fisheries and Oceans
EIS	Environmental Impact Statement
el	elevation
FSL	Full Supply Level
GS	Generating Station
km	kilometres
m	metre
mg/L	milligrams per litre
m/s	metres per second
PR	Provincial Road
PMF	Probable Maximum Flood
ROW	Right of Way
TSS	Total Suspended Solids

Keeyask GS – Stage IV Engineering

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References:

GN-9.2.15 Draft 1

**Keeyask GS EIS PD SV* http://keeyask.com/wp/wp-content/uploads/Keeyask_EIS-Project_Description_SV.pdf

1 INTRODUCTION

Originally intended as a primary reference guideline for contractors, this memorandum has become a secondary reference being replaced by environmental protection and management plans that have been, or are being, developed by Manitoba Hydro and which will describe methods and requirements for sediment management. The more recent documents may also include additional requirements for sediment management or drainage control plans. Environmental protection and sediment management plans identified for the project include:

"Keeyask Generation Project: Generating Station Construction Environmental Protection Plan"

"Keeyask Generation Project: South Access Road Construction Environmental Protection Plan"

"Keeyask Generation Project: In-stream Construction Sediment Management Plan"

"Keeyask Infrastructure Project: Construction Environmental Protection Plan"

The current intended use of this document is as a reference providing ***an overview of the various types of construction activities and methods of sediment management that will be or may be*** *undertaken in the construction of the Keeyask GS. The memorandum remains largely unchanged from its initial Draft 1 issue in 2009, and it contains some information that is out of date. Accordingly,* ***caution should be exercised*** *in extracting text or figures from the memorandum for use in other documentation. Since the initial draft was prepared in 2009, the development of the Keeyask project has advanced in many areas, including:*

- *Details pertaining to the project description, including the construction of infrastructure under the Keeyask Infrastructure Project, which involves construction of the North Access Road and Phase 1 construction camp before construction of the Keeyask GS begins.*
- *Details presented in most of the attached tables and figures.*
- *The selection of working areas and buffer zones around various structures.*
- *The identification of suitable excavated material placement areas.*
- *The identification of a number of mitigation initiatives associated with the construction of fish habitat and wetlands habitat.*
- *The development of a detailed in-stream TSS monitoring program.*

- *Revisions to the project construction schedule.*
- *Other details that typically evolve as a construction project moves forward.*

1.1 GENERAL

This report outlines preliminary considerations of methods to manage sediment during construction of the support facilities and the generating station at the Keeyask Site. This preliminary plan has been developed to minimize the impacts of construction activities on Total Suspended Solids (TSS) in the Nelson River and its contributory streams, to the extent practicable. The construction methods and final sediment management plans, including all required details, will be developed by the project owner and/or contractors that are awarded the various construction contracts.

The specifications for the construction of the support facilities and the generating station will require that the contractors meet the water quality target levels specified in the environmental protection and management plans, or as subsequently required by the project's license, and that the proposed final plans be approved by Manitoba Hydro and, if required, the applicable regulatory agencies, prior to the start of construction.

Environmental surveillance and monitoring during construction are fundamental components of the project, and as stated in the Environmental Impact Statement, Manitoba Hydro's Project Manager will be accountable for environmental protection compliance. Individual environmental inspectors will be assigned to monitor discrete areas of work. The Project Manager, together with the environmental inspectors, will monitor that the actual work carried out complies with the regulatory requirements, approval conditions, and the pending Environmental Protection Plan.

1.2 PROJECT DESCRIPTION

The Keeyask Generating Station Site (Keeyask Site) is located in the vicinity of Gull Rapids, on the Nelson River, approximately 30 km west of Gillam and 190 km northeast of Thompson, and 730 km north of Winnipeg, Manitoba (Figure 1).

The local area and general arrangement of the proposed Keeyask Generating Station (Keeyask GS) is shown in Figures 2 and 3. The layout corresponds to that proposed for Axis GR-4, with a Full Supply Level (FSL) at el 159.0 m. The main components of the Keeyask GS consist of two long river bank dykes (North and South Dykes) totaling 22.7 km in length which connect to the Principal Structures that span the Nelson River. These Principal Structures consist of a Powerhouse Complex constructed on the north side of the river that is connected to the North Dyke via the North Dam which has a length of 99 m. The Central Dam stretches 1,600 m and will connect the Spillway to the Powerhouse. The 565 m long South Dam closes off the river channel to the south

of the Spillway and connects to the South Dyke. Five retaining walls, Walls A to E, will also be constructed. In addition to these structures, channels will be excavated for the Spillway, Intakes, and Tailrace.

The infrastructure required to support the construction activities, shown in Figure 3, includes the North Access Road from PR 280 to the Keeyask Site, South Access Road from the Keeyask Site to Gillam, the Main Construction Camp (Main Camp), various on-site roads and work areas (Manitoba Hydro Work Area, Contractor's Work Area), a water supply system, sewage collection and treatment system, and electrical power supply.

In addition to the construction infrastructure, a series of cofferdams will be constructed in the Nelson River to divert the flow away from the in-stream construction area, in order to allow construction activities for the generating station to take place in-the-dry. The location of the cofferdams is shown in Figure 3.

1.3 CONSTRUCTION SEQUENCE[1]

NOTE:

The description of the construction sequence was based on the construction schedule as it was in 2009 and has not been updated, although a revised schedule from Apr 5, 2012 is shown in Figure 13. Please refer to the Keeyask EIS Project Description Supporting Volume (Sec. 3.1) for a more current description of the construction sequence. Since 2009 there was a substantial change in the construction sequence in that the North Access Road and phase one of the Keeyask GS construction camp are currently under construction (i.e., the Keeyask Infrastructure Project) and should be completed by the time Keeyask GS construction starts.

The plan is to construct the Keeyask GS over a period of approximately 7½ years, as shown in the Summary Project Schedule (Figure 13, Sheets 1 & 2). Assuming that construction starts early in the winter of Year 1, the sequence of activities can be summarized as follows:

1. Initial activities will be associated with the development of supporting infrastructure for the Project and will start with the construction of the North Access Road and a transmission line to supply construction power. The proposed all-weather access road to the site will pass over extensive granular deposits and near two granular borrow sources, which could be utilized for the construction of the North Access Road, the Main Camp and the Work Area. At this early stage of construction it is planned that workers will be housed in one of two commercial Starter Camps.
2. By March 1 of Year 1, a winter trail to the site will have been constructed, a "Startup Camp" installed, and the first phase of site clearing commenced. Water for

1 Since the in-service date for the Project may be delayed for a number of reasons, specific dates have been replaced with Year 1, Year 2, Year 3, and so on. Using this nomenclature, construction commences in Year 1, and December 21st of Year 6 is the date that the first unit is scheduled to go on-line.

the Starter Camp is planned to be sourced from an existing well located within Deposit G-1. Phase I of the Main Construction Camp construction will also begin with a proposed capacity of 500 persons.

3. During the spring and summer of Year 1, Main Camp Phase I construction will continue, work on the North Access Road and camp site infrastructure will also continue, with a "fair weather" pilot road having been completed.

4. Mobilization of the Ice Boom Contractor and the Stage I Cofferdam Contractor will begin early in the summer of Year 1.

5. The construction power support infrastructure will be completed by the end of the summer of Year 1. The Quarry Cofferdam will also be completed by this time. Once access is obtained, excavation of Quarry Q-7 will begin and materials will be placed for the North Channel Rock Groin and North Channel Stage I Cofferdam. By late summer of Year 1, the entire flow of the Nelson River will be directed through the South Channel of the Nelson River.

6. By the end of Year 1, the North Access Road will be near completion and the installation of the Main Camp Phase I buildings will also have been completed. Upon completion of the Main Camp Phase I, Phase II will begin with a proposed capacity of 1,500 units. It is planned that the Main Camp will source its water from the fresh water intake located within the North Channel Rock Groin. The Ice Boom is also scheduled to be completed by the fall of Year 1 in preparation for the first ice season.

7. Activities associated with the Main Construction Camp and Manitoba Hydro Work Area will be ongoing during this period and will consist of local roadwork and site grading, construction of water and sewer services, installation of works and camp buildings, communications system, and a power distribution system for the site infrastructure.

8. Year 2 will see the continuation of construction on the Main Construction Camp as well as the completion of the Island, Powerhouse and Spillway Stage I Cofferdams. Initial earthworks and excavation will also begin over the summer of Year 2 and continue for the next five years.

9. In the summer of Year 2 the General Civil Contractor will mobilize to site.

10. The Main Camp will be completed by the spring of Year 3 in time for First Concrete. The final Stage I Cofferdam, the Central Dam Stage I Downstream Cofferdam, is scheduled to be complete in the summer of Year 3 by the General Civil Contractor; by this time the Stage I Cofferdam Contractor will have completed its scope of work and demobilized.

11. Beginning in the summer of Year 3, activities undertaken will be associated with manufacturing and placement of concrete for the Principal Structures. Work will

begin on the Spillway, Intakes and Powerhouse, Service Bay, Transition Structures, Walls and Transmission Tower Spur. Dyke construction will also commence in this time and continue for the next four years.

12. In Year 4, work will continue on the Principal Structures. The Spillway structure's concrete is scheduled for completion to its Stage II Diversion configuration by the summer.

13. By the fall of Year 4, work will begin on the Powerhouse and Service Bay superstructures and the Powerhouse Crane will be operational. As the Powerhouse and Service Bay are being enclosed, the Electrical and Mechanical Installation Contractor will start the installation of the electrical and mechanical systems. Work will also begin on installing the Turbines and Generators. In preparation for Stage II Diversion in the following year, work will start on the Central Dam and Stage II Cofferdams adjacent to the Spillway. A short portion of the Stage II Island Cofferdam will also be constructed in-the-dry. The Spillway Gates will be completed by the spring of Year 5.

14. Major activities that would be accomplished in the summer of Year 5 include; the continuation of the forebay clearing, Tailrace Channel excavation, completion of the Central Dam, and finally removal of those portions of the Stage I Spillway Cofferdam required for Stage II River Diversion through the partially completed Spillway. Stage II Diversion will begin in late summer of Year 5. Upon River Closure, via construction of the South Dam Stage II Upstream Cofferdam, the North and South Dykes will be linked and the road access link between PR 280 and Gillam will be complete.

15. Year 6 will see the completion of the South Dam as well as the North and South Dykes completing the necessary closure before Forebay Impoundment. Any remaining forebay clearing will be completed as well. North Channel Stage I Cofferdam, North Channel Rock Groin and the Stage I Island Cofferdam will be lowered for forebay improvements, and the Powerhouse and Spillway Cofferdams will be removed.

16. Later in the summer of Year 6, the impoundment of the forebay will commence and progress through to the fall of Year 7. The Tailrace Channel Cofferdam will also be removed and preparations for First Power will begin. Concreting of the Spillway Final Rollways will begin and continue into Year 7.

17. The Spillway Final Rollways will be completed in Year 7. As well, Units 1, 2, 3, 4, and 5 will all have gone on-line by the end of Year 7.

18. In the final year of construction (Year 8), key activities will include the completion and commissioning of electrical and mechanical systems installations, final assembly and commissioning of the last turbines and generators (Units 6 and 7).

19. Upon completion of construction of the generating station in the summer of Year 8, all buildings and temporary infrastructure will be removed from the site, and the site will be decommissioned, including the reclamation of all temporary roads and borrow areas that will not be required for the operation of the generating station. This is expected to be completed by the summer of Year 8.

The schedule contained herein, shown in Figure 13, as well as the dates referred within the text of this document are current to the date of issue for this document. The construction sequence, methodology and durations assigned to activities are considered to be the most likely given the expected conditions for the Project; however, they are likely to change if unexpected conditions are encountered.

2 POTENTIAL SOURCES OF SEDIMENT

2.1 SEDIMENT MANAGEMENT MEASURES

2.1.1 GENERAL

The planning, design, scheduling of work, and methods of construction for the Keeyask GS have been based on a pro-active rather than a reactive approach to sediment control. The following two objectives are fundamental to this approach:

1. Minimize the amount of soil erosion that may occur due to construction activities.
2. Minimize the amount of sediment entering watercourses as a result of construction activities.

These two objectives are closely linked, with the first objective considered to be of prime importance in meeting the requirements of the second.

Various alternatives have been considered in the design and proposed construction methods for the Keeyask GS and the construction support facilities. The final design and construction methods that will be used at Keeyask GS consist of best management practices that balance impacts to the environment and costs. The primary and secondary mitigation strategies to minimize the amount of sediment entering watercourses as a result of construction activities are presented in the following sections.

The strategies employed during construction may vary from the preliminary strategies that are discussed below, as the actual construction methods and final sediment management plan will be developed by the various contractors involved in the construction of the site infrastructure and the generating station.

2.1.2 PRIMARY MITIGATION STRATEGIES

The primary mitigation strategies employed in the planning, design, and construction of Keeyask GS are intended to minimize the amount of soil erosion that may occur due to construction activities and to minimize the amount of sediment that may enter a watercourse.

General measures employed to control soil erosion include:

- Minimizing the area disturbed by construction activities.
- Management of surface water flow and outflow from or within site activities.
- Locating facilities to suit topography and soil conditions.
- Providing for the diversion of surface runoff away from disturbed areas.
- Providing adequate ditches.
- Providing erosion protection.

General measures employed to minimize the release of sediment into a watercourse include:

- Providing a buffer zone of undisturbed vegetation adjacent to water courses.
- Carefully controlling in-stream work to minimize stream disturbance.
- Adopting construction methods and sequences of in-stream activities to minimize the release of sediment into the main flow.
- Controlling the quantity of sediment contained in surface runoff by utilizing sediment settling ponds, silt fences, straw waddles, erosion control blankets, re-vegetation, etc.

The primary mitigation methods that will be employed to control erosion and minimize the release of sediment into a watercourse for the on-land construction activities for the Keeyask GS are believed to be widely accepted best management practices and based on the "Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat" [Ref 1] and "Manual of Erosion and Sedimentation Control During Highway Construction" by Manitoba Infrastructure and Transportation [Ref 2], supplemented by "Land Development Guidelines for the Protection of Aquatic Habitat" by the Department of Fisheries and Oceans [Ref 3].

The primary mitigation methods that will be employed to control erosion and minimize the release of sediment into a watercourse for the in-stream construction activities for the Keeyask GS include components incorporated in the design of the structure (e.g., cross-section adopted), method of construction, and sequence of construction.

The primary mitigation methods for both on-land and in-stream construction activities are discussed in detail in Sections 2.2 to 2.4.

2.1.3 SECONDARY MITIGATION STRATEGIES

The mitigation methods that will be employed to control erosion and minimize the release of sediment into a watercourse for the on-land construction activities for the Keeyask GS, are based on best management practices to effectively manage this issue. In addition, any requirement or additional means to further reduce erosion can be readily implemented. Consequently, no secondary mitigation methods have been identified for the on-land construction activities.

The primary mitigation methods that will be employed to control erosion and minimize the release of sediment into a watercourse for the in-stream construction activities for the Keeyask GS have been developed on a site-specific basis. Uncertainties exist in the amount of riverbed sediment that may be re-suspended and the amount of fine material that may be washed away from the impervious fill, granular transition fill, and rockfill during cofferdam fill placement/removal in-the-wet. Due to these uncertainties, secondary mitigation strategies have been developed for in-stream construction activities in order to further mitigate sediment release into a watercourse, should the need arise.

The possible secondary mitigation strategies that may be employed to minimize the release of sediment into a watercourse for the in-stream construction activities for the Keeyask GS consist of installing a turbidity or silt curtain and/or slowing down or temporarily stopping construction activities to reduce the sediment concentrations in the water. The selection of the most practical secondary mitigation strategy will depend on the type of construction activity, when and where it is occurring, and local water conditions in the area of the activity.

In-stream construction activities are discussed in Subsection 2.3. The discussion contained therein is related to individual activities and their potential impact. There are a number of these activities that occur concurrently, as illustrated on the construction schedule (Figure 13). Should the combined impacts of these activities result in exceedance of the guideline criteria discussed below in Subsection 2.1.4, appropriate secondary mitigation strategies will be employed to limit the levels of TSS in the Nelson River.

Note: The Keeyask Generation Project In-stream Construction Sediment Management Plan contains additional information regarding potential erosion of shorelines upstream of the construction site due to water level increases that will occur progressively during the construction period.

2.1.4 WATER QUALITY TARGET LEVELS FOR IMPLEMENTATION OF SECONDARY MITIGATION STRATEGIES

Note: The following was largely based on recent experience at the Wuskwatim Generation Project and has not been revised. Refer to the Keeyask Generation Project In-stream Construction Sediment Management Plan for the current target levels, monitoring requirements, monitoring sites, and other information on sediment management for in-stream construction.

The Keeyask Construction Sediment Management Plan has assessed TSS levels of up to 25 mg/L above background conditions in the fully mixed zones of Gull Rapids is not expected to cause a significant change in the aquatic biota due to its short-term nature and given the range of existing background conditions of TSS in the Nelson River. A monitoring site at a fully mixed location downstream of the site at the downstream of Gull Rapids/entrance to Stephens Lake will monitor TSS levels throughout construction. If TSS is above 25 mg/L above background for 1 hour an investigation into the reason for the increase will commence. If it can be shown to be short term (i.e., less than 24 hours) no corrective measure is required. If TSS is 200 mg/L above background levels instantaneously then immediate corrective actions to reduce TSS levels are required.

A second fully mixed zone will be monitored at the exit of Stephens Lake, near Kettle GS; if TSS is 25 mg/L above background for a moving 24 hour period then an investigation into the reason for the increase will be triggered. If the increase in TSS can be shown to be short term (i.e., less than a day) then no corrective measure is required. This site is also required to monitor effects on water supply for the town of Gillam.

To provide further context to the overall assessment of potential impacts, the reader is directed to Subsection 5.4.1 of Volume 5 of the Keeyask Generation Project Environmental Impact Statement.

Subsequent to the preparation of the Environmental Impact Statement, further discussions with the Department of Fisheries and Oceans has led to the development of TSS action levels for monitoring sites located in closer proximity to the construction site than the fully mixed zone. Exceedance of these action levels would initiate an action plan and, depending on the antecedent levels, initiation of secondary mitigation strategies.

Monitoring of the construction activities and their impacts on TSS levels will be undertaken throughout the construction period by the Manitoba Hydro construction management team's Environmental Protection Practitioners. Details of the Protection Practitioner's responsibilities and tasks, as well as of the Sediment Monitoring Program, are discussed further in Section 4 of this document.

2.2 CONSTRUCTION INFRASTRUCTURE, BORROW AREAS & QUARRIES

2.2.1 GENERAL

The supporting infrastructure for the construction of the Keeyask GS includes an access road to the Keeyask Site from Provincial Road (PR) 280, construction camp (including potable water supply and sewerage), work areas for Manitoba Hydro and the various construction contractors, temporary on-site roads, and construction power services. The development of granular and impervious fill borrow sources, as well as rockfill quarries, located remote from the immediate Keeyask site, will be required to supply material for the construction of the supporting infrastructure, and temporary and permanent structures for the generating station.

The potential sources of sediment and the primary mitigation strategy to be employed for the supporting infrastructure are discussed in the following subsections.

2.2.2 MATERIAL SOURCES AND EXPLOITATION

Material for construction of the Keeyask GS will include impervious fill, granular fill, rockfill, riprap, and coarse and fine concrete aggregates. Site investigations carried out during various exploration programs and in consideration of various alignments, have identified a number of natural sources for production of rockfill as well as for impervious and granular materials (Figure 5), however only a few will be considered for this project. Their selection is based on a function of the haul distance to the site, material quality and required stripping quantity of material, depth of ground water, and environmental considerations.

The probable sources for construction materials for each type of material and structure are shown in Table 1. It is important to note that while the sources of materials and exploitation methods described herein are likely to be those used during construction, based on engineering and economic considerations, the actual sources of materials and methods of exploitation will be determined by the contractors.

Impervious materials are anticipated to be sourced from overburden excavations of Deposits N-5 and N-21 to the North, and S-2, S-17, and S-18 to the South. This fill will consist of sandy silt or silty clay to be used to seal cofferdam and dyke material.

Rockfill (Class C) and riprap (Class D) are expected to be sourced mainly from Quarry Q-7 within the cofferdam enclosure as well as from the Principal Structures excavations. The bulk of the rockfill will be comprised of Class C1 with larger sizes to be used at cofferdam closure areas. Riprap (Class D) will be placed for erosion protection along vulnerable shorelines as well as along the spillway and powerhouse cofferdams.

Quarry Q-7 is expected to provide a majority of the total rockfill material necessary for the Keeyask GS with the remainder being sourced from the Principal Structure excavations. Material for construction of the South Access Road may also be quarried from a source on the south side of the river.

Transition fill (Class B) obtained from these sites will be well graded pervious to semi-pervious mixtures of sand and gravel containing less than 10% fines. No sorting is expected to occur except selective exploration to remove oversize material.

Granular material for construction of the support infrastructure, temporary structures, and permanent structures of the Keeyask GS will be obtained from two primary granular Deposits, G-1 and G-3, which are both located on the north side of the Nelson River (Figures 4 and 5). No suitable sources of granular material have been found within reasonable proximity of the site on the south side of the river.

Deposit G-1 is located approximately 8 km northwest of the Keeyask site. Deposit G-3 is much closer and is located approximately 3 km north of the Keeyask site. Deposit G-2 and G-5 may also be considered for granular material in the construction of the North Access Road since the road traverses close to or along these deposits. The material requirements and utilization plan for the North Access Road is preliminary and is subject to change.

All of these deposits are located on topographic highs that are typically higher than the surrounding terrain. It is anticipated that the depth of exploitation within any of the deposits that are utilized will be limited to the top 4 m to 8 m; however, the actual depth of exploration will be determined by the contractors. It is not anticipated that groundwater will be encountered, other than as isolated perched water tables in the deposits, therefore, minimal pumping and discharge of water will occur to the surrounding environment.

Appropriate rehabilitation measures for the borrow areas will be undertaken after consultation with the representatives of Manitoba Natural Resources. This may include infilling with unclassified disposal materials and re-vegetation of the borrow sources.

Table 1
Keeyask GS – Stage IV Studies, Axis GR-4
Probable Sources of Construction Materials

Structure	Granular				Impervious						Rock			Other Quarries
	G-1	G-2	G-3	G-5	N-5	N-6	N-21	S-2	S-17	S-18	Q-1	Q-7	PH/SW Exc	
	NB	NB	NB	NB	NB	NB	NB	SB	SB	SB	SB	NB	-	-
Camp and Work Areas	X						x					x		
Quarry Cofferdam	X											x		
North Channel Rock Groin												x		
North Channel Stage I Cofferdam	X						x					x		
Stage I Island Cofferdam	X						x					x		
Powerhouse Stage I Cofferdam	X				x		x					x		
Central Dam Stage I D/S Cofferdam			x		x		x					x	x	
Spillway Stage I Cofferdam	X						x					x		
Tailrace Summer Level Cofferdam			x		x							x	x	
South Dam Stage II D/S Cofferdam	X							x				x	x	
South Dam Stage II U/S Cofferdam	X							x				x	x	
Stage II Island Cofferdam	X						x					x	x	
North Dyke	X		x		x	x	x					x	x	
South Dyke	X		x					x	x	x	x		x	
North Dam			x		x								x	
Central Dam	X		x		x								x	
Transmission Tower Spur			x									x	x	
South Dam within Spillway Cofferdam			x				x						x	
South Dam River Section	X		x					x					x	
Parking Lot and Access Ramps			x										x	
North Access Road	X	x		X										
South Access Road											x			x

Notes:
PH Powerhouse excavation
SW Spillway excavation
NB – North Bank
SB – South Bank

2.2.3 MAIN CAMP, WORK AREAS AND ON-SITE ROADS

The site camp facilities and infrastructure are temporary works and have been provisionally designed to initially accommodate a 500-person start-up work force and finally a 1,500-person construction work force, as well as to provide other construction support facilities. The work force estimate and facility requirements are based on previous experience at Manitoba Hydro construction sites, and as determined by Manitoba Hydro. Upon completion of construction, these temporary facilities will be removed and the area reinstated. Work areas for use by Manitoba Hydro and the construction contractors, as well as a series of on-site roads, will also be constructed (Figure 3).

All infrastructure will be adequately drained through a series of ditches, culverts and swales, into low lying areas in the vicinity of the camp and worksites as discussed in the Drainage Management Plan contained in Section 3 of this report.

The Manitoba Hydro Work Area will likely contain an engineering office, a warehouse storage building and yard, a fuel storage and vehicle refueling facility, field offices, a soil and concrete laboratory, a maintenance building, and an Allied Hydro Council building.

The Contractors' Work Area will likely contain office trailers, storage facilities, maintenance shops, a fuel storage and vehicle refueling facility, toilet facilities, a concrete batch plant, an aggregate processing area, a carpenters' shop, and a precast concrete yard. In addition, a magazine for storing explosives will be located away from work and camp areas.

A Construction Power Substation will provide electric power for all camp services and the construction works and will be located within a portion of the area allocated for the permanent switching yard. The locations for the switchyard, transmission line corridor and the communication facilities, which are all considered permanent, have yet to be determined.

The erosion and sediment control measures that will be employed will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS. The activities required to construct these infrastructure areas, along with the proposed Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Primary Mitigation Techniques
Clearing and Grubbing	Clearing will be maintained at the minimum required for construction and safety. The majority of clearing is scheduled to be completed during the winter season.
Strip Organics and Topsoil	Topsoil generally will be removed only from designated excavation and fill placement areas. Topsoil will not be stripped when submerged. Topsoil will be temporarily stockpiled for later use in construction and/or rehabilitation of the areas.
Roadway Embankment Construction Roadway embankments will be constructed using a combination of side-casting, hauling excess material from other nearby work areas, and borrow pit excavation. Usable material excavated from the ditches and backslopes within the ROW (side-casting) will be compacted into the roadway embankment. This side-cast material will be supplemented by material excavated from other work areas and borrow pits located outside of the ROW and hauled to the embankment fill areas as required.	Ditches will be excavated to proper grades and will be maintained to ensure good drainage. Ditches will have flat bottoms (not v-shaped), to minimize potential for scouring. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. This may include the use of settling ponds prior to discharging water into the low-lying areas. The bottom of the ditches will be lined, as required, with low silt content gravel and/or crushed rock of sufficient size to resist being displaced by flows in the ditches. Riprap will be placed in the ditches at culvert inlet and outlets to provide erosion protection, as required. The riprap will be underlain with non-woven geotextile filter fabric to prevent erosion of the natural materials. Borrow pits will only be used as a source of common fill after all excess material from the required excavations has been utilized. Vegetation will be seeded on the shoulders of the newly constructed roadway embankment and on the backslope of the ditches, as required.

Description of Construction Activities	Primary Mitigation Techniques
Yard and Pad Construction Yard and pad areas will be constructed using a combination of excavation of the high sections and fill placement in the low sections of the area. Additional sources of fill for construction of these areas will include ditch excavation, excess material from other nearby work areas, and borrow source excavation. Usable material excavated from within the yard or pad area, the ditches and adjacent backslopes will be compacted to form the yard/pad area.	Ditches will be excavated to proper grades and will be maintained to ensure good drainage. Ditches will have flat bottoms (not v-shaped), to minimize scouring. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. This may include the use of settling ponds prior to discharging water into the low-lying areas. The bottom of the ditches will be lined, as required, with low silt content gravel and/or crushed rock of sufficient size to resist being displaced by flows in the ditches. Riprap will be placed in the ditches at culvert locations to provide erosion protection. The riprap will be underlain by non-woven geotextile filter cloth to prevent erosion of the natural materials. Borrow pits will only be used as a source of common fill after all excess material from the required excavations has been utilized. The yard and pad areas will be covered by a layer of granular material to improve traffic ability and prevent erosion. Vegetation will be seeded on the shoulders of the newly constructed yard/pad and on the backslope of the ditches.

2.2.4 MAIN ACCESS ROADS

Two main access roads are planned for the Keeyask GS project. One road will be located to the north of the Nelson River and will be connected to PR 280. The other road will be located on the south side of the Nelson River and it will be connected to the west end of Butnau Dam at the existing Kettle GS development. The proposed access road routes are shown in Figure 2.

2.2.5 NORTH ACCESS ROAD

The North Access Road will be the primary source of access and is required to transport materials, equipment and workers from between the existing PR 280 and the project site.

The proposed access road will begin at km 174.7 of PR 280, east of Thompson. It will be a gravel surfaced, all-weather road approximately 24 km long stretching from the intersection at PR 280 north-west to the Keeyask site. The North Access Road passes through or close to the designated granular borrow areas.

There will be one stream crossing and one creek crossing at Looking Back Creek; the location of the crossing over Looking Back Creek is shown on Figure 3. At the stream crossing, the banks will be riprapped to prevent erosion and sedimentation.

The Looking Back Creek Bridge will have no in-stream piers and will not require in-stream work for its construction. The bridge structure will be constructed entirely above the ordinary high water mark, with the exception of driven precast piles. A temporary crossing is likely to be required until the bridge construction is completed.

The bridge will be designed such that storm water runoff from the bridge deck, side slopes and approaches is directed into a retention pond or vegetated area to remove suspended solids, dissipate velocity and prevent sediment and other deleterious substances from entering the watercourse.

The methods that will be employed to construct the North Access Road generally will follow Manitoba Transportation and Government Services "Grading and Surfacing Specifications" [Ref 4]. Erosion control measures will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS. The activities required to construct the North Access Road, along with proposed Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Primary Mitigation Techniques
Clearing and Grubbing Clear and grub the road right of way (ROW). The ROW is 100 m wide.	Clearing and grubbing will be maintained at the minimum required for construction and safety. The cleared width will vary from 60 m to 100 m, depending on the curvature of the road and terrain. Grubbing generally will be restricted to within the areas of the excavations for the road, and where the road embankment grade is less than 1 m above the original ground level.

Description of Construction Activities	Primary Mitigation Techniques
Strip Organics and Top Soil	Topsoil generally will be removed only from designated excavation and roadway embankment areas. Organics and topsoil will be excavated not more than 2 km in advance of the roadway embankment construction. Organics and topsoil will be temporarily stockpiled for use in covering slopes and ditch inverts.
Roadway Embankment The roadway embankment will be constructed using a combination of side-casting and borrow pit excavation. Usable material excavated from the ditches and backslopes within the ROW (side-casting) will be compacted into the roadway embankment. This side-cast material will be supplemented by material excavated from local borrow pits located outside of the ROW and hauled to the embankment fill areas as required.	Ditches will be excavated to proper grades and will be maintained to ensure good drainage. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. Vegetation will be seeded from the shoulder of the newly constructed roadway embankment to the tree line, as required. Erosion control blankets will be placed on the grade or backslopes of the newly constructed roadway embankment, on the ditch bottoms, and at various locations, as required. The erosion control blankets will be installed on all designated areas after the final seeding operations have been completed. Riprap will be placed in the ditches to provide erosion protection, as required. Silt Fence Barriers will be placed at all stream crossings. Erosion control measures will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1], and may include: - placing slash material on stockpiled topsoil and borrow pit slopes and bottom.

Description of Construction Activities	Primary Mitigation Techniques
	- placing commercially manufactured erosion control blankets on stockpiled topsoil or borrow pit slopes and bottoms. - flattening slopes to reduce velocities of surface water runoff. - diverting runoff to vegetated areas that will act to filter it en-route to adjacent natural watercourses.
Stream Crossing and Looking Back Creek Crossings Corrugated steel culverts will be installed at stream crossings. A total of 18 culverts will be installed across the road for local drainage, with diameters of 750 mm to 1,200 mm corrugated steel. The Looking Back Creek Crossing will use a single span precast-prestressed concrete girder bridge approximately 12 m wide by 30 m long. The abutments for the bridge will be constructed with cast-in-place concrete and likely supported by precast driven piles. The Looking Back Creek Crossing is the only crossing identified for the North Access Road that has the potential to pass fish.	Embankments at stream crossings will be constructed with a low silt content granular fill material. Effective sediment and erosion control measures will be installed before commencing work to prevent the entry of sediment into the watercourse. Construction procedures at stream crossings such as temporary crossing procedures for equipment will minimize in-stream time and disturbance of the watercourse bed and banks. Such activities will be scheduled to avoid periods of high fish movement, sensitive fish life stages, and high water flow to protect from erosion and riverbed degradation. Machinery fording the watercourse, to bring equipment required for construction to the opposite side of the creek, will be limited to a one-time event (over and back) and will only occur if other means of crossing are not practicable. Fording will be scheduled to avoid periods of high fish movement, sensitive fish life stages, and high water flow to protect from erosion and riverbed degradation. If minor rutting is likely to occur during fording, stream bank and bed protection methods (e.g., swamp mats, pads) will be used provided they do not constrict flows or block fish passage. Modification of the banks to accommodate machinery fording should not occur.

Description of Construction Activities	Primary Mitigation Techniques
	Care will be taken during construction to minimize loss and disturbance to the riparian vegetation. Care will be taken to ensure that only the vegetation required to accommodate operational and safety concerns for the crossing structure and approaches, within the right-of-way. Removal of plants within the road right-of-way will be completed to meet operational and/or safety concerns. When practicable, vegetation will be pruned or topped instead of uprooted. Disturbed, unstable debris in the stream resulting from the construction activities at the location of the creek crossings will be removed to prevent it from being washed downstream during high flow events. Disturbed areas will be re-vegetated or provided with suitable protection to prevent erosion. Banks will be restored to original condition if any disturbance occurs.
Borrow Pits	A strip of at least 30 m of undisturbed terrain will be left between the edges of the ROW and the borrow pit. Borrow pits will be located at least 100 m from any active stream channels. Borrow material will not be excavated below the normal high water mark on streams, or in areas where flows in adjacent streams, or effect on local water tables will occur. The borrow pits will be excavated as uniformly as possible, and the side slopes generally will be maintained at a slope of 1V:4H or flatter. The borrow pits will be trimmed and leveled after the excavation has been completed. Stripped organics and topsoil which were stockpiled will be uniformly placed over the slope and bottom of the borrow pits and seeded.

Description of Construction Activities	Primary Mitigation Techniques
Quarries	Prior to drilling, trees and shrubs will be disposed of in accordance with Manitoba Infrastructure and Transportation's Forest Service Provisions of the Standard Construction Specifications. A minimum width of 30 m of undisturbed rock or treed buffer will be left between the ROW and the quarry operations.

2.2.5.1 South Access Road

The 15 km long South Access Road, shown in Figures 2, 4 and 5, will connect the construction site with the end of the existing Butnau Dam.

The methods that will be employed to construct the South Access Road generally will follow Manitoba Infrastructure and Transportation "Grading and Surfacing Specifications" [Ref 4]. Erosion control measures will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS. The activities required to construct the South Access Road, along with proposed Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Primary Mitigation Techniques
Clearing and Grubbing Clear and grub the road right of way (ROW). The ROW is 100 m wide.	Clearing and grubbing will be maintained at the minimum required for construction and safety. The cleared width will vary from 60 to 100 m, depending on the curvature of the road and terrain. Grubbing generally will be restricted to within the areas of the excavations for the road, and where the road embankment grade is less than 1 m above the original ground level.
Strip Organics and Top Soil	Topsoil generally will be removed only from designated excavation and roadway embankment areas.

Description of Construction Activities	Primary Mitigation Techniques
	Organics and topsoil will be excavated not more than 2 km in advance of the roadway embankment construction. Organics and topsoil will be temporarily stockpiled for later use in covering slopes and ditch inverts.
Roadway Embankment The roadway embankment will be constructed using a combination of side-casting and borrow pit excavation. Usable material excavated from the ditches and backslopes within the ROW (side-casting) will be compacted into the roadway embankment. This side-cast material will be supplemented by material excavated from borrow pits located outside of the ROW and hauled to the embankment fill areas as required.	Ditches will be excavated to proper grades and will be maintained to ensure good drainage. Ditches will have flat bottoms (not V-shaped), to minimize scouring. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. This may include the use of settling ponds prior to discharging water into the low-lying areas. Vegetation will be seeded from the shoulder of the newly constructed roadway embankment to the tree line, as required. Erosion control blankets will be placed on the grade or backslopes of the newly constructed roadway embankment, on the ditch bottoms, and at various locations, as required. The erosion control blankets will be installed on all designated areas after the final seeding operations have been completed. Riprap will be placed in the ditches to provide erosion protection, as required. Silt Fence Barriers will be placed at all stream crossings. Erosion control measures will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1], and may include: - placing slash material on stockpiled topsoil and borrow pit slopes and bottom.

Description of Construction Activities	Primary Mitigation Techniques
	- placing commercially manufactured erosion control blankets on stockpiled topsoil or borrow pit slopes and bottoms. - flattening slopes to reduce velocities of surface water runoff. - diverting runoff to vegetated areas that will act to filter it en-route to adjacent natural watercourses.
Stream Crossings Corrugated steel culverts will be installed at stream crossings. The diameter of the culverts will range from 0.9 m to 1.8 m in diameter.	Embankments at stream crossings will be constructed with a low silt content granular fill material. Construction of stream crossings will be scheduled to avoid periods of high fish movement and high water flow. Streams that have the potential to pass fish will have a minimum 1.0 m diameter culvert, regardless of stream flow, and be in compliance with DFO and Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat. Construction procedures at stream crossings will minimize in-stream time and disturbance of the watercourse bed and banks. Highest priority will be given to areas with the greatest potential for erosion. A minimum buffer zone of 10 m of undisturbed vegetation will be left at stream crossings. Impacted areas containing eroding or eroded banks will be stabilized. Disturbed, unstable debris in the stream in the vicinity of the road crossing will be removed to prevent it from being washed downstream during high flow events. Disturbed areas will be revegetated or provided with suitable protection to prevent erosion.

Description of Construction Activities	Primary Mitigation Techniques
Borrow Pits	A strip of at least 30 m of undisturbed terrain will be left between the edges of the ROW and the borrow pit. Borrow pits will be located at least 100 m from any active stream channels. Borrow material will not be excavated below the normal high water mark on streams, or in areas where flows in adjacent streams, or effect on local water tables will occur. The borrow pits will be excavated as uniformly as possible, and the side slopes generally will be maintained at a slope of 1V:4H or shallower. The borrow pits will be trimmed and leveled after the excavation has been completed. Stripped organics and topsoil which were stockpiled will be uniformly placed over the slope and bottom of the borrow pit.
Quarries	Prior to drilling, trees and shrubs will be disposed of in accordance with the Forest Services Provisions of the standard construction specifications. A minimum width of 30 m of undisturbed rock or treed buffer will be left between the ROW and the quarry operations.

2.2.6 SEWAGE TREATMENT LAGOON

Note: Subsequent to this report it was determined that sewage would be treated in a package treatment plant. Please refer to the Keeyask EIS Project Description Supporting Volume for additional information.

The sewage treatment facility was designed based on a 1,500-person summer capacity. Sewage will be conveyed by gravity from most of the facilities except Work Area C which will be pumped to a Sewage Lagoon, shown in Figure 6. The lagoon will be constructed from the locally available clays. Sewage will be treated in the two-cell lagoon and operated in accordance with Provincial regulations. Final discharge will be through a gravity outfall (Lagoon Effluent Ditch) into a backwater marshy area of Stephens Lake (Figure 6).

The design and construction methods that will be employed to construct the Sewage Lagoon have been developed to minimize the amount of sediment produced. The erosion and sediment control measures that will be employed will be consistent with Manitoba Infrastructure and Transportation "Manual of Erosion and Sedimentation Control" [Ref 2] and "Manitoba Stream Crossing Guidelines for Protection of Fish and Fish Habitat" [Ref 1]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS. The activities required to construct the lagoon, along with the proposed Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Primary Mitigation Techniques
Clearing and Grubbing	Clearing will be maintained at the minimum required for construction and safety. The majority of clearing will be completed during the winter season.
Strip Organics and Topsoil	Topsoil generally will be removed only from designated excavation and fill placement areas. Topsoil will not be stripped when wet. Topsoil will be temporarily stockpiled for later use in construction and/or rehabilitation of the area.
Lagoon Construction The lagoon will be constructed using a combination of excavation of the high sections and fill placement in the low sections and cell embankments. Additional sources of fill for construction of the lagoon will include ditch excavation, excess material from other nearby work areas, and borrow pit excavation. Usable material excavated from within the lagoon area, the ditches and adjacent backslopes will be compacted to form the lagoon.	Ditches will be excavated to proper grades and will be maintained to ensure good drainage. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. Borrow pits will only be used after all suitable material from the required excavations has been utilized. Topsoil will be placed on the exterior slopes and vegetation will be seeded on the newly constructed lagoon embankments and on the backslopes of the ditches. The bottoms of the ditches will be lined, as required, with low silt content gravel and/or crushed rock of sufficient size to resist being displaced by flows in the ditches.

Description of Construction Activities	Primary Mitigation Techniques
	Riprap will be placed in the ditches at culvert locations to provide erosion protection. The riprap will be underlain by non-woven geotextile filter cloth to prevent erosion of the native materials.
Lagoon Effluent Ditch	The ditch will be excavated to proper grades and will be maintained to ensure good drainage. Temporary soil erosion and sedimentation control measures will be implemented at all times during the construction, as dictated by local conditions. The lower portions of the side slopes and bottom of the ditch will be lined with low silt content cobbles and/or crushed rock of sufficient size to resist being displaced by flow in the ditch. The rockfill will be underlain by non-woven geotextile filter cloth to prevent erosion of the natural materials. The upper portions of the side slopes of the ditch will be dressed with topsoil, seeded and covered by an Erosion Control Blanket (ECB), consisting of coconut fiber. Energy dissipation structures will be constructed along the ditch to control flow velocities, as required. Riprap will be placed in the ditch at the location of the energy dissipating structures and where the ditch discharges, to provide erosion protection. All riprap will be underlain by non-woven geotextile filter cloth to prevent erosion of the native materials.

2.2.7 MAIN CAMP RAW WATER INTAKE CONSTRUCTION

Note: Subsequent to this report it was determined that the camp water would be obtained from wells located in Deposit G-1. Please refer to the Keeyask EIS Project Description Supporting Volume.

Potable water supply for the camp and work areas will be taken from the Nelson River at an intake located upstream of the North Channel Rock Groin in the vicinity of Deposit N-21 (see Figure 6) and will be pumped to a treatment plant facility located onshore. The intake line will pass through a road section of the North Dyke axis where

the original elevation ground surface is above the FSL, el 159 m. This simplifies the construction of the dyke where the pipeline crosses the dyke.

Potable water supply for the camp and work areas prior to construction of the North Channel Rock Groin and Raw Water Intake will be from a well located in Deposit G-1. The well located in Deposit G-1 will be temporary in nature as Deposit G-1 will be used as a source of materials during construction and therefore contamination of the water supply may be an issue. As this well is planned to be constructed on dry land it will not have an affect on TSS and is therefore not discussed further in this report.

The design and construction methods that will be employed to construct the Camp Raw Water Intake have been developed to minimize the amount of sediment produced. The erosion and sediment control measures that will be employed will be consistent with Manitoba Transportation and Government Services "Manual of Erosion and Sedimentation Control" [Ref 2] and "Protection of Fish and Fish Habitat" [Ref 1]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS. The construction steps for this structure, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Primary Mitigation Techniques
Concrete Work	All necessary concrete work is planned to be completed in-the-dry above water levels existing at the time of construction.
Laying Pipe	The 900 m of 200 mm diameter pipe is to be assembled on a barge and laid on the river bed with assistance by divers to minimize the disturbance of the river bottom.
Anchorage	The number of anchorage points will be minimized.

2.3 GENERATING STATION – IN-STREAM CONSTRUCTION ACTIVITIES

2.3.1 GENERAL

Ten cofferdams and two rock groins will be constructed to divert the Nelson River during construction and enable construction of the Principal Structures of the Generating Station. The cofferdams and rock groins are listed as follows:

- Quarry Cofferdam.
- North Channel Rock Groin.
- North Channel Stage I Cofferdam.
- Powerhouse Stage I Cofferdam.
- Spillway Stage I Cofferdam.
- Central Dam Stage I Downstream Cofferdam.
- Central Dam Rock Groin.
- Stage I Island Cofferdam.
- Tailrace Summer Level Cofferdam.
- South Dam Stage II Upstream Cofferdam.
- South Dam Stage II Downstream Cofferdam.
- Stage II Island Cofferdam.

The locations of the cofferdams and rock fill groins are shown in Figure 3. Each cofferdam's type of construction, removal requirements, and type of removal are summarized in Table 2.

Table 2 - Cofferdams
Keeyask GS – Stage IV Studies, Axis GR-4

Cofferdam	Type of Construction	Required Removal	Type of Removal
Quarry Cofferdam	Mostly in-the-wet.	No removal required.	
North Channel Rock Groin	Mostly in-the-wet.	Lowered to el 153.0 m.	In-the-dry and in-the-wet.
North Channel Stage I Cofferdam	Mostly in-the-wet.	Lowered to el 152.5 m.	In-the-dry and in-the-wet.

Cofferdam	Type of Construction	Required Removal	Type of Removal
Powerhouse Stage I Cofferdam	In-the-dry and in-the-wet.	Partial removal of eastern leg. South leg will be incorporated into the Transmission Tower Spur.	All in-the-dry.
Spillway Stage I Cofferdam	In-the-dry and in-the-wet.	Partial removal of approach and discharge channels.	In-the-dry and in-the-wet.
Central Dam Stage I Downstream Cofferdam	Mostly in-the-wet.	No removal required. Incorporated into the Central Dam.	
Central Dam Rock Groin	Mostly in-the-wet.	No removal required. Incorporated into the Central Dam.	
Stage I Island Cofferdam	In-the-dry.	Lowered to el 152.0 m.	Mostly in-the-dry.
Tailrace Summer Level Cofferdam	Mostly in-the-wet.	Complete removal to riverbed.	In-the-dry and in-the-wet.
South Dam Stage II Upstream Cofferdam	Mostly in-the-wet.	No removal required. Incorporated into the South Dam.	
South Dam Stage II Downstream Cofferdam	Mostly in-the-wet.	No removal required. Incorporated into the South Dam.	
Stage II Island Cofferdam	In-the-dry.	Lowered to el 151.0 m.	In-the-dry.

Primary mitigation of potential sediment releases into the river will be accomplished by the choice of cofferdam cross-sections, the incorporation of measures to reduce flow velocities across the faces of the cofferdams, and developing construction sequences that will provide tranquil water areas, where possible, for placement of granular and impervious fill for the cofferdams. Details of the preliminary primary mitigation strategies for each of the river diversion structures are presented in Subsections 2.3.2 to 2.3.11. The actual strategies employed during construction may vary from the preliminary strategies that are presented below, as the construction methods and final sediment management plan(s) will be developed by the Stage I

Cofferdam Contractor and General Civil Contractor, in consultation with Manitoba Hydro.

2.3.2 RIVER MANAGEMENT DURING CONSTRUCTION

The arrangement of the Keeyask GS and the management of the river during the construction are designed to take advantage of the natural configuration of Gull Rapids. The Nelson River is divided into three channels across the project site.

As shown in Figure 3, the channels are separated by two large islands; one to the north (North Island) and one to the south (Central Island) of Gull Rapids. A third island (West Island) which is immediately upstream of the Central Island initially divides the river into a North and South Channel, with a small cross-over channel located between the Central Island and the West Island.

A two stage River Diversion approach will be used to manage the Nelson River and allow the construction of the project along Axis GR-4. Stage I Diversion, shown in Figure 7, will block off the two channels that are north of the Central Island. This will be accomplished by means of an upstream rock groin and cofferdam, a downstream cofferdam adjacent to the Powerhouse and another cofferdam across the Central Channel. These structures will allow excavation works for the Powerhouse and Central Dam areas to be undertaken "in-the-dry". A final Stage I Cofferdam will be constructed in a U-shape on the southeast side of the Central Island to allow the Spillway excavation to be undertaken "in-the-dry".

Stage II Diversion will involve the partial removal of the Spillway Stage I Cofferdam, to allow water to flow through the partially complete Spillway while construction of the South Dam is undertaken. River Closure will be accomplished by advancing two parallel cofferdams across the South Channel from the south leg of the remaining portion of the Spillway Cofferdam.

2.3.3 QUARRY COFFERDAM CONSTRUCTION

The Quarry Cofferdam is required to allow for the initial exploitation of Rock Quarry No. 7 which is the material source for the construction of the North Channel Rock Groin and North Channel Stage I Cofferdam. The Quarry Cofferdam will consist of an upstream impervious blanket of silty sand till (Class A), and a downstream rockfill groin (Class C1). No road topping will be applied to this cofferdam as road access is not required. A cross-section of the Quarry Cofferdam is shown in Figure 9. This cofferdam will be maintained only until the North Channel Stage I Cofferdam is completed. This cofferdam will become submerged during forebay impoundment.

The Quarry Cofferdam will be constructed by placing fill material in the river "in-the-wet". The erosion of the impervious fill material is anticipated to be minimal, as this area of the channel will consist of relatively still waters following the placement of the rockfill zone. The Quarry Cofferdam will be founded almost entirely on the riverbed bedrock surface. [Note: Field explorations of cofferdam abutment areas are scheduled for the spring of 2009.]

The cross-section, design, and methods that will be employed to construct the Quarry Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Advance Rockfill Groin Construct the downstream rockfill groin (Figure 9) across the west end of the channel that is north of North Island. The rockfill will be placed from the North Shore directly across to the North Island near the entrance of the North Channel. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from a quarry excavation. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The only potential source of sediment that may enter the river is the washing of fine grain size material due to flow through and along the face of the rockfill. Since the rockfill will have a low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 2 – Impervious Fill Placement Place impervious fill on the upstream face of the rockfill (Figure 9). The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in the dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. Since this fill zone will be placed in an area with little flow, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

* Please note that, although the construction steps are described sequentially, it is likely that the construction activities will take place concurrently.

2.3.4 NORTH CHANNEL ROCK GROIN CONSTRUCTION

The North Channel Rock Groin will be a flow-through zoned granular and rock fill embankment, consisting of an upstream zone of granular fill and a downstream rockfill groin consisting of Classes C1, C2 and C3. The Rock Groin will be used to impede the flow through the northern river channel and divert it through the river's south channel. This will result in a small increase of water levels to the upstream that are required to ensure the successful operation of the Ice Boom. The structure also serves as a component of the water intake for the camp infrastructure. Its construction will assist in reducing the sediment erosion on the North Channel Stage I Cofferdam during construction and throughout the project lifecycle. Further, this structure will decrease the currently active erosion in the "cross over channel" which interconnects the north and south channels at a location just downstream of the North Channel Rock Groin. The cross-section of the North Channel Rock Groin is shown in Figure 9.

The North Channel Rock Groin will be constructed by placing the fill material in the river "in-the-wet". The North Channel Rock Groin will be founded directly on the riverbed. Erosion of the riverbed is not anticipated, as the riverbed in this area is predominantly bedrock controlled, with occasional cobbles and boulders. [Note: Field explorations of the cofferdam abutment areas are scheduled for the spring of 2009. Depending upon the findings, it may be necessary to armour the shore of the island or construct a non-erodible structure against which channel closure can be completed.]

Most of the rockfill used in the construction of the North Channel Rock Groin will be Class C1. However, larger rockfill (Classes C2 and C3) may be required when higher velocities (south side of the channel) are encountered in the river during closure. To minimize the required quantities of the larger rockfills, they will be dumped and pushed at the leading edge of the advancing face and finer rockfill will then be used to complete the balance of the section which is sheltered from the high velocity flows. The North Channel Rock Groin will have its crest elevation lowered and the remnant will be submerged during the forebay impoundment.

The cross-section, design, and methods that will be employed to construct the North Channel Rock Groin have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Advance Rockfill Groin Construct the rockfill groin across the North Channel (Figure 9). The rockfill will be placed from the north shore to the North Shore of the West Island. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from a quarry excavation. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The only potential source of sediment that may enter the river is the washing of fine grain size material due to flow through and along the face of the rockfill. Since the rockfill will have a low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 2 – Advance Transition Fill Place granular transition fill on the upstream slope of the rockfill groin (Figure 9). The transition fill will be placed on the upstream side of the rockfill groin. The granular transition fill will lag behind the advancing rockfill groin described in Step 2. The lag distance will be sufficient to create an area of low velocity water in which to place the transition fill. The transition fill will be end-dumped on top of the transition fill that has already been placed and then pushed into the water. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam and from velocities along the surface of this slope. Initially, a small portion of even the coarse grained material may be washed through the rockfill, with the size of the particles being washed gradually reducing until essentially no washing occurs. Since the granular transition fill will have low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

2.3.5 NORTH CHANNEL STAGE I COFFERDAM CONSTRUCTION

The North Channel Stage I Cofferdam will be a zoned earth and rockfill embankment. The cofferdam will consist of an upstream impervious blanket of silty sand till (Class A), a transition zone of granular fill (Class B) and a downstream rockfill groin (Class C1). A zone of larger rockfill material (Class C) will be placed on the upstream slope of the impervious fill for erosion protection. A layer of road topping material (Class RT) will be placed on the crest of the cofferdams. The cross-section of the North Channel Stage I Cofferdam is shown in Figure 9.

The lower portions of the North Channel Stage I Cofferdam will be constructed in the water. The water level during construction will vary depending on the flow in the Nelson River. The fill for the lower portions will be placed to a level slightly above the water level that will exist at the time of construction. The remaining upper portions of this cofferdam will be constructed in-the-dry.

There are two potential sources of flow during advancement of the North Channel Stage I Cofferdam's rockfill groin; flow through the North Channel Rock Groin and northward flow from the channel between the West and Central Islands. However, erosion during construction of the impervious and transition fill zones of the cofferdam is not anticipated to be a major concern, as there will be very little flow in the channel, due to the earlier construction of the North Channel Rock Groin and the construction of the North Channel Stage I Cofferdam rock groin. The North Channel Stage I Cofferdam will be founded directly on the riverbed.

The cross-section, design, and methods that will be employed to construct the North Channel Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Clearing Heavy vegetation located within the footprint of the rock groin will be stripped.	Since these activities will be conducted on dry ground, using best management practices to control erosion and sediment, it is anticipated that they will not contribute to sediment in the river.

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 2 – Advance Rockfill Groin Construct the rockfill groin of the North Channel Stage I Cofferdam. The rockfill will be placed from the North Shore to the north shore of the Central Island. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill groin will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from the Quarry Q7 and from required excavations for the Principal Structures. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The only potential source of sediment that may enter the river is washing of fine grain size material due to flow through and along the face of the rockfill. Since the rockfill will have low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 3 – Advance Transition Fill Place granular transition fill on the upstream slope of the rockfill groin (Figure 9). The transition fill will be placed on the wet-side of the rockfill groin. The granular transition fill will lag behind the advancing rockfill groin described in Step 2. The lag distance will be sufficient to create an area of low velocity water in which to place the transition fill. The transition fill will be end-dumped on top of the transition fill that has already been placed and then pushed into the water. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam and from velocities along the surface of this slope. Initially, a small portion of even the coarse grained material may be washed through the rockfill, with the size of the particles being washed gradually reducing until essentially no washing occurs. Since the granular transition fill will have low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 4 – Advance Impervious Fill Place impervious fill on the upstream slope of the transition fill (Figure 9). The impervious fill will be placed against previously advanced transition zones of the cofferdam. The impervious fill will be end-dumped on top of the impervious fill that has already been placed and then pushed into the water so that it will enter the water as a sliding mass. The impervious fill will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. Placement of impervious fill will be sheltered by the previously constructed North Channel Rock Groin, as well; flow will be impeded due to the prior advancement of the rockfill and transition zones of the cofferdam. Since this fill zone will be placed in relatively tranquil water and with a granular filter zone located on the downstream side, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

*Please note that, although the construction steps are described sequentially, it is likely that the construction activities will take place concurrently.

2.3.6 STAGE I ISLAND COFFERDAM CONSTRUCTION

The Stage I Island Cofferdam will be a zoned earthfill embankment, consisting of a central impervious core (Class A), with granular and rockfill zones located to the upstream and downstream. Finally, a layer of road topping material (Class RT) will be placed on the crest of the cofferdam (Figure 9). This cofferdam is located on the Central Island in a depression between the edge of the North Channel Stage I Cofferdam and the hill crest of the island.

This cofferdam will be placed 'in-the-dry' and will be founded on prepared overburden surface, to a crest governed by open water conditions during Stage II Diversion.

The cross-section, design, and methods that will be employed to construct the Stage I Island Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 Strip the surface organics and remove any unsuitable overburden located within the footprint of the cofferdam on the Central Island.	Since these activities will be conducted on dry ground, using management practices to control erosion and sediment, it is anticipated that they will not contribute to sediment in the river.
Construction Step 2 – Fill Placement Construct the earth fill portion of the Stage I Island Cofferdam (Figure 9). As work is to be completed "in-the-dry" it is likely that the cofferdam fill material will be placed in successive lifts with impervious, granular and rockfill zones being placed concurrently.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass obtained from the excavations required for the Primary Structures and along the north shore. The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The rockfill (Class C1 fill) will consist of low silt content rock obtained from the required excavations for the Principal Structures. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow against the cofferdam. The riprap zone (Class C2) will consist of low silt content rock obtained from the required excavations for the Principal Structures. The Class C2 fill will consist of rock fragments of sufficient size to resist being carried downstream by ice and water flow against the cofferdam. Since these activities will be conducted on dry ground, using management practices to control erosion and sediment, it is anticipated that they will not contribute to sediment in the river.

*Please note that, although the construction steps are described sequentially, it is likely that the construction activities will take place concurrently.

2.3.7 POWERHOUSE STAGE I COFFERDAM CONSTRUCTION

The Powerhouse Stage I Cofferdam will be a zoned earth and rockfill embankment. In its final configuration, shown in Figure 9, the cofferdam will consist of a wet-side impervious blanket of silty sand till (Class A), a transition zone of granular (Class B) and a dry-side

rockfill groin (Class C1). The dry-side rockfill groin is only required on the river leg of the cofferdam and will not be constructed for the downstream leg of the cofferdam where water velocities are reduced. A layer of road topping material (Class RT) will be placed on the crest of the cofferdam. A protective outer zone of rockfill (Class C) will cover the wet-side impervious blanket to prevent damage caused by overtopping of water and ice during winter months. On the river leg of the cofferdam the toe of the impervious fill will be confined by the outer rockfill groin comprised of rockfill (Class C).

The initial step in the construction of the Powerhouse Stage I Cofferdam is the construction of an outer protective rockfill groin (Class C) which will be advanced prior to construction of the remainder of the river leg of the Powerhouse Stage I Cofferdam. The Powerhouse Stage I Cofferdam will encroach into the path of flow downstream of the river's southern channel/Spillway Discharge Channel during Stage I and II Diversion, respectively. The groin will be constructed in-the-wet, to the wet-side of the balance of the fill for the cofferdam. This outer groin will be used to confine the toe of the impervious fill and protect the fine grained materials from direct contact with the river velocities and thus permit the placement of the finer grained materials in relatively tranquil water. The cross-section of the Powerhouse Stage I Cofferdam in this initial construction stage is shown in Figure 9 – Powerhouse Stage I Cofferdam Step 1.

The lower portions of the Powerhouse Stage I Cofferdam will be constructed in-the-wet. The water level during construction will vary depending on the flow in the Nelson River. The fill for the lower portions will be placed to a level slightly above the water level that will exist at the time of construction. The remaining upper portions of this cofferdam will be constructed in-the-dry.

Once the placement of fine grained impervious materials is completed in an area, a portion of the outer protective rockfill groin (Class C) will be removed and placed on the impervious fill, thereby providing protection from ice and flowing water during winter months, shown in Figure 9 – Powerhouse Stage I Cofferdam Step 2.

The cross-section, design, and methods that will be employed to construct the Powerhouse Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 - Clearing Heavy vegetation located within the footprint of the rock groin will be stripped. The footprints of the cofferdam should be stripped of organics and topsoil prior to placement of materials.	Since these activities will be conducted on dry ground, using management practices to control erosion and sediment, it is anticipated that they will not contribute to sediment in the river.
Construction Step 2 – Advance Outer Rockfill Groin Construct the upstream rockfill groin of the southern leg of the Powerhouse Stage I Cofferdam (Figure 8, Year 2(1) and Figure 9). The rockfill will be placed from the east shore of the North Island parallel to the southern leg and extend to the point where the Powerhouse Stage I Cofferdam turns northward. The rockfill (wet-side) will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from a nearby quarry excavation. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The potential sources of sediment that may enter the river are the washing of fine grain size material due to flow through and along the face of the rockfill and the washing of the finer grain size material by water seeping through the cofferdam rockfill groin or due to the low velocities present along the downstream leg. Since the rockfill will have low fines content it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 3 – Advance Inner Rockfill Groin Construct the inner rockfill groin of the cofferdam (Figure 9), including the portion on the North Island. The rockfill will be placed from the east side of North Island and will extend east toward the centre of the channel before turning northward to the North Shore. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C fill) will consist of low silt content rock obtained from a quarry or the required excavations for the Principal Structures. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow through the rockfill zones. The only potential source of sediment that may enter the river is the washing of fine grain size material due to flow through the rockfill groin. Since the rockfill will have low silt content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 4 – Advancing Transition Fill Against Inner Rockfill Groin Place granular transition fill on the wet-side slope of the inner rockfill groin (Figure 9). The transition fill will be placed between the previously constructed outer rockfill groin and inner rockfill groin on the southern leg of the cofferdam. The outer rockfill groin will be advanced ahead of the placement of transition fill enough to provide adequate protection from river flow. Transition fill will also be placed on the wet-side portion of the cofferdam's downstream leg, where the flow velocities are expected to be low. The transition fill will be end-dumped close to the water's edge and subsequently pushed into the water. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam and from the relatively low velocities present along the exposed slope of the fill. Since the granular transition fill will have a low silt content, it is anticipated that this activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 5 – Impervious Fill Placement Place impervious fill between the transition fill of the inner rockfill groin and the rockfill of the outer rockfill groin (Figure 9). The impervious fill will be placed between the previously constructed inner and outer rockfill groins of the south leg. The outer rockfill groin will be advanced ahead of the placement of impervious fill enough to provide adequate protection from river flow. Impervious fill will also be placed on the wet side of the downstream leg where low flow velocities are expected. The impervious fill will be end-dumped on top of the impervious fill that has already been placed and then pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. Since this fill zone will be placed in relatively tranquil water with low flow velocity and with a granular filter zone located on the downstream side, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 6 – Excavate Rockfill from the Outer Protective Rockfill Groin (Wet-Side) and Placement onto Impervious Zone of Cofferdam	
Excavate excess rockfill from the wet-side of the outer protective rockfill groin (wet-side, Figure 9) along select portions of the cofferdam and place on top of the impervious zone of the cofferdam. The rockfill will be excavated from the outer protective rockfill groin, by excavator, and placed on the exposed impervious fill zone. Placement activities will likely be completed using an excavator and dozer.	The rockfill (Class C fill) will consist of clean blast rock or boulders having a low fines content rock obtained from the outer protective rockfill groin. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow. Since the rockfill will have low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

*Please note that, although the construction steps are described sequentially, it is likely that some of the construction activities will take place concurrently.

2.3.8 SPILLWAY STAGE I COFFERDAM CONSTRUCTION

The Spillway Stage I Cofferdam will be a zoned earth and rockfill embankment. In its final configuration, shown in Figure 9, the cofferdam will consist of a wet-side impervious blanket of silty sand till (Class A), a transition zone of granular fill (Class B) and dry-side rockfill groin (Classes C1, C2 and C3). A layer of road topping material (Class RT) will be placed on the crest of the cofferdam. A protective outer zone of rockfill (Class C) will cover the wet-side impervious blanket to prevent damage caused by overtopping of water and ice during winter months. The toe of the impervious fill will be confined by an outer rockfill groin comprised of a transition zone of granular fill (Class B) and rockfill (Class C).

The initial step in the construction of the upstream and river legs of the cofferdam will be the construction of a protective outer rockfill groin (Classes C1, C2 and C3, and Class B). It will be constructed in-the-wet, to the wet-side of the balance of the fill. This outer groin will be used to confine the toe of the impervious fill and protect the fine grained materials from direct contact with the river velocities and thus permit the placement of the finer grained fill materials in relatively tranquil water. During construction of the rockfill groin a transition zone of granular fill (Class B) will be placed on the dry-side of the groin to prevent migration of fine grained materials from the cofferdam that will be placed on the cofferdam subsequent steps.

As velocities along the face of the downstream leg will be relatively low, erosion of the fine grained materials during placement is not anticipated to be significant and therefore no outer protective rockfill groin is required.

The lower portions of the Spillway Stage I Cofferdam will be constructed in the water. The water level during construction will vary depending on the flow in the Nelson River. The fill for the lower portions will be placed to a level slightly above the water level that will exist at the time of construction. The remaining upper portions of this cofferdam will be constructed in-the-dry.

Once the placement of fine grained impervious materials is completed in an area, rockfill (Class C) placed on the impervious fill, thereby providing protection from ice and flowing water during winter months.

The cross-section, design, and methods that will be employed to construct the Spillway Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Please note that, although the construction steps, below, are described sequentially, it is likely that the construction activities will take place concurrently. During construction, the outer protective rockfill groin will be advanced first; construction of the inner rockfill zone of the cofferdam will likely follow close behind. Placement of the granular transition fill zones will follow the advancing fronts of the rockfill groins and placement of impervious fill will lag behind farther still. The outer protective rockfill groin will lead the construction of the remaining zones by a distance that will ensure fine grained materials placed later will have adequate protection from river flow. When the placement of impervious fill is complete; rockfill from the outer protective rockfill groin will be excavated and placed on top of the completed impervious blanket.

There are some lenses of sandy gravels present in the riverbed alluvium and overburden. The extent of these lenses is believed to be limited. If these lenses are encountered in the foundation of a cofferdam, where they are accessible, they will be excavated to the extent possible. Seepage through any unexcavated zones is expected to be minor, and will be handled by sump and pumping, and by the placement of reverse filters.

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1– Advance the Outer Protective Rock Groin Construct the outer (wet side) rockfill groin of the upstream and river legs of the Spillway Stage I Cofferdam (Figures 8 Year 2(1) and 9). The rock groin will be advanced from the south shore of the Central Island parallel to the upstream and river legs of the cofferdam. The rockfill (wet-side) and transition fill (dry-side) will be end-dumped close to the water's edge and subsequently pushed into the water. The rockfill zone will initially be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The rockfill (Class C1 fill) will consist of low silt content rock obtained from the required excavations for the Principal Structures. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current of the river. Prior to placing fill on the Central Island shore, the surface organics will be stripped. (This activity will occur on dry ground using management practices to control erosion and sediment production and will not contribute to sediment in the river). The potential sources of sediment that may enter the river are the washing of fine grain size material due to flow through and along the face of the rockfill and the washing of the finer grain size material by water seeping through the cofferdam rockfill groin or due to the low velocities present along the downstream leg. Since the rockfill and granular transition fill will have low silt content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 2 – Advance Inner Rock Groin Construct the inner rockfill zone of the cofferdam (Figure 9) extending in a U-shape around the proposed Spillway excavation and closing at both ends on the Central Island. The rockfill could be placed starting from either the upstream or downstream ends of the u-shape or from both ends simultaneously. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the river. The rockfill zone will initially be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-	The rockfill (Class C1 and C2 fill) will consist of clean blast rock or boulders having a low fines content rock obtained from the required excavations for the Principal Structures. The Class C1 and C2 fill will consist of rock fragments of sufficient size to resist being carried downstream by flow through the rockfill zones. The only potential source of sediment that may enter the river is the washing of fine grain size material due to flow through the rockfill groin. Since the rockfill will have low fines content, it is anticipated that this construction activity will not

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
the-dry.	produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 3 – Placement of Granular Transition Fill Place granular transition fill on the slope of the inner rockfill groin (wet-side, Figure 9). The transition fill will be placed between the previously constructed outer rockfill groin and inner rockfill zone on the river and upstream legs of the cofferdam. The outer rockfill groin will be advanced ahead of the placement of transition fill enough to provide adequate protection from river flow. Transition fill will also be placed on the wet-side of the downstream leg of the inner rockfill zone where the flow velocities are predicted to be low according to hydraulic physical model results and no protective outer rockfill groin is required. The transition fill will be end-dumped on top of the transition fill that has already been placed and pushed into the water. The transition fill zone will be initially constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam rockfill groin or due to the low velocities present along the downstream leg. Since the granular transition fill will have a low fines content, it is anticipated that this activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 4 – Placement of Impervious Fill Place impervious fill on the slope of the transition fill (wet-side, Figure 9). Along the upstream and river legs of the cofferdam, the impervious fill will be placed against the transition zone between the previously constructed outer rockfill groin and inner rockfill zones. The outer rockfill groin will be advanced ahead of the placement of impervious fill enough to provide adequate protection from river flow. The only flow will be the water leaking through the outer rockfill groin. Impervious fill will also be placed on the wet-side of the downstream rockfill leg where the flow velocities are predicted to be low. The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass obtained from the excavations required for the Principal Structures. Since this fill zone will be placed in a very tranquil water area with almost no flow velocity and with a granular filter zone located on the downstream side, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
water as a sliding mass. The impervious fill zone will initially be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	
Construction Step 5 – Place Rockfill onto Impervious Zone of Cofferdam Place rockfill on top of the impervious zone of the cofferdam. The rockfill will be sourced from either quarries or the required excavations, hauled by truck and placed on the exposed impervious fill zone. Placement activities will likely be completed using an excavator and dozer.	The rockfill (Class C fill) will consist of clean blast rock or boulders having a low fines content rock obtained either quarries or the required excavations. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow. Since the rockfill will have low fines content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

2.3.9 CENTRAL DAM STAGE I DOWNSTREAM COFFERDAM & ROCK GROIN CONSTRUCTION

The Central Dam Stage I Cofferdam will be a zoned earth and rockfill embankment consisting of a wet-side impervious blanket of silty sand till (Class A), a transition zone of granular fill (Class B) and a dry-side rockfill groin (Class C1). Rockfill material will be placed on the wet-side slope of the cofferdam, as required, to provide erosion protection. A layer of road topping material (Class RT) will be placed on the crest of the cofferdam. The cross-section of the Central Dam Stage I Cofferdam is shown in Figure 9. The cofferdam will be incorporated into the Central Dam fill section.

The lower portions of the Central Dam Stage I Cofferdam will be constructed in the water. The water level during construction will vary depending on the flow in the Nelson River. The fill for the lower portions will be placed to a level slightly above the water level that will exist at the time of construction. The remaining upper portions of this cofferdam will be constructed in-the-dry.

The Central Dam Rock Groin will be constructed prior to placing the impervious and transition fill for the Central Dam Stage I Cofferdam.

The Central Dam Rock Groin, shown in Figure 7, is constructed to reduce velocities along the wet-side slope of the Central Dam Stage I Cofferdam during placement of transition and impervious fill materials. The Central Dam Rock Groin extends from the Central Dam Stage I Cofferdam to a downstream island that is close by.

The cross-section, design, and methods that will be employed to construct the Central Dam Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Construction Activities*	**Potential Sediment Problem and/or Primary Mitigation Techniques**
Construction Step 1 – Advance Rock Groins Construct the rockfill zone of the Central Dam Stage I Downstream Cofferdam (Figure 9) between the North and Central Islands and the Central Dam Rock Groin. The portion of the cofferdam and rock groin that will be placed in the water will be founded on bedrock, while the portions constructed on the North and Central Islands will be founded on bedrock and/or overburden. The rockfill will be placed between the south shore of the North Island and the north shore of the Central Island, cutting off the downstream end of the river's central channel. There will be no flow through the channel at this time as the upstream entrance will have been blocked by the construction of the North Stage I Cofferdam. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the river. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from nearby quarry excavation. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. Prior to placing fill on the Central Island shore, the surface organics will be stripped. (This activity will occur on dry ground using management practices to control erosion and sediment production and will not contribute to sediment in the river). The potential sources of sediment that may enter the river are: (i) the washing of fine grain size material due to flow along the face of the rockfill; (ii) the disturbance of possible overburden below the zone. Since the rockfill will have low silt content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 2 – Placement of Transition Fill Place granular transition fill on the wet-side slope of the rockfill zone of the Central Dam Stage I Downstream Cofferdam (Figure 9). The transition fill will be end-dumped close to the water's edge and subsequently pushed into the river. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently	The Central Dam Rockfill Groin, constructed in Step 1, will reduce velocities along the wet-side slope of the cofferdam during the placement of transition fill materials. The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential source of sediment that may

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
be raised to final elevations in-the-dry.	enter the river is the washing of the finer grain size material by flow along the fill's surface. Since the granular transition fill will have a low fines content, it is anticipated that this activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 3 – Placement of Impervious Fill Place impervious fill on the wet-side surface of the transition fill (Figure 9). The impervious fill will be placed against the previously constructed groin and transition zone. The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. Since this fill zone will be placed in a very tranquil water area with low flow velocity, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection. 2.1.4).
Construction Step 4 – Placement of Wet-Side Protective Rockfill Zone Construct the protective rockfill zone against the impervious zone on the wet-side of the Central Dam Stage I Downstream Cofferdam. The portion of the cofferdam requiring erosion protection constructed above water level will likely be constructed in lifts, concurrent with the placement of transition and impervious fill. The portion constructed below the water level at the time of construction will be end dumped close to the water's edge and subsequently pushed into the river.	The rockfill (Class C1 fill) will consist of clean blast rock or boulders having a low fines content obtained from nearby quarry excavation. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The potential sources of sediment that may enter the river are: (i) the washing of fine grain size material due to flow along the face of the rockfill; (ii) the disturbance of possible overburden below the zone. Since the rockfill will have low silt content, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

2.3.10 TAILRACE SUMMER LEVEL COFFERDAM CONSTRUCTION

The Tailrace Summer Level Cofferdam will be a zoned earth and rockfill embankment. In its final configuration, shown in Figure 9, the cofferdam will consist of a wet-side impervious blanket of silty sand till (Class A), a transition zone of granular fill (Class B) and a dry-side rockfill groin (Class C1). A layer of road topping material (Class RT) will be placed on the crest of the cofferdam. A protective outer zone of rockfill (Class C) will cover the wet-side impervious blanket to prevent damage caused by overtopping of water and ice during winter months. The toe of the impervious fill will be confined by an outer rockfill groin comprised of rockfill (Class C). The location of the Tailrace Summer Level Cofferdam is shown in Figure 8, Year 5(1) and Year 5(2).

The initial step in the construction of the river leg of the cofferdam will be the construction of a protective outer rockfill groin (Class C). It will be constructed in-the-wet, to the wet-side of the balance of the fill. This outer groin will be used to confine the toe of the impervious fill and protect the fine grained materials from direct contact with the river velocities and thus permit the placement of the finer grained fill materials in relatively tranquil water.

As velocities along the face of portions of the downstream leg will be relatively low, erosion of the fine grained materials during placement is not anticipated to be significant and therefore no outer protective rockfill groin is required. The extent of the outer protective rockfill groin is shown in Figure 8, Year 5(1).

The Tailrace Summer Level Cofferdam will be primarily constructed in-the-wet. The water level during construction will vary depending on the flow in the Nelson River but is mainly governed by the Full Supply Level of the Kettle GS forebay (Stephen's Lake).

Once the placement of fine grained impervious materials is completed in an area, a rockfill (Class C) will be placed on the impervious fill, thereby providing protection from ice and flowing water during winter months.

The construction of the Tailrace Summer Level Cofferdam and excavation of the Tailrace Channel is scheduled over two summer construction seasons. During the winter season the cofferdam will be overtopped by ice and water; it is expected that the cofferdam will sustain damage due to overtopping that will require repairs prior to dewatering for the second summer construction season.

It is expected that any damage that occurs to the cofferdam will occur during spring breakup of ice. At this time the river is already laden heavily with non-project related

suspended solids due to the scouring of ice along the river's edge. It is expected that TSS increases due to damage of the cofferdam will be relatively low.

The cross-section, design, and methods that will be employed to construct the Tailrace Summer Level Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Please note that although the construction steps, below, are described sequentially it is likely that the construction activities will take place concurrently. During construction the outer protective rockfill groin will be advanced first; construction of the inner rockfill zone of the cofferdam will likely follow close behind. Placement of the granular transition fill will follow the advancing fronts of the rockfill groins and placement of impervious fill will lag behind further still. The outer protective rockfill groin will lead the construction of the remaining zones by a distance that will ensure adequate protection from river flow. When the placement of impervious fill is complete; rockfill obtained from either quarries or from required excavations will be placed on top of the completed impervious blanket.

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Advance Outer Protective Rockfill Groin Construct the outer (wet-side) rockfill groin of the Tailrace Summer Level Cofferdam (Figure 9). This portion of the cofferdam will be founded on bedrock and some overburden. The rockfill will be placed in a downstream direction from the Powerhouse Stage I Cofferdam in an L-shape back towards the north shore. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the river. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of low silt content rock obtained from the required excavations for the Principal Structures. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam rockfill groin are due to the low velocities present along the downstream leg. Since the rockfill will have a low silt content and be placed in an area of low velocities, it is anticipated that the placement of rockfill will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 2 – Advancement of the Inner Rockfill Groin Construct the inner (dry-side) rockfill groin of the Tailrace Summer Level Cofferdam (Figure 9). This portion of the cofferdam will be founded on bedrock and some overburden. Construction of this rockfill zone will take place in an area of relatively tranquil water in the area upstream of the cofferdam's outer protective rock groin. The rockfill will be placed in a downstream direction from the Powerhouse Stage I Cofferdam in an L-shape back towards the north shore. The rockfill will be end-dumped close to the water's edge and subsequently pushed into the river. The rockfill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of low silt content rock obtained from the required excavations for the Principal Structures. The only potential sources of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam rockfill groin are due to the low velocities present along the downstream leg. Since the rockfill will have a low silt content and be placed in an area of low velocities, it is anticipated that the placement of rockfill will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 3 – Placement of Granular Transition Fill Place the granular transition fill on the wet-side slope of the Tailrace Summer Level Cofferdam inner rockfill groin (Figure 9). The granular transition fill will be placed between the previously constructed outer protective rockfill groin and the inner rockfill zone. The outer rockfill groin will be advanced ahead of the placement of transition fill enough to provide adequate protection from river flow. The only flow will be from the water leaking through the outer rockfill groin. Transition fill will also be placed on the wet side of the downstream leg of the inner rockfill zone beyond the extent of the outer protective rockfill groin, where the velocities are predicted to be low according to physical model results. The transition fill will be end-dumped on top of the impervious fill that has already been placed and	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential source of sediment that may enter the river is the washing of the fine grain size material by water flowing along the surface of this zone. Since these fill zones will be placed in a relatively tranquil water area located between the two rockfill groins, with little to no flow velocity, it is anticipated that this construction activity will not contribute any significant amount of sediment to the river.

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	
Construction Step 4– Placement of Impervious Fill Place impervious fill on the wet-side slope of the Tailrace Cofferdam (Figure 9). The impervious fill will be placed between the outer protective rockfill groin and the inner rockfill groin's zone of granular transition fill. The outer rockfill groin will be advanced ahead of the placement of impervious fill enough to provide adequate protection from river flow. The only flow will be from the water leaking through and around the outer protective rockfill groin. Impervious fill will also be placed on the wet-side of the downstream leg of the inner rockfill zone beyond the extent of the outer protective rockfill groin where the velocities are predicted to be low according to physical model studies. The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. Since these fill zones will be placed in a tranquil water area located between the two rockfill groins, with little to no flow velocity, it is anticipated that this construction activity will not contribute any significant amount of sediment to the river.
Construction Step 5 – Place Rockfill onto the Impervious Zone of the Cofferdam Place rockfill obtained from either quarries or required excavations on top of the impervious zone of the cofferdam. The rockfill will be placed on the exposed impervious fill zone likely by using an excavator and dozer.	The rockfill (Class C fill) will consist of clean blast rock or boulders having a low fines content obtained from the outer protective rockfill groin. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow. Since the rockfill will have low fines content,

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
	it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

*Please note that, although the construction steps are described sequentially, it is likely that the construction activities will take place concurrently.

2.3.11 SPILLWAY STAGE I COFFERDAM REMOVAL

Prior to Stage II Diversion, portions of the Spillway Stage I Cofferdam will be removed to the riverbed in a controlled fashion by the use of excavators. The maximum volume of the cofferdam that can be removed in-the-dry will vary depending on the flow in the Nelson River. Material excavated from the cofferdam will be loaded into trucks and hauled to excavated material placement areas in the vicinity.

In order to reduce the potential for uncontrolled erosion, the area between the upstream leg of the Spillway Stage I Cofferdam and Spillway will be flooded through a controlled breach of the cofferdam. This will create balanced head conditions across the upstream leg of the cofferdam and an area of tranquil water between the Spillway and the cofferdam. At this point in time the Spillway Gates are in their closed position.

Following the breaching of the cofferdam, excavation of the cofferdam's impervious, transition and inner rock fill zones will commence behind the cofferdam's outer protective rock groin. The outer protective rock groin will be excavated concurrent with the other zones but will lag far enough behind to give protection from the river's currents.

Prior to the removal of downstream leg of the Spillway Stage I Cofferdam, the Spillway Gates will be opened slightly; allowing the area downstream of the Spillway within the Spillway Stage I Cofferdam to be flooded. Excavation of the downstream leg of the cofferdam's impervious, transition and inner rock fill zones will commence with balanced head conditions across the cofferdam. The downstream leg is in an area of low velocity and the loss of fines into the river should be minimal.

As shown in Figure 8 Year 5(2), the Spillway Stage I Cofferdam upstream and downstream of the Spillway will only have to be partially removed to achieve the necessary approach and discharge flow conditions for long term Spillway operation. The section of the Spillway Stage I Cofferdam to the south of the Spillway will be left in place permanently. This south section will be used to connect the South Dam Stage II Upstream and Downstream Cofferdams to the northern portion of the South Dam.

The methods that will be employed to remove the Spillway Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for removal of this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Step 1 - Removal in-the-dry Excavate the top portion and dry side portions of the cofferdam in-the-dry. Most of the material will be excavated with an excavator and trucked to an excavated material placement area.	Since this activity will be conducted on dry ground, using best management practices to control erosion and sediment, it is anticipated that it will not contribute to sediment in the river.
Step 2 – Flooding of the Area Between the Upstream Leg of the Cofferdam and the Spillway Excavate through the rockfill, impervious fill and transition fill sections of the upstream leg of the Spillway Stage I Cofferdam, to create a small armoured channel to permit flooding of the area between the cofferdam and the Spillway. Material will be excavated by excavator and trucked to an excavated material placement area.	The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam. The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass.
Step 3 - Removal in-the-wet Excavate the balance of the cofferdam material to be removed in-the-wet, down to the riverbed. This portion of the cofferdam will be removed using excavators and hauled to excavated material placement areas.	Prior to commencing removal of the cofferdam in-the-wet, the area between the cofferdam and the Spillway will be flooded. This will create balanced head conditions on both sides of the cofferdam for wet excavation. The Spillway Gates will be in the closed position during this operation to reduce the potential for uncontrolled erosion. The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam. The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The potential sources of sediment that may enter the river are: (i) the disturbance of non-colloidal fine grain size material in the impervious and transition

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
	fill zones upstream and downstream, and possibly in the riverbed, by the excavation activities, and the washing of this material due to flow along the face of the cofferdam; (ii) saturated non-colloidal fine grain size material escaping from the excavator bucket and re-entering the water. Little to no sediment is anticipated to be created from the rockfill.
Step 4 – Armouring the Cofferdam Remnants Portions of the Stage I Cofferdam will not be removed. The newly exposed impervious and transition fill areas of these permanent cofferdam remnants, created during Step 1 and 3, will be armoured with riprap.	The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam. The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass.

2.3.12 STAGE II ISLAND COFFERDAM CONSTRUCTION

The Stage II Island Cofferdam will be a zoned earth and rockfill embankment. The cofferdam will consist of a central impervious core (Class A), with granular shells upstream and downstream. An upstream zone of riprap (Class C2) will be placed to provide erosion protection. A layer of road topping material (Class RT) will be placed on the crest of the cofferdam. This cofferdam will allow for the upstream areas of the Central Dam and Powerhouse to be retained in-the-dry during the Stage II Diversion period and the pre-impoundment stages of forebay impoundment that are linked to closure and construction of the Spillway Rollways.

The Stage II Island Cofferdam will be constructed by placing the fill material in-the-dry, founded directly on prepared overburden. The Stage II Island Cofferdam's impervious core will be tied into the Central Dam's impervious core.

The cross-section, design, and methods that will be employed to construct the Stage II Island Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Fill Placement	
Construct the impervious zone of the Stage II Island Cofferdam (Figure 9). The impervious fill will be placed from the high ground on Central Island and will tie into the core of the Central Dam. As work is to be completed in-the-dry, it is likely that the cofferdam fill material will be placed in successive lifts with impervious, granular and rockfill being placed concurrently.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The rockfill (Class C fill) will consist of low silt content rock. The Class C fill will consist of rock fragments of sufficient size to resist being carried downstream by flow against the cofferdam. The riprap zone (Class C2) will consist of low silt content rock. The Class C2 fill will consist of rock fragments of sufficient size to resist being carried downstream by ice and water flow against the cofferdam. Since these activities will be conducted on dry ground, using management practices to control erosion and sediment, it is anticipated that they will not contribute to sediment in the river.

*Please note that, although the construction steps are described sequentially, it is likely that the construction activities will take place concurrently.

2.3.13 SOUTH DAM STAGE II COFFERDAMS

The South Dam Stage II Upstream and Downstream Cofferdams will be zoned earth and rockfill embankments. The cofferdams will each consist of an impervious blanket (wet side) of silty sand till (Class A), a transition zone of granular fill (Class B) and a downstream rockfill groin (Classes C1, C2 and C3). A zone of riprap material will be placed on the upstream slope of the impervious blanket to provide erosion protection.

A layer of road topping material (Class RT) will be placed on the crest of the cofferdams. The cross-sections of the South Dam Stage II Cofferdams are shown in Figure 9.

The river is planned to be closed by advancing a single rockfill groin from the Spillway Stage I Cofferdam to tie-in to the south shore of the river. Minimal riverbed scour is expected in the gap between the closure leg and the river's south bank since high velocity water and ice have likely already removed the majority of the overburden in this area and

left the bed as being bedrock controlled. A bedrock outcrop is present on the south shore to elevations above that at which closure will take place. The closure rock groin will ultimately be incorporated into the cross-section of the South Dam Stage II Upstream Cofferdam which in-turn will be incorporated into the cross-section of the South Dam.

The rockfill groin for the South Dam Stage II Downstream Cofferdam could be advanced concurrently with the upstream rockfill groin; however, the groin needs to lag behind the upstream groin to stay out of high velocity flows. Analysis has determined that by advancing two-legs concurrently the head differential across the closure leg would be reduced, however, modeling has indicated that this approach is not required unless river flows are substantial at the time of river closure.

Following River Closure, the placement of the remaining fill material on the wet-side slope of the South Dam Stage II Upstream Cofferdam rockfill groin will commence. At this point the rockfill groin of the South Dam Stage II Downstream Cofferdam can be completed to the south shore.

Once the upstream cofferdam is sealed the construction of the transition and impervious zones of the South Dam Stage II Downstream Cofferdam will commence. Prior to this stage in construction, if the downstream cofferdam were sealed, pressure would build acting the opposite direction from that which the cofferdam was designed creating undesirable erosion.

The lower portions of the South Dam Stage II Cofferdams will be constructed in the water. The water levels during construction will vary, depending on the flow in the Nelson River. The fill for the lower portions will be placed to a level slightly above the water level that will exist at the time of construction. The remaining upper portions of this cofferdam will be constructed in-the-dry.

The cross-section, design, and methods that will be employed to construct the South Dam Stage II Cofferdams have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for these cofferdams, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 1 – Advance Rockfill Groins Construct the rockfill groins of the South Dam Stage II Upstream and Downstream Cofferdams (Figure 9). These cofferdams will be located across the South Channel of the Nelson River between the Central Island and the south shore of the Nelson River. The rockfill groins will be advanced from the Central Island and close against the south shore of the Nelson River. The South Dam Stage II Downstream Cofferdam's rockfill groin will lag behind the construction of the upstream cofferdam's rockfill groin by at least 30 m. The rockfill will be end-dumped near the river's edge and subsequently pushed into the water. The rockfill groin will initially be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The rockfill (Class C1 fill) will consist of low silt content rock. The Class C1 fill will consist of rock fragments of sufficient size to resist being carried downstream by the current. The only potential source of sediment that may enter the river is the washing of fine grain size material due to flow through and along the face of the rockfill. Construction of the upstream rockfill closure groin, along with the remnants of the Spillway Stage I Cofferdam, will create an area of relatively tranquil water immediately upstream and downstream of the cofferdam. Since the rockfill will have a low fines content and is founded on bedrock, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 2 – Place Transition Fill on South Dam Stage II Upstream Cofferdam Place granular transition fill on the wet-side slope of the South Dam Stage II Upstream Cofferdam rockfill groin (Figure 9). The transition fill will be end-dumped on top of the transition fill that has already been placed and pushed into the water. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry. The transition fill will further reduce seepage through the rockfill groin of the cofferdam.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. The only potential source of sediment that may enter the river is the washing of the finer grain size material by water seeping through the cofferdam or flowing along the slope. Initially, a small portion of even the coarse grained material may be washed through the rockfill, with the size of the particles being washed gradually reducing until essentially no washing is occurring. Since for the South Dam Stage II Upstream Cofferdam the transition fill will be placed in an area of relatively tranquil water in front of its completed rockfill groin, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

Description of Construction Activities*	Potential Sediment Problem and/or Primary Mitigation Techniques
Construction Step 3 – Place Impervious Fill on the South Dam Stage II Upstream Cofferdam Place impervious fill on the upstream face of the transition fill (Figure 9). The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The only potential source of sediment that may enter the river is the washing of silt size material contained in the impervious material that is exposed to the water as the mass of fill slides into the water. Since the impervious fill will be placed in an area of relatively tranquil water on top of granular filter material, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 4 – Place Transition Fill on South Dam Stage II Downstream Cofferdam Place granular transition fill on the wet-side face of the rockfill groin (Figure 9). The transition fill will be end-dumped on top of the transition fill that has already been placed and pushed into the water. The transition fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The granular transition fill (Class B fill) will consist of low silt content gravelly sand or gravel. Since the transition fill will be placed in an area of tranquil water in front of the rockfill zone of the Stage II Downstream Cofferdam, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Construction Step 5 - Place Impervious Fill on the South Dam Stage II Downstream Cofferdam Place impervious fill on the wet-side face of the transition fill (Figure 9). The impervious fill will be end-dumped on top of the impervious fill that has already been placed and pushed into the water so that it will enter the water as a sliding mass. The impervious fill zone will be constructed to a level that is slightly above the water level as it is advanced; it will subsequently be raised to final elevations in-the-dry.	The impervious fill (Class A fill) will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The only potential source of sediment that may enter the river is the washing of silt size material that is exposed to the water as the mass of fill slides into the water. Since the impervious fill will be placed in an area of tranquil water on top of granular filter material, it is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).

*Please note that, although the construction steps are described sequentially, it is likely that some of the construction activities will take place concurrently.

2.3.14 NORTH CHANNEL STAGE I COFFERDAM LOWERING & BREACHING

Prior to final impoundment of the Keeyask GS forebay, the North Channel Stage I Cofferdam will be lowered, in a controlled fashion, from a crest elevation of 154.0 m to 152.5 m by the use of excavators. The maximum volume of the cofferdam that can be removed in-the-dry will vary depending on the flow in the Nelson River. Prior to the removal of materials in-the-wet the area to the downstream of the cofferdam will be flooded in a controlled manner. This will allow for partial impoundment of the forebay area downstream and wet removal of remaining cofferdam materials under balanced head conditions. Allowing the river to flow through and over the rockfill portion of the North Channel Stage I Cofferdam during impoundment instead of overflowing the Stage II Island Cofferdam will minimize the scouring of the overburden and minimize the impacts on TSS levels in the river. The riverbed in the area of the North Channel Stage I Cofferdam is generally bedrock controlled.

Due to the configuration of the North Channel Stage I Cofferdam's layout, especially the position of the North Channel Rock Groin located to the upstream, the river's velocities in the vicinity of the North Channel Stage I Cofferdam will be relatively low. This layout, coupled with the controlled breaching of the cofferdam, and the subsequent resulting low water velocity, will help to minimize the dispersion of any fine material that is disturbed by the excavator during the cofferdam wet removal process. Since the cofferdam remnant below elevation 152.5 m will remain in place after forebay flooding, in addition to lowering the cofferdam, rockfill may be excavated from the downstream side of the cofferdam and placed on the upstream impervious zone in-the-wet to armour it against erosion.

The methods that will be employed to remove the North Channel Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for removal of this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Step 1 - Lowering in-the-dry Excavate the top portion of the cofferdam in-the-dry, above the water level in the river at the time of excavation.	Since this activity will be conducted on dry ground, using management practices to control erosion and sediment, it is anticipated that it will not contribute to sediment in the river.
Step 2 – Breaching the Cofferdam Excavate a trench through the rockfill, impervious and transition fill sections of the cofferdam, and armour as necessary prior to	The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam.

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
creating a controlled breach and flooding of the area downstream of the cofferdam.	The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The potential sources of sediment that may enter the river are the disturbance of non-colloidal fine grain size material in the impervious and transition fill zones by the excavation activities and the washing of this material due to flows through the breach trench.
Step 3 – Lowering in-the-wet and Armouring the Exposed Upstream Impervious Zone Excavate the balance of the cofferdam material to be removed in-the-wet, down to el 152.5 m (down to el 151.0 m at the upstream impervious zone). Excavate rockfill from the 'dry-side' of the cofferdam and place it overtop of the exposed upstream impervious zone (to el 152.5 m) in-the-wet to armour it against erosion.	The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam. The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The potential sources of sediment that may enter the river are: (i) the disturbance of non-colloidal fine grain size material in the impervious and transition fill zones, by the excavation activities, and the washing of this material due to flow along the face of the cofferdam; (ii) saturated non-colloidal fine grain size material escaping from the excavator bucket and re-entering the water.

2.3.15 NORTH CHANNEL ROCK GROIN LOWERING

The Final Design Phase will determine the elevations to which the cofferdam will be lowered. The elevation in this section is preliminary and may change.

The portion of the North Channel Rock Groin south of the wet well pump station and intake will be lowered from a crest elevation of 156.0 m to 153.0 m during impoundment of the reservoir but prior to impoundment to Full Supply Level. The work will be undertaken in a controlled fashion by the use of excavators. The maximum volume of the rock groin that can be removed in-the-dry will vary depending on the flow conditions in the Nelson River.

The lowering of the North Channel Rock Groin will be completed during Stage II Diversion likely prior to completion of the final concreting of the Spillway Rollways. During this period of river diversion water levels are not regulated by the Spillway Gates.

The portion of the North Channel Rock Groin from the wet well intake north to the North Dyke will remain at el 161.0 m to permit access for maintenance during operation of the well.

The methods that will be employed to lower the North Channel Rock Groin have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for lowering this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Step 1 - Removal in-the-dry Excavate the top portion of the groin in-the-dry.	Since this activity will be conducted on dry ground, using best management practices to control erosion and sediment, it is anticipated that it will not contribute to sediment in the river.
Step 2 - Removal in-the-wet Excavate cofferdam material in-the-wet, down to el 153.0 m, by means of an excavator.	Excavated materials will consist of low silt content rock obtained that has been subjected to flowing water during construction of the groin. Little to no sediment is anticipated to be created from the rockfill deposited downstream of the groin due to the clean nature of this material.

2.3.16 POWERHOUSE STAGE I COFFERDAM REMOVAL

The downstream leg of the Powerhouse Stage I Cofferdam will be removed in-the-dry in a controlled fashion behind the Tailrace Summer Level Cofferdam. It is anticipated that the removal of this portion of the cofferdam can be accomplished with both excavators and wheel loaders. The southern leg of the Powerhouse Stage I Cofferdam will be incorporated into the Transmission Tower Spur.

The methods that will be employed to remove the Powerhouse Stage I Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. Since this activity will be conducted

completely in-the-dry, using best management practices to control erosion and sediment, it is anticipated that it will not contribute to sediment in the river.

2.3.17 TAILRACE SUMMER LEVEL COFFERDAM REMOVAL

Upon completion of the Tailrace Channel and removal of as much of the cofferdam's fill as possible in-the-dry, the Tailrace Summer Level Cofferdam will be breached in a controlled fashion to equalize water levels on either side of the cofferdam. This will be followed by removal of the balance of the fill in-the-wet. The maximum volume of the cofferdam that can be removed in-the-dry will vary depending on the flow conditions in the Nelson River and the level of Stephens Lake which is regulated by Kettle GS. The inner fine grained zones of the cofferdam are planned to be removed behind the protective outer rockfill groin in relatively tranquil water. When the protective outer rockfill groin is no longer needed for protection, it will also be removed. Excavation is expected to be completed with excavators; material removed is to be hauled to nearby excavated material placement areas. The general arrangement after the removal of this cofferdam is shown in Figure 8, Year 6.

The methods that will be employed to remove the Tailrace Summer Level Cofferdam have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The construction steps for removal of this cofferdam, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Construction Activities	Potential Sediment Problem and/or Primary Mitigation Techniques
Step 1 - Removal in-the-dry Excavate the top portion of the cofferdam in-the-dry, above the water level at the time of excavation, as well as some of the rockfill material from the dry-side portion of the cofferdam.	Since this activity will be conducted on dry ground, using best management practices to control erosion and sediment, it is anticipated that it will not contribute to sediment in the river.
Step 2 – Breaching the Cofferdam Excavate and construct an armoured channel through the cofferdam to permit a controlled flooding of the area between the cofferdam and Powerhouse. Material will be excavated by excavator and trucked to an excavated material placement area.	Minimal excavation will be required therefore; the excavation is not anticipated to contribute significant sediment to the river. The armoured channel is designed to permit controlled flooding of the area enclosed by the cofferdam without erosion of the cofferdam fill and thus it is not anticipated to result in increases of TSS levels within the river.
Step 3 - Removal in-the-wet Prior to commencing the removal of the cofferdam in-the-wet, the area between the cofferdam and the Powerhouse will be flooded to reduce the potential for uncontrolled erosion (Step 2). The balance of the cofferdam material will be removed, in-the-wet, down to the riverbed. This remaining portion of the cofferdam will be removed using excavators. The impervious and granular transition zones will be excavated while protected from high velocity flows by the rockfill zone located along the outer side of the cofferdam.	The rockfill will consist of low silt content rock that has been subjected to flowing water during construction of the cofferdam. The granular transition fill will consist of low silt content gravelly sand or gravel, portions of which have been subjected to flowing water during construction of the cofferdam. The impervious fill will consist of well graded, unfrozen silty sand/sandy till that will form a homogeneous impervious mass. The potential sources of sediment that may enter the river are: (i) the disturbance of non-colloidal fine grain size material in the impervious and transition fill zones, by the excavation activities, and the washing of this material due to flow along the face of the cofferdam; (ii) saturated non-colloidal fine grain size material escaping from the excavator bucket and re-entering the water.

2.3.18 OPERATION OF THE SPILLWAY DURING CONSTRUCTION

Prior to Stage II Diversion, the upstream and downstream portions of the Spillway Stage I Cofferdam will be removed down to the riverbed to begin passage of river flow through the Spillway. The end slopes of the remnants of the Stage I Cofferdams will be armoured, as required, to prevent unacceptable erosion due to operation of the Spillway. Once the Stage II Cofferdams are complete, all of the river's flow will be diverted through the sluices in the partially complete Spillway.

During operation of the Spillway, the flow from the Spillway will discharge directly into the Spillway Discharge Channel excavated in the bedrock immediately to the downstream of the structure.

The Spillway will be operated during construction in a manner that will minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. The stages of operation for the Spillway, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Construction Activity	**Potential Sediment Problem and/or Primary Mitigation Techniques**
Stage 1 – Operation of Spillway during Stage II Diversion	
Erosion of overburden due to high flow velocities may occur due to operation of the Spillway during construction.	Due to the natural erosion of the Nelson River, very little river alluvium remains in the South Channel. Velocities have traditionally been high in this region causing very little sedimentation to occur along the channel path. It is possible that pockets of silts and sands do occur in the river channel. Since the overburden in this area is minimal, no increased erosion or sediment creation is anticipated.

2.4 GENERATING STATION – ON-LAND CONSTRUCTION ACTIVITIES

2.4.1 GENERAL

On-land construction activities for the Keeyask GS will include overburden excavation and the placement of fill for the North, Central and South Dams, as well as for the North and South Dykes. Overburden and rock excavation will be required for the Powerhouse and Spillway and their associated channels. Concrete placement operations will be required for the construction of the Powerhouse and Spillway structures. In addition to the work on the generating station facilities, on-land construction activities will be required for the completion of the North and South Access Roads linking the Principal Structures to PR 280 and the town of Gillam.

The potential sources of sediment release due to construction of the temporary and permanent structures include:

- overburden excavation,
- concrete production,
- construction material stockpiles,
- excavated slopes in soil,
- excavated materials disposal areas, and
- excavation dewatering.

The erosion and sediment control measures that will be employed for the on-land construction activities for the generating station will be consistent with Manitoba Infrastructure and Transportation's, "Manual of Erosion and Sedimentation Control" [Ref 2] and Department of Fisheries and Oceans' "Land Development Guidelines for the Protection of Aquatic Habitat" [Ref 3]. These documents emphasize the prevention of erosion, which comprises the first of the Primary Mitigation Techniques that will be employed for the Keeyask GS.

The potential sources of sediment and primary mitigation strategy for these on-land activities are discussed in the following sections.

2.4.2 OVERBURDEN EXCAVATION

Overburden excavation for the Keeyask GS considered in this study refer to the following: (i) the Powerhouse and Spillway structures (including channel excavations); (ii) the North and South Dykes, (iii) excavation to the foundation levels of the dams

(North, Central and South Dams), and (iv) improvements to forebay via lowering of local high areas.

The area of the Powerhouse and Tailrace Channel will require the excavation of significant depths and quantities of overburden. This will result in slopes of up to 9 m in height. Depending upon location, soil strength, groundwater conditions and height of cut, the slopes of the overburden excavations will generally vary from 1V:3.5H to 1V:4H but may be flatter if required. Once the forebay is at FSL el 159 m, the excavated upstream slope of the Intake Channel will be entirely submerged.

The Spillway and its channels will be cleared of overburden. Overburden depths are typically less than 1 m, consisting of cobbles and boulders overlying a thin layer of glacial tills of sand and gravel.

Discontinuous permafrost is present in the postglacial clays and silts and locally extends into the underlying tills at the axes selected for the North and South Dykes. Average depths of excavations of postglacial clays will be around 2 m and most of the dyke will be constructed on the surface of the tills. Typical slopes of the overburden excavations will vary from 1V:2H in deep excavations to 1V:4H with most being around 1V:3.5H to 1V:4H.

The North Dam is a very short earth and rockfill structure located on the north side of the Powerhouse. Overburden excavation slopes on the North Abutment for the North Dam will have a maximum height of approximately 8 m to10 m and will be excavated at a slope of 1V:4H.

The Central Dam is the longest of the three earth and rockfill dams, extending approximately 1.6 km between the Powerhouse and the Spillway. The thickness of the overburden deposits along the axis of the Central Dam is variable, ranging from zero up to approximately 14 m near the Powerhouse. Discontinuous permafrost is present in the postglacial clays and silts and locally extends into the underlying tills. The dam traverses several ridges and topographic lows covered by a 0.5 m to 1.5 m thick layer of peat, underlain by up to 2 m of postglacial clays and between 5 m to 11 m of sandy silty till deposits. Overburden will be excavated to the bedrock surface and will generally be at a slope of 1V:4H.

The South Dam extends 565 m between the Spillway and the south shore. High velocities and ice scour have likely removed most of the overburden deposits within the river channel along this axis but information is limited due to the rapid flow preventing field investigations. The South Abutment consists of recent deposits of peat overlying silty sands and gravels, over glacial till and/or inter-till deposits. The thickness of the overburden ranges from approximately 1 m to over 15.5 m and will be excavated at a slope

of 1V:4H. The majority of the South Dam will be founded on bedrock with the exception of the south bank where it will rise through the glacial till to tie into the South Dyke.

The erosion control measures will be consistent with recommendations contained in the "Manual of Erosion and Sedimentation Control During Highway Construction" [Ref 2] and "Land Development Guidelines for the Protection of Aquatic Habitat" [Ref 3]. Runoff from the excavation areas will be collected and passed through settling ponds (if required) prior to being discharged into the river.

2.4.3 CONCRETE PRODUCTION

The construction of the Keeyask GS will require the manufacture and placement of approximately 363,000 m^3 of concrete. In the current schedule, shown in Figure 13, most of the concrete will be placed during the summers of Years 3, 4, 5 and 6. Some concrete will be placed over the winters of Years 4/5, 5/6, and 6/7. The concrete will be manufactured in a concrete batch plant located in the Contractor's Work Area. The batch plant will be used to mix water, cement, aggregates and additives to produce the concrete. Water for the operation will be obtained from the Nelson River, while the cement will be delivered to site by truck throughout the duration of the project and may be stored temporarily in silos located adjacent to the batch plant. The aggregates required for use in the concrete will be produced from local rock and regional gravel sources. Fine aggregates will be sourced from Deposits G-1 and G-3 while coarse aggregates will be obtained from the excavations required for the Principal Structures or from quarries.

Fine aggregate for concrete will be a uniform medium sand with no more than 3% finer than silt size. Coarse aggregate for concrete will consist of crushed rock particles containing no more than 5% finer than coarse sand.

The production of aggregates for the concrete will involve the crushing, screening, washing and stockpiling of bedrock obtained from the project's required excavations, as well as granular material obtained from one of the investigated granular deposits. In order to permit the manufacture of concrete during the winter months, a heating system may be employed to maintain the concrete aggregate in an unfrozen state and at suitable temperatures.

The production of aggregate and concrete will involve a number of processes. These processes, along with potential sources of sediment and proposed additional Primary Mitigation Techniques, are summarized below:

Description	Potential Sediment Problem and/or Primary Mitigation Techniques
Concrete Fine Aggregate Production Fine aggregate for the production of concrete is anticipated to be obtained from Deposit G-1 and G-3. Screening, washing and stockpiling will be required to process the fine aggregates. The material from Deposit G-1 and G-3 are relatively clean, having an average of less than 5% finer than silt size.	Since the material from Deposits G-1 and G-3 are relatively clean, the washing process is not anticipated to produce wash water with high TSS levels. Since wash water from the concrete fine aggregate washing will be collected and passed through an adequately sized, two-cell settling pond prior to discharge back into the river, it is anticipated that aggregate washing will not contribute to sediment in the river.
Concrete Coarse Aggregate Production Rock from the required excavations for the concrete structures and channels as well as Quarry Q-7 will be suitable for use as coarse aggregate. Crushing, screening, washing and stockpiling will be required to process the coarse aggregates.	Since wash water from the concrete coarse aggregate washing will be collected and passed through an adequately sized, two-cell settling pond prior to discharge back into the river, it is anticipated that aggregate washing will not contribute to sediment in the river.
Concrete Production Wastewater Wastewater from the manufacture of concrete will be created as a result of washing of the batch plant and concrete hauling equipment.	Since wastewater from the manufacture of concrete will be collected and passed through an adequately sized, two-cell settling pond prior to discharge back into the river, it is anticipated that concrete production will not contribute to sediment in the river. Testing will also be undertaken before discharging to ensure that the pH levels are within the acceptable range.

2.4.4 CONSTRUCTION MATERIAL STOCKPILES

Stockpiles will be required for aggregate and concrete production, crushed/processed rock for fill and for organic materials for rehabilitation of the site after construction. These stockpiles will be utilized over a number of years.

Stockpiles of construction materials will be located in the Contractors' Work Area and within the Principal Structures area. Management of the stockpiles will be the responsibility of the Contractors.

The stockpiling methods that will be employed at Keeyask GS will be developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. A description of the required stockpiles, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below:

Description of Stockpiles	Potential Sediment Problem and/or Primary Mitigation Techniques
Aggregate for Concrete Production Stockpiles will be required for unwashed and washed/processed aggregate for concrete. The washed/processed aggregate will contain very small amounts of silt (<3%). Precipitation runoff will affect only a thin layer of material at the surface of the stockpiles. Since the stockpile material will be unsaturated, there will be very little percolation into the stockpiles.	The only potential source of sediment that may enter the river is the washing of fines from the stockpiles due to precipitation runoff. Since surface runoff will be directed to settling pond(s), as required, before being discharged into the river, it is anticipated that these stockpiles will not contribute to sediment in the river.
Unprocessed and Crushed/Processed Rockfill Stockpiles will be required for both unwashed/unprocessed and washed/processed rock. The crushing operation is anticipated to produce small amounts of silt size material, while the washed/processed aggregate will contain very small amounts of silt. Precipitation will tend to percolate into the stockpiles of coarse rockfill; however, the coarse rockfill will contain extremely small amounts of silt.	The only potential source of sediment that may enter the river is the washing of fines from the stockpiles due to precipitation runoff. Since surface runoff will be directed to settling pond(s), as required, before being discharged into the river, it is anticipated that these stockpiles will not contribute to sediment in the river.

Description of Stockpiles	Potential Sediment Problem and/or Primary Mitigation Techniques
Granular Fill Materials Stockpiles of granular fill for use in construction of the earthworks will be required. The granular material will have a low silt content, having no more than 5% or 7% silt size and finer material.	The only potential source of sediment that may enter the river is the washing of fines from the stockpiles due to precipitation runoff. Since surface runoff will be directed to settling pond(s), as required, before being discharged into the river, it is anticipated that these stockpiles will not contribute to sediment in the river.
Stockpiles for Organic Materials Stockpiles of organic materials will be required to supply material for re-vegetation of various areas of the site.	These stockpiles will have flat side slopes and will be protected from erosion, as required. The only potential source of sediment that may enter the river is the washing of fines from the stockpiles due to precipitation runoff. Since the stockpiles will incorporate erosion protection, as required and surface runoff will be directed to settling pond(s), as required, before being discharged into the river, it is anticipated that these stockpiles will not contribute to sediment in the river.
Stockpiles for Impervious Fill Stockpiles of impervious fill for use in the construction of the Main Dam and North Dyke will be required. These stockpiles will be relatively low in height and will have flat side slopes.	In order to prevent erosion, these stockpiles will have a compacted surface, with a slight crown on the top of the stockpile to prevent ponding of water on the surface. The only potential source of sediment that may enter the river is the washing of fines from the stockpiles due to precipitation runoff. Since the stockpile will have a compacted surface, and surface runoff will be directed to settling pond(s), as required, before being discharged into the river, it is anticipated that these stockpile will not contribute to sediment in the river.

2.4.5 DISPOSAL OF SURPLUS UNCLASSIFIED MATERIALS DURING CONSTRUCTION

Approximately 3 million m^3 of all types of earth and rock materials, mostly obtained from the excavations which are required for the construction for the generating station, or from removal of the cofferdams at the end of their use, will be required to be disposed of in the immediate site vicinity. These materials will be placed in a controlled manner, within designated disposal areas. Unclassified overburden materials disposal areas will

be located both inside and outside the forebay. Disposal sites identified are shown in Figures 10 and 11.

The disposal sites will consist primarily of postglacial till and clays, blast rock, and small amounts of granular material. Disposal of these materials will be directed so as to ensure runoff will not increase TSS within the Nelson River and its tributaries. The disposed materials will be placed, spread and compacted, where appropriate, however, general compaction in the disposal areas will be accomplished through the random movements of the placing equipment. The outer shapes of the area will be made such as to ensure its long-term stability and to accommodate the topography of the area. The final slopes will be flattened in order to reduce the potential for erosion.

(a) Surplus Material Disposal Sites Outside the Dyke Lines

Disposal areas will be chosen based primarily on their proximity to the project and areas being excavated. Topographic lows will be filled first, thereby taking advantage of the natural confinement that natural low areas offer. Where natural confinement does not exist berms will be constructed to confine disposed surplus materials to the disposal areas. Surface runoff will drain to a settling pond contained within each disposal site with controlled outflow to natural drainage courses or purpose made channels.

Where possible, buffer zones of approximately 100 m will be maintained between any dykes and creeks and the disposal areas when disposed outside of the forebay. Slopes of deposited materials will be kept to a maximum grade of 5H:1V to prevent excessive washout and erosion and will have a maximum height of 3 m.

After final grading, the exposed overburden slopes will be re-vegetated.

(b) Surplus Material Disposal Sites Inside the Dyke Lines

Disposal sites selected within the Keeyask forebay area are low lying areas in close proximity to the areas being excavated. Topographic lows will be filled to confine the unclassified materials to the chosen areas. Where natural confinement is not available berms will be constructed to confine disposed materials to the disposal areas.

The built-up maximum elevation of the disposal sites will not exceed the elevations shown in Figure 11. These elevations represent the maximum elevation of the disposal sites before there is a potential for erosion and transportation of the deposited material by the flowing water during minimum forebay level operation of the plant.

Where possible and prior to impoundment, a buffer zone of approximately 50 m will be maintained between the river edge and the disposal areas. Slopes of deposited materials will be kept to a stable grade to prevent excessive scouring and erosion.

The disposal areas that will be created at Keeyask GS have been developed to minimize the amount of sediment produced. These measures comprise the first of the Primary Mitigation Techniques. A description of the required disposal areas, along with potential sediment problems and proposed additional Primary Mitigation Techniques, are summarized below.

Description of Disposal Area	Potential Sediment Problem and/or Primary Mitigation Techniques
Disposal Areas DN-A1, DN-A2 & DN-A3 Disposal of surplus unclassified materials into Disposal Areas DN-A1, DN-A2 and DN-A3. The disposal sites will be flooded upon impoundment of the forebay.	Placement of materials will be completed in-the-dry prior to impoundment of the forebay behind the Stage I Cofferdams and Stage II Cofferdams and Principal Structures. Due to the velocity of water present in the area, these disposal areas may need to be armoured prior to submergence to prevent erosion of the disposal material. The amount and location of armouring required will depend on the final height of disposal area, velocity of water, and composition of fill placed in the disposal area. Drainage will be contained within the Stage I and Stage II Cofferdams and the Principal Structures. Surface water will be drained to adequately sized settling ponds prior to its controlled release into the river.
Disposal Areas DS-A, DN-B & DN-C Disposal of surplus unclassified materials into Disposal Areas DS-A, DN-B and DN-C. The areas that are defined for the disposal sites are naturally low lying and are within the planned dyke area. The disposal sites will be flooded upon impoundment of the forebay.	Placement of materials will be completed in-the-dry prior to impoundment of the forebay behind the Stage I Cofferdams and Stage II Cofferdams and the Principal Structures. Surface water will be drained to adequately sized settling ponds within the disposal sites prior to its controlled release into the river.
Disposal Area DN-A4 Controlled disposal of surplus unclassified materials into Disposal Area DN-A4. The surplus unclassified materials will be placed in-the-wet in low velocity water. The surplus unclassified fill will be end-dumped close to the	Placement of surplus unclassified materials will be protected locally, as required, to prevent TSS laden water from entering the watercourse. Due to the velocity of water present in the area, these disposal areas may need to be armoured

Description of Disposal Area	Potential Sediment Problem and/or Primary Mitigation Techniques
water's edge and subsequently pushed into the river.	prior to submergence to prevent erosion of the disposal material. The amount and location of armouring required will depend on the final height of disposal area, velocity of water, and composition of fill placed in the disposal area. It is anticipated that this construction activity will not produce increases in TSS levels above those that would trigger the implementation of secondary mitigation measures (see Subsection 2.1.4).
Disposal Areas Outside the Dyke Lines (DN1A, DN1B, DN1C, DN2, DN3, DN4, DN5, DS1, DS3, DS3, DS4 & DS5) Place surplus unclassified materials into disposal areas outside of the dyke line. Placement of the materials will be completed in-the-dry into the areas that are defined for the disposal sites are generally naturally low lying. The disposal sites will not be flooded upon impoundment of the forebay.	Placement of materials will be completed in-the-dry prior to throughout all stages of the construction of the generating station. Surface water will be drained to adequately sized settling ponds within each of the disposal sites prior to its controlled release.

3 DRAINAGE MANAGEMENT

Drainage of surface runoff at the Keeyask Site will be controlled to minimize the amount of sediment produced and the potential for sediment to enter watercourses. In general, the erosion control measures will be consistent with recommendations contained in the "Manual of Erosion and Sedimentation Control During Highway Construction" [Ref 2] supplemented by "Land Development Guidelines for the Protection of Aquatic Habitat" [Ref 3] and will include, but not necessarily limited to, the installation of:

- settling ponds,
- silt fences,
- check dams,
- ditch armoring (riprap),
- water turnout,
- erosion control blankets, and
- vegetation buffer zones.

The control of surface drainage will be accomplished by design of the earthworks, installation of temporary and permanent erosion protection measures, and by selection of

the methods that will be employed during construction. These measures comprise the first of the Primary Mitigation Techniques, and are described below.

The ditches at the Keeyask Site have been designed to prevent erosion of the silts and sands that generally blanket the area, as described in Subsection 2.2.4, consequently no significant erosion of the ditches is anticipated.

The drainage ditches will be set parallel to the centerline of the road embankment and will have off-take ditches, at or near perpendicular to the road alignment. The off-take ditches will transport the surface runoff (snowmelt/rainfall events) to low lying areas such as marshes, and settling ponds, eventually draining into natural streams and ponds. For areas where a low-lying area is not near a natural drainage outfall, a "water turnout" will be installed to disperse the runoff into nearby vegetated areas. Undisturbed natural vegetation has a high resistance to erosion from discharged water and is very effective in removing sediment loads. Small rock berms and deltas will be used at the outlet of the water turnouts to diffuse the water into the undisturbed terrain.

Surface runoff at the Keeyask Site will be managed by means of a network of swales, culverts and ditches within and around the roadways and structures. Ditch capacities will be sized to accommodate the extreme daily rainfall event. Flow volumes and velocities will be kept low enough to minimize erosion and scouring of the drainage area. Riprap energy dissipaters and ditch lining will be installed in areas where velocities are deemed to be excessive. The surface runoff from the remaining site area will be collected and discharged onto a vegetated buffer zone wherever possible, from where it will flow into a drainage inlet of the Nelson River.

Settling ponds will be built to permit the silt and fine particles to settle out. Clarified water from the ponds will be reused in the aggregate production, equipment washing, site cleaning, and dust control. Waste sediments resulting from aggregate processing will be conveyed to and disposed of in a designated area adjacent to the plants. Where necessary, an oil/water separator will also be employed to conform to provincial regulations. Waste water being discharged into the river will be free of excessive amounts of sediment and other contaminants, as required by provincial regulations.

River water that has been contained within a cofferdamed area will be discharged into the river after allowing excess sediment to settle.

Surface runoff and seepage entering the excavations required for construction of the generating station will be allowed to pond in selected areas of the excavation to allow sediment to settle prior to the water being discharged into the river.

The dyke alignments were selected along higher ground and as such the ponding of water against the downstream toe of the dykes is not anticipated to occur. However, any seepage and surface runoff (snowmelt/rainfall events) will be intercepted with a surface drainage ditch, located at the downstream toe of the dyke.

The drainage ditches will be set parallel to the centerline of the dyke line and will have off-take ditches, at near perpendicular to the dyke's axis, which will transport water into low-lying areas such as marshes, natural streams and ponds. Some of the potential low-lying areas in which the drainage ditches may outfall into are identified in Table 3. For the cases where a low-lying area is not near the drainage outfall, a "Water Turnout" will be installed. A Water Turnout is defined as an outfall of a drainage ditch set into a naturally vegetated area to provide dispersion and filtration of the discharged water. Undisturbed natural terrain has a high resistance to erosion and is very effective in removing sediment loads. In addition, rocks will be used at the outlet of the Water Turnout to diffuse the water into the undisturbed terrain.

Table 3
Keeyask GS – Stage IV Studies, Axis GR-4
Potential Low-Lying Areas Adjacent to the North and South Dykes

Outfall Area (watercourse)	Description
Ephemeral Stream	A watercourse generally without a well-defined channel that flows only in response to rainfall or snowmelt. Ephemeral streams flow less than 20% of the year during normal rainfall conditions.
Headwater Swamp	An area of potentially saturated land at the source of a flowing water body.
Intermittent Stream	A watercourse that flows in a well-defined channel for 20% to 90% of the year during normal rainfall conditions.
Muck Swamp	A very poorly drained area, usually with standing water, characterized by heavy organic matter accumulation.
Wetlands	Areas, which are generally concave or low-lying topographic forms that collect, store or flow water frequently.
Perennial Stream	A watercourse that flows continuously (at least 90% of the year) in a well-defined channel.
Peat Swamp	A poorly drained area with heaving accumulations of raw organic matter, resembling muck swamps.

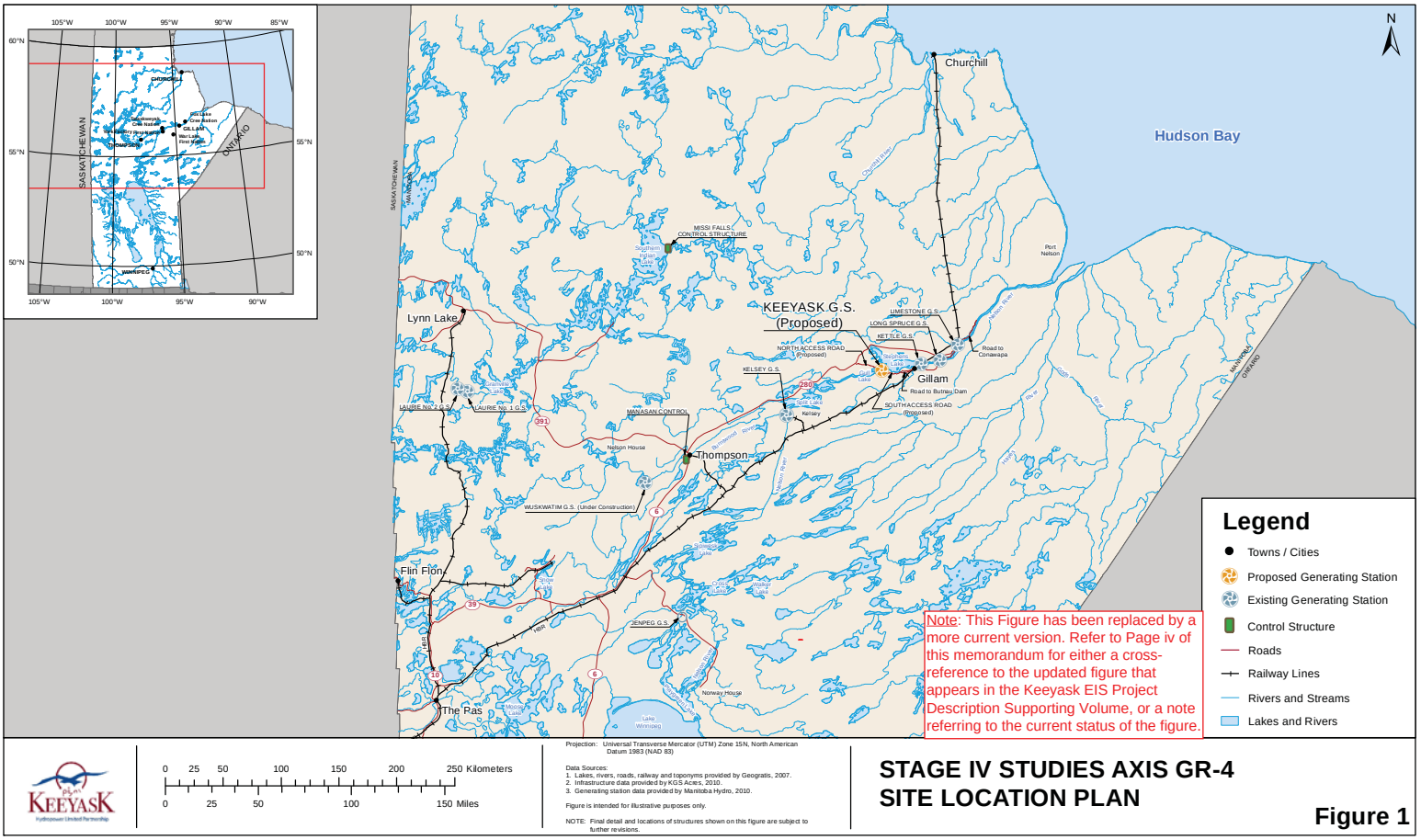
4 SEDIMENT MONITORING PROGRAM

Manitoba Hydro is preparing a detailed in-stream TSS monitoring program that will be implemented by the Manitoba Hydro construction management team throughout the course of construction to monitor the magnitude, spatial, and temporal scope of TSS increases in the Nelson River. The monitoring program will be used to help manage in-stream construction activities to insure that effects on TSS do not exceed limits imposed by regulators (limits not yet specified). Manitoba Hydro has developed a draft of the Instream Construction Sediment Management Plan which should be referenced.

REFERENCES

1. Department of Fisheries and Oceans and Manitoba Natural Resources 1996 – "Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat", May 1996.
2. Manitoba Transportation and Government Services 1995 – "Manual of Erosion and Sedimentation Control During Highway Construction".
3. Department of Fisheries and Oceans 1993 – "Land Development Guidelines for the Protection of Aquatic Habitat", Second Printing, September 1993.
4. Manitoba Transportation and Government Services 2004 – "Grading and Surfacing Specifications".

FIGURES



Note: This Figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Lakes, rivers, roads, railway and toponyms provided by Geogratis, 2007.
2. Infrastructure data provided by KGS Acres, 2010.
3. Generating station data provided by Manitoba Hydro, 2010.

Figure is intended for illustrative purposes only.

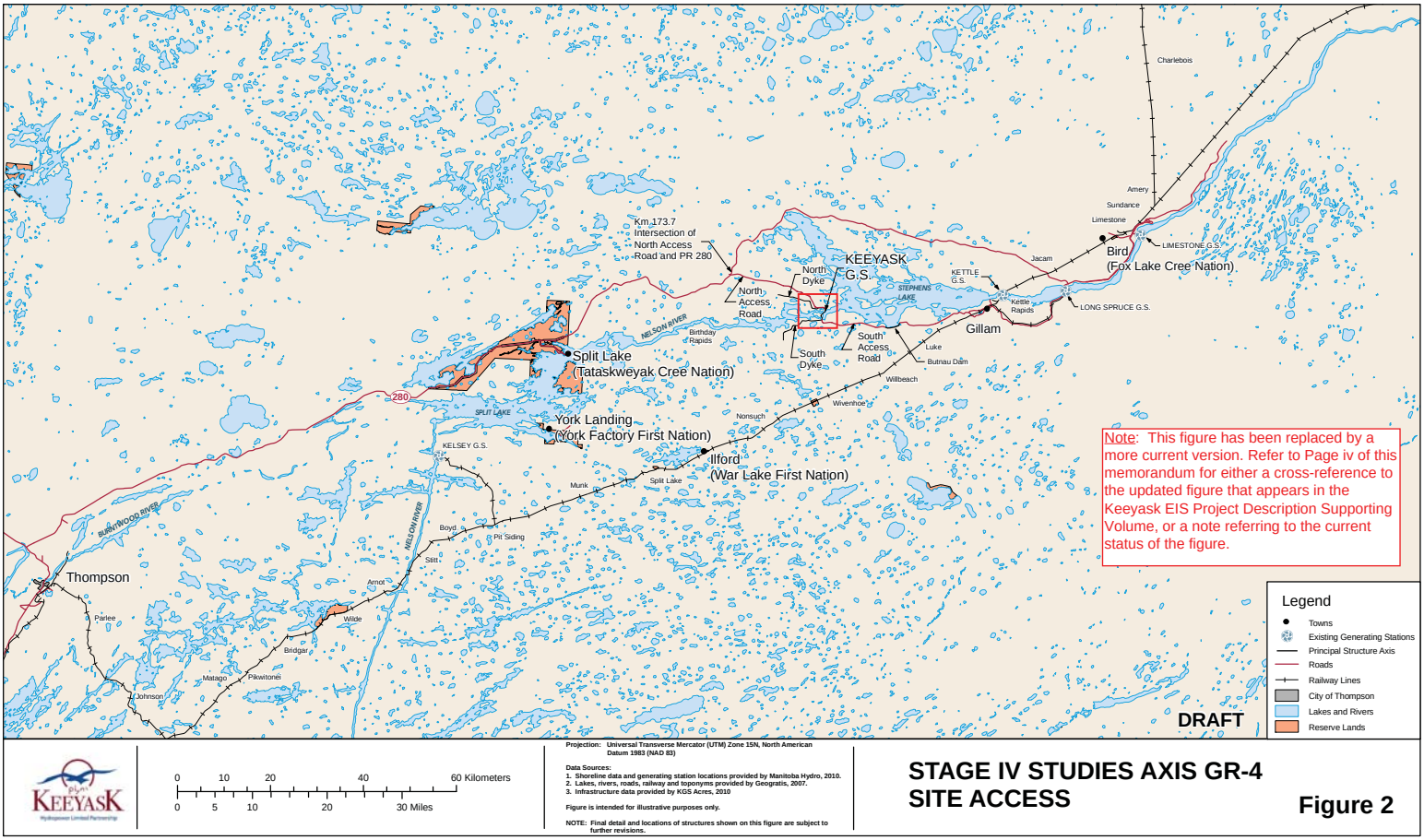
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 SITE LOCATION PLAN

Figure 1

Created By: KGS ACRES

Date Created: December 20, 2011



0 10 20 40 60 Kilometers

0 5 10 20 30 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data and generating station locations provided by Manitoba Hydro, 2010.
2. Lakes, rivers, roads, railway and toponyms provided by Geogratis, 2007.
3. Infrastructure data provided by KGS Acres, 2010

Figure is intended for illustrative purposes only.

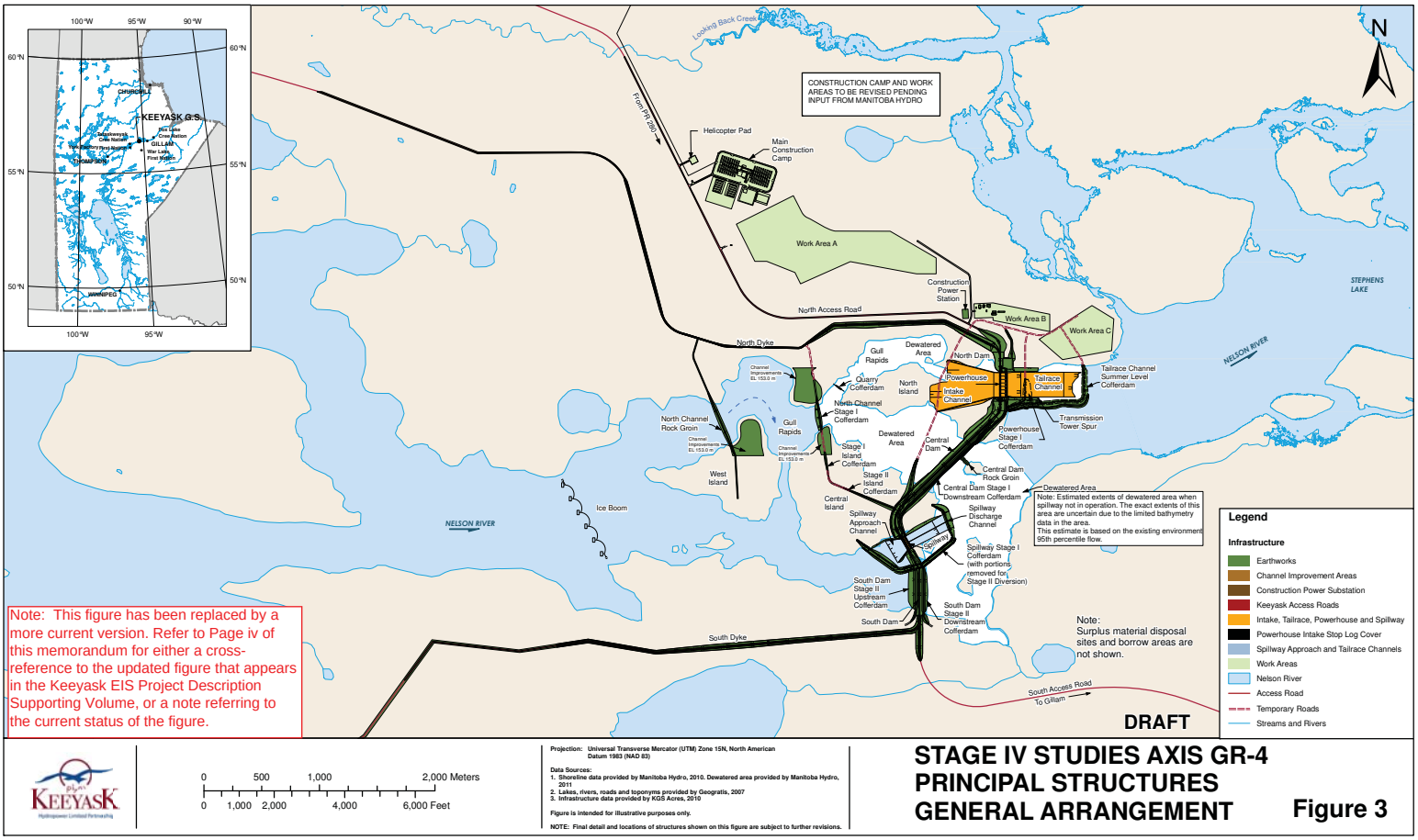
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 SITE ACCESS

Figure 2

Created By: KGS ACRES

Date Created: February 04, 2011



Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

0 500 1,000 2,000 Meters
0 1,000 2,000 4,000 6,000 Feet

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010. Dewatered area provided by Manitoba Hydro, 2011.
2. Lakes, rivers, roads and toponyms provided by Geogratis, 2007
3. Infrastructure data provided by KGS Acres, 2010

Figure is intended for illustrative purposes only.

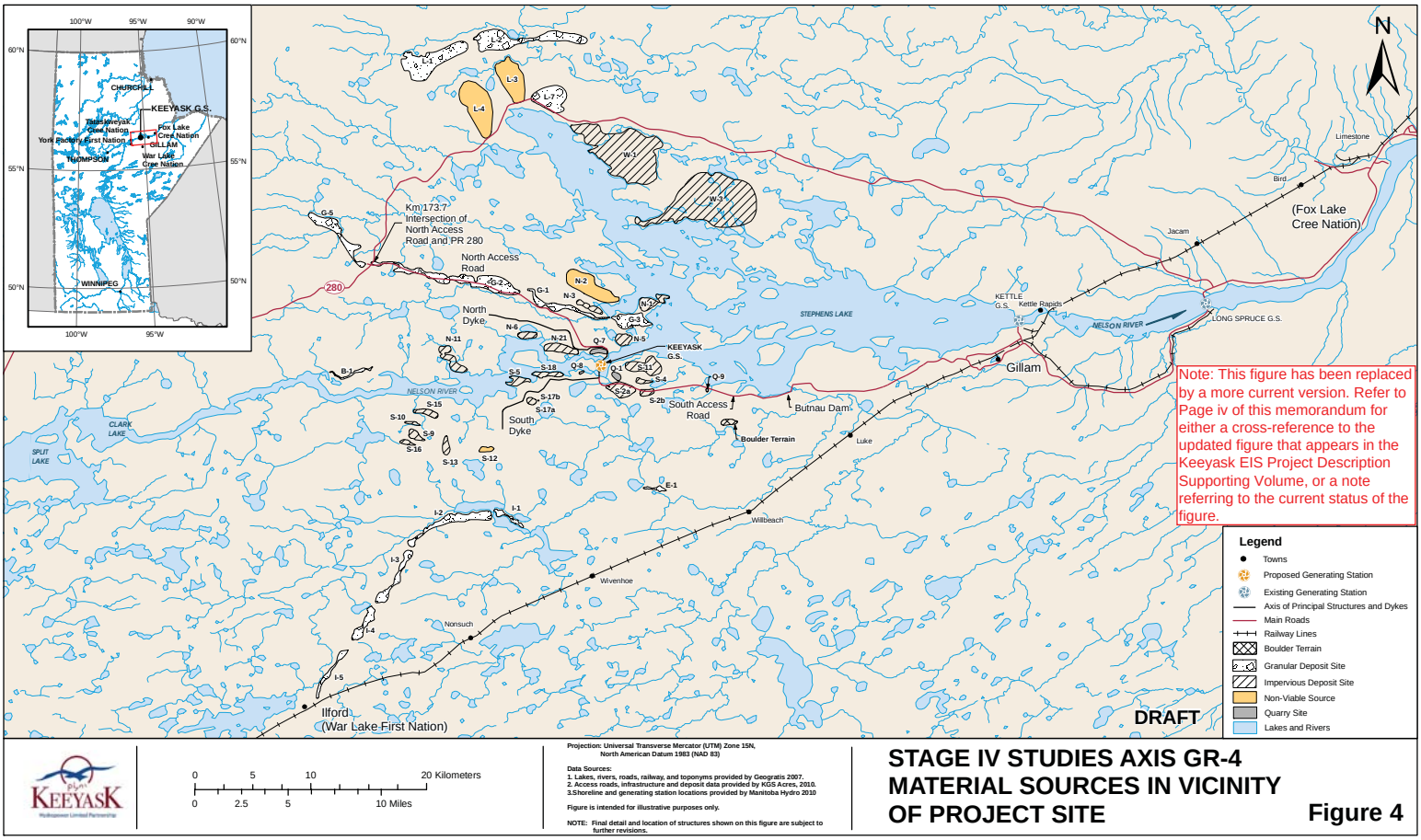
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 PRINCIPAL STRUCTURES GENERAL ARRANGEMENT

Figure 3

Created By: KGS ACRES

Date Created: February 04, 2011



0 5 10 20 Kilometers

0 2.5 5 10 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Lakes, rivers, roads, railway, and toponyms provided by Geogratis 2007.
2. Access roads, infrastructure and deposit data provided by KGS Acres, 2010.
3.Shoreline and generating station locations provided by Manitoba Hydro 2010

Figure is intended for illustrative purposes only.

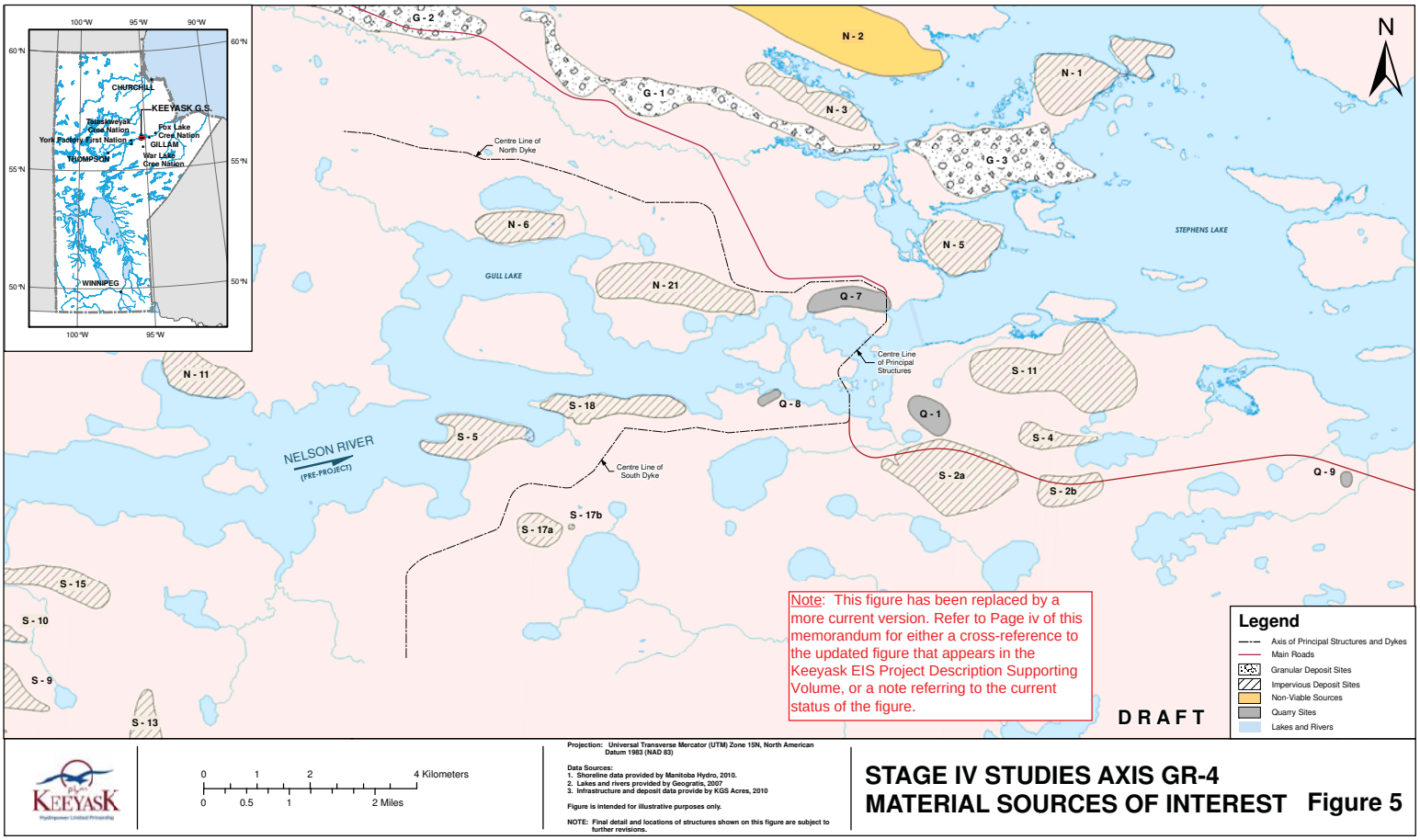
NOTE: Final detail and location of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 MATERIAL SOURCES IN VICINITY OF PROJECT SITE

Figure 4

Created By: KGS ACRES

Date Created: February 04, 2011



Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010.
2. Lakes and rivers provided by Geogratis, 2007
3. Infrastructure and deposit data provide by KGS Acres, 2010

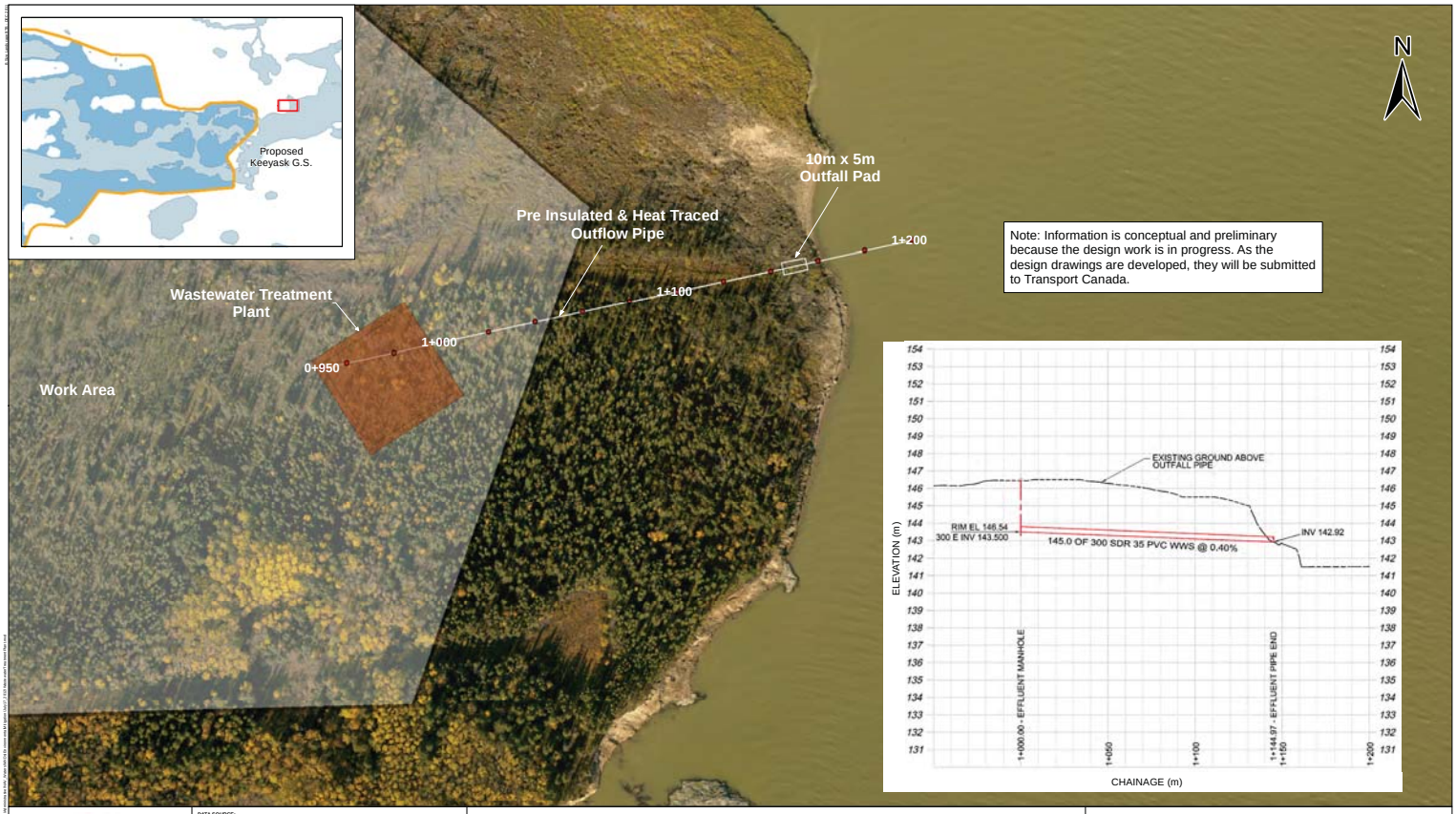
Figure is intended for illustrative purposes only.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 MATERIAL SOURCES OF INTEREST Figure 5

Created By: KGS ACRES

Date Created: February 04, 2011



KEEYASK
Hydropower Limited Partnership

DATA SOURCE:
KGS Acres; Manitoba Hydro
Manitoba Hydro 2010 10cm Colour DOI

CREATED BY:
Manitoba Hydro - Hydro Power Planning Department

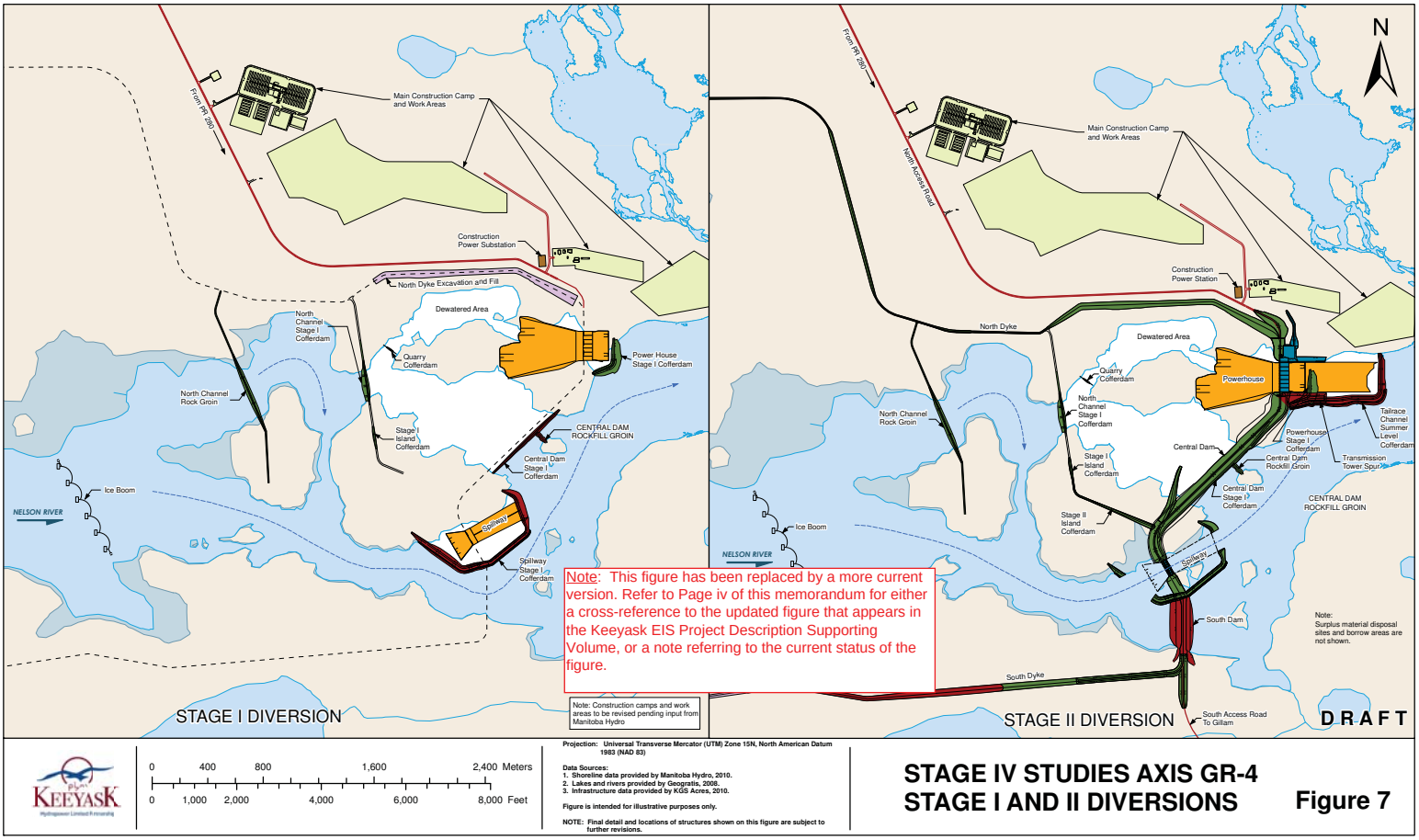
COORDINATE SYSTEM:	DATE CREATED:	REVISION DATE:
UTM NAD 1983 Z15N	04-APR-12	19-JUL-12
0 0.015 0.03 Kilometres 0 0.01 0.02 Miles	VERSION NO: 1.0	QA/QC: APPROVED

Legend

- Wastewater Treatment Plant
- Work Area

Effluent Outflow From Wastewater Treatment Plant

Figure 6



Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

D R A F T

0 400 800 1,600 2,400 Meters
0 1,000 2,000 4,000 6,000 8,000 Feet

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010.
2. Lakes and rivers provided by Geogratis, 2008.
3. Infrastructure data provided by KGS Acres, 2010.

Figure is intended for illustrative purposes only.

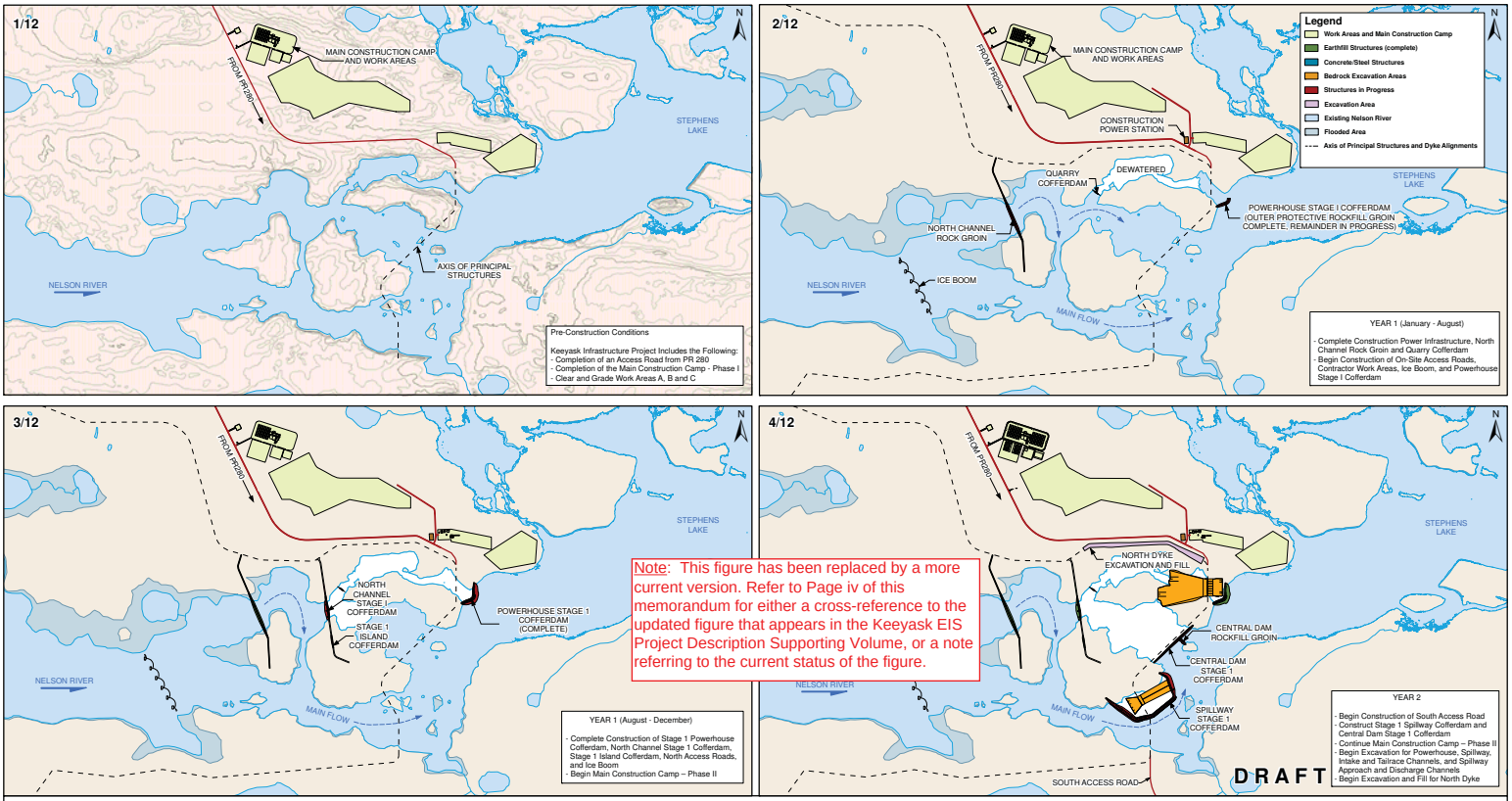
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4
STAGE I AND II DIVERSIONS

Figure 7

Created By: KGS ACRES

Date Created: February 04, 2011



Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

0 0.5 1 2 3 4 Kilometers
0 0.5 1 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010.
2. Lakes and rivers provided by Geogratis 2007
3. Monitor point, access roads and infrastructure data provided by KGS Acres, 2010.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

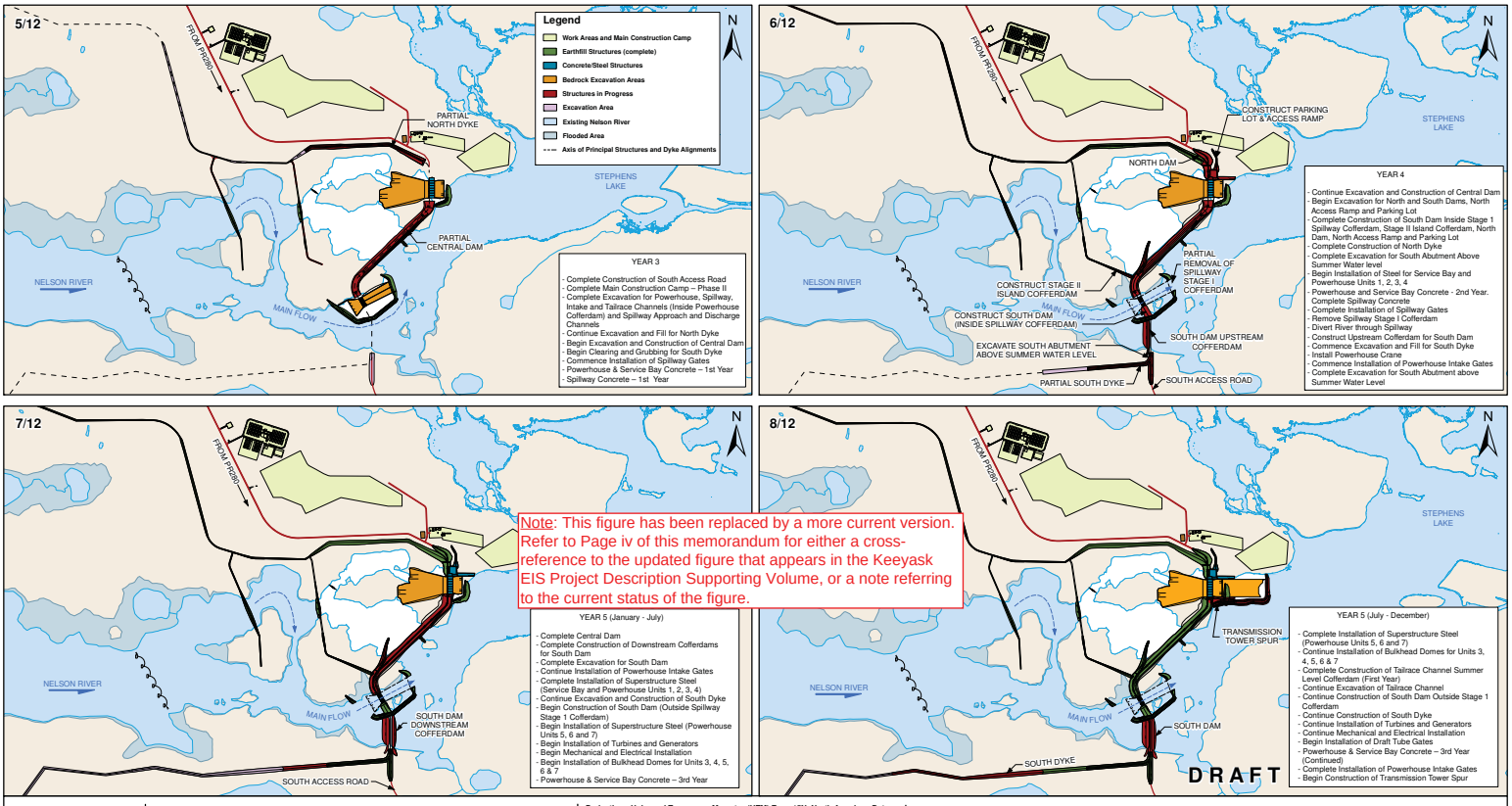
Figure is intended for illustrative purposes only.

STAGE IV STUDIES AXIS GR-4 CONSTRUCTION SEQUENCE

Figure 8 (Sheet 1 of 3)

Created By: KGS ACRES

Date Created: February 04, 2011



Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

KEEYASK Hydropower Limited Partnership

0 0.5 1 2 3 4 Kilometers
0 0.5 1 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010.
2. Lakes and rivers provided by Geogratis 2007
3. Monitor point, access roads and infrastructure data provided by KGS Acres, 2010.

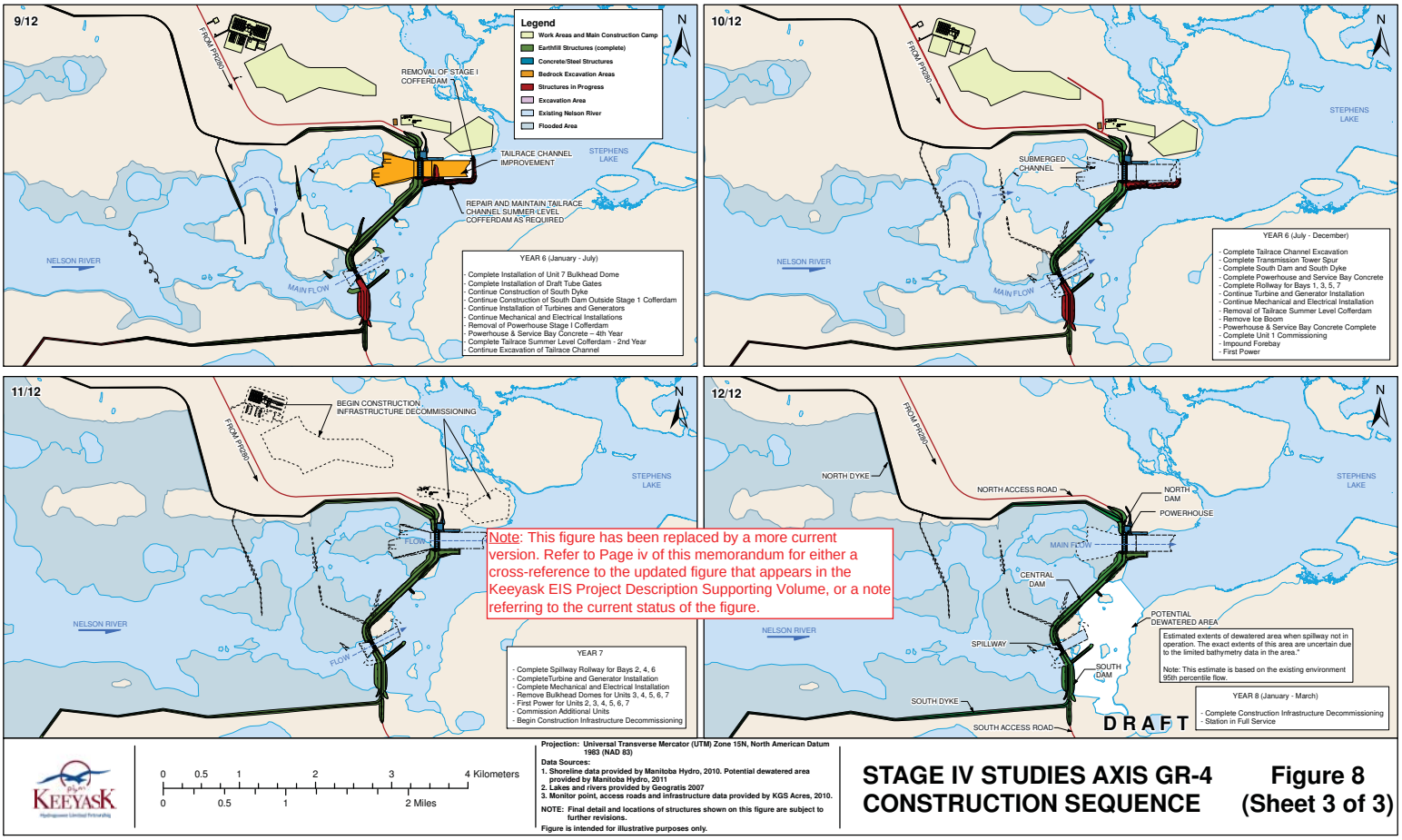
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.
Figure is intended for illustrative purposes only.

STAGE IV STUDIES AXIS GR-4 CONSTRUCTION SEQUENCE

Figure 8 (Sheet 2 of 3)

Created By: KGS ACRES

Date Created: February 04, 2011



Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

KEEYASK

0 0.5 1 2 3 4 Kilometers
0 0.5 1 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010. Potential dewatered area provided by Manitoba Hydro, 2011
2. Lakes and rivers provided by Geogratis 2007
3. Monitor point, access roads and infrastructure data provided by KGS Acres, 2010.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

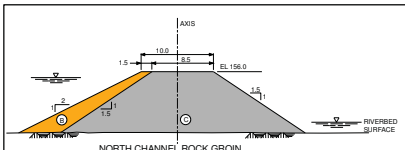
Figure is intended for illustrative purposes only.

STAGE IV STUDIES AXIS GR-4 CONSTRUCTION SEQUENCE

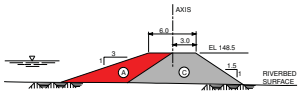
Figure 8 (Sheet 3 of 3)

Created By: KGS ACRES

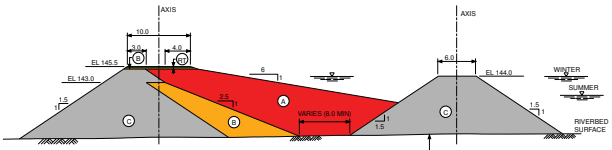
Date Created: February 04, 2011



NORTH CHANNEL ROCK GROIN

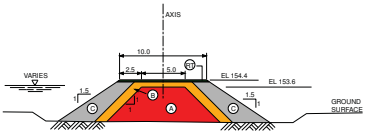


QUARRY COFFERDAM

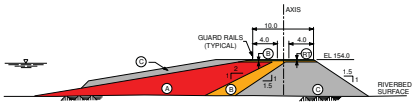


POWERHOUSE STAGE I COFFERDAM
STEP I (INITIAL)

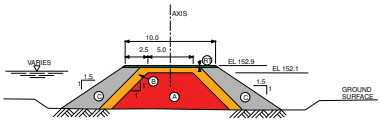
INITIAL PROTECTIVE ROCK GROIN (SEE NOTE 3)



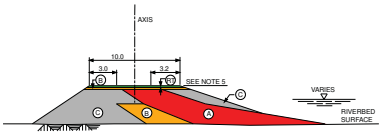
ISLAND STAGE I COFFERDAM



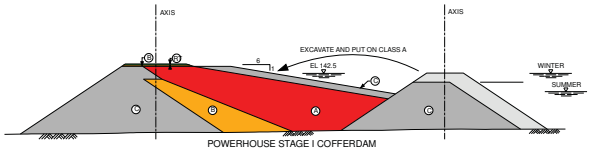
NORTH CHANNEL STAGE I COFFERDAM



ISLAND STAGE II COFFERDAM

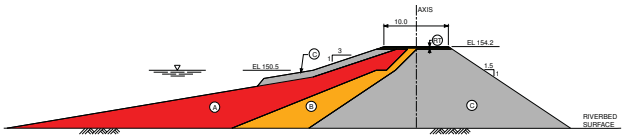


CENTRAL DAM STAGE I DOWNSTREAM COFFERDAM

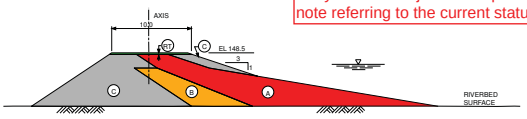


POWERHOUSE STAGE I COFFERDAM
STEP II (FINAL)

Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.



SOUTH DAM STAGE II UPSTREAM COFFERDAM



SOUTH DAM STAGE II DOWNSTREAM COFFERDAM

DRAFT

LEGEND

- A — Impervious Fill
- B — Granular Transition Fill
- C — Rock Fill (See Note 4)
- RT — Road Topping
- Water Level

Notes:

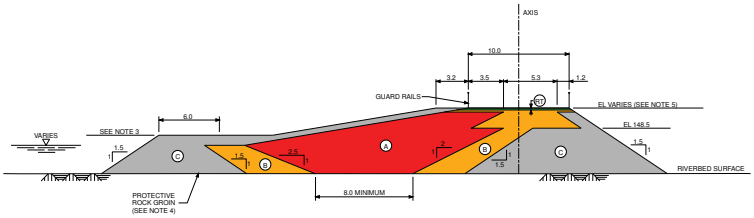
1. Dimensions and elevations are in metres.
2. Elevations are based on GS OF C, CGVD28 and 1929 adjustment.
3. Initial protective rock groin not required for powerhouse cofferdam, downstream of tailrace channel.
4. Various sizes and gradations of rockfill (classes C1, C2 and C3, will be utilized. Precise locations of utilization will be based on flow conditions.
5. Crest elevation varies from EL 146.5 at its downstream abutment to EL 147.5 at its upperstream abutment.

STAGE IV STUDIES AXIS GR-4 COFFERDAM CROSS-SECTIONS

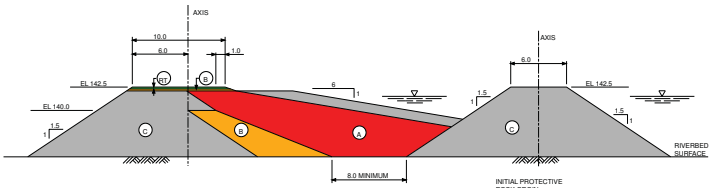
Figure 9 (Sheet 1 of 2)

Created By: KGS ACRES

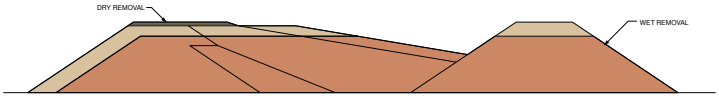
Date Created: February 04, 2011



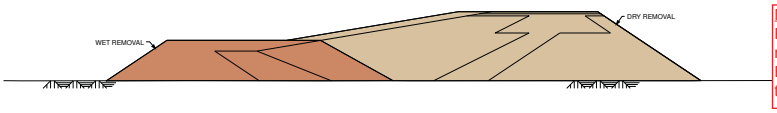
SPILLWAY STAGE I COFFERDAM
STEP 1



TAILRACE SUMMER LEVEL COFFERDAM
STEP 1



TAILRACE SUMMER LEVEL COFFERDAM
STEP 2



SPILLWAY STAGE I COFFERDAM
STEP 2

Note: This figure has been replaced by a more current version. Refer to Page iv of this memorandum for either a cross-reference to the updated figure that appears in the Keeyask EIS Project Description Supporting Volume, or a note referring to the current status of the figure.

LEGEND

- A — Impervious Fill
- B — Granular Transition Fill
- C — Rock Fill (See Note 6)
- RT — Road Topping
- Dry Removal
- Wet Removal
- Water Level

DRAFT

NOTES:

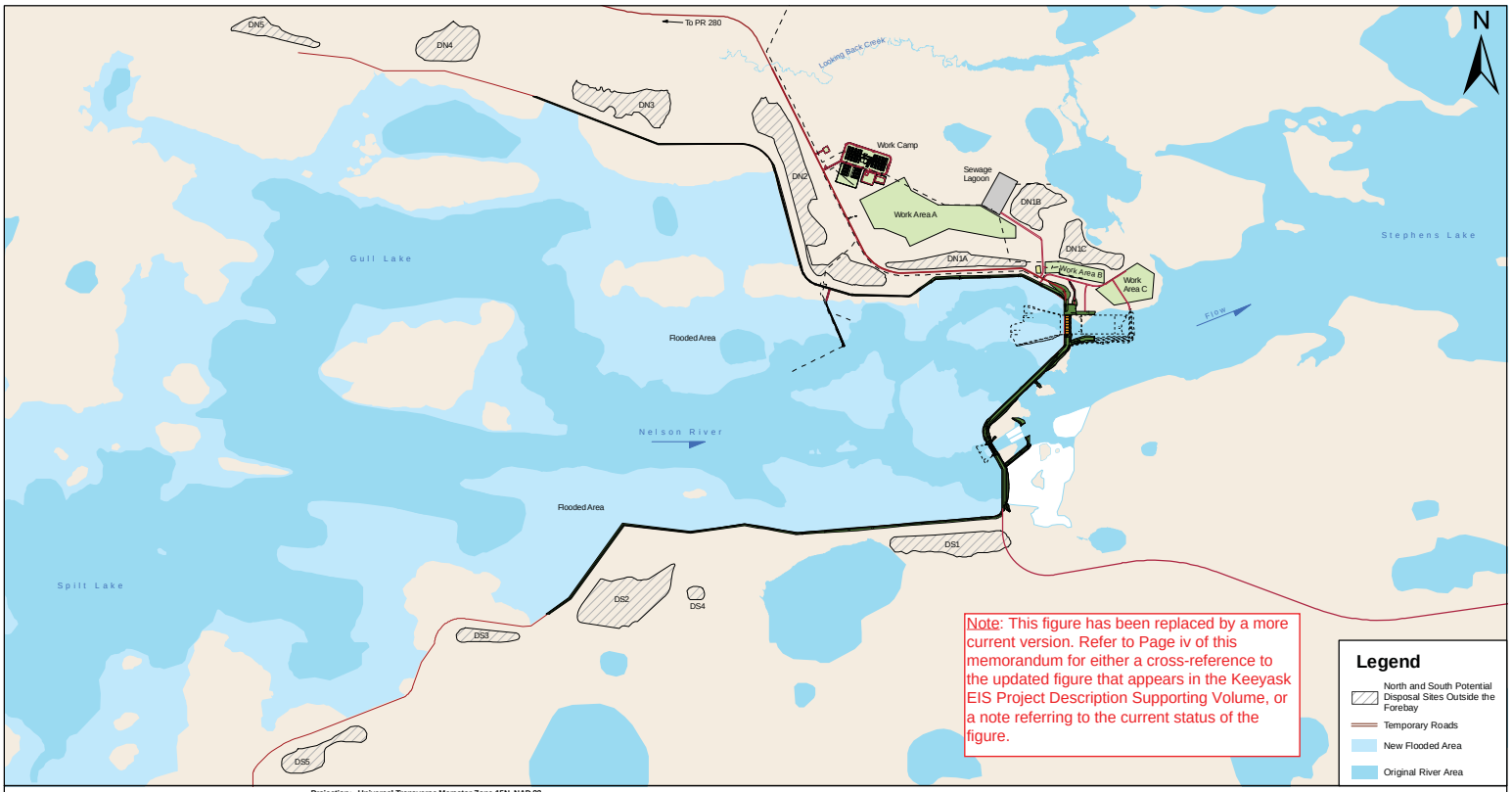
1. Dimensions and elevations are in metres.
2. Elevations are based on GS OF C, CGVD28 and 1929 adjustment.
3. Crest elevation set at 1.0 metre above the prevailing water level at time of construction.
4. Protective rock groin not required for spillway stage I cofferdam, downstream of discharge channel.
5. The crest shall be EL 151.0 at the left abutment of the upstream leg sloping to EL 150.5 at the right corner of this leg, then sloping along the river leg to EL 148.0 at the right corner of the downstream leg. The downstream leg shall have a crest at EL 148.0 for its entire length.
6. Various sizes and gradations of rockfill (classes C1, C2 and C3) will be utilized. Precise locations of utilization will be based on flow conditions.

STAGE IV STUDIES AXIS GR-4 COFFERDAM CROSS-SECTIONS

Figure 9 (Sheet 2 of 2)

Created By: KGS ACRES

Date Created: February 04, 2011



0 500 1,000 2,000 Meters

0 1,500 3,000 6,000 Feet

Scale 1 : 41 000

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Lakes and Rivers Provided by Manitoba Hydro, 2008
Access Roads and Infrastructure Data Provided by KGS Acres, 2008
Water Main Lines, Lagoon and Water Treatment StationData Provided by ININEW, 2005

Figure is intended for illustrative purposes only.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

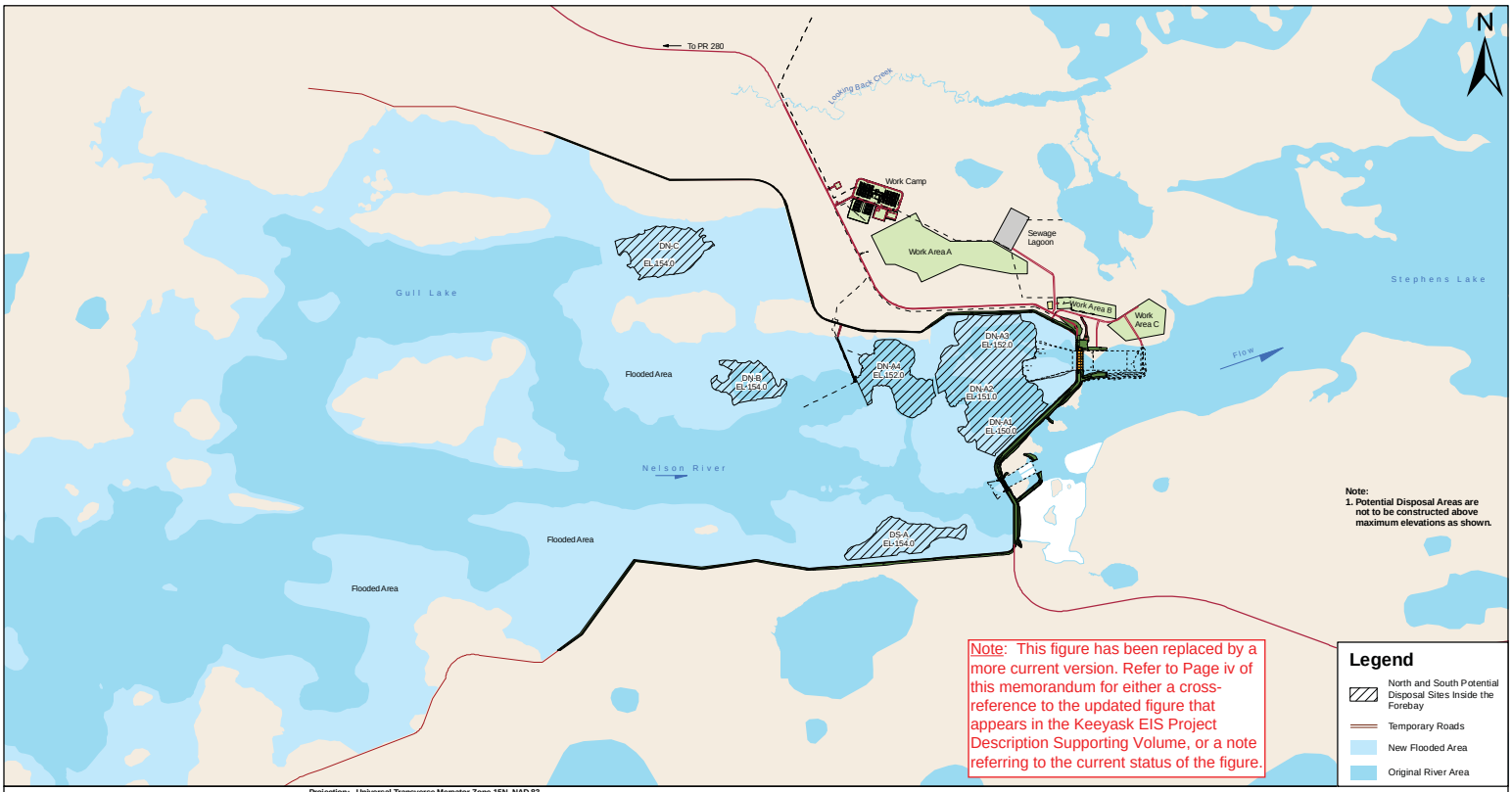
KEEYASK

STAGE IV STUDIES AXIS GR-4 POTENTIAL DISPOSAL SITES OUTSIDE OF FOREBAY

Figure 10

Created By:
KGS ACRES

Date Created:
May 7, 2009



0 500 1,000 2,000 Meters
0 1,500 3,000 6,000 Feet

Scale 1 : 41 000

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Lakes and Rivers Provided by Manitoba Hydro, 2008
Disposal Areas and Infrastructure Data Provided by KGS Acres, 2008
Water Main Lines, Lagoon and Water Treatment StationData Provided by ININEW, 2005

Figure is intended for illustrative purposes only.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

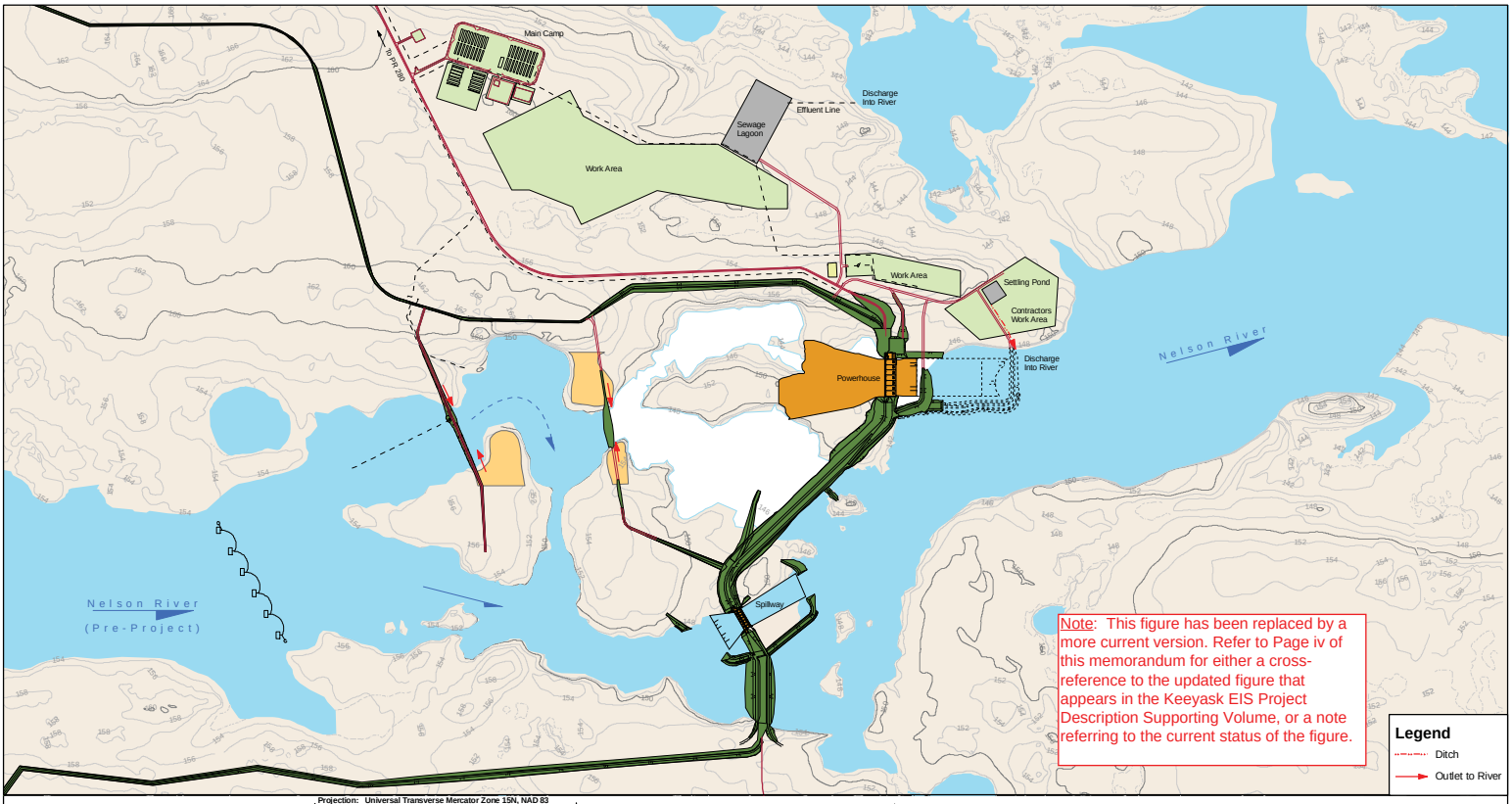
KEEYASK

STAGE IV STUDIES AXIS GR-4 POTENTIAL DISPOSAL SITES INSIDE FOREBAY

Figure 11

Created By:
KGS ACRES

Date Created:
May 7, 2009



0 350 700 1,400 Meters

0 1,000 2,000 4,000 Feet

Scale 1 : 21 000

Projection: Universal Transverse Mercator Zone 15N, NAD 83

Data Source: Lakes and Rivers Provided by Manitoba Hydro, 2008
Access Roads and Infrastructure Data Provided by KGS Acres, 2008
Water Main Lines, Lagoon and Water Treatment StationData Provided by ININEW, 2005

Figure is intended for illustrative purposes only.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

KEEYASK

STAGE IV STUDIES AXIS GR-4 SITE DRAINAGE PLAN

Figure 12

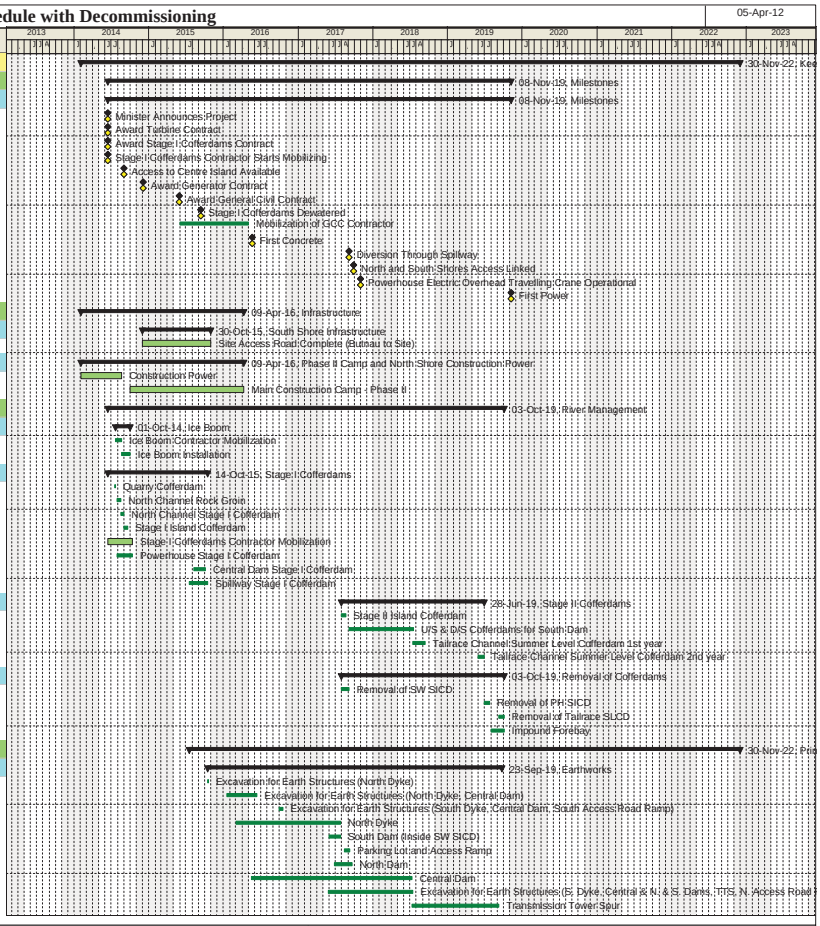
Created By:
KGS ACRES

Date Created:
May 7, 2009

Keeyask GS Summary Schedule with Decommissioning

05-Apr-12

Activity ID	Activity Type	Activity Name	Start	Finish
Keeyask GS Summary Schedule			03-Feb-14	30-Nov-22
Milestones			15-Jun-14	08-Nov-19
Milestones			15-Jun-14	08-Nov-19
A000	Finish Milestone	Minister Announces Project		15-Jun-14
A002	Finish Milestone	Award Turbine Contract		15-Jun-14
A004	Finish Milestone	Award Stage I Cofferdams Contract		15-Jun-14
A006	Finish Milestone	Stage I Cofferdams Contractor Starts Mobilizing		16-Jun-14
A008	Finish Milestone	Access to Centre Island Available		03-Sep-14
A010	Finish Milestone	Award Generator Contract		05-Dec-14
A012	Finish Milestone	Award General Civil Contract		31-May-15
A016	Finish Milestone	Stage I Cofferdams Dewatered		12-Sep-15
A014	Level of Effort	Mobilization of GCC Contractor	01-Jun-15	30-Apr-16
A018	Finish Milestone	First Concrete		21-May-16
A020	Finish Milestone	Diversion Through Spillway		06-Sep-17
A022	Finish Milestone	North and South Shores Access Linked		27-Sep-17
A024	Finish Milestone	Powerhouse Electric Overhead Travelling Crane Operational		31-Oct-17
A026	Finish Milestone	First Power		08-Nov-19
Infrastructure			03-Feb-14	09-Apr-16
South Shore Infrastructure			01-Dec-14	30-Oct-15
A100	Task Dependent	Site Access Road Complete (Butnau to Site)	01-Dec-14*	30-Oct-15
Phase II Camp and North Shore Construction Power			03-Feb-14	09-Apr-16
A105	Task Dependent	Construction Power	03-Feb-14	21-Aug-14
A110	Task Dependent	Main Construction Camp - Phase II	02-Oct-14	09-Apr-16
River Management			16-Jun-14	03-Oct-19
Ice Boom			22-Jul-14	01-Oct-14
A115	Level of Effort	Ice Boom Contractor Mobilization	22-Jul-14	20-Aug-14
A116	Level of Effort	Ice Boom Installation	21-Aug-14	01-Oct-14
Stage I Cofferdams			16-Jun-14	14-Oct-15
A205	Level of Effort	Quarry Cofferdam	17-Jul-14	22-Jul-14
A210	Level of Effort	North Channel Rock Groin	28-Jul-14	18-Aug-14
A220	Level of Effort	North Channel Stage I Cofferdam	18-Aug-14	03-Sep-14
A225	Level of Effort	Stage I Island Cofferdam	03-Sep-14	18-Sep-14
A200	Task Dependent	Stage I Cofferdams Contractor Mobilization	16-Jun-14	11-Oct-14
A215	Level of Effort	Powerhouse Stage I Cofferdam	28-Jul-14	14-Oct-14
A235	Level of Effort	Central Dam Stage I Cofferdam	10-Aug-15	06-Oct-15
A230	Level of Effort	Spillway Stage I Cofferdam	16-Jul-15	14-Oct-15
Stage II Cofferdams			01-Aug-17	28-Jun-19
A240	Level of Effort	Stage II Island Cofferdam	01-Aug-17	23-Aug-17
A245	Level of Effort	U/S & D/S Cofferdams for South Dam	07-Sep-17	19-Jul-18
A250	Level of Effort	Tailrace Channel Summer Level Cofferdam 1st year	16-Jul-18	14-Sep-18
A255	Level of Effort	Tailrace Channel Summer Level Cofferdam 2nd year	01-Jun-19	28-Jun-19
Removal of Cofferdams			01-Aug-17	03-Oct-19
A260	Level of Effort	Removal of SW SICD	01-Aug-17	06-Sep-17
A270	Level of Effort	Removal of PH SICD	29-Jun-19	25-Jul-19
A275	Level of Effort	Removal of Tailrace SLCD	09-Sep-19	03-Oct-19
A265	Level of Effort	Impound Forebay	02-Aug-19	03-Oct-19
Principal Structures			16-Jul-15	30-Nov-22
Earthworks			16-Oct-15	23-Sep-19
A300	Level of Effort	Excavation for Earth Structures (North Dyke)	16-Oct-15	21-Oct-15
A301	Level of Effort	Excavation for Earth Structures (North Dyke, Central Dam)	18-Jan-16	10-Jun-16
A302	Level of Effort	Excavation for Earth Structures (South Dyke, Central Dam, So...	30-Sep-16	17-Oct-16
A310	Level of Effort	North Dyke	04-Mar-16	25-Jul-17
A325	Level of Effort	South Dam (Inside SW SICD)	02-Jun-17	27-Jul-17
A335	Level of Effort	Parking Lot and Access Ramp	14-Aug-17	08-Sep-17
A330	Level of Effort	North Dam	26-Jun-17	19-Sep-17
A315	Level of Effort	Central Dam	18-May-16	12-Jul-18
A304	Level of Effort	Excavation for Earth Structures (S. Dyke, Central & N. & S. D...	29-May-17	13-Jul-18
A355	Level of Effort	Transmission Tower Spur	13-Jul-18	07-Sep-19



Remaining Level of Effort | Remaining Work | Milestone
Actual Work | Baseline Milestone | Summary

TASK filter: All Activities

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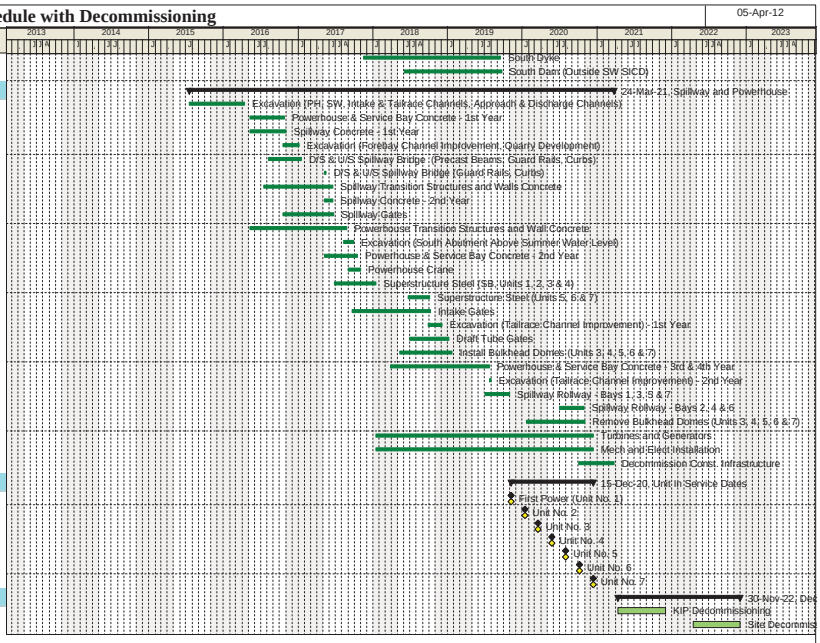
Figure 13
(Sheet 1 of 2)

Keeyask GS Summary Schedule with Decommissioning

05-Apr-12

Activity ID	Activity Type	Activity Name	Start	Finish
A345	Level of Effort	South Dyke	17-Nov-17	16-Sep-19
A350	Level of Effort	South Dam (Outside SW SICD)	04-Jun-18	23-Sep-19
Spillway and Powerhouse			16-Jul-15	24-Mar-21
A500	Level of Effort	Excavation (PH, SW, Intake & Tailrace Channels, Approach &...	16-Jul-15	12-Apr-16
A505	Level of Effort	Powerhouse & Service Bay Concrete - 1st Year	09-May-16	24-Oct-16
A510	Level of Effort	Spillway Concrete - 1st Year	09-May-16	01-Nov-16
A501	Level of Effort	Excavation (Forebay Channel Improvement, Quarry Develop...	17-Oct-16	05-Jan-17
A521	Level of Effort	D/S & U/S Spillway Bridge (Precast Beams, Guard Rails, Cur...	08-Aug-16	17-Jan-17
A522	Level of Effort	D/S & U/S Spillway Bridge (Guard Rails, Curbs)	08-May-17	17-May-17
A520	Level of Effort	Spillway Transition Structures and Walls Concrete	16-Jul-16	17-Jun-17
A511	Level of Effort	Spillway Concrete - 2nd Year	08-May-17	17-Jun-17
A527	Level of Effort	Spillway Gates	17-Oct-16	21-Jun-17
A515	Level of Effort	Powerhouse Transition Structures and Wall Concrete	09-May-16	26-Aug-17
A502	Level of Effort	Excavation (South Abutment Above Summer Water Level)	12-Aug-17	27-Sep-17
A506	Level of Effort	Powerhouse & Service Bay Concrete - 2nd Year	08-May-17	16-Oct-17
A535	Level of Effort	Powerhouse Crane	02-Sep-17	31-Oct-17
A530	Level of Effort	Superstructure Steel (SB, Units 1, 2, 3 & 4)	27-Jun-17	16-Jan-18
A531	Level of Effort	Superstructure Steel (Units 5, 6 & 7)	22-Jun-18	06-Oct-18
A540	Level of Effort	Intake Gates	20-Sep-17	10-Oct-18
A503	Level of Effort	Excavation (Tailrace Channel Improvement) - 1st Year	27-Sep-18	06-Dec-18
A560	Level of Effort	Draft Tube Gates	29-Jun-18	07-Jan-19
A555	Level of Effort	Install Bulkhead Domes (Units 3, 4, 5, 6 & 7)	11-May-18	21-Jan-19
A507	Level of Effort	Powerhouse & Service Bay Concrete - 3rd & 4th Year	27-Mar-18	24-Jul-19
A504	Level of Effort	Excavation (Tailrace Channel Improvement) - 2nd Year	26-Jul-19	31-Jul-19
A525	Level of Effort	Spillway Rollway - Bays 1, 3, 5 & 7	02-Jul-19	30-Oct-19
A526	Level of Effort	Spillway Rollway - Bays 2, 4 & 6	30-Jun-20	28-Oct-20
A556	Level of Effort	Remove Bulkhead Domes (Units 3, 4, 5, 6 & 7)	21-Jan-20	04-Nov-20
A545	Level of Effort	Turbines and Generators	16-Jan-18	15-Dec-20
A550	Level of Effort	Mech and Elect Installation	16-Jan-18	15-Dec-20
A565	Level of Effort	Decommission Const. Infrastructure	05-Oct-20	24-Mar-21
Unit In Service Dates			08-Nov-19	15-Dec-20
A901	Finish Milestone	First Power (Unit No. 1)		08-Nov-19
A902	Finish Milestone	Unit No. 2		13-Jan-20
A903	Finish Milestone	Unit No. 3		19-Mar-20
A904	Finish Milestone	Unit No. 4		26-May-20
A905	Finish Milestone	Unit No. 5		01-Aug-20
A906	Finish Milestone	Unit No. 6		08-Oct-20
A907	Finish Milestone	Unit No. 7		15-Dec-20
Decommissioning			15-Apr-21	30-Nov-22
A1010	Task Dependent	KIP Decommissioning	15-Apr-21*	30-Nov-21
A1020	Task Dependent	Site Decommissioning/Rehabilitation	15-Apr-22*	30-Nov-22



Remaining Level of Effort | Remaining Work | Milestone
Actual Work | Baseline Milestone | Summary

TASK filter: All Activities

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Figure 13
(Sheet 2 of 2)

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - PHYSICAL ENVIRONMENT

COFFERDAM EROSION DURING CONSTRUCTION

REV 3

DELIVERABLE GN 9.2.17

MANITOBA HYDRO FILE: 00195-11100-0191_04

2013/02/22

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

KGS ACRES

Hydropower Limited Partnership

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-9.2.17

Rev. 3, February 22, 2013

Cofferdam Erosion During Construction

Manitoba Hydro File 00195-11100-0191_04

Prepared by Edward Sikora (EDWARD SIKORA) 2013-02-22
KGS Acres Ltd.

Checked by P. Pantel (P. PANTEL) 2013.02.22
KGS Acres Ltd.

Approved by I.R. Dewar (I.R. Dewar) 2013-02-22
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

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Sketch A-1 Cofferdam Construction Flow Velocity Schematics Plans

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Sketch B-1 North Channel Rock Groin - Plan

Sketch B-2 North Channel Stage I Cofferdam – Plan

1 INTRODUCTION

This memorandum provides estimates of cofferdam fill material losses during cofferdam construction and removal at the Keeyask Generating Station, and of the resulting increase of Total Suspended Solids (TSS) concentrations in the Nelson River. The summary results are presented in Tables 9.0, 9.1, and 9.2. Cofferdam arrangements and design descriptions are covered in Memorandum GN-4.2.2.11, "Earthworks Structures, Cofferdam Arrangement and Design" [Ref. 1] as well as Memorandum GN-9.2.15, "Preliminary Consideration of Methods to Manage Sediment During Construction" [Ref. 2]. The sources of construction materials and their utilization factors are discussed in Memorandum GN-1.9.3, "Construction Materials Sources and Utilization" [Ref. 3].

Total Suspended Solids (TSS), in mg/l, is defined as the concentration of solids in suspension in the river at a particular point or location and time. Extreme values of TSS could have negative environmental effects such as clogging the gills of fish, blocking off light to algal photosynthesis required in the food chain, and smothering fish spawning beds as the material settles out of suspension. Hence, it is considered important to calculate the increase in TSS resulting from construction activities in the river to demonstrate its relative magnitude. In this memorandum, TSS refers to increased levels above background and is calculated only for mineral soils.

Estimates of material losses which will generate increases in TSS levels during cofferdam fill placement and removal in the water are complex and very difficult to quantify on a theoretical basis. In order to estimate TSS level increases for the purposes of this memorandum assumptions regarding construction scheduling and methodology, and the amount of erosion of the various fills by water current action during fill placement were made, as described in Sections 4, 5, 6 and 7.

The procedure used to estimate increases in TSS levels in this memorandum was similar to that used for the Wuskwatim GS [Ref. 4], for which the increase in TSS during construction was generally within the predicted range as of March, 2011 [Ref 10]. The analysis method was altered to suit special cases differing from conditions at the Wuskwatim Generating Station (Wuskwatim GS), with the rationale described herein for each case.

This memorandum has been organized as follows:

- Basic background information on project location, site conditions, and cofferdam construction materials are presented in Sections 2 to 4.
- The method of analysis used to estimate increases in TSS levels due to cofferdam construction and removal are presented in Section 5.

- The estimated flow conditions and water levels during cofferdam construction and removal are presented in Sections 6 and 7.
- Cofferdam construction sequence, cofferdam removal, and TSS calculation details are presented in Section 8.
- A summary and discussion of results are given in Section 10, with the estimated increases in TSS levels presented in Tables 9.0, 9.1, and 9.2.

2 PROJECT LOCATION

The site of the proposed Keeyask Generating Station (Keeyask GS) is located on the Nelson River approximately 730 km north of Winnipeg, at the head of Stephens Lake. Stephens Lake is the reservoir for the existing Kettle Generating Station, the upstream limit of which is the Keeyask site. The proposed axis for the structures crosses the Nelson River near the foot of the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam. A project site location plan is presented in Figure 1.

The general arrangement for the proposed Keeyask GS is presented in Figure 2. The selected arrangement consists of two long riverbank dykes, on the north and on the south sides of the river, linked to the Principal Structures which span the Nelson River at Gull Rapids. The Principal Structures consist of a Powerhouse Complex constructed on the north side of the river, which will be connected to the North Dyke through a 99 m long North Dam and a short concrete transition structure. The Spillway will be located on the South Island, approximately 1.5 km south of the Powerhouse. A Central Dam will connect the Spillway and the Powerhouse structures. The Spillway will be connected to the South Dam by means of a short concrete transition structure. The South Dam, approximately 565 m long, will connect to the South Dyke on the south riverbank.

3 SITE CONDITIONS

The site for the Keeyask GS lies within the Canadian Shield near the boundary between the Churchill, and Superior provinces. The glacial and post-glacial geological overburden thickness is estimated as being as much as 30 m over the Precambrian bedrock. The Precambrian bedrock generally consists of greywacke gneisses, granite gneisses and granites. The overburden stratigraphy is a result of the multiple glacial advances and retreats, followed by the inundation of much of Manitoba by Glacial Lake Agassiz after the last glacial retreat. Some preglacial sands and silty sands are found immediately above the bedrock formation but generally the overburden consists of a thick layer(s) of deposited glacial material (till) overlain by postglacial deposits in the form of alluvium (cobbles and boulders overlying sands and gravels) and Lake Agassiz silts and

clays; the latter of which are commonly varved and relatively thin in nature (except in topographic lows) or absent (e.g., on nearby ridges and knolls).

4 COFFERDAM CONSTRUCTION

The construction of the cofferdams will consider material gradation, construction sequencing, material placement techniques, and certain mitigation techniques to minimize the TSS levels in the Nelson River.

4.1 MATERIALS

The cofferdams will be constructed of Class A Impervious Fill, Class B Transition Fill, and Class C rockfill, as described below.

4.1.1 IMPERVIOUS FILL (CLASS A)

Class A fill will be used as cofferdam seal material in the construction of the Stage I and Stage II Cofferdams. It will be a uniform to well-graded, unfrozen silty sand/sandy silt till that will generally be placed in-the-wet by end dumping, with little or no compaction [Ref 3].

4.1.2 TRANSITION FILL (CLASS B)

Class B fill will be a well graded, pervious to semipervious mixture of sand and gravel containing less than 10% fines by weight. It shall be used to form a transition between the Class A impervious cofferdam seal and the Class C rockfill [Ref 3].

4.1.3 ROCKFILL (CLASS C)

Class C rockfill, consisting of Classes C1, C2, C3 and C4 fill, will be composed of well graded excavated quarry run Precambrian bedrock, assumed to contain 0.5% fines by weight. The bulk of cofferdam rockfills will be comprised of Class C1, having an average D_{50} size of about 170 mm. Larger sizes, Classes C2, C3 and C4, may be required for cofferdam closure and at the leading edge of the advancement leg in those locations where the flow velocities are relatively high [Ref 3].

4.2 CONSTRUCTION METHODOLOGY

The general method of construction for cofferdams consists of placing the fill material in the river “in-the-wet” up to an elevation of approximately 1 m above the prevailing water level. The construction of the structures will proceed by end dumping the rockfill directly into the water from a point as close to the water as possible. Then the Class B and Class A materials will be dumped on the existing fill near the front of the advancing face of these zones and pushed into the river by utilizing dozers so that the fill enters the

water as a sliding mass, thereby minimizing the amount of fill that will come in direct contact with the water. The rockfill groin will be kept some 20 to 30 m ahead of the placement front of the finer grained materials.

For cofferdams which incorporate both inner and outer rockfill groins, the inner and outer rockfill groins will be kept some 20 m to 30 m ahead of the placement front of the finer grained materials to create tranquil flows in the areas where these finer grained materials are to be placed in-the-wet.

5 DESCRIPTION OF ANALYSIS

Estimates of material losses which will generate increases in the river's TSS levels during cofferdam material placement and removal are complex and very difficult to quantify on a strictly theoretical basis. Hence, they must be based on the envisioned construction methodology, contract specifications, engineering judgment, and previous construction project experiences, as well as conservative assumptions.

Although riverbed erosion is not part of the Scope for this study, it should be noted that there is very little concern for erosion of the riverbed materials at the Keeyask site during cofferdam construction, as was the case for the Wuskwatim GS. The riverbed at Keeyask in the cofferdam areas is generally bedrock controlled with cobbles and boulders, possible trapped gravel and sands, and exposed hard dense glacial tills which have a high resistivity to erosional forces.

The estimated TSS values presented in this report are based on the assumption that both the Stage I Cofferdam Contractor and the General Civil Contractor will supply and place the cofferdam materials in accordance with the technical specifications, the detailed drawings, and the Preliminary Sediment Management Plan [Ref. 2].

5.1 SUSPENSION OF FILL MATERIALS DURING COFFERDAM CONSTRUCTION

When fill is placed directly into the flowing water of the river, part of the silt and clay fraction of the fill that is in contact with the flowing water will be entrained at a rate proportional to the fill placement rate. This is referred to as the "Entrainment Rate" (ERate).

The ERate was taken as being directly proportional to:

- The rate of placement of the material, Placement Rate = PRate, in m^3/s.
- The total portion of the material that is susceptible to be entrained by the flow = Material Factor = MF, in percent.

- The degree of exposure to the water that the fill will experience = Exposure Factor = EF, in percent

The ERate was converted from a volume of eroded material into mass by using the dry unit weight of the material (DUW), in kN/m^3. The various factors used are described below.

- **Material Factor (MF)**: the fraction of material being placed that is susceptible to becoming entrained into the water during the interval while it is directly exposed to flow. The MF was based on the general method of cofferdam construction where the Class C rockfill is advanced across a river channel, with the placement of Class B following slightly behind the advancing face of the Class C, and the placement of Class A following slightly behind the advancing face of the Class B as illustrated in Sketch A-1 in Appendix A. This general method of placement is Condition II as described in Section 5.2 below. The assumed MF for the cofferdam fill materials were:

Material Class	Material Factor (MF)
Class A Impervious Fill	30%
Class B Granular Transition Fill	10%
Class C Rockfill	0.5%

- **Exposure Factor (EF)**: the proportion of the time that the material will actually be exposed to direct erosion by flowing water. This factor also captures the generation of TSS along the upstream slope of the advancing cofferdam materials, and takes into account self armoring action of the materials with its coarse material content and subsequent cover protection with successive fill layers. The EF was based on the general method of cofferdam construction described under Material Factor (MF), above. The assumed EF for the cofferdam materials were:

Material Class	Exposure Factor (EF)
Class A Impervious Fill	33%
Class B Granular Transition Fill	33%
Class C Rockfill	100%

- **Fill Placement Rate (PRate):** the volume of material placed/removed during cofferdam construction/removal per unit time. The PR was calculated for each class of material in each structure by dividing the volume of material by the estimated time required to place the material.

- **Reduction Factor (RF):** an adjustment factor to account for changes in conditions from the general cofferdam construction case, described above and as Condition II in Section 5.2, below. This factor includes allowances for reduced velocity adjacent to the eroding material in the case where flow in the vicinity of the fill is only due to leakage through the Class C rockfill, which would occur after the river channel is closed by the Class C.

Material Class	Reduction Factor (RF)
Class A Impervious Fill	3.3%
Class B Granular Transition Fill	5%
Class C Rockfill	100% (i.e., no reduction)

The Entrainment Rate (ERate) was calculated as follows:

ERate (mg/sec) = [PRate (m^3/sec) x DUW (kN/m^3) x MF x EF x RF x 10^6 (mg/kg) x 10^3 (N/kN)/9.81] (m/sec^2)

= [PRate (m^3/sec) x DUW (kN/m^3) x LF x 10^6 (mg/kg) x 10^3 (N/kN)] / 9.81 (m/sec^2)

Where LF = Loss Factor = MF x EF x RF

The Loss Factors for each cofferdam are summarized in Table 1.

The resulting entrainment rate expressed in mg/sec, was then divided by the channel flow (Q), expressed in l/sec, to arrive at the Total Suspended Solids (TSS), in mg/l, anticipated during construction. The daily and weekly TSS values were calculated by factoring this figure by 20/24 for daily and (20x6)/ (24x7) for weekly. These values are based on two ten hour shifts per day and a six day week. The analysis method is identical to that employed for Wuskwatim GS.

The above analysis provides results for the totally mixed case of full dilution by channel flow. "Local" temporarily elevated TSS which would occur in partial flow channels and "partially exposed" cases described below, which would subsequently become fully mixed when they re-enter the main stream, have also been estimated. Potential TSS plumes or local higher concentrations which may occur immediately adjacent to the equipment performing the work will be very temporary in nature.

5.2 FILL EXPOSURE CASES

Three fill exposure cases (discussed below as Conditions I, II and III) will occur at Keeyask GS as described below, and as illustrated in Appendix A, Sketch A-1.

Condition I - River Channel Closed by Rockfill Prior to Class A & B Placement

Condition I occurs when a Class C rockfill groin has been advanced across the entire channel and cuts off the channel flow. This condition occurs during construction of the Quarry Cofferdam, North Channel Rock Groin, North Channel Stage I Cofferdam, Central Dam Stage I Cofferdam and South Dam Stage II Upstream Cofferdam.

During construction of the Class C rockfill groin, the finer particles in the rockfill will be entrained in the flow by currents running around the upstream corner of the advancing face, along the advancing face, and running parallel to the rockfill groin. Additional material will be entrained by the leakage through the rockfill. The highest flow velocities will be experienced at the upstream corner of the advancing fill, with the flow velocities along the upstream face of the rockfill decreasing towards the closure abutment.

Although there will be no flow through the channel, or portion of the channel, closed by the rock groin, there will be leakage through the rockfill. This leakage will result in flow currents along the advancing faces and upstream slopes of the Class B and Class A fills. During the placement of the Class B and A materials, some of the fines will be entrained in these currents and will increase local TSS levels.

Condition II – Class A and B Placement Concurrent with Class C Placement

Condition II occurs where the placement of Class B and Class A fill materials follow slightly behind the advancing face of the Class C rockfill. Although this case does not occur in the assumed construction methodology, it is a method that the Contractor may decide to use. In this case, the river continues to flow between the advancing face of the Class C and the closure shoreline. There will be significant flow velocities along the advancing face of the Class C with lower flow velocities running along the upstream slopes of the Class B and Class A fills. The finer material sizes of the Class A and B fills will be entrained by the current and be either taken around the advancing face of the Class C by the main flow in the river channel or will pass through the Class C fill by leakage through this rockfill zone.

For Condition II, it is expected that the higher flow velocities will be experienced at the upstream corner of the advancing fill, with the flow velocities along the upstream slope of the advancing cofferdam decreasing towards the starting shoreline. Therefore, by staggering the advancement of the Class C, B and A cofferdam materials by 20 to 30 m the

exposure of the finer materials to the flow velocities along the upstream slope of the cofferdam will be minimized.

It is expected that the entrainment rate of the Class B materials by the leakage through the cofferdam will increase as the rockfill groin closes against the shoreline. Once the rockfill groin has closed against the shoreline the entrainment of the Class A and B materials along upstream slope and through the cofferdam is expected to decrease as these materials are advanced to the closing shoreline.

The Preliminary Sediment Management Plan [Ref. 2] describes these activities as taking place sequentially and notes that the construction activities may take place concurrently. The construction methodology will directly influence the flow volume and flow velocity on a given material, thus affecting the amount of TSS that can be generated.

Condition III – Double Rockfill Groin

Condition III occurs when a double rockfill groin design has been utilized and the subsequent Class A & B fill placement is partially sheltered from the river's velocity. This occurs during construction of a portion of the Powerhouse Cofferdam, the Tailrace Summer Level Cofferdam, and the Spillway Stage I Cofferdam. However, it is assumed that there will still be water seeping through the rockfill which will flow along the face of the Class A & B during their placement. The velocities in this instance would be much lower than where Class A & B are exposed directly to the main flow of the river, hence the above Reduction Factors would be applied to material fraction lost into flow. There is no reduction in exposure factor in this case.

6 HYDRAULIC CONDITIONS

Three flow conditions were used to estimate increases in TSS levels. They consist of three ranges of flow frequency, the 1:20 year monthly low daily flow; the mean monthly flow and the 1:20 year monthly peak daily flow (see Table 2). Total river flow rates for TSS analysis were selected based on the months of construction shown on the Summary Construction Schedule in Table 3 [Ref 6]. Where the work period is scheduled for more than one month, the lower monthly flow rates of each of the three flow frequencies was assumed for the calculation, as this will result in a higher estimated increase in TSS levels (i.e., same amount of material exposed to lower total flows). Similarly, where the schedule has significant float for a structure, the construction period for each material placement was assumed to occur in the lower flow month(s).

The layout of the cofferdams is presented in Figure 2, the construction sequence of the project is shown in Figure 3 as outlined in Reference 2, and typical cofferdam cross-sections are shown in Figures 4 and 5. A more detailed description of the cofferdam arrangement and design is provided in References 1 and 2.

The proportions of total river flow rate assumed for use in the TSS calculations for each cofferdam were selected as outlined in the following sub-sections.

6.1 STAGE I DIVERSION – FLOW DISTRIBUTIONS

6.1.1 QUARRY COFFERDAM

At the start of Class C placement, based on relative channel widths, it was assumed that 10% of the full river flow would be flowing in the North Channel of the river. Following complete closure with Class C, the channel flow would be greatly reduced. During Class A placement, the flow would be limited to seepage through the rockfill. This is assumed to be 1% of the full river flow.

6.1.2 NORTH CHANNEL ROCK GROIN

Based on relative channel widths, it was assumed that flow in the north channels would be approximately 30% of the full river flow at the start of Class C placement. Following complete closure with Class C, the channel flow would be greatly reduced. During Class B placement, the flow would be limited to seepage through the rockfill. This is assumed to be 1% of the full river flow.

6.1.3 NORTH CHANNEL STAGE I COFFERDAM

Flow passing through the North Channel Cofferdam area is anticipated to be in the order of 10% of normal river flow. This flow will consist of seepage flow through the North Channel Rock Groin as well as flow coming from the south river channel through the cross-over channel[1]. Following Class C closure of North Channel Cofferdam, it is assumed that the TSS generated by the placement of Class B material will be impacted by seepage through the Class C embankment, assumed to be 1% of the full river flow. The TSS generated by placement of Class A material is not anticipated to be carried through the Class B and C embankment, as it is assumed the Class B material layer will be advancing about 20 m ahead of the Class A. Therefore, it is assumed that relatively calm water conditions will exist during Class A placement [Ref 2]. However, to be conservative, the Class A material has been included in the calculation of TSS

[1] Under normal conditions, flow through the Cross-Over Channel is generally from the Middle Channel to the South Channel. However, the flow direction will reverse after the Middle Channel has been blocked by the construction of the North Channel Rock Groin.

generation. TSS generated from the Class A will be transported through the cross-over channel and be fully mixed in the South Channel prior to reaching Monitoring Station K-Tu-2 (see Figure 2).

6.1.4 POWERHOUSE STAGE I COFFERDAM

It was assumed that the zoned embankment materials within the area enclosed by the two rockfill groins would be impacted by seepage entering through the protective groin, assumed to be about 1% of the full river flow (i.e. Condition III). It is noted that while the protective rock groin does not extend along the entire cofferdam, the exposed portion of zoned embankment extending beyond the groin (i.e. the North/South leg) is not subject to significant erosion as it will likely be a very calm area. To be conservative, this material was included in the calculation of TSS generation. Following completion of the cofferdam, rockfill materials are anticipated to be placed onto the Class A layer to provide erosion protection during winter months. The cofferdam construction stages are shown in Figure 3.

6.1.5 SPILLWAY STAGE I COFFERDAM

It was assumed that the zoned embankment materials within the area enclosed by the two rockfill groins would be impacted by seepage entering through the upstream leg of the protective groin and exiting the downstream area, assumed to be about 1% of the full river flow. It is noted that while the protective rock groin is terminated at the downstream leg, the exposed portion of zoned embankment extending beyond the groin to the North Abutment is not subject to significant erosion as it will likely be a calmer area, shallow and sheltered from the main flow [see Ref 7 for details]. To be conservative, this material was included in the calculation of TSS generation. The cofferdam construction stages are shown in Figure 3.

6.1.6 CENTRAL DAM STAGE I COFFERDAM

The construction of the North Channel Rock Groin and the North Channel Cofferdam will have diverted the flow from the north and central channels into the South Channel (see Figure 3). Hence, there will be no river flow in the north and central channels. In addition, most of the channels will have drained out naturally with the exception of the deeper portions of the river channel (i.e. channel depressions).

The site of the Central Dam Stage I Cofferdam will be somewhat sheltered from the full river flow. It was assumed that 20% of the full river flow from Stage I Diversion would impact the Central Dam Stage I Cofferdam area and that the velocities will be much lower than those in the main channel. This is a conservative assumption as velocities along the downstream slope of the cofferdam were observed to be negligible in the

Comprehensive Model Study [Ref 9]. In addition, the flow will generally be moving roughly parallel to the cofferdam's alignment.

It is assumed that the TSS generated by the placement of the Class C material will be impacted from the flows coming from the South Channel and going around the little island. It has been conservatively assumed that 100% of the Class C material placed in-the-wet will be exposed to the flow. Following the closure of the central channel with the Class C material, the channel flow will generally be moving parallel to the cofferdam alignment in a northeasterly direction.

A Central Dam Rock Groin will be constructed using Class C materials. This rock groin will be located perpendicular to the Central Stage I Cofferdam between the cofferdam and the small island just to the east, as shown in Figure 2. It is assumed that the placement of this groin will be exposed to about 20% of the full river flow. This groin will be installed before the placement of the Class B and Class A materials and is designed to minimize the flow and TSS impacts during placement of the finer materials. This structure will be excavated locally to complete the cofferdam seal, but this will be undertaken after all other materials are placed. Once installed, the flow along the face of the Central Dam Cofferdam will be reduced, and it will create a relatively calm area for the final placement of the Class B and C materials. The cofferdam and associated rock groin are shown in Figures 2 and 3.

It is envisioned that the Class B will be advancing ahead of the Class A. In addition, as mentioned above, the river flow will be parallel to the structure and further reduced by the presence of the rock groin. Furthermore, it is anticipated that very little TSS generated from the placement of the Class B and A materials will be transported out into the main flow of the river based on the geometry of the site.

For this study it is assumed that all of the TSS generated from the placement of the Class B will be transported downstream. Since river flow passes across a series of rapids, the silts and clays will likely be fully mixed in the river flow prior to reaching the monitoring station at K-Tu-2.

6.1.7 TAILRACE SUMMER LEVEL COFFERDAM

It is assumed that there will be essentially zero flow through the inner rock groin as there will be a balance of head water. However, it is assumed that the flow inside of the rock groin will be about 1% of the full river flow. It is noted that while the protective rock groin does not extend along the entire length of the cofferdam, the exposed portion of zoned embankment extending beyond the groin to the North Abutment is not subject to significant erosion as it is expected to be a relatively calm area. To be conservative, this

material was included in the calculation of TSS generation. The cofferdam construction stages are shown in Figure 3.

6.2 STAGE II DIVERSION – FLOW DISTRIBUTION

6.2.1 SOUTH DAM STAGE II U/S COFFERDAM

Based on the relative widths of the Spillway and the river channel, it was assumed that the channel flow at the start of Class C placement would be 70% of river flow. Following complete closure with Class C material, it was assumed that the TSS generated by the placement of Class B material would be impacted by seepage through the Class C embankment, assumed to be 1% of the full river flow. The TSS generated by placement of Class A material is not anticipated to be carried through the Class B and C embankment, as it is envisioned that the Class B material will be advancing ahead of the Class A placement. It is also envisioned that relatively calm water conditions will exist along the upstream of the cofferdams, and that some of the TSS may settle out locally during Class A placement [Ref 2]. However, to be conservative, the Class A material has been included in the calculation of TSS generation.

The partially removed Spillway Stage I Cofferdam and South Dam Stage II U/S Cofferdam are shown in Figure 3.

6.2.2 SOUTH DAM STAGE II D/S COFFERDAM

It has been assumed that construction of the South Dam Stage II Upstream Cofferdam will be completed prior to the construction of the South Dam Stage II Downstream Cofferdam. Based on the construction sequence and river diversion through the Spillway structure it is assumed that there will be no river flow at the South Dam Stage II Downstream Cofferdam location.

In addition it is anticipated that the area immediately downstream of the Stage II Upstream Cofferdam will drain out. The extent of this drained area is unknown as bathymetry data in the vicinity of the rapids to the east of the Spillway Discharge Channel is not available. It is assumed that the water levels will drain down to the lowest point in the upstream section of the rapids. It is also assumed that water discharge from the Spillway will not back up into this area. Therefore, it is anticipated that a small wetted area of the river will remain. Under these assumed conditions, the placement of the Class A, B and C materials would be placed in calm water (i.e. no river flow) and that the any TSS generated from placing materials in-the-wet during the construction of the Stage II Downstream Cofferdam will be contained within the immediate area and will settle out locally. Therefore, there should be no impact on TSS at monitoring station K-Tu-2.

It is noted that based on the uncertainty in the bathymetry of the rapids and of the water levels following river closure and diversion through the Spillway, the final design of the South Dam Stage II Downstream Cofferdam will be reviewed during Stage II River Diversion. The site conditions will be assessed to determine if there are any opportunities to minimize the cofferdam volumes or potential for TSS generation.

If a two leg closure is required (i.e. Condition III - concurrent rockfill placement for upstream and downstream cofferdams), the TSS calculation would have assumed 70% river flow during the Class C placement and no flow during the placement of the Class A and B materials for the Stage II D/S Cofferdam.

6.3 COFFERDAM REMOVAL – FLOW DISTRIBUTION

Generally, full exposure to river channel flow was conservatively assumed for the TSS calculations for removal of Powerhouse Stage I Cofferdam, Spillway Stage I Cofferdam and Tailrace Summer Level Cofferdam, even though the Class A and B materials will most likely be removed inside the protection of the outer rockfill groins. However, it is assumed that the Tailrace Summer Level Cofferdam Class A material located within the protective rock groin will be removed while the double rock groin is still in place. The Powerhouse Stage I Cofferdam Class A material is also assumed to be removed while the double rock groin is still in place. The seepage flow passing through the rock groins is assumed to be about 1% of river flow for both cases.

7 WATER LEVELS DURING CONSTRUCTION

The increases in TSS levels were calculated based on submerged volumes of material corresponding with the assumed water levels during construction, as presented in Table 4. The assumed water level during construction has a direct effect on the quantity of fill placed in-the-wet, as utilized in the calculation of the estimated increases in TSS levels. This quantity, in turn, is used to calculate the placement rate during the construction period using cofferdam construction intervals from Table 3. Calculated TSS values are directly proportional to the placement rate.

The water levels utilized to determine the submerged volumes of materials are maximum water levels anticipated during construction of the Cofferdams. These are higher than the levels that would correspond to the flow rates given in Section 6. This approach was taken to provide for added conservatism in TSS calculations.

8 COFFERDAM CONSTRUCTION SEQUENCE AND TSS CALCULATION DETAILS

Table 1 lists the assumed Reduction, Material and Exposure Factors, as well as the calculated Loss Factors for each work phase, as further described below. The TSS calculations were then carried out as described in Section 5, using the assumed flows discussed in Section 6 and water levels identified in Section 7. Cofferdam construction sites are shown in the appended photographs (taken at a period of very low flow), and cofferdam layouts and cross-sections are shown in Figures 2, 4 and 5. The project construction sequence is shown in Figure 3.

8.1 STAGE I DIVERSION

Construction of the Stage I cofferdams for Stage I Diversion is required to permit the excavation works for the principal structures to be undertaken and construction of the Spillway and Powerhouse.

8.1.1 QUARRY COFFERDAM (PHOTOS 1 AND 2)

This temporary structure, constructed in-the-wet, will consist of a Class C rockfill embankment with upstream Class A impervious seal. It has a short service life and will permit the excavation of rockfill from Quarry No. 7, which is the planned source of the Class C rockfill to be used in construction of the balance of Stage I Cofferdams. The Class A material is considered to be a Condition I as outlined in Subsection 5.2.

8.1.2 NORTH CHANNEL ROCK GROIN (PHOTOS 3 AND 4)

The North Channel Rock Groin (NCRG) will be located across the Middle Channel approximately 1 km downstream of where the Nelson River divides into the Middle and South Channels at Gull Rapids (Figure 3, Photos 3 & 4). The NCRG will consist of a downstream rockfill groin with a granular fill zone placed as a blanket on the upstream slope of the groin (Figure 4). The NCRG has been designed to be a semi flow-through structure.

Based on available information the river channel in the vicinity of the NCRG consists of bedrock or thin layers of coarse granular material overlying bedrock. This portion of the river experienced sustained high flows during the summer of 2005 with flow characteristics as noted below:

- Total flow= 6500 m^3/s
- Flow through the Middle Channel =1033 m^3/s
- Energy Grade Line elevation at gauge SGNL12 = el 152.9 m

The total flow of 6500 m^3/s is almost 500 m^3/s more than the maximum 1:20 peak daily inflow (see Table 2). Aerial photographs, taken during low flow conditions in 2003, show that erosion of the overburden has occurred below or approximately up to the el 151 m topographic contour (Sketch B-1 in Appendix B). Aerial photographs taken in 2010 indicate that little further erosion has occurred since 2003 even after the passage of the high flows of the summer of 2005.

On the south abutment, where closure of the Middle Channel will occur, the stratigraphy generally consists of very stiff to hard clays overlying bedrock (based on explorations conducted in 2010). Further inland, these clays overlie soft silts, clayey sands, or bedrock.

Water levels and flow through the Middle Channel at the location of the NCRG will depend on the water/energy grade line levels in the South Channel and are summarized in Table 5.

Construction of the NCRG will occur during the months of July and August. Due to its small size, the following construction sequence has been assumed:

- The Middle Channel will be closed by the rockfill groin prior to commencing the placement of the upstream granular fill zone. The initial crest level of the rockfill groin will be constructed to just above (approximately 0.5 m to 1 m above) the existing water/energy grade line level at the time of construction.
- The granular fill zone will be completed to the same level as the initial rockfill groin.

The cofferdam will be completed to its design crest elevation by placement (in-the-dry) of the rockfill and granular fill.

Potential sources of finer materials that may increase the TSS levels monitored at Site K-Tu-2 are summarized in Table 6.

For the purposes of this memorandum, it has been assumed that erosion of the abutment will be prevented by the construction of erosion protection measures and/or re-alignment of the groin, as discussed in Ref 13. Also, material eroded from the riverbed has not been included in the estimated increase in TSS.

8.1.3 NORTH CHANNEL STAGE I COFFERDAM (PHOTO 5)

The North Channel Stage I Cofferdam (NCS1CD) will be located across the Middle Channel just downstream of the Cross-Over Channel (Figure 3, Photo 5). The NCS1CD will consist of a downstream rockfill groin with granular transition and impervious fill zones placed as blankets on the upstream slope of the groin (Figure 4).

Based on available information, the river channel in the vicinity of the NCS1CD consists of bedrock or thin layers of coarse granular material overlying bedrock. This portion of the river experienced sustained high flows during the summer of 2005, namely:

- Total flow= 6500 m^3/s
- Flow through the South Channel =5467 m^3/s
- Flow through the Middle Channel =1033 m^3/s
- Energy Grade Line elevation in the South Channel at gauge SGNL19D = el 149.8 m
- Energy Grade Line elevation in the Middle Channel at gauge SGNL10 = el 149.5 m

Aerial photographs, taken during low flow conditions in 2003, show that erosion of the overburden has occurred below or approximately up to the el 150 m topographic contour (Sketch B-2 in Appendix B).

Water levels and flow through the Middle Channel at the location of the NCS1CD will depend primarily on the water/energy grade line levels in the South Channel and the flow through the Cross-Over Channel, and are summarized in Table 7. Leakage through the previously constructed North Channel Rockfill Groin will be small and will not affect the water levels a significant amount.

Construction of the NCS1CD will occur during the months of July and August. Due to its small size, the following construction sequence has been assumed:

- The Middle Channel will be closed by the rockfill groin prior to commencing the placement of the upstream granular transition and impervious fill zones. The initial crest level of the rockfill groin will be constructed to just above (approximately 0.5 m to 1 m above) the existing water/energy grade line level at the time of construction.
- The granular transition and impervious fill zones will be completed to the same level as the initial rockfill groin.
- The cofferdam will be completed to its design crest elevation by placement (in-the-dry) of the rockfill, granular transition, and impervious fill.

Potential sources of finer materials that may increase the TSS levels monitored at Site K-Tu-2 are summarized in Table 8.

8.1.4 STAGE I ISLAND COFFERDAM

A zoned earth and rockfill embankment is assumed to be constructed in-the-dry as shown in Figure 4 to provide protection of the principal structures area against design flows and

water levels during Stage I and Stage II Diversion. No TSS calculation is required for this cofferdam.

8.1.5 POWERHOUSE STAGE I COFFERDAM (PHOTO 6)

An initial Class C protective rock groin is assumed to be placed in-the-wet as shown in Figure 4 to provide shelter from erosion while constructing the zoned earth and rockfill embankment in-the-wet. Following completion of the cofferdam, additional Class C materials will be placed on the Class A layer to provide erosion protection during the winter months. The structure will provide closure along the east side of the Powerhouse construction area. The Class A and B material are considered to be Condition III cases as outlined in Subsection 5.2.

8.1.6 SPILLWAY STAGE I COFFERDAM (PHOTO 7)

A U-shaped cofferdam will be constructed in the South Channel as shown in Figure 4 to enable rock excavation and construction of the Spillway. It is assumed that it will utilize an initial rockfill protection groin, followed by placement in-the-wet of the zoned earth and rockfill embankment within the sheltered enclosure. The Class A and B material are considered to be Condition III cases as outlined in Subsection 5.2.

8.1.7 CENTRAL DAM STAGE I COFFERDAM (PHOTO 8)

An initial Class C rockfill embankment is assumed to be placed in-the-wet as shown in Figure 4, followed by placement of Class B and Class A zones on-the-wet side and rip rap on the upper upstream slope. The structure will complete the closure for the principal structures area. A Class C rockfill groin is assumed to be placed in-the-wet prior to placement of Class B and A materials to prevent further erosion due to high flows. The Class A and B material are considered to be Condition I cases as outlined in Subsection 5.2.

8.1.8 TAILRACE SUMMER LEVEL COFFERDAM (PHOTO 6)

A cofferdam will be constructed to enable rock excavation and construction of the tailrace channel as shown in Figure 3. It is assumed to utilize an initial rockfill protection groin, followed by placement in-the-wet of the zoned earth and rockfill embankment within the sheltered enclosure. The Class A and B material are considered to be Condition III cases as outlined in Subsection 5.2.

8.2 STAGE II DIVERSION

Construction of the Stage II Diversion cofferdams will close off the south river channel, diverting all flow though the Spillway.

8.2.1 SOUTH DAM STAGE II U/S COFFERDAM (PHOTOS 9 AND 10)

Closure of the South Channel is assumed to be made with a Class C rockfill embankment, placed in-the-wet, followed by placement of Class B and Class A zones on-the-wet side. The Class A and B material are considered to be Condition I cases as outlined in Subsection 5.2.

8.2.2 SOUTH DAM STAGE II D/S COFFERDAM (PHOTOS 9 AND 10)

The initial operation is assumed to be placement of a Class C rockfill embankment, placed in-the-wet, followed by placement of Class B, Class A and riprap zones on-the-wet side as shown in Figure 5.

It is assumed that there will already have been a single leg closure and completion of the upstream cofferdam zones at this stage, in which case there would be no losses or TSS generated during the placement of Class A, B, and C materials which would then be placed in calm water.

In the extreme, if a two leg closure were to be required, the TSS calculation would assume a 0.5 % loss factor for initial Class C placement, with 70% river flow. This would give identical TSS results shown in Table 9.0, 9.1 and 9.2 for the upstream Cofferdam, resulting in a doubling of TSS while simultaneously placing both rockfills. There would be no losses or TSS for placement of Class A and B material since they would be placed in calm water after the upstream cofferdam is sealed.

8.2.3 STAGE II ISLAND COFFERDAM

A zoned earth and rockfill embankment will be constructed in-the-dry, to provide closure for the principal structures area. No TSS calculation is required for this cofferdam.

8.3 COFFERDAM REMOVAL

Removal of the cofferdams in-the-dry will likely be done using backhoes, dozers and loaders. Cofferdam removal in-the-wet is assumed to be undertaken using backhoes and a dragline.

8.3.1 SPILLWAY STAGE I COFFERDAM

It is assumed that materials will be removed by a backhoe, with the Spillway gates closed to minimize velocities in the vicinity of the materials being removed. The limits of the removal of the cofferdam are shown in Figure 3. Although the cofferdam is intended to be completely removed in the required area, it is likely that some material will remain in place after removal. Erosion of this remaining material is discussed in Section 9.

8.3.2 SOUTH DAM STAGE II U/S COFFERDAMS

The South Dam Stage II U/S Cofferdam will not be removed, but will be incorporated into the South Dam, as shown in Figure 3.

8.3.3 POWERHOUSE STAGE I COFFERDAM

The north/south leg of the Powerhouse Stage I Cofferdam is assumed to be removed in-the-dry inside the confines of the Tailrace Summer Level Cofferdam, as shown in Figure 3, resulting in no TSS generation. The Class A and much of the Class B materials within the east/west leg of the Powerhouse Stage I Cofferdam are assumed to be removed within the shelter of the protective rock groin, which will result in a partially exposed Condition III as outlined in Subsection 5.2. It is conservatively assumed that the Condition III does not apply to the Class B or C materials in the east/west leg of the cofferdam, which are assumed to be removed in-the-wet by backhoe and/or dragline. Although it is intended that the cofferdam be completely removed within the limits shown on Figure 3, it is likely that some material will remain in place after removal due to irregularities in the riverbed. Any such remaining material will be quickly eroded by flows from the Spillway once the outside protective rockfill groin has been removed. The erosion of this material is discussed in Section 9.

8.3.4 CENTRAL DAM COFFERDAM AND SOUTH DAM STAGE II D/S COFFERDAM

These structures will remain in place and will be incorporated into the permanent earthwork structures (i.e. completed dams) as shown in Figure 3. It is assumed that there will be partial removal of impervious and transition materials from the downstream sides of the dams to facilitate embankment drainage. Excavation is assumed to be in calm water in both cases, resulting in no TSS generation

8.3.5 TAILRACE SUMMER LEVEL COFFERDAM

The north/south leg of the Tailrace Summer Level Cofferdam is assumed to be removed in-the-wet by backhoe and/or dragline in relatively calm water. If required, the use of a silt curtain between the outer protective rock groin of the Tailrace Summer Level Cofferdam and the North Abutment of the Nelson River which would further mitigate the introduction of TSS from this cofferdam into the Nelson River may be possible. The suitability of a silt curtain would have to be determined during final design.

The removal of material from the east/west leg of the cofferdam is assumed to occur in two phases in a similar manner to the Powerhouse Stage I Cofferdam. In the east/west leg, the Class A and much of the Class B material is assumed to be removed within the shelter of the protective rock groin, resulting in a partially exposed Condition III as outlined in Subsection 5.2. This approach will minimize the amount of TSS introduced

into the full river flow. It is conservatively assumed that the Condition III does not apply to the Class B or C materials in the east/west leg of the cofferdam, which are assumed to be removed in-the-wet by backhoe and/or dragline.

8.3.6 ALL OTHER COFFERDAMS

The structures will remain in place permanently, with their crests partly excavated in-the-dry, resulting in no TSS. The crest excavations of the North Channel Rock Groin, the North Channel Stage I Cofferdam and the Stage I and II Island Cofferdams may involve some excavation in-the-wet, but are assumed to be in essentially calm water conditions due to zero flow in the north channel at that time, with the Powerhouse closed.

9 INITIAL OPERATION OF THE SPILLWAY AND POWERHOUSE UNITS

First order estimates of the TSS generated during first Spillway operation, and for each Powerhouse Unit as it is tested and becomes operational, were made using simplified calculations.

Two approaches were used to estimate the TSS increases:

1. Simplified Average Method (SAM), which consisted of:
   - Estimating the total weight of material that could be eroded for the case being considered.
   - Assuming that the material would be completely eroded in a single day.
   - Assuming that all of the erodible material was eroded, even in areas where the bed shear stress was less than the critical shear stress for the material.
   - All of the eroded material would be carried from the eroding area to K-Tu-2.
   - Determining the cumulative volume of water that would flow past K-Tu-2 for a given elapsed time as:

     Total Volume of Water = River Flow * Elapsed Time

   - Calculating the average fully mixed TSS, in mg/l, as:

     TSS = 1000 * [Total Weight of Material] / [Total Volume of Water]

   This approach provides an estimate of the maximum average daily increase in TSS when the Elapsed Time is taken as 86 400 seconds (one day).

2. Simplified estimation of the erosion as a function of bed shear stress and time (referred to as the SET method), where
   - The sediment carrying capacity of the flow in the eroding areas calculated using the following methods: Ackers-White (1993), Engelund – Hansen, and Yang

- The material entrainment rate was defined as:

$$Ce = 1.368 – \exp(-L/(30 * D)), \text{ for } L <= 30 * D$$

$$Ce = 1.0 \text{ for } L > 30 * D$$

Where:

Ce = entrainment coefficient

L = upstream-downstream length of material exposed to flow

D = flow depth

The estimated increases in TSS due to erosion of the cofferdam remnants were based on the following:

- The estimated maximum particle size that will remain in suspension at K-Tu-2 is 0.5 mm, based on the Rouse equation and the average velocity at K-Tu-2.
- The representative particle size used in the sediment transport functions was determined from a revised gradation of the eroding material. The material gradation was revised such that 100% is finer than the coarsest particle that will remain in suspension at K-Tu-2 for the assumed river flow. This resulted in average particle diameters of 0.02 mm and 0.15 mm for the Class A and Class B materials, respectively.
- The erosion of each remnant was assumed to be independent of the TSS of the incoming water.
- The increase in TSS as K-Tu-2 was based on fully mixed conditions.
- The top of the eroding remnant was assumed to be the average elevation of the riverbed (i.e. bedrock) plus the remnant thickness.
- The permanently exposed excavation faces at the ends of the removed portions of the cofferdam will be over-excavated and protected by rockfill or riprap, in order to reduce the potential erosion.
- Once in suspension, all of the eroded material would be carried to K-Tu-2, i.e. there would be no deposition of the eroded material.

9.1 INITIAL OPERATION OF THE SPILLWAY:

Potential sources of increases in TSS during initial operation of the Spillway are:

- Suspension of material remaining on the invert of the Approach and Discharge Channels after final clean-up (i.e., debris).
- Erosion of cofferdam fill remnants.

Although the channels will be cleaned to the maximum extent practical during construction, some material will remain on the channel inverts. The portions of the Approach and Discharge Channels that are excavated into bedrock cover an area of approximately 73,000 m^2. Assuming an average thickness of 5 mm of erodible material[2] would remain after final clean-up about 657,000 kg of material could be suspended from the channel inverts.

Cofferdam removal is intended to be to riverbed level. The bedrock surface has been scoured by glacial action and is generally smooth and undulating, however, local irregularities in the bedrock surface will likely result in some material remaining in "pockets" that cannot be cleaned by an excavator or dragline. Based on engineering judgment, the best possible removal by excavator or dragline under favourable site characteristics would result in a volume of material equal to an average thickness of 0.1 m of cofferdam fill remaining in place. For the calculation of increases in TSS levels an average thickness of remnant of 1 m has been adopted to allow for local irregularities.

It is envisioned that for the Spillway Diversion, the Spillway gates will be actively managed to control and minimize the TSS generation during the initiation of Stage II Diversion. It has been assumed that the Spillway Gates will be adjusted on an hourly basis, similar to what was done at Wuskwatim GS. Average flows during river closure through the Spillway for three cases were examined:

i. One gate open for 1 hour.
ii. All gates open 1 m for 1 hour.
iii. All gates fully open, with the following hydraulic conditions for a total flow of 3,743 m^3/s:

2 The average thickness of debris on the channel invert will be greater than 5 mm, however, 5 mm represents the equivalent thickness of material that would remain in suspension at K-Tu-2. The remaining debris will either settle out of the flow prior to reaching K-Tu-2, will be bedload, or will not get suspended.

Description	Flow Through South River Channel (m^3/s)	Flow Through Spillway (m^3/s)	Energy Grade Line Elevation (m)	
			Upstream	Downstream
1 Bay Open 1 m	3680	63	148.0	145.2
7 Bays Open 1 m	3321	422	147.7	145.2
7 Bays Open Full	2432	1311	146.6	145.2

9.2 POWERHOUSE UNIT TESTING AND INITIAL OPERATION

Potential sources of increases in TSS during initial operation of the Powerhouse Units are:

- Suspension of material remaining on the invert of the Intake and Tailrace Channels after final clean-up.
- Erosion of cofferdam fill remnants.
- Erosion of the overburden downstream of the Tailrace Channel (which is beyond the scope of this memorandum, see Ref. 11.
- Re-suspension of sediment deposited upstream of the Intakes during initial impoundment and Stage II Diversion.

The main increases in TSS are expected during passage of the initial flow through each unit in the Powerhouse during testing of the unit.

During unit testing, the turbine will be operated at speed-no-load for around 4 to 6 hours. For Keeyask, the flow through the Intake-Powerhouse at speed-no-load is estimated to be 233 m^3/s. Following the completion of this initial testing, it is anticipated that the unit will be stepped up to rated flow capacity (585 m^3/s) over a period of 1 or 2 days. For estimating the increase in TSS during initial unit start-up, it was assumed that the testing will occur over a period of 24 hours with the flow through the unit being increased by approximately 15 m^3/s every hour.

Although the channels will be cleaned to the maximum extent practical during construction, some fine material will remain on the channel inverts. The portions of the Intake and Tailrace Channels that are excavated into bedrock cover an area of approximately 207,000 m^2. For 5 mm of erodible material remaining after final clean-up, about 1.865×10^6 kg of material would available to be suspended from the channel inverts.

Most of the cofferdam is founded on overburden, consequently the riverbed is anticipated to be relatively smooth and can be excavated with a hoe or dragline. This should permit a more complete removal of the cofferdam than in areas where the cofferdam is founded on bedrock. The area where the cofferdam is founded on bedrock is located immediately downstream of the Transmission Tower Spur which will be subjected to relatively high velocity flow coming from the Spillway. Erosion of the cofferdam remnant in this area due to flow from the Spillway has been assumed to be complete prior to commencing unit startup in the Powerhouse. Consequently, there will be no cofferdam remnant that could erode due to flow through the Powerhouse.

During Stage II Diversion, it is expected that some sediment will be deposited in the still water area immediately upstream of the Intakes. This sediment will be flushed downstream during unit testing and will result in spikes in TSS levels. The estimated amount of sediment that will be deposited upstream of the Intakes was based on the following assumptions:.

- The water had an initial TSS of 20 mg/l
- All of the sediment (20 mg/l) in the entire water column, from FSL to channel invert), will have settled out prior to commencing unit testing.

It is estimated that a maximum thickness of 0.3 mm of sediment that may be deposited in the Intake Channel during Stage I Diversion, based on a 30 m deep column of water initially having a TSS of 20 mg/l.

10 RESULTS OF ANALYSES

10.1 TSS INCREASES DUE TO COFFERDAM CONSTRUCTION/REMOVAL

The calculations presented in Tables 9.0, 9.1 and 9.2 of this report predict that the TSS concentrations introduced into the Nelson River due to cofferdam construction and removal will be negligible compared to the River's background TSS levels of 5 mg/l to 30 mg/l [Ref 12] and the TSS generated from shoreline erosion during construction, which ranges from 3 mg/l to 7 mg/l for Stage I Diversion and 4 mg/l to 14 mg/l for Stage II Diversion [Ref 11]. Estimated values of TSS, presented in Tables 9.0, 9.1 and 9.2, were calculated based on engineering judgment and experience, utilizing worst case scenarios. The estimated TSS concentrations are generally far less than those obtained for the Wuskwatim GS [Ref. 4]. This is mainly due to the river channel's physical characteristics, river flows and the dilution effect of the river flow at the Keeyask GS being three to four times greater than at the Wuskwatim GS.

The TSS values will generally be more elevated in "local" flow areas, which will occur in partial flow channels and in "partially exposed" cases; however, these concentrations will

decrease upon being mixed into the main flow of the river. It is anticipated that placement and removal of fine clay and silt material will take place in calm water conditions, such as behind the protective rock groins. This material will then settle out next to the cofferdams or enter the main flow at very slow rates thereby resulting in little increase in TSS. However, to be conservative, it has been assumed that the full sediment volume enters the main flow of the river. When mixed completely with the main river flow, the TSS generated by this material is calculated to be negligible, as shown in Table 9.0. Consideration should be given to the use of silt curtains at the end of the protective rock groins as an added measure for mitigation against TSS levels.

Estimated values of TSS were calculated at Monitoring Station K-Tu-2, located downstream of the Keeyask GS, as seen on Figure 2. The values shown in Tables 9.0, 9.1 and 9.2 indicate the estimated TSS levels that may impact aquatic systems downstream of the project. Rockflour and silt sized materials entrained in the flow from placement or removal of Class B and C materials are expected to reach Monitoring Station K-Tu-2, and some Class A material will also likely remain in suspension at Monitoring Station K-Tu-2. However, these TSS levels are not expected to be cumulative and the levels will be very small. The Spillway Stage I Cofferdam removal and the Powerhouse Stage I Cofferdam removal are predicted to contribute an increase of about 2 mg/l and 1 mg/l respectively in the TSS levels at Monitoring Station K-Tu-2. Somewhat more elevated levels of 4 mg/l are estimated during removal of the Tailrace Summer Level Cofferdam Class A material that is located outside the area protected by the outer rock groin. However, the use of a silt curtain between the protective rock groin of the Tailrace Summer Level Cofferdam and the North Abutment of the Nelson River would mitigate the introduction of TSS from this cofferdam removal into the Nelson River.

Tables 9.0, 9.1 and 9.2 cover only the increase in TSS due to construction and removal of individual cofferdams in isolation. Based on the Summary Construction Schedule (Table 3), the only anticipated overlaps of activities are: 1) concurrent construction of the Powerhouse Stage I Cofferdam with the North Channel Rock Groin; 2) concurrent construction of the Powerhouse Stage I Cofferdam with the North Channel Stage I Cofferdam and 3) concurrent construction of the Spillway Stage I Cofferdam with the Central Dam Stage I Cofferdam. The estimated increases in TSS incorporating the effects of overlapping construction activities are shown in Figure 7. The increases in TSS due to these overlapping construction activities peak at 4 mg/l for periods of 1 or 13 days (Figure 7).

10.1.1 LOCAL INCREASES IN TSS

The calculated local values of 23 mg/l and 51 mg/l, for low flow conditions, for the Class C (armour), placed on top of the impervious core of the Spillway Stage I Cofferdam and of the Tailrace Summer Level Cofferdam respectively, are anticipated to be locally high values that will be contained between the dual rock groins of the cofferdams (see Table 9.0). These TSS concentrations are generated from the fine material content in the Class C material (i.e. Rockflour, dust etc.) and are elevated values resulting from the assumed local flow of 1% of the total river flow acting inside the protective rock groins, and may occur during the entire time of fill placement (29 days for the Spillway Stage I Cofferdam and 20 days for the Tailrace Summer Level Cofferdam.

In addition, a local hourly value of 28 mg/l, for a period of 2 days, was calculated for the removal of the Class A material from inside the dual rock groin of the Summer Level Cofferdam. This value is for low flow conditions and is based on the assumed local flow of 1% of the total river flow acting inside the protective rock groins (see Table 9.0).

The local daily value of 7 mg/l (Table 9.0), estimated for removal of the Class A from outside the rock fill groin, will have a duration of 5 to 10 days.

10.2 SENSITIVITY TO ASSUMED WATER LEVEL

In order to check the sensitivity of the calculated increases in TSS values to the assumed construction water levels, calculations were carried out for the hypothetical maximum case of the entire fill quantities being placed in-the-wet. The results were compared to the TSS values obtained with the above assumed water levels. The largest difference was 6 mg/l, for Powerhouse Stage I Cofferdam Class C placed at low flow, with a hypothetical maximum TSS of 12 mg/l versus an expected maximum value of 6 mg/l. All the other differences ranged from 0 to 5 mg/l, and averaged 1 mg/l. The relatively small differences obtained indicate a low sensitivity of the predicted TSS increases to the assumed construction water levels. All of the above predicted TSS increases are for fully mixed conditions.

10.3 TSS INCREASES DUE TO INITIAL OPERATION OF SPILLWAY AND POWERHOUSE UNITS

10.3.1 SPILLWAY

The estimated increases in TSS discussed in this section are above background levels, for fully mixed conditions at K-Tu-2, with a total river flow of 3743 m^3/s.

The material remaining on the invert of the Approach and Discharge Channels after final clean-up (i.e. debris) would be eroded within a few minutes if the Spillway Gates were to

be open fully. Although there would be an initial extremely high spike in TSS levels for a short period of time, the increase in TSS would average about 2 mg/l for a day, using the Simplified Average Method (SAM). Increases in TSS using SAM vary in direct proportion to the assumed thickness of erodible debris on the channel inverts. For example, if the thickness of erodible debris on the channel inverts were to be doubled to 10 mm, the estimated increases in TSS would also double, becoming 4 mg/l for 24 hours. Due to the small average daily increase in TSS using SAM, calculations using the SET method were not considered necessary.

Increases in TSS due to cofferdam remnant erosion were estimated using both the SAM and SET methods. For the assumed case where an average thickness of cofferdam remnant is 1 m, with 100% of the Class A and 35% of the Class B being eroded, resulting in a total mass of 9.1836×10^6 kg of remnant material, the average daily TSS increase estimated using SAM was about 28 mg/l, assuming that all of the cofferdam remnants were eroded in a single day. When bed shear and time are considered in the remnant erosion, the TSS increases are as high as about 250 mg/l for a period of less than 25 minutes, with a daily average increase of about 24 mg/l, based on the Engelund-Hansen sediment transport model (Figure 6).

Both of the SAM and SET methods indicate a short duration for any significant sediment increases during initial Spillway operation for a flow of 3743 m^3/s with all seven gates fully open, with average daily increases in the vicinity of 28 mg/l. It must be noted that this scenario (i.e. all Spillway gates open) will not be employed during initial operation of the Spillway. Instead, this opening of the Spillway gates will be actively managed to minimize TSS generation during initial operation.

10.3.2 POWERHOUSE

The estimated increases in TSS discussed in this section are above background levels, for an average particle diameter of 0.02 mm, assuming fully mixed conditions at K-Tu-2, with a total river flow of 1787 m^3/s, which is the 1:20 year low flow for October (Table 2).

Suspension of most of the material remaining on the invert of the Intake and Tailrace Channels after final clean-up will occur during testing and initial operation of the first unit in the Powerhouse. If the sediment carrying capacity of the flow is ignored, the 5 minute average increase in TSS would be about 41 mg/l at the initial start-up of Unit 1, which would decrease to increases of less than 19 mg/l (5 minute average) during the first increment in flow through the Unit, with 5 minute average increases in TSS decreasing with each incremental increase of flow through the unit (see Figure 8(d.)).

The estimated TSS increases due to re-suspension of deposited sediment is less than 1 mg/l (5 minute average, Figure 8).

These increases in TSS above background levels represent the worst case for the start-up of any unit in the Powerhouse since the flow from Unit No.1 will affect approximately 80% of the bedrock portions of the Intake and Approach Channels. The remaining 20% of the channels will be affected by the start-up of Units 2 to 7. The estimated increases in TSS during the testing of all units is shown in Figures 9 and 10 for the case when the sediment is fully mixed into the flow through the Powerhouse.

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1. KGS Acres Ltd., "Keeyask Generating Station, Stage IV Studies - Axis GR-4, Earthworks Structures, Cofferdam Arrangement and Design, Memorandum GN-4.2.2.11 Rev 0", June 10, 2011. Manitoba Hydro File 00195-21600-0004-01.
2. Draft KGS ACRES Report "Keeyask Generating Project, Stage IV Studies – Physical Environment, Preliminary Consideration of Methods to Manage Sediment During Construction, Deliverable GN-9.2.15". Manitoba Hydro File 00195-11100-0188_00.
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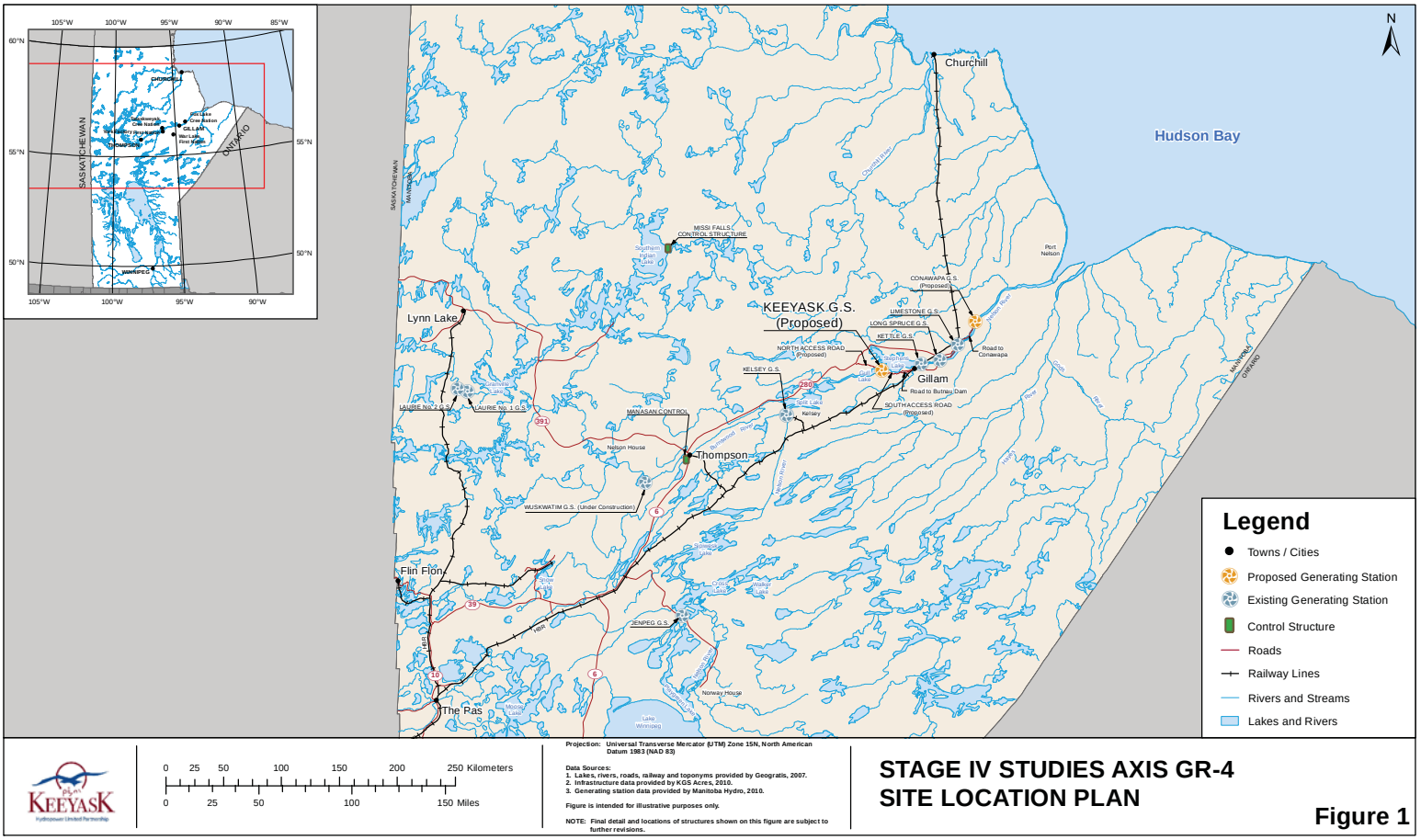
4. Acres Office Memorandum "Wuskwatim Generating Station, Stage IV Studies, Burntwood River, Cofferdam and Riverbed Erosion During Construction, Memorandum W-2.9.1 Rev 1" April 28, 2003. Manitoba Hydro File 00184-104.
5. Manitoba Hydro "Nelson River Studies, Gull Generating Station, 1991 Summer Subsurface Investigation Report, Report No. PSPD 95-3" November 1993. Manitoba Hydro File 00195-11600.
6. Email from W. DeWit, Manitoba Hydro, "Keeyask Construction Timeline Sedimentation 2011 11 16 – Charts for SV, Summary Construction Schedule, January 12, 2012.
7. KGS Acres Ltd., "Keeyask Generating Station, Stage IV Studies – Axis GR-4, River Management During Construction, Memorandum GN-2.8 Rev 0", June 10, 2011. Manitoba Hydro 00195-10420-0002_03.
8. KGS Acres Ltd., "Keeyask Generating Station, Stage IV Studies – Axis GR-4, Hydrology Review Construction Design Flood, Memorandum GN-2.3 Rev 0", December 2010. Manitoba Hydro 00195-11350-0007_00.
9. LaSalle Consulting Group Inc., "Keeyask Generating Station, Stage IV Studies, Comprehensive Model Study", August 2005.
10. Manitoba Hydro, "Wuskwatim Monitoring Overview for the Period Ending March 31, 2011.
11. Manitoba Hydro, "Keeyask Generating Station, Stage IV Studies, Shoreline Erosion and Sedimentation Effects During Construction, Memorandum GN-9.2.10 Draft 2", July 13, 2009. Manitoba Hydro 00195-11100-0161_01.
12. KGS Acres Ltd., "Keeyask Generating Station, Stage IV Studies, Existing Environment Sedimentation, Memorandum GN-9.2.3 Rev 0", June 10, 2011. Manitoba Hydro 00195-11100-0154_03.

13. KGS Acres Ltd., “Keeyask Generating Station Stage IV Studies, River Management During Construction Memorandum GN-2.8 Rev 1”, March 23, 2012 Manitoba Hydro 00195-10420-0002_04.

FIGURES



Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Lakes, rivers, roads, railway and toponyms provided by Geogratis, 2007.
2. Infrastructure data provided by KGS Acres, 2010.
3. Generating station data provided by Manitoba Hydro, 2010.

Figure is intended for illustrative purposes only.

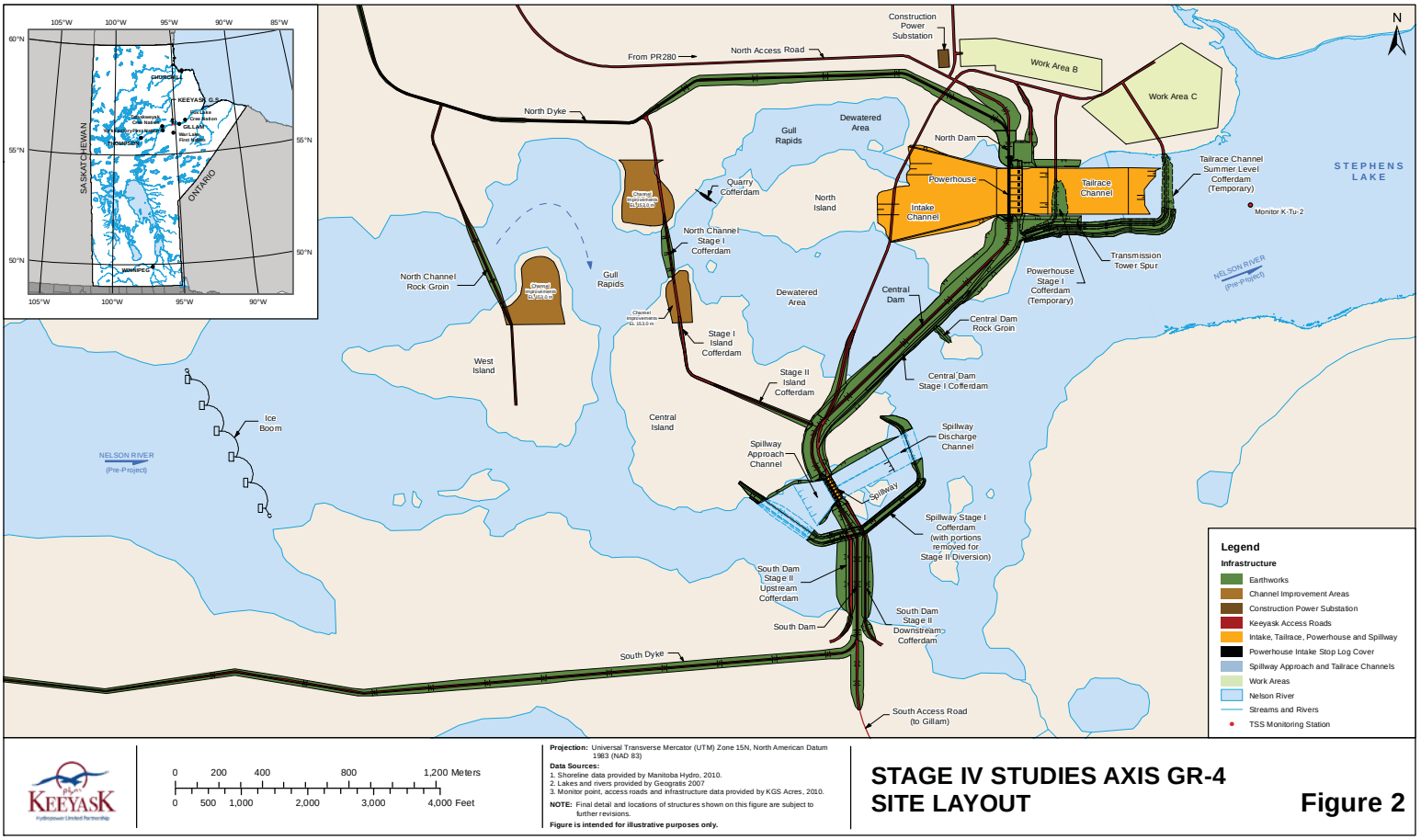
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 SITE LOCATION PLAN

Figure 1

Created By: KGS ACRES

Date Created: March 23, 2012



0 200 400 800 1,200 Meters

0 500 1,000 2,000 3,000 4,000 Feet

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Shoreline data provided by Manitoba Hydro, 2010.
2. Lakes and rivers provided by Geogratis 2007
3. Monitor point, access roads and infrastructure data provided by KGS Acres, 2010.

NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

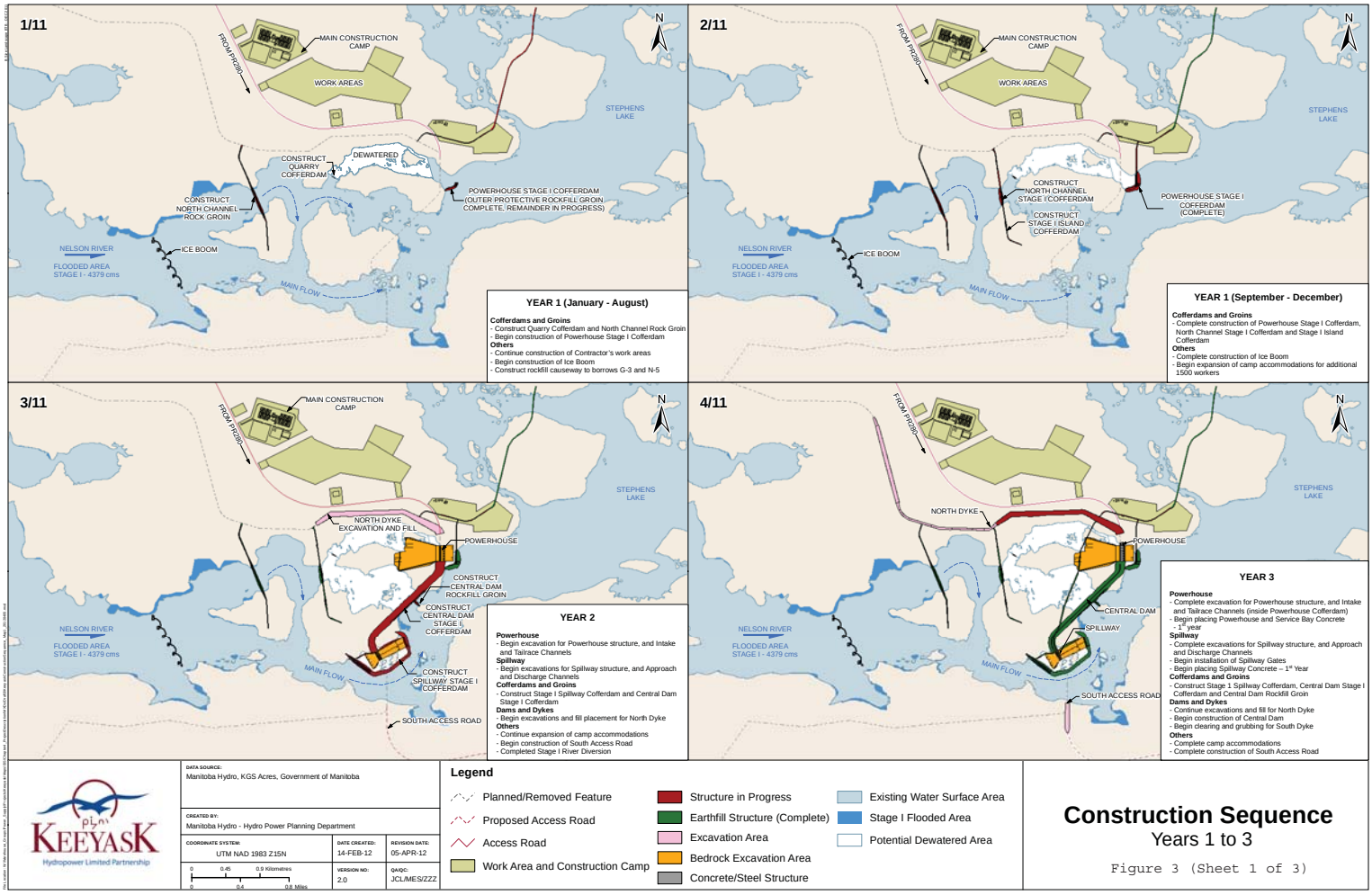
Figure is intended for illustrative purposes only.

STAGE IV STUDIES AXIS GR-4 SITE LAYOUT

Figure 2

Created By: KGS ACRES

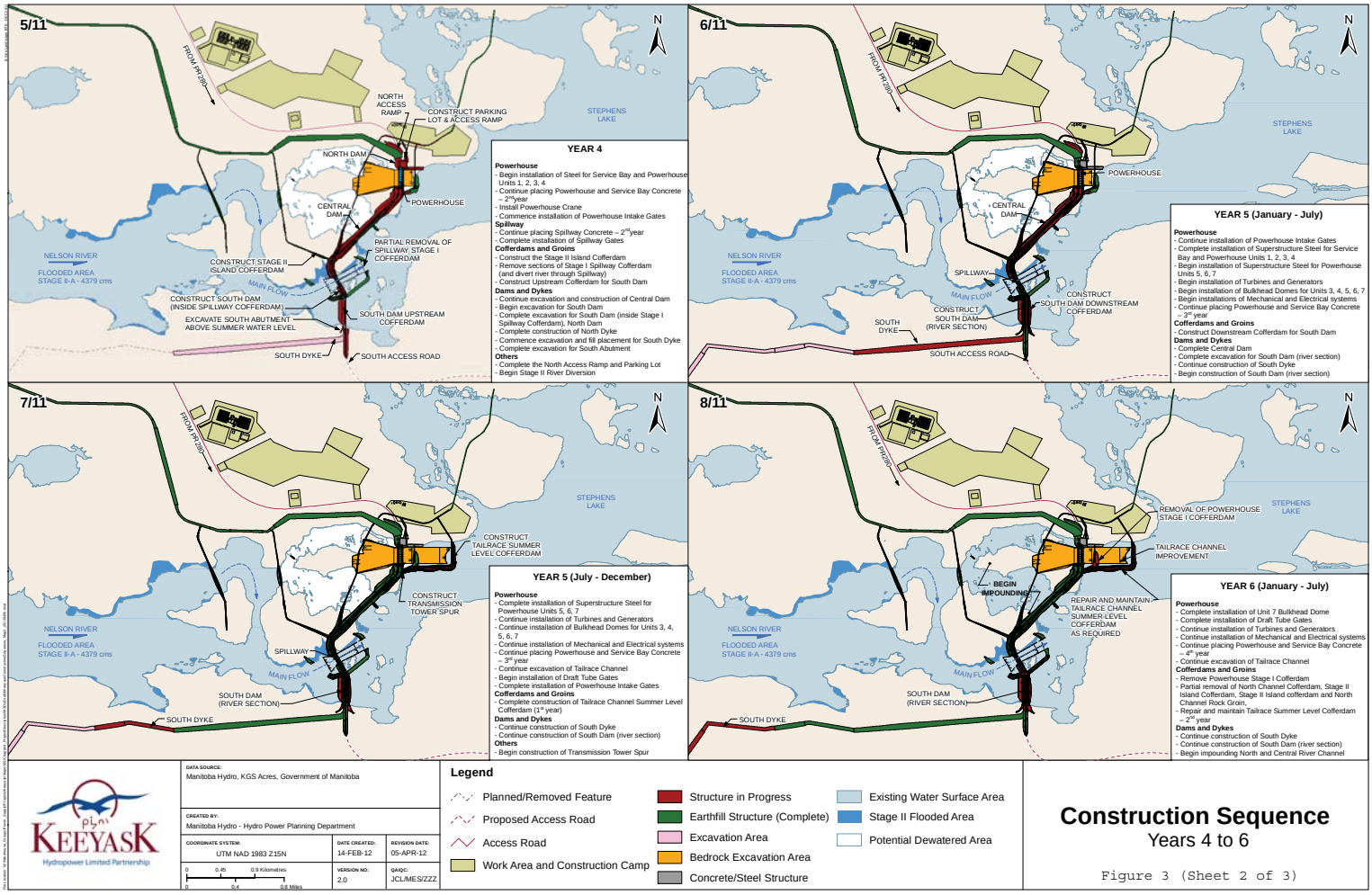
Date Created: May 27, 2011



Construction Sequence

Years 1 to 3

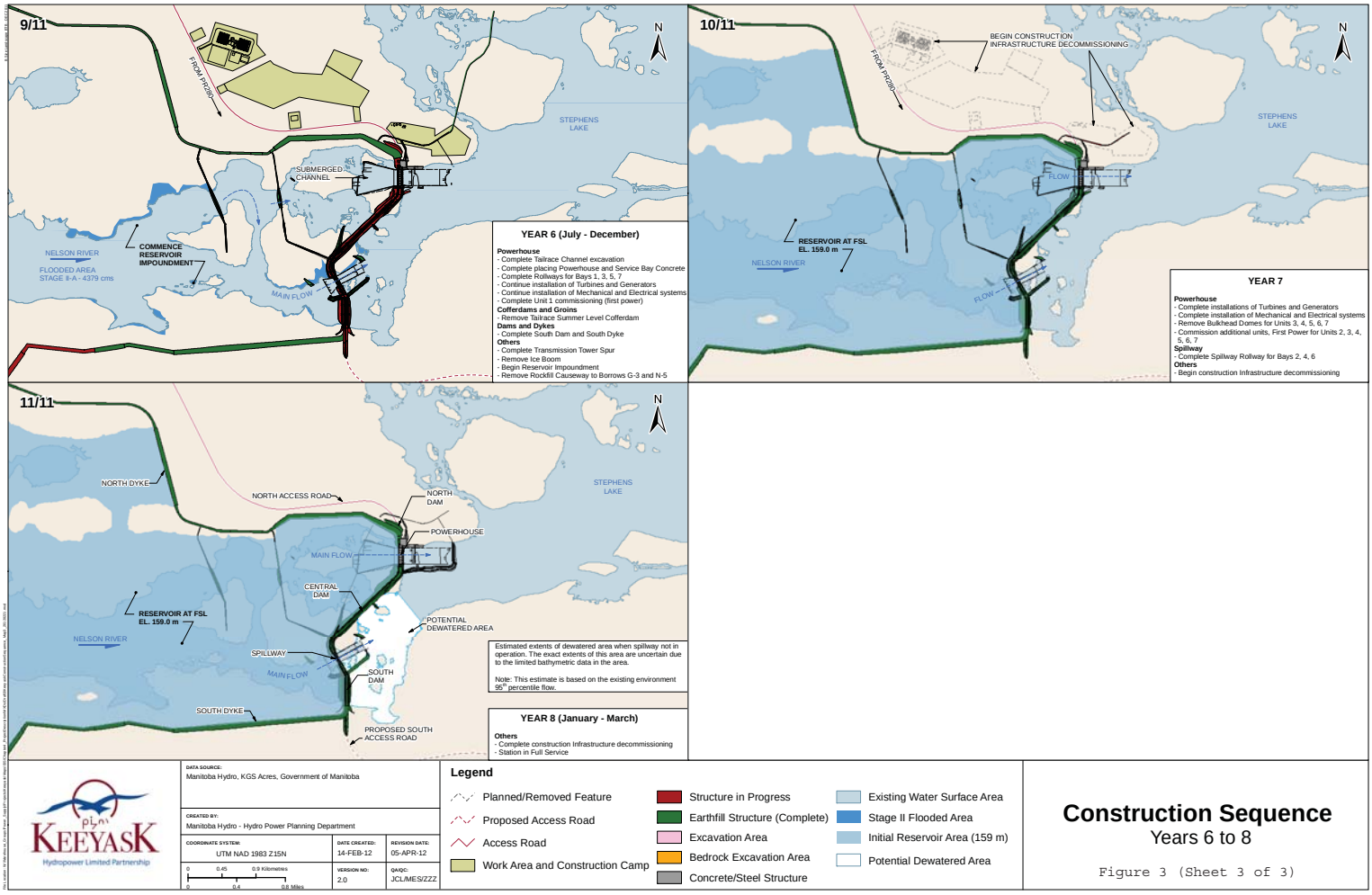
Figure 3 (Sheet 1 of 3)



Construction Sequence

Years 4 to 6

Figure 3 (Sheet 2 of 3)



KEEYASK Hydropower Limited Partnership

DATA SOURCE: Manitoba Hydro, KGS Acres, Government of Manitoba

CREATED BY: Manitoba Hydro - Hydro Power Planning Department

COORDINATE SYSTEM:	DATE CREATED:	REVISION DATE:
UTM NAD 1983 Z15N	14-FEB-12	05-APR-12
0 0.45 0.9 Kilometres / 0 0.4 0.8 Miles	VERSION NO: 2.0	QA/QC: JCL/MES/ZZZ

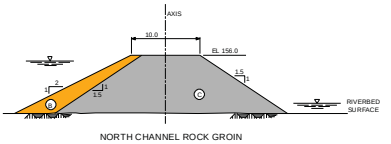
Legend

- Planned/Removed Feature
- Proposed Access Road
- Access Road
- Work Area and Construction Camp
- Structure in Progress
- Earthfill Structure (Complete)
- Excavation Area
- Bedrock Excavation Area
- Concrete/Steel Structure
- Existing Water Surface Area
- Stage II Flooded Area
- Initial Reservoir Area (159 m)
- Potential Dewatered Area

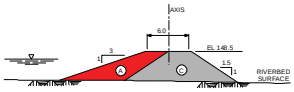
Construction Sequence

Years 6 to 8

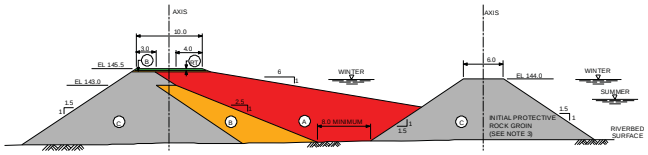
Figure 3 (Sheet 3 of 3)



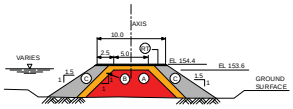
NORTH CHANNEL ROCK GROIN



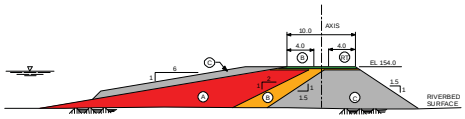
QUARRY COFFERDAM



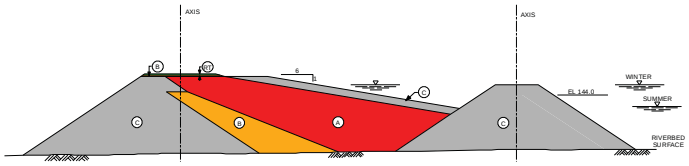
POWERHOUSE STAGE I COFFERDAM
STEP I (INITIAL)



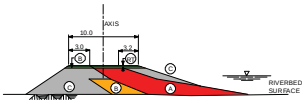
STAGE I ISLAND COFFERDAM



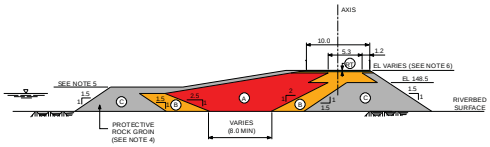
NORTH CHANNEL STAGE I COFFERDAM



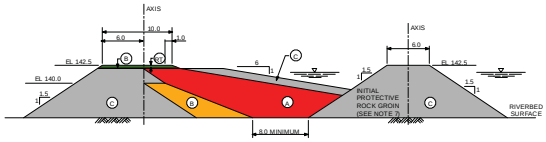
POWERHOUSE STAGE I COFFERDAM
STEP II (FINAL)



CENTRAL DAM STAGE I DOWNSTREAM COFFERDAM



SPILLWAY STAGE I COFFERDAM



TAILRACE SUMMER LEVEL COFFERDAM

Legend

- A — Impervious Fill
- B — Granular Transition Fill
- C — Rock Fill (See Note 8)
- RT — Road Topping
- Water Level

NOTES:

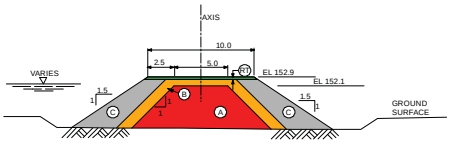
1. DIMENSIONS AND ELEVATIONS ARE IN METRES.
2. ELEVATIONS ARE BASED ON GS OF C_CGVD28_1929 ADJUSTMENT.
3. INITIAL PROTECTIVE ROCK GROIN NOT REQUIRED FOR POWERHOUSE COFFERDAM DOWNSTREAM OF TAILRACE CHANNEL.
4. PROTECTIVE ROCK GROIN NOT REQUIRED FOR SPILLWAY STAGE I COFFERDAM DOWNSTREAM OF DISCHARGE CHANNEL
5. CREST ELEVATION SET AT 1.0 METRE ABOVE THE PREVAILING WATER LEVEL AT TIME OF CONSTRUCTION
6. THE CREST SHALL BE EL 151.0 AT THE LEFT ABUTMENT OF THE UPSTREAM LEG SLOPING TO EL 150.5 AT THE RIGHT CORNER OF THIS LEG, THEN SLOPING ALONG THE RIVER LEG TO EL 148.0 AT THE RIGHT CORNER OF THE DOWNSTREAM LEG. THE DOWNSTREAM LEG SHALL HAVE A CREST AT EL 148.0 FOR ITS ENTIRE LENGTH.
7. PROTECTIVE ROCK GROIN NOT REQUIRED FOR DOWNSTREAM LEG OF TAILRACE SUMMER LEVEL COFFERDAM.
8. VARIOUS SIZES AND GRADATIONS OF ROCKFILL (CLASSES C1, C2 AND C3) WILL BE UTILIZED. PRECISE LOCATIONS OF UTILIZATION WILL BE BASED ON FLOW CONDITIONS.

STAGE IV STUDIES AXIS GR-4
STAGE I COFFERDAM CROSS-SECTIONS

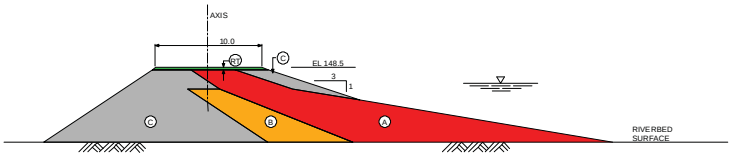
Figure 4

Created By: KGS ACRES

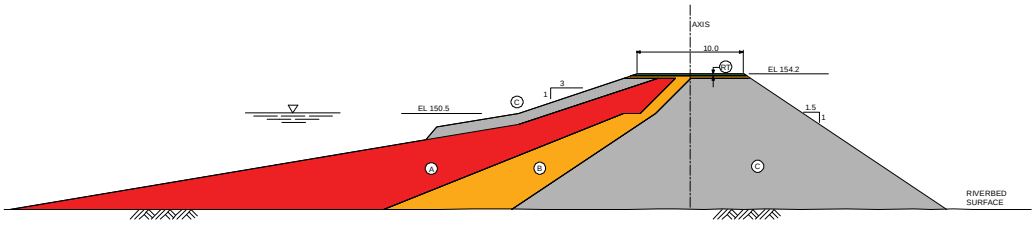
Date Created: March 23, 2012



STAGE II ISLAND COFFERDAM



SOUTH DAM STAGE II DOWNSTREAM COFFERDAM



SOUTH DAM STAGE II UPSTREAM COFFERDAM

Legend

- A — Impervious Fill
- B — Granular Transition Fill
- C — Rock Fill (See Note 3)
- RT — Road Topping
- Water Level

NOTES:

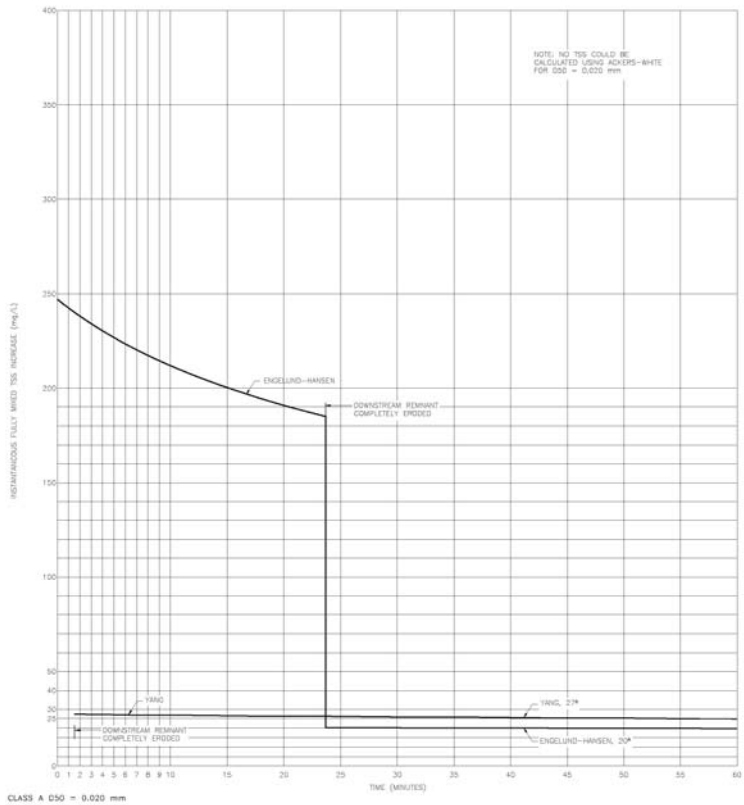
1. DIMENSIONS AND ELEVATIONS ARE IN METRES.
2. ELEVATIONS ARE BASED ON GS OF C, CGVD28, 1929 ADJUSTMENT.
3. VARIOUS SIZES AND GRADATIONS OF ROCKFILL (CLASSES C1, C2 AND C3) WILL BE UTILIZED. PRECISE LOCATIONS OF UTILIZATION WILL BE BASED ON FLOW CONDITIONS.

STAGE IV STUDIES AXIS GR-4
STAGE II COFFERDAM CROSS-SECTIONS

Figure 5

Created By: KGS ACRES

Date Created: May 27, 2011



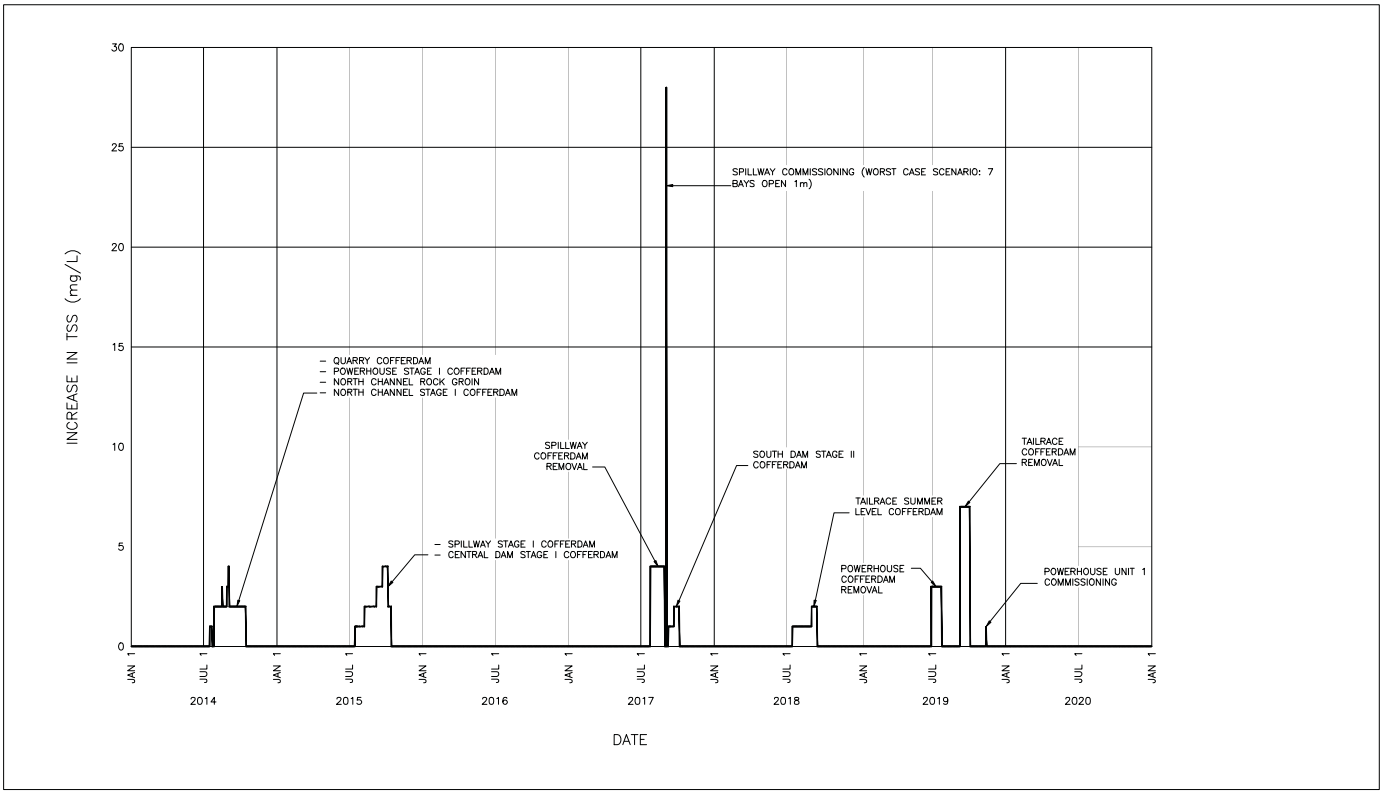
CLASS A D50 = 0.020 mm
CLASS B D50 = 0.150 mm

7 BAYS OPEN 1 m

NOTES:

1. TOTAL FLOW = 3743 m3/s.

		31 JAN 3 02 20 ISSUED AS A FIGURE FOR MEMO REFERENCE ON 4.2.11
NO.	DATE	REVISION
KGS ACRES		KGS ACRES LTD MANITOBA HYDRO KEEYASK GENERATING STATION SPILLWAY STAGE I COFFERDAMS REMNANT EROSION ESTIMATED TSS LEVELS NELSON RIVER FLOW 3743 m3/S
SCALE: AS SHOWN		FIGURE 6 — SHEET 01 of 01 — REV. D1



NO.	DATE	REVISIONS
D1	2013 02 22	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-9.2.17.

KGS ACRES

KGS ACRES LTD

MANITOBA HYDRO

KEEYASK GENERATING STATION

COFFERDAM
EROSION DURING CONSTRUCTION
ESTIMATED TSS INCREASES
VERSUS TIME

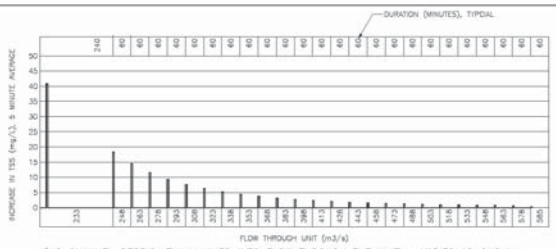
SCALE: AS SHOWN

FIGURE 7

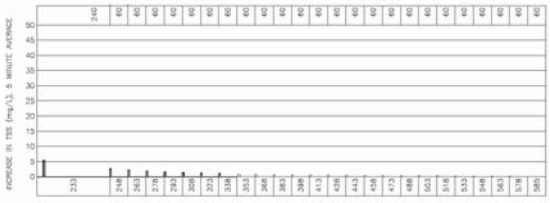
SHEET: 01 of 01

REV: D1

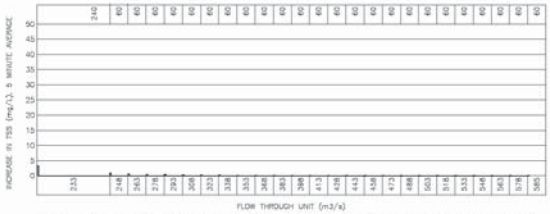
FOR INFORMATION USE ONLY



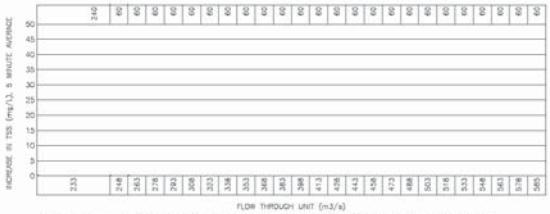
(a.) CHANNEL DEBRIS, FULLY MIXED INTO FLOW THROUGH THE UNIT = VARIES AS SHOWN



(b.) CHANNEL DEBRIS, FULLY MIXED INTO TOTAL RIVER FLOW = 1787 m3/s



(c.) SEDIMENT RE-SUSPENSION, FULLY MIXED INTO FLOW THROUGH UNIT = VARIES AS SHOWN



(d.) SEDIMENT RE-SUSPENSION, FULLY MIXED INTO TOTAL RIVER FLOW = 1787 m3/s

NOTES:

1. RESERVOIR ELEVATION FSL 159.0 m.
2. TOTAL RIVER FLOW 1787 m3/s, WHICH IS THE 1:20 YEAR FLOW FOR OCTOBER.
3. ONLY SEDIMENT DEPOSITED DURING IMPOUNDMENT ERODES. EROSION OF THE ORIGINAL PEAT AND MINERAL SOILS DOES NOT OCCUR.
4. DEPOSITED SEDIMENT IS CALCULATED AS THE DEPTH OF WATER MULTIPLIED BY THE INITIAL TSS IN THE WATER.
5. INITIAL TSS IN THE WATER ASSUMED TO BE 20 mg/L.
6. THE DURATION OF THE SPIKES IN TSS LEVELS HAVE BEEN AVERAGED OVER 5 MINUTE PERIODS.
7. FLOW CHANGES ARE INSTANTANEOUS.
8. THE VELOCITY OF EXPANSION OF THE AREA AFFECTED BY CHANGES IN FLOW = (GRAVITY * DEPTH)^(1/2).
9. CHANNEL DEBRIS/DEPOSITED SEDIMENT ERODE INSTANTANEOUSLY AND COMPLETELY IN ANY AREA WHERE THE BED SHEAR IS >= THE CRITICAL SHEAR = 0.3 Pa.
10. ERODIBLE CHANNEL DEBRIS ASSUMED TO BE 5 mm THICK.

NO.	DATE	REVISION
01	2013 03 22	ISSUED AS A FIGURE FOR MDMS REFERENCE EN-8.2.17

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KEEYASK GENERATING STATION

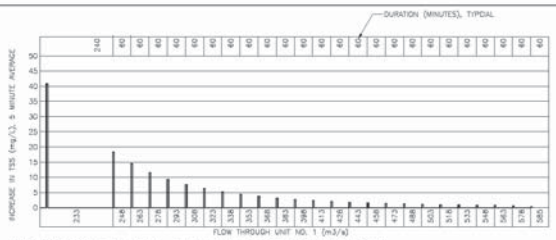
POWERHOUSE
ESTIMATED TSS INCREASES
DURING TESTING OF UNIT NO. 1

SCALE: AS SHOWN

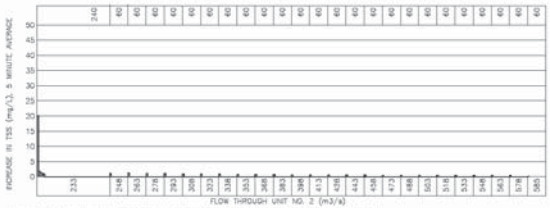
FIGURE 8

SHEET: 01 of 01

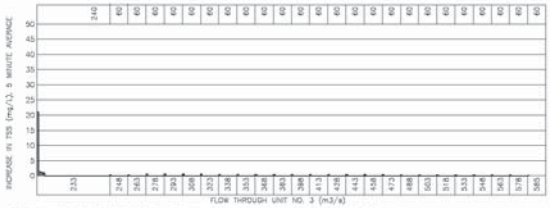
REV: 01



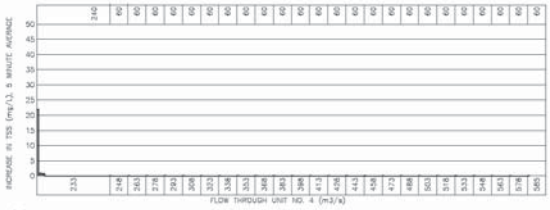
(a.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = THE FLOW THROUGH UNIT NO. 1



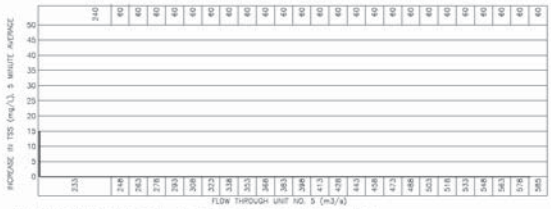
(b.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 585 m3/s PLUS THE FLOW THROUGH UNIT NO. 2



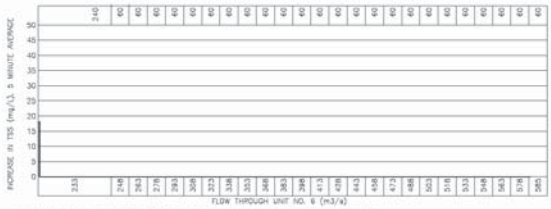
(c.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 1170 m3/s PLUS THE FLOW THROUGH UNIT NO. 3



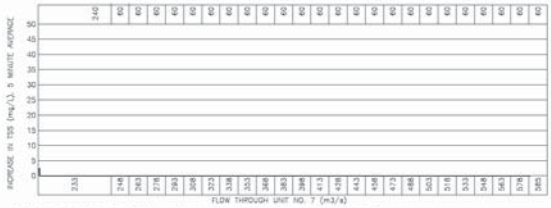
(d.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 1755 m3/s PLUS THE FLOW THROUGH UNIT NO. 4



(e.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 2340 m3/s PLUS THE FLOW THROUGH UNIT NO. 5



(f.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 2925 m3/s PLUS THE FLOW THROUGH UNIT NO. 6



(g.) CHANNEL DEBRIS SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 3510 m3/s PLUS THE FLOW THROUGH UNIT NO. 7

NOTES:

1. RESERVOIR ELEVATION FSL 159.0 m.
2. THE DURATION OF THE SPIKES IN TSS LEVELS HAVE BEEN AVERAGED OVER 5 MINUTE PERIODS.
3. FLOW CHANGES ARE INSTANTANEOUS.
4. THE VELOCITY OF EXPANSION OF THE AREA AFFECTED BY CHANGES IN FLOW = (GRAVITY * DEPTH)^0.5.
5. SEDIMENT ERODES INSTANTANEOUSLY AND COMPLETELY IN ANY AREA WHERE THE BED SHEAR IS >= THE CRITICAL SHEAR = 0.2 Pa.
6. ERODIBLE CHANNEL DEBRIS ASSUMED TO BE 5 mm THICK.

BY	DATE	REVISION
01	2013 03 22	ISSUED AS A FIGURE FOR MDMS REFERENCE EN-8.2.17

KGS ACRES LTD.
MANITOBA HYDRO
KEEYASK GENERATING STATION

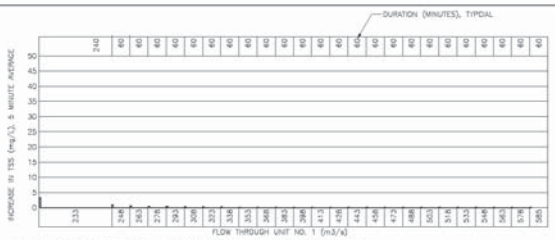
POWERHOUSE
ESTIMATED TSS INCREASES DUE TO
CHANNEL DEBRIS SUSPENSION
DURING TESTING OF UNITS

SCALE: AS SHOWN

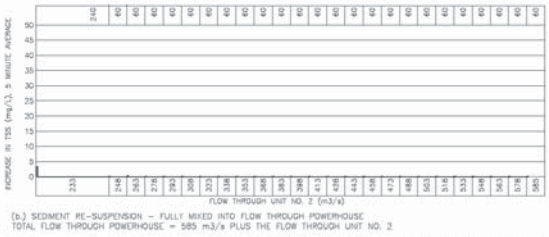
FIGURE 9

SHEET: 01 of 01

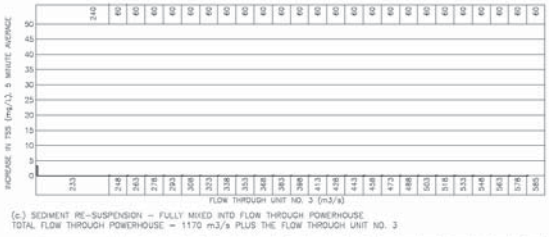
REV: 01



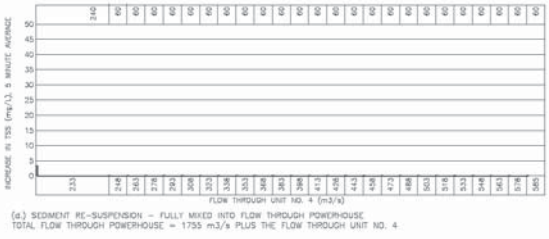
(a.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = THE FLOW THROUGH UNIT NO. 1



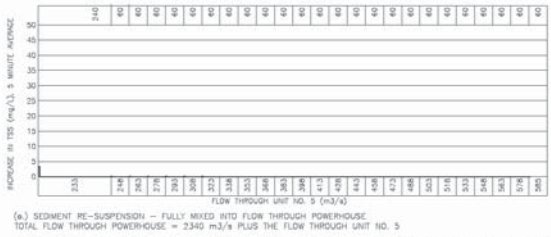
(b.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 585 m3/s PLUS THE FLOW THROUGH UNIT NO. 2



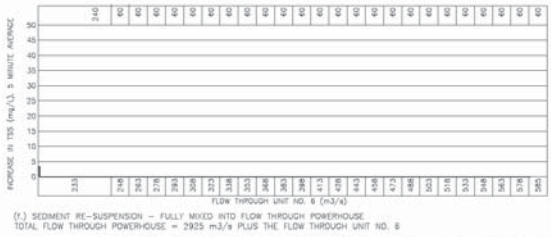
(c.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 1170 m3/s PLUS THE FLOW THROUGH UNIT NO. 3



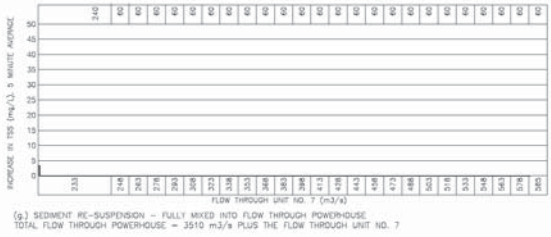
(d.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 1755 m3/s PLUS THE FLOW THROUGH UNIT NO. 4



(e.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 2340 m3/s PLUS THE FLOW THROUGH UNIT NO. 5



(f.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 2925 m3/s PLUS THE FLOW THROUGH UNIT NO. 6



(g.) SEDIMENT RE-SUSPENSION – FULLY MIXED INTO FLOW THROUGH POWERHOUSE
TOTAL FLOW THROUGH POWERHOUSE = 3510 m3/s PLUS THE FLOW THROUGH UNIT NO. 7

NOTES:

1. RESERVOIR ELEVATION FSL 159.0 m.
2. ONLY SEDIMENT DEPOSITED DURING IMPOUNDMENT ERODES. EROSION OF THE ORIGINAL PEAT AND MINERAL SOILS DOES NOT OCCUR.
3. DEPOSITED SEDIMENT IS CALCULATED AS THE DEPTH OF WATER MULTIPLIED BY THE INITIAL TSS IN THE WATER.
4. INITIAL TSS IN THE WATER ASSUMED TO BE 20 mg/L.
5. THE DURATION OF THE SPIKES IN TSS LEVELS HAVE BEEN AVERAGED OVER 5 MINUTE PERIODS.
6. FLOW CHANGES ARE INSTANTANEOUS.
7. THE VELOCITY OF EXPANSION OF THE AREA AFFECTED BY CHANGES IN FLOW = (GRAVITY * DEPTH)^(1/2).
8. DEPOSITED SEDIMENT ERODES INSTANTANEOUSLY AND COMPLETELY IN ANY AREA WHERE THE BED SHEAR IS >= THE CRITICAL SHEAR = 0.2 Pa.

BY	YYY.MM.DD	REVISION
00	2013.02.22	ISSUED AS A FIGURE FOR MDMS REFERENCE EN-8.2.17

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KEEYASK GENERATING STATION

POWERHOUSE
ESTIMATED TSS INCREASES DUE TO
SEDIMENT RE-SUSPENSION
DURING TESTING OF UNITS

SCALE: AS SHOWN

FIGURE 10

SHEET: 01 of 01

REV: 01

TABLES

Table 1
Keeyask Generating Station, Stage IV Studies – Physical Environment
GN-9.2.17, Rev 3 – Cofferdam Erosion During Construction
Calculation of Loss Factors

Construction Stage	Structure	Material Class	Comments	Exposure Case	Material Factor (MF), %[3]	Exposure Factor (EF), %[4]	Reduction Factor (RF), %[2]	Loss Factor (LF), %[5]
Stage I	Quarry Cofferdam	A	Initial Class C closure followed by Class A seal.	Partially Exposed – Condition I[1]	30	33	3.3	0.1
		C		Fully exposed	0.5	100	100	0.5
	North Channel Rock Groin	B	Initial Class C closure followed by Class B	Partially Exposed – Condition I[1]	10	33	5	0.05
		C		Fully exposed	0.5	100	100	0.5
	North Channel Stage I Cofferdam	A	Initial Class C closure followed by Class B and Class A.	Condition I[1]	30	33	3.3	0.1
		B			10	33	5	0.05
		C		Fully exposed	0.5	100	100	0.5
	Powerhouse Stage I Cofferdam	A	Protective Class C groin. Assumed erosion within enclosure due to groin seepage.	Condition III	30	10	3.3	0.33
		B			10	10	5	0.17
		C		Fully exposed	0.5	100	100	0.5
		C Groin			0.5	100	100	0.5
	Spillway Stage I Cofferdam	A	Protective Class C groin. Assumed erosion within enclosure due to groin seepage.	Condition III	30	10	3.3	0.33
		B			10	10	5	0.17
		C		Fully exposed	0.5	100	100	0.5
		C Groin			0.5	100	100	0.5
	Central Dam Stage I Cofferdam	A	Initial Class C closure followed by Class B and Class A. Construct Class C groin to prevent further erosion.	Condition I[1]	30	33	3.3	0.1
		B			10	33	5	0.05
		C		Fully exposed	0.5	100	100	0.5
		C Groin			0.5	100	100	0.5
	Tailrace Summer Level Cofferdam	A	Protective Class C groin. Assumed erosion within enclosure due to groin seepage.	Condition III	30	10	3.3	0.33
		B			10	10	5	0.17
		C		Fully exposed	0.5	100	100	0.5
		C Groin			0.5	100	100	0.5
Stage II	South Dam Stage II Upstream Cofferdam	A	Initial Class C closure followed by Class B and Class A.	Condition I[1]	30	33	3.3	0.1
		B			10	33	5	0.1
		C		Fully exposed	0.5	100	100	0.5
Removal	Powerhouse Stage I Cofferdam	A	Class A material excavated within protective Class C groin. Assumed erosion within enclosure due to groin seepage. Remaining excavation by backhoe/dragline.	Condition III	30	33	3.3	0.33
		B		Fully exposed	10	33	100	3.3
		C			0.5	100	100	0.5
	Spillway Stage I Cofferdam	A	Excavation by backhoe/dragline.	Fully exposed	30	33	100	9.9
		B			10	33	100	3.3
		C			0.5	100	100	0.5
	TRC Summer Level Cofferdam	A Inside Groin	Some Class A material excavated within protective Class C groin. Assumed erosion within enclosure due to groin seepage. Remaining excavation by backhoe/dragline.	Condition III	30	33	3.3	0.33
		A Outside Groin		Fully exposed	30	33	100	9.9
		B			10	33	100	3.3
		C			0.5	100	100	0.5

Notes:
(1) Condition I assumes a single Class C rockfill groin having been advanced completely prior to placement of the Class A or B materials.
(2) Reduction Factor, (RF), which represents the fraction of materials being assumed to be contributing to generating TSS.
(3) Material Factor, (MF), which represents the fraction of materials being placed that is comprised of fines which are susceptible to becoming entrained into the water during the interval while it is directly exposed to flow.
(4) Exposure Factor, (EF), which is the proportion of time that the material will actually be exposed to direct erosion by flowing water.
(5) Loss Factor, (LF), which represents the fraction of materials assumed lost in the flow as suspended material = MF x EF x RF.

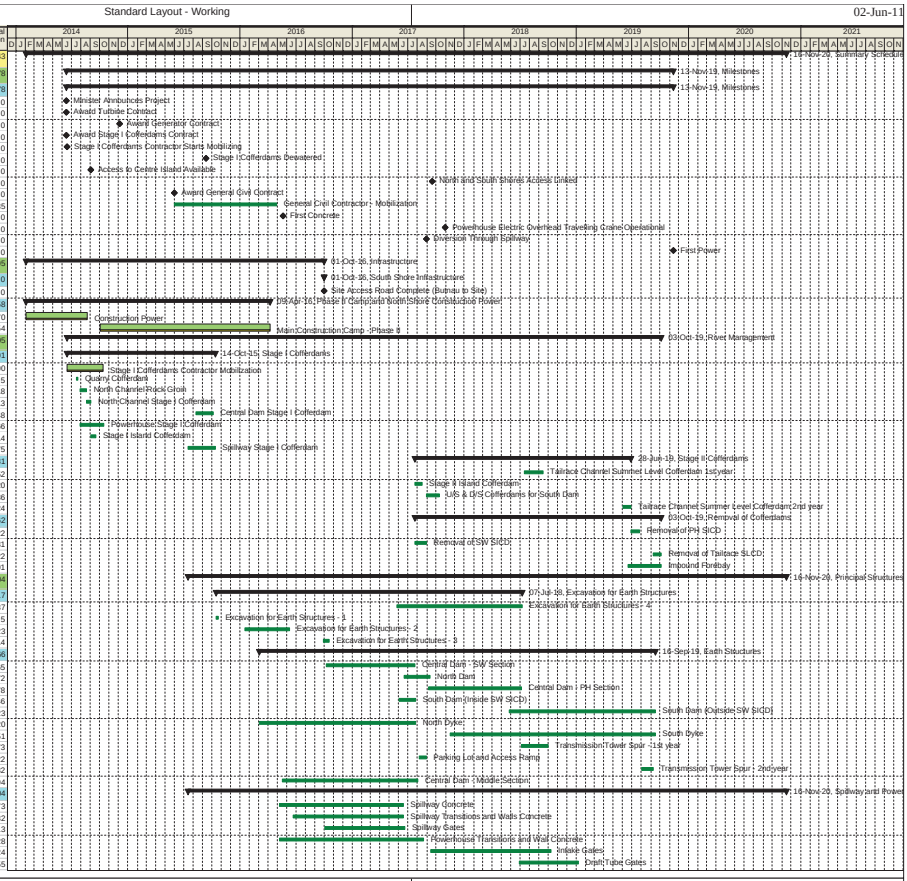
Table 2
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17, Rev 3 – Cofferdam Erosion During Construction
Nelson River Flows
1:20 Statistical Daily Flows and Mean Monthly Flow[(1)]

	1:20 Low Daily Flow in Month m^3/s	Mean Monthly Flow m^3/s	1:20 Peak Daily Flow in Month m^3/s
January	2501	3271	4080
February	2284	3069	3799
March	1999	2906	3684
April	1675	2932	4505
May	1628	3125	5846
June	1860	3410	6003
July	1835	3508	5922
August	1769	3262	4708
September	1601	2898	4341
October	1787	3041	4745
November	2096	3142	4370
December	2285	3285	4292

Note:
(1) See Reference 8.

Summary Schedule | Standard Layout - Working | 02-Jun-11

Activity ID	Activity Type	Activity Name	Start	Finish	Original Duration
Summary Schedule			03-Feb-14	16-Nov-20	2043
Milestones			15-Jun-14	13-Nov-19	1978
Milestones			15-Jun-14	13-Nov-19	1978
S-03	Finish Milestone	Minister Announces Project		15-Jun-14	0
S-04	Finish Milestone	Award Turbine Contract		15-Jun-14	0
S-05	Finish Milestone	Award Generator Contract		05-Dec-14	0
S-08	Finish Milestone	Award Stage I Cofferdams Contract		15-Jun-14	0
S-09	Finish Milestone	Stage I Cofferdams Contractor Starts Mobilizing		16-Jun-14	0
S-10	Finish Milestone	Stage I Cofferdams Dewatered		12-Sep-15	0
S-11	Finish Milestone	Access to Centre Island Available		03-Sep-14	0
S-12	Finish Milestone	North and South Shores Access Linked		19-Sep-17	0
S-13	Finish Milestone	Award General Civil Contract		31-May-15	0
S-14	Level of Effort	General Civil Contractor - Mobilization	01-Jun-15	30-Apr-16	335
S-15	Finish Milestone	First Concrete		21-May-16	0
S-16	Finish Milestone	Powerhouse Electric Overhead Travelling Crane Operational		31-Oct-17	0
S-17	Finish Milestone	Diversion Through Spillway		29-Aug-17	0
S-18	Finish Milestone	First Power		13-Nov-19	0
Infrastructure			03-Feb-14	01-Oct-16	805
South Shore Infrastructure			01-Oct-16	01-Oct-16	0
S-23	Finish Milestone	Site Access Road Complete (Butnau to Site)		01-Oct-16	0
Phase II Camp and North Shore Construction Power			03-Feb-14	09-Apr-16	658
S-104	Task Dependent	Construction Power	03-Feb-14	21-Aug-14	170
S-24	Task Dependent	Main Construction Camp - Phase II	02-Oct-14	09-Apr-16	454
River Management			16-Jun-14	03-Oct-19	1595
Stage I Cofferdams			16-Jun-14	14-Oct-15	401
S-32	Task Dependent	Stage I Cofferdams Contractor Mobilization	16-Jun-14	11-Oct-14	100
S-34	Level of Effort	Quarry Cofferdam	17-Jul-14	22-Jul-14	5
S-37	Level of Effort	North Channel Rock Groin	28-Jul-14	18-Aug-14	18
S-38	Level of Effort	North Channel Stage I Cofferdam	18-Aug-14	03-Sep-14	13
S-39	Level of Effort	Central Dam Stage I Cofferdam	10-Aug-15	06-Oct-15	48
S-40	Level of Effort	Powerhouse Stage I Cofferdam	28-Jul-14	14-Oct-14	66
S-41	Level of Effort	Stage I Island Cofferdam	03-Sep-14	18-Sep-14	14
S-42	Level of Effort	Spillway Stage I Cofferdam	16-Jul-15	14-Oct-15	75
Stage II Cofferdams			25-Jul-17	28-Jun-19	581
S-44	Level of Effort	Tailrace Channel Summer Level Cofferdam 1st year	16-Jul-18	14-Sep-18	52
S-45	Level of Effort	Stage II Island Cofferdam	25-Jul-17	16-Aug-17	20
S-46	Level of Effort	U/S & D/S Cofferdams for South Dam	30-Aug-17	12-Oct-17	36
S-54	Level of Effort	Tailrace Channel Summer Level Cofferdam 2nd year	01-Jun-19	28-Jun-19	24
Removal of Cofferdams			25-Jul-17	03-Oct-19	662
S-48	Level of Effort	Removal of PH SICD	29-Jun-19	25-Jul-19	22
S-50	Level of Effort	Removal of SW SICD	25-Jul-17	29-Aug-17	31
S-52	Level of Effort	Removal of Tailrace SLCD	09-Sep-19	03-Oct-19	22
S-53	Level of Effort	Impound Forebay	18-Jun-19	03-Oct-19	91
Principal Structures			16-Jul-15	16-Nov-20	1604
Excavation for Earth Structures			16-Oct-15	07-Jul-18	817
S-106	Level of Effort	Excavation for Earth Structures - 4	26-May-17	07-Jul-18	337
S-56	Level of Effort	Excavation for Earth Structures - 1	16-Oct-15	21-Oct-15	5
S-66	Level of Effort	Excavation for Earth Structures - 2	18-Jan-16	10-Jun-16	123
S-96	Level of Effort	Excavation for Earth Structures - 3	30-Sep-16	17-Oct-16	14
Earth Structures			04-Mar-16	16-Sep-19	1066
S-108	Level of Effort	Central Dam - SW Section	08-Oct-16	24-Jul-17	65
S-57	Level of Effort	North Dam	19-Jun-17	12-Sep-17	72
S-58	Level of Effort	Central Dam - PH Section	06-Sep-17	06-Jul-18	78
S-60	Level of Effort	South Dam (Inside SW SICD)	02-Jun-17	27-Jul-17	46
S-61	Level of Effort	South Dam (Outside SW SICD)	28-May-18	16-Sep-19	223
S-62	Level of Effort	North Dyke	04-Mar-16	25-Jul-17	420
S-63	Level of Effort	South Dyke	16-Nov-17	16-Sep-19	551
S-64	Level of Effort	Transmission Tower Spur - 1st year	07-Jul-18	01-Oct-18	73
S-65	Level of Effort	Parking Lot and Access Ramp	07-Aug-17	31-Aug-17	22
S-94	Level of Effort	Transmission Tower Spur - 2nd year	01-Aug-19	07-Sep-19	32
S-98	Level of Effort	Central Dam - Middle Section	18-May-16	03-Aug-17	194
Spillway and Powerhouse			16-Jul-15	16-Nov-20	1604
S-68	Level of Effort	Spillway Concrete	09-May-16	17-Jun-17	173
S-71	Level of Effort	Spillway Transitions and Walls Concrete	23-Jun-16	15-Jun-17	132
S-72	Level of Effort	Spillway Gates	04-Oct-16	21-Jun-17	213
S-74	Level of Effort	Powerhouse Transitions and Wall Concrete	09-May-16	21-Aug-17	228
S-75	Level of Effort	Intake Gates	13-Sep-17	10-Oct-18	324
S-76	Level of Effort	Draft Tube Gates	29-Jun-18	07-Jan-19	155



Remaining Level of Effort | Primary Baseline | Remaining Work
Actual Level of Effort | Actual Work | Critical Remaining Work

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TASK filter: All Activities

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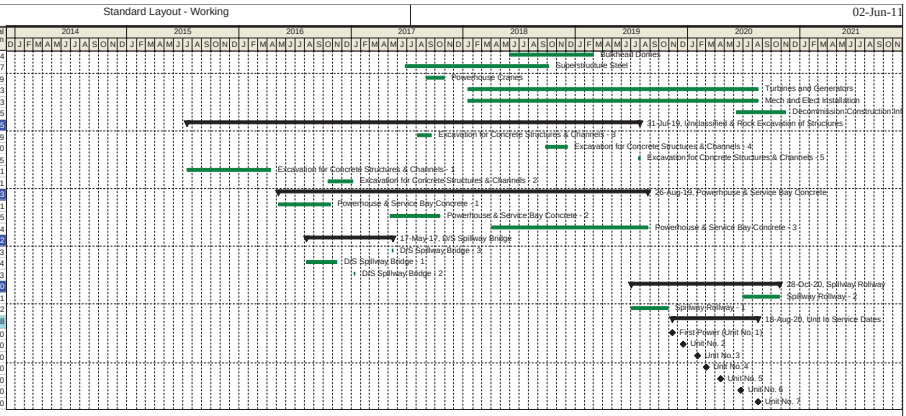
Summary Schedule | Standard Layout - Working | 02-Jun-11

Activity ID	Activity Type	Activity Name	Start	Finish	Original Duration
S-77	Level of Effort	Bulkhead Domes	01-Jun-18	01-Mar-19	224
S-78	Level of Effort	Superstructure Steel	27-Jun-17	06-Oct-18	387
S-79	Level of Effort	Powerhouse Cranes	02-Sep-17	31-Oct-17	49
S-80	Level of Effort	Turbines and Generators	16-Jan-18	18-Aug-20	783
S-81	Level of Effort	Mech and Elect Installation	16-Jan-18	18-Aug-20	783
S-82	Level of Effort	Decommission Construction Infrastructure	06-Jun-20	16-Nov-20	135
Unclassified & Rock Excavation of Structures			16-Jul-15	31-Jul-19	1215
S-107	Level of Effort	Excavation for Concrete Structures & Channels - 3	04-Aug-17	19-Sep-17	39
S-117	Level of Effort	Excavation for Concrete Structures & Channels - 4	27-Sep-18	06-Dec-18	60
S-127	Level of Effort	Excavation for Concrete Structures & Channels - 5	26-Jul-19	31-Jul-19	5
S-67	Level of Effort	Excavation for Concrete Structures & Channels - 1	16-Jul-15	12-Apr-16	221
S-97	Level of Effort	Excavation for Concrete Structures & Channels - 2	17-Oct-16	06-Jan-17	61
Powerhouse & Service Bay Concrete			09-May-16	26-Aug-19	993
S-73	Level of Effort	Powerhouse & Service Bay Concrete - 1	09-May-16	24-Oct-16	141
S-83	Level of Effort	Powerhouse & Service Bay Concrete - 2	08-May-17	16-Oct-17	135
S-93	Level of Effort	Powerhouse & Service Bay Concrete - 3	03-Apr-18	26-Aug-19	424
D/S Spillway Bridge			08-Aug-16	17-May-17	232
S-109	Level of Effort	D/S Spillway Bridge - 3	15-May-17	17-May-17	3
S-69	Level of Effort	D/S Spillway Bridge - 1	08-Aug-16	15-Nov-16	84
S-99	Level of Effort	D/S Spillway Bridge - 2	10-Jan-17	12-Jan-17	3
Spillway Rollway			02-Jul-19	28-Oct-20	400
S-100	Level of Effort	Spillway Rollway - 2	30-Jun-20	28-Oct-20	101
S-70	Level of Effort	Spillway Rollway - 1	02-Jul-19	30-Oct-19	102
Unit In Service Dates			13-Nov-19	18-Aug-20	278
S-84	Finish Milestone	First Power (Unit No. 1)		13-Nov-19	0
S-85	Finish Milestone	Unit No. 2		18-Dec-19	0
S-86	Finish Milestone	Unit No. 3		03-Feb-20	0
S-87	Finish Milestone	Unit No. 4		03-Mar-20	0
S-88	Finish Milestone	Unit No. 5		18-Apr-20	0
S-89	Finish Milestone	Unit No. 6		22-Jun-20	0
S-90	Finish Milestone	Unit No. 7		18-Aug-20	0



Remaining Level of Effort | Primary Baseline | Remaining Work
Actual Level of Effort | Actual Work | Critical Remaining Work

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TASK filter: All Activities

(c) Primavera Systems, Inc.

GN-9.2.17 Rev 3
Table 3

Table 4
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17, Rev 3 – Cofferdam Erosion During Construction
Assumed Water Levels During Construction

Construction Stage	Structure	Assumed Construction Water Level Elevation (m)	Comments
Stage I	Quarry Cofferdam	147.5	Ref. 1
	North Channel Rock Groin	155.0	Assumed 1 m below crest Shown in Ref. 1
	North Channel Stage I Cofferdam	152.2	Ref. 1
	Powerhouse Stage I Cofferdam	144.5	Assumed 1 m below crest Shown in Ref. 2
	Spillway Stage I Cofferdam	Varies between el. 150 m and el. 147 m	Assumed 1 m below crest Shown in Ref.1
	Central Dam Stage I Cofferdam	Average 142.7 (Assumed lower 0.5 m of dam submerged)	Ref.1
	Tailrace Summer Level Cofferdam	141.5	Assumed 1 m below crest Level shown in Ref. 2
Stage II	South Dam Upstream Stage II Cofferdam	150.9	Ref. 1
Removal	Powerhouse Stage I Cofferdam	144.5	Assumed 1 m below crest Shown in Ref. 2
	Spillway Stage I Cofferdam	Varies between el. 150 m and el. 147 m	Assumed 1 m below crest Shown in Ref.1
	Tailrace Summer Level Cofferdam	141.5	Assumed 1 m below crest Level in Ref. 2

Table 5
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17 Rev 3 – Cofferdam Erosion During Construction
North Channel Rock Groin Estimated Flows and Energy Grade Line Levels

Return Period[3]	Total Flow (m^3/s)	Existing Conditions			After Closure of Middle Channel by NCRG (m^3/s)		
		Flow in Middle Channel (m^3/s)	Flow in South Channel (m^3/s)	EGL Elevation at NCRG (m) [1]	Flow in Middle Channel (m^3/s)	Flow in South Channel (m^3/s)	EGL Elevation at NCRG (m) [2]
Q05	1835	116	1719	150.5	0	1835	151.9
Qm	3508	725	2783	152.3	0	3508	153.2
Q95	5922	974	4948	152.8	0	5922	154.6

Notes:

(1) Energy grade line elevation upstream of the North Channel Rock Groin (NCRG) is based on the rating curve for gauge SGNL12.

(2) Energy grade line (EGL) with the Middle Channel closed by the NCRG assumed to be equal to the EGL elevation at gauge SGNL20 with the total flow going through the South channel.

(3) Q05 = Low Flow Conditions = 1:20 Low Daily > Flow in month(s) during which the rock groin is constructed.

Qm = Average Flow Conditions = mean monthly flow.

Q95= Peak Flow Conditions = 1:20 peak daily flow in month(s) during which the rock groin is constructed.

Table 6
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17 Rev 3 – Cofferdam Erosion During Construction
North Channel Rock Groin Potential Sources Contributing to Increases in TSS

Potential Sources		
Fill Materials	**Riverbed**	**Abutments**
The loss of the finer fractions of the fill materials during construction of the cofferdam will contribute to increases in the TSS.	In general, erosion of the river bed will be minimal since the area is mostly bedrock controlled. There is a potential for some erosion as the river channel becomes more constricted by the construction of the rock groin. TSS increases due to riverbed erosion were not included in this study.	Erosion of the south abutment during closure will depend on how resistant the top clay layer is to erosion. It has been assumed that erosion protection measures will be put in place to prevent erosion of the abutment.

Table 7
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17 Rev 3 – Cofferdam Erosion During Construction
North Channel Stage I Cofferdam Estimated Flows and Energy Grade Line Levels

Return Period[3]	Total Flow (m^3/s)	Existing Conditions			After Closure of Middle Channel by NCS1CD (m^3/s)		
		Flow in Middle Channel (m^3/s)	Flow in South Channel (m^3/s)	EGL Elevation at NCS1CD (m) [1]	Flow in Middle Channel (m^3/s)	Flow in South Channel (m^3/s)	EGL Elevation at NCS1CD (m) [2]
Q05	1835	116	1719	147.7	0	1835	147.5
Qm	3508	725	2783	149.1	0	3508	148.7
Q95	5922	974	4948	149.5	0	5922	150.0

Notes:
(1) Energy grade line elevation upstream of the North Channel Stage I Cofferdam (NCS1CD) is based on the rating curve for gauge SGNL10.
(2) Energy grade line (EGL) with the Middle Channel closed by the NCS1CD assumed to be equal to the EGL elevation at gauge SGNL19D with the total flow going through the South Channel.
(3) Q05 = Low Flow Conditions = 1:20 low daily flow in month(s) during which the Cofferdam is constructed.
Qm = Average Flow Conditions = mean monthly flow.
Q95 = Peak Flow Conditions = 1:20 peak daily flow in month(s) during which the Cofferdam is constructed.

Table 8
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17 Rev 3 – Cofferdam Erosion During Construction
North Channel Stage I Cofferdam, Potential Sources Contributing to Increases in TSS

Potential Sources		
Fill Materials	**Riverbed**	**Abutments**
The loss of the finer fractions of the fill materials during construction of the cofferdam will contribute to increases in the TSS.	Erosion of the river bed will be minimal and should not contribute to increases in the TSS since the river bed has experienced high flows in the past and all of the finer material has been already eroded.	Erosion of the south abutment during closure is expected to be small to minor depending on the precise elevation to which the overburden has already been eroded by the previously experienced high flows in the summer of 2005.

Table 9.0
Keeyask Generating Station
Stage IV Studies - Physical Environment
GN-9.2.17, Rev 3 - Cofferdam Erosion During Construction
Estimated Increases in TSS due to Cofferdam Fill Material Dispersion into River Flow During Cofferdam Construction and Removal
Low Flow Conditions

										Increase in TSS Above Background Levels								
										Low Flow								
										Hourly During Actual Construction[2]			Daily[3]			Weekly[4,5]		
Construction Stage	Structure	Material Placed	D_{50} range[1] mm	Assumed Month of Placement	Placement/ Removal Rate m^3/s	Duration (Days)	River Flow Rate Low Flow m^3/s	% of River Flow Rate Impacting Fill Site	Loss Factor	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l
Stage I	Quarry Cofferdam[6]	Class A	0.03 - 0.4	July	0.008	3	1835	1%	0.10%	2	0-1	0-1						
		Class C	40 - 300	July	0.008	3		10%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	North Channel Rock Groin[6]	Class B	1 - 9	August	0.078	3	1769	1%	0.05%	12	0-1	0-1	10	0-1	0-1			
		Class C	40 - 300	August	0.078	19		30%	0.50%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
	North Channel Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.086	8	1601	1%	0.10%	22	0-1	0-1	18	0-1	0-1			
		Class B	1 - 9	September	0.080	2		1%	0.05%	14	0-1	0-1	12	0-1	0-1			
		Class C	40 - 300	September	0.082	7		10%	0.50%	5	0-1	0-1	4	0-1	0-1	3	0-1	0-1
	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	September	0.012	41	1601	1%	0.33%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	September	0.011	8		1%	0.17%	1	0-1	0-1	0	0-1	0-1	0-1	0-1	0-1
		Class C	40 - 300	September	0.011	22		1%	0.50%	6	0-1	0-1	5	0-1	0-1	4	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.022	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.042	32	1601	1%	0.33%	3	0-1	0-1	3	0-1	0-1	2	0-1	0-1
		Class B	1 - 9	September	0.039	12		1%	0.17%	2	0-1	0-1	2	0-1	0-1	1	0-1	0-1
		Class C - not including rock groin	40 - 300	September	0.039	29		1%	0.50%	23	0-1	0-1	19	0-1	0-1	16	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.044	18		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Central Dam Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.007	21	1601	1%	0.10%	2	0-1	0-1	2	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	September	0.007	4		1%	0.05%	1	0-1	0-1	1	0-1	0-1			
		Class C	40 - 300	September	0.005	30		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.005	3		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	Tailrace Summer Level Cofferdam	Class A	0.03 - 0.4	September	0.090	17	1601	1%	0.33%	7	0-1	0-1	6	0-1	0-1	5	0-1	0-1
		Class B	1 - 9	September	0.087	6		1%	0.17%	5	0-1	0-1	4	0-1	0-1			
		Class C - not including rock groin	40 - 300	September	0.088	20		1%	0.50%	51	1	0-1	43	0-1	0-1	36	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.088	18		100%	0.50%	1	1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Stage II	South Dam Upstream Stage II Cofferdam[6]	Class A	0.03 - 0.4	September	0.078	33	1601	1%	0.10%	20	0-1	0-1	16	0-1	0-1	14	0-1	0-1
		Class B	1 - 9	September	0.078	12		1%	0.05%	14	0-1	0-1	11	0-1	0-1	10	0-1	0-1
		Class C	40 - 300	September	0.074	43		70%	0.50%	1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Removal	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	June/July	0.013		1835	1%	0.33%	3	0-1	0-1	2	0-1	0-1	2	0-1	0-1
		Class B	1 - 9	June/July	0.013	19		100%	3.30%	0-1	0-1	0-1	0-1	0-1	0-1			
		Class C	40 - 300	June/July	0.013	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.040		1601	100%	9.90%	3	3	0-1	2	2	2	2	2	2
		Class B	1 - 9	September	0.036	17		100%	3.30%	1	1	0-1	1	1	1	1	1	1
		Class C	40 - 300	September	0.038	19		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Tailrace Summer Level Cofferdam	Class A - inside rock groin	0.03 - 0.4	September	0.114	2	1601	1%	0.33%	28	0-1	0-1	24	0-1	0-1	20	0-1	0-1
		Class A - outside rock groin[10]		September	0.115	5		100%	9.90%	9	9	4	7	7	7	6	6	6
		Class B	1 - 9	September	0.111	3		100%	3.30%	4	4	0-1	3	3	3			
		Class C	40 - 300	September	0.112	15		100%	0.50%	1	1	0-1	1	1	1	0-1	0-1	0-1

Notes:
(1) Range for Class C includes only Class C1 materials as majority of rockfill used is Class C1.
(2) Estimated increase in TSS during actual working operation.
(3) Average increase in TSS, on a daily basis, based on work day shifts of 20 hours.
(4) Average increase in TSS, on a weekly basis, based on word day shifts of 20 hours and a 6 day work week.
(5) Fields are blank where placement of material takes less than one week to complete.
(6) TSS calculation assumes that Class C rockfill base will be placed before Class A or B material placement begins.
(7) "Local Flow" represents temporarily elevated TSS levels resulting from cofferdam activities that would occur in partial flow channels and "partially exposed" cases described in Section 5, which would subsequently become fully mixed when they re-enter the main stream.
(8) "Full River Flow" represents the TSS levels resulting from cofferdam activities in the main stream directly adjacent to the cofferdams.
(9) "Monitor Tu-K-2" represents the TSS levels downstream of the Keeyask GS resulting from cofferdam activities. This monitoring station is shown on Figure 2.
(10) Use of a silt curtain brings the grey shaded values to 0-1 mg/l.

Table 9.1
Keeyask Generating Station
Stage IV Studies - Physical Environment
GN-9.2.17, Rev 3 - Cofferdam Erosion During Construction
Estimated Increases in TSS due to Cofferdam Fill Material Dispersion into River Flow During Cofferdam Construction and Removal
Average Flow Conditions

										Increase in TSS Above Background Levels								
										Average Flow								
										Hourly During Actual Construction[2]			Daily[3]			Weekly[4,5]		
Construction Stage	Structure	Material Placed	D_{50} range[1] mm	Assumed Month of Placement	Placement Rate m^3/s	Duration (Days)	River Flow Rate Average Flow m^3/s	% of River Flow Rate Impacting Fill Site	Loss Factor	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l
Stage I	Quarry Cofferdam[6]	Class A	0.03 - 0.4	July	0.008	3	3508	1%	0.10%	1	0-1	0-1						
		Class C	40 - 300	July	0.008	3		10%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	North Channel Rock Groin[6]	Class B	1 - 9	August	0.078	3	3262	1%	0.05%	7	0-1	0-1	6	0-1	0-1			
		Class C	40 - 300	August	0.078	19		30%	0.50%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
	North Channel Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.086	8	2898	1%	0.10%	12	0-1	0-1	10	0-1	0-1			
		Class B	1 - 9	September	0.080	2		1%	0.05%	8	0-1	0-1	6	0-1	0-1			
		Class C	40 - 300	September	0.082	7		10%	0.50%	3	0-1	0-1	2	0-1	0-1	2	0-1	0-1
	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	September	0.012	41	2898	1%	0.33%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class B	1 - 9	September	0.011	8		1%	0.17%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C	40 - 300	September	0.011	22		1%	0.50%	3	0-1	0-1	3	0-1	0-1	2	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.022	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.042	32	2898	1%	0.33%	2	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	September	0.039	12		1%	0.17%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class C - not including rock groin	40 - 300	September	0.039	29		1%	0.50%	12	0-1	0-1	10	0-1	0-1	9	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.044	18		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Central Dam Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.007	21	2898	1%	0.10%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	September	0.007	4		1%	0.05%	1	0-1	0-1	1	0-1	0-1			
		Class C	40 - 300	September	0.005	30		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.005	3		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	Tailrace Summer Level Cofferdam	Class A	0.03 - 0.4	September	0.090	17	2898	1%	0.33%	4	0-1	0-1	3	0-1	0-1	3	0-1	0-1
		Class B	1 - 9	September	0.087	6		1%	0.17%	3	0-1	0-1	2	0-1	0-1			
		Class C - not including rock groin	40 - 300	September	0.088	20		1%	0.50%	28	0-1	0-1	23	0-1	0-1	20	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.088	18		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Stage II	South Dam Upstream Stage II Cofferdam[6]	Class A	0.03 - 0.4	September	0.078	33	2898	1%	0.10%	11	0-1	0-1	9	0-1	0-1	8	0-1	0-1
		Class B	1 - 9	September	0.078	12		1%	0.05%	8	0-1	0-1	6	0-1	0-1	5	0-1	0-1
		Class C	40 - 300	September	0.074	43		70%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Removal	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	June/July	0.013		3410	1%	0.33%	2	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	June/July	0.013	19		100%	3.30%	0-1	0-1	0-1	0-1	0-1	0-1			
		Class C	40 - 300	June/July	0.013	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.040		2898	100%	9.90%	2	2	2	1	1	1	1	1	1
		Class B	1 - 9	September	0.036	17		100%	3.30%	1	1	1	1	1	1	1	1	1
		Class C	40 - 300	September	0.038	19		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Tailrace Summer Level Cofferdam	Class A - inside rock groin	0.03 - 0.4	September	0.114	2	2898	1%	0.33%	16	0-1	0-1	13	0-1	0-1	11	0-1	0-1
		Class A - outside rock groin[10]		September	0.115	5		100%	9.90%	5	5	5	4	4	4	3	3	3
		Class B	1 - 9	September	0.111	3		100%	3.30%	2	2	2	2	2	2			
		Class C	40 - 300	September	0.112	15		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1

Notes:
(1) Range for Class C includes only Class C1 materials as majority of rockfill used is Class C1
(2) Estimated increase in TSS during actual working operation
(3) Average increase in TSS, on a daily basis, based on work day shifts of 20 hours
(4) Average increase in TSS, on a weekly basis, based on word day shifts of 20 hours and a 6 day work week
(5) Fields are blank where placement of material takes less than one week to complete
(6) TSS calculation assumes that Class C rockfill base will be placed before Class A or B material placement begins
(7) "Local Flow" represents temporarily elevated TSS levels resulting from cofferdam activities that would occur in partial flow channels and "partially exposed" cases described in Section 5, which would subsequently become fully mixed when they re-enter the main stream.
(8) "Full River Flow" represents the TSS levels resulting from cofferdam activities in the main stream directly adjacent to the cofferdams.
(9) "Monitor Tu-K-2" represents the TSS levels downstream of the Keeyask GS resulting from cofferdam activities. This monitoring station is shown on Figure 2.
(10) Use of a silt curtain brings the grey shaded values to 0-1 mg/l.

Table 9.2
Keeyask Generating Station
Stage IV Studies - Physical Environment
GN-9.2.17, Rev 3 - Cofferdam Erosion During Construction
Estimated Increases in TSS due to Cofferdam Fill Material Dispersion into River Flow During Cofferdam Construction and Removal
Peak Flow Conditions

										Increase in TSS Above Background Levels								
										Peak Flow								
										Hourly During Actual Construction[2]			Daily[3]			Weekly[4,5]		
Construction Stage	Structure	Material Placed	D_{50} range[1] mm	Assumed Month of Placement	Placement Rate m^3/s	Duration (Days)	River Flow Rate Peak Flow m^3/s	% of River Flow Rate Impacting Fill Site	Loss Factor	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l	Local Flow[7] mg/l	Full River Flow[8] mg/l	Monitor K-Tu-2[9] mg/l
Stage I	Quarry Cofferdam[6]	Class A	0.03 - 0.4	July	0.008	3	5922	1%	0.10%	1	0-1	0-1						
		Class C	40 - 300	July	0.008	3		10%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	North Channel Rock Groin[6]	Class B	1 - 9	August	0.078	3	4708	1%	0.05%	5	0-1	0-1	4	0-1	0-1			
		Class C	40 - 300	August	0.078	19		30%	0.50%	1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	North Channel Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.086	8	4341	1%	0.10%	8	0-1	0-1	7	0-1	0-1			
		Class B	1 - 9	September	0.080	2		1%	0.05%	5	0-1	0-1	4	0-1	0-1			
		Class C	40 - 300	September	0.082	7		10%	0.50%	2	0-1	0-1	1	0-1	0-1	1	0-1	0-1
	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	September	0.012	41	4341	1%	0.33%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class B	1 - 9	September	0.011	8		1%	0.17%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C	40 - 300	September	0.011	22		1%	0.50%	2	0-1	0-1	2	0-1	0-1	2	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.022	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.042	32	4341	1%	0.33%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	September	0.039	12		1%	0.17%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class C - not including rock groin	40 - 300	September	0.039	29		1%	0.50%	8	0-1	0-1	7	0-1	0-1	6	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.044	18		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Central Dam Stage I Cofferdam[6]	Class A	0.03 - 0.4	September	0.007	21	4341	1%	0.10%	1	0-1	0-1	1	0-1	0-1	0-1	0-1	0-1
		Class B	1 - 9	September	0.007	4		1%	0.05%	0-1	0-1	0-1	0-1	0-1	0-1			
		Class C	40 - 300	September	0.005	30		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.005	3		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1			
	Tailrace Summer Level Cofferdam	Class A	0.03 - 0.4	September	0.090	17	4341	1%	0.33%	3	0-1	0-1	2	0-1	0-1	2	0-1	0-1
		Class B	1 - 9	September	0.087	6		1%	0.17%	2	0-1	0-1	1	0-1	0-1			
		Class C - not including rock groin	40 - 300	September	0.088	20		1%	0.50%	19	0-1	0-1	16	0-1	0-1	13	0-1	0-1
		Class C - rock groin only	40 - 300	September	0.088	18		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Stage II	South Dam Upstream Stage II Cofferdam[6]	Class A	0.03 - 0.4	September	0.078	33	4341	1%	0.10%	7	0-1	0-1	6	0-1	0-1	5	0-1	0-1
		Class B	1 - 9	September	0.078	12		1%	0.05%	5	0-1	0-1	4	0-1	0-1	4	0-1	0-1
		Class C	40 - 300	September	0.074	43		70%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
Removal	Powerhouse Stage I Cofferdam	Class A	0.03 - 0.4	June/July	0.013		5922	1%	0.33%	1	0-1	0-1	1	0-1	0-1	1	0-1	0-1
		Class B	1 - 9	June/July	0.013	19		100%	3.30%	0-1	0-1	0-1	0-1	0-1	0-1			
		Class C	40 - 300	June/July	0.013	8		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Spillway Stage I Cofferdam	Class A	0.03 - 0.4	September	0.040		4341	100%	9.90%	1	1	1	1	1	1	1	1	1
		Class B	1 - 9	September	0.036	17		100%	3.30%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
		Class C	40 - 300	September	0.038	19		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1
	Tailrace Summer Level Cofferdam	Class A - inside rock groin	0.03 - 0.4	September	0.114	2	4341	1%	0.33%	10	0-1	0-1	9	0-1	0-1	7	0-1	0-1
		Class A - outside rock groin[10]		September	0.115	5		100%	9.90%	3	3	3	3	3	3	2	2	2
		Class B	1 - 9	September	0.111	3		100%	3.30%	1	1	1	1	1	1			
		Class C	40 - 300	September	0.112	15		100%	0.50%	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1	0-1

Notes:
(1) Range for Class C includes only Class C1 materials as majority of rockfill used is Class C1
(2) Estimated increase in TSS during actual working operation
(3) Average increase in TSS, on a daily basis, based on work day shifts of 20 hours
(4) Average increase in TSS, on a weekly basis, based on word day shifts of 20 hours and a 6 day work week
(5) Fields are blank where placement of material takes less than one week to complete
(6) TSS calculation assumes that Class C rockfill base will be placed before Class A or B material placement begins
(7) "Local Flow" represents temporarily elevated TSS levels resulting from cofferdam activities that would occur in partial flow channels and "partially exposed" cases described in Section 5, which would subsequently become fully mixed when they re-enter the main stream.
(8) "Full River Flow" represents the TSS levels resulting from cofferdam activities in the main stream directly adjacent to the cofferdams.
(9) "Monitor Tu-K-2" represents the TSS levels downstream of the Keeyask GS resulting from cofferdam activities. This monitoring station is shown on Figure 2.
(10) Use of a silt curtain brings the grey shaded values to 0-1 mg/l.

Table 10
Keeyask Generating Station
Stage IV Studies – Physical Environment
GN-9.2.17 Rev 3 – Cofferdam Erosion During Construction
Spillway Cofferdam Remnant Erosion During Initial
Spillway Operation, Total River Flow of 3743 m^3/s

Flow Condition	Material	Increase in TSS(1)		
		Ackers-White	Engelund-Hansen	Yang
1 Bay open 1 m	Class A	Could not be calculated, particle diameter too small	< 1 mg/L	<1 mg/L
	Class B	< 1 mg/L	< 1 mg/L	< 1 mg/L
7 Bays open 1 m	Class A	See Figure 6		
	Class B			
7 Bays open full	Class A	Could not be calculated, particle diameter too small	Extreme TSS with erosion complete in under a few minutes	
	Class B	Extreme TSS with erosion complete in under a few minutes		

Note:

(1) Potential deposition of eroded material between the eroding area and K-Tu-2 has been ignored.

PHOTOGRAPHS

Photo 1: View looking northeast at site for Quarry Cofferdam and downstream rapids in Quarry Q7.

Photo 2: View looking northwest at Quarry Cofferdam location.

Photo 3: View looking at south closure area of North Channel Rock Groin

Photo 4: View looking north at north abutment area for North Channel Rock Groin

Photo 5: View looking southwest at location of North Channel Stage I Cofferdam.

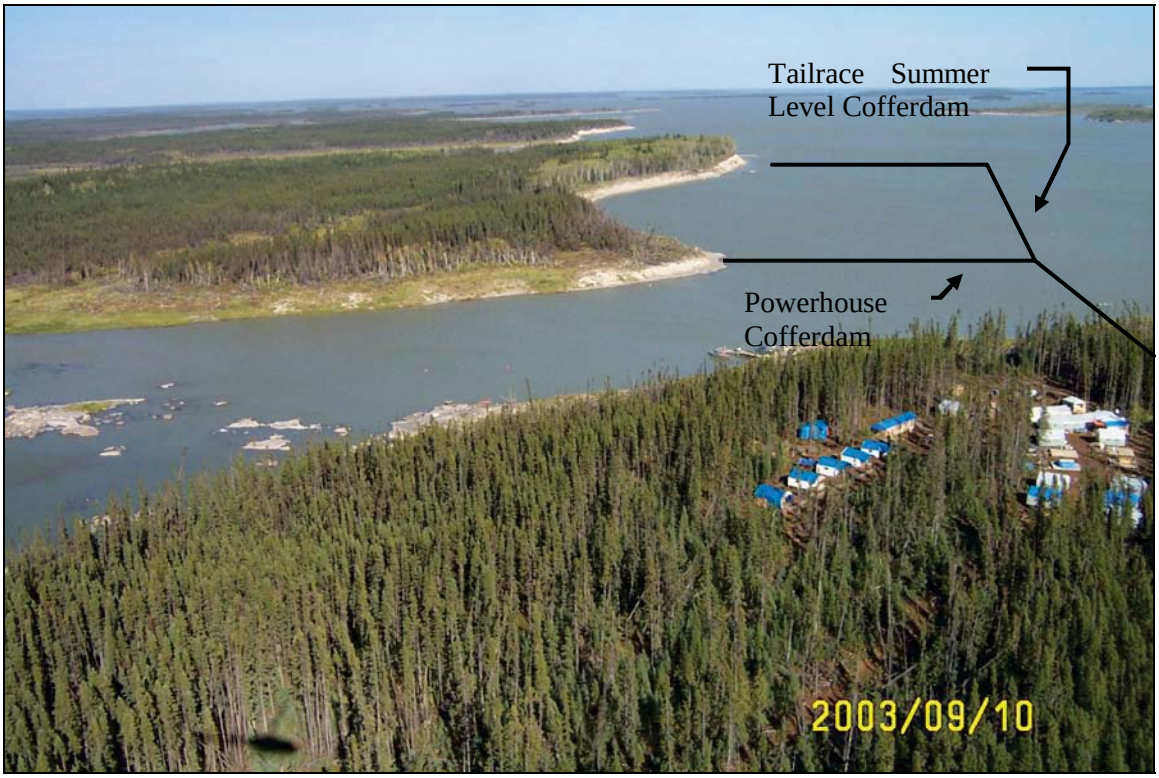


Photo 6: View looking east at sites for Powerhouse Cofferdam and Tailrace Summer Level Cofferdam.

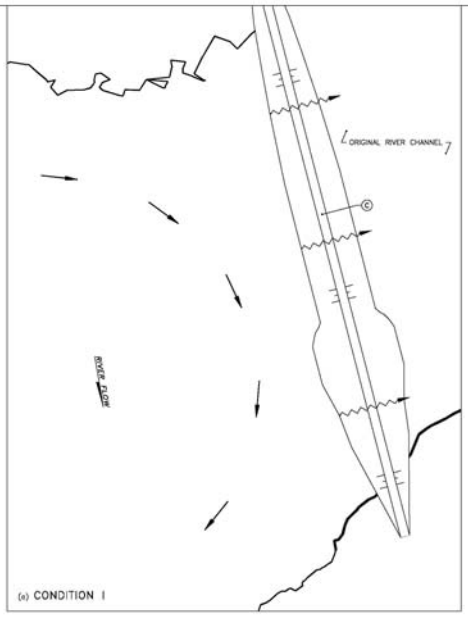
Photo 7: View looking northwest at Spillway Cofferdam area.

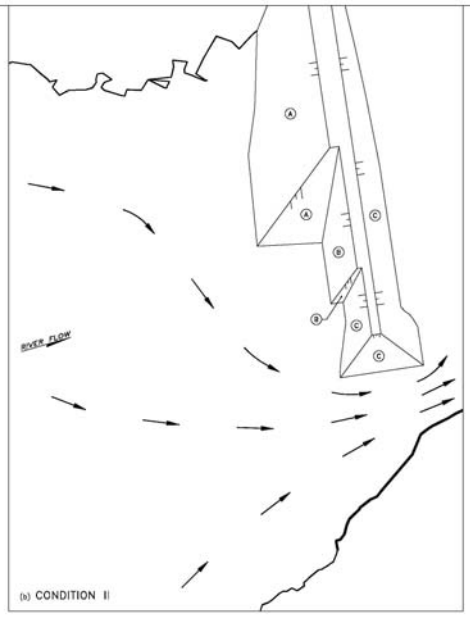
Photo 8: View looking west at Central Dam area, showing North and South Cofferdam abutment areas.

Photo 9: View looking north towards Spillway area from south abutment of South Dam.

Photo 10: View looking south at South Dam abutment area. Note the upstream Stage II South Dam Cofferdam will close onto the diabase bedrock outcrop in the center of the photo.

APPENDIX A



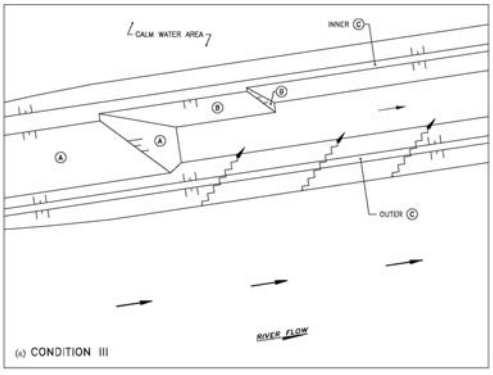


LEGEND:

- (A) IMPERVIOUS FILL
- (B) GRANULAR (TRANSITION) FILL
- (C) ROCKFILL
- FLOW DIRECTION, SCHEMATIC
- LEAKAGE
- SHORELINE

NOTE:

ROCKFILL EROSION PROTECTION ON CLASS (A) IMPERVIOUS FILL NOT SHOWN.



NO.	DATE	REVISIONS
01	2013 02 22	ISSUED AS A FIGURE FOR MEMO REFERENCE GN-9.2.17.

KGS ACRES

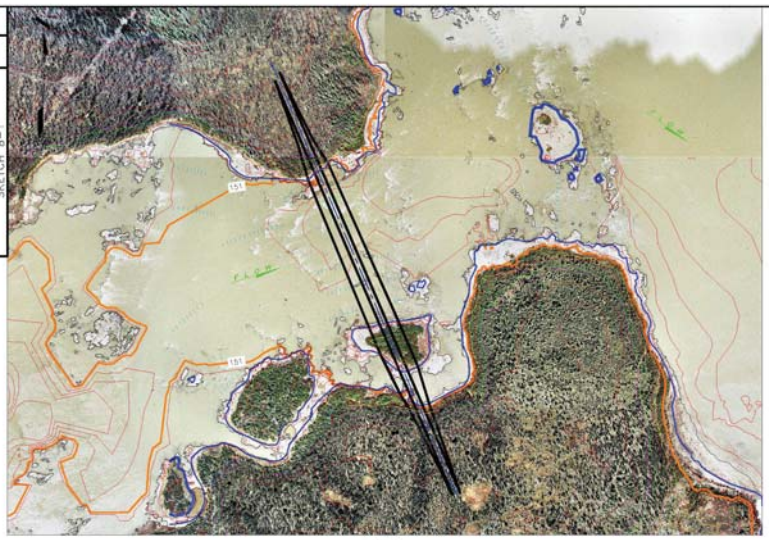
KGS ACRES LTD
MANITOBA HYDRO
KEEYASK GENERATING STATION

COFFERDAM CONSTRUCTION
FLOW VELOCITY SCHEMATICS
PLANS

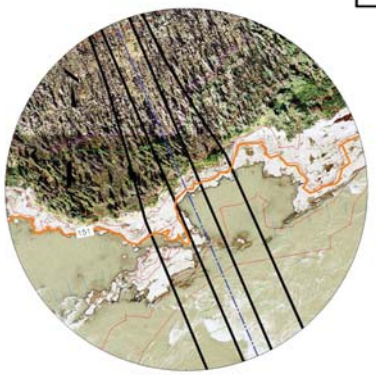
SCALE		SHEET	REV.
N.T.S.	SKETCH A-1	01 of 01	D1

APPENDIX B

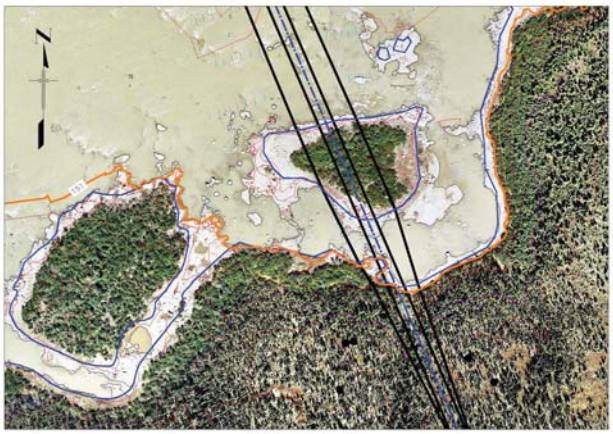
SKETCH B-1



PLAN



NORTH ABUTMENT



SOUTH ABUTMENT

LEGEND:

151 CONTOUR 151

KGS ACRES

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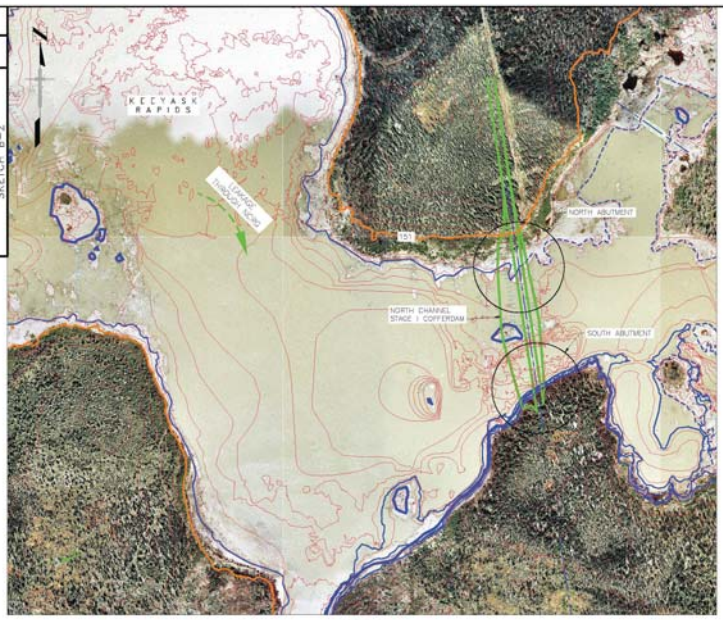
MANITOBA HYDRO

KEEYASK GENERATING STATION

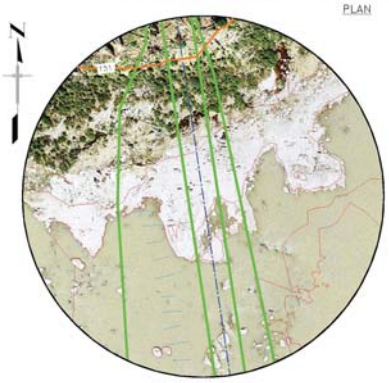
STAGE I DIVERSION
NORTH CHANNEL
ROCK GROIN
PLAN

SKETCH B-1

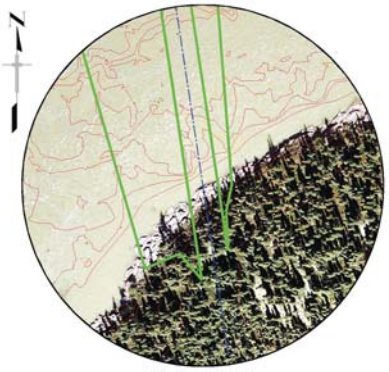
SKETCH B-2



PLAN



NORTH ABUTMENT



SOUTH ABUTMENT

LEGEND:

151 CONTOUR 151

NOTE:

NCRG – NORTH CHANNEL ROCK GRDN

NOTES	1	2021-3-02 ISSUED AS A FIGURE FOR MEMO REFERENCE QA-18-3-17	BY: MCK APP:
	NO.	DATE	REVISION
KGS ACRES	DESIGN BY	KGS ACRES LTD	
	DESIGN CHECK	MANITOBA HYDRO	
	SUBMITTED	KEEYASK GENERATING STATION	
	APPROVED	STAGE I DIVERSION NORTH CHANNEL STAGE I COFFERDAM PLAN	
	DRAWN		
	CHECK		
PROJECT MANAGER	SCALE 1:1000		
DATE	DATE	SKETCH B-2	NO.

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - PHYSICAL ENVIRONMENT

HISTORICAL CLIMATE ANALYSIS

REV 0

DELIVERABLE GN 9.5.1

MANITOBA HYDRO FILE: 00195-11100-00175_01

02/29/2012

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

WATER RESOURCES ENGINEERING DEPARTMENT

KEEYASK
Hydropower Limited Partnership

Manitoba Hydro

Keeyask Generating Station (Axis GR-4)
Physical Environment Studies

Technical Memorandum GN-9.5.1
Rev. 0, February 29, 2012

Historical Climate Analysis
Manitoba Hydro File 00195-11100-00175_01

Prepared by _____
Mark Gervais, Manitoba Hydro

_____
Michael Vieira, Manitoba Hydro

Checked by _____
Kristina Koenig, Manitoba Hydro

Reviewed by _____
Marco Braun, Ouranos Consortium

Approved by _____
Kristina Koenig, Manitoba Hydro

Accepted by _____
Efrem Teklemariam, Manitoba Hydro

Noted by _____ (WILLIAM DEWIT) SEP 5/2012
Manitoba Hydro, Hydro Power Planning

Manitoba Hydro
Winnipeg, Manitoba

D1910

MANITOBA HYDRO

INTEROFFICE MEMORANDUM

FROM	Kristina Koenig, M.Sc., P.Eng. Hydrologic and Hydroclimatic Studies Water Resources Engineering Dept. Power Planning Division Power Supply	TO	Marc St. Laurent, M.Sc., P.Eng. Keeyask/Burntwood River Planning Hydro Power Planning Department Power Planning & Dev. Division Power Supply

DATE 02/29/2012

FILE 00195-11100-00175_01

SUBJECT Memo # GN-9.5.1

Enclosed is the Historical Climate Analysis Technical Memorandum # GN-9.5.1 for the Keeyask Generation Project Stage IV Studies- Physical Environment. The objective of this study was to characterize the existing climate and identifying any significant climate trends in the immediate Keeyask Generating Station study area. The data to undertake this study was obtained from the Gillam and Thompson Environment Canada Airport meteorological stations.

This technical memorandum is to be used in support of the Keeyask Generating Station Environmental Impact Statement. In order to provide appropriate interpretation and guidance, please consult the Water Resources Engineering Department prior to external distribution.

If you have any questions regarding this report please feel free to contact me at 204-360-6318 or at kkoenig@hydro.mb.ca.

Regards,

Kristina Koenig, M.Sc., P.Eng.

Cc. Efrem Teklemariam, M.Sc., P.Eng.
Mark Gervais, M.Sc., A.Sc.T.
Michael Vieira, B.Sc., E.I.T.
Wil Dewit, P.Eng.

EXECUTIVE SUMMARY

Historical Climate Analysis memo No. GN-9.5.1 characterizes the climate in terms of precipitation, temperature and wind and identifies any significant climate trends with respect to temperature, precipitation and wind in the immediate Keeyask Generating Station (Keeyask) study area. The two meteorological stations closest to Keeyask that maintain a comprehensive long-term record are the Gillam and Thompson Airport meteorological stations (Gillam A and Thompson A). Gillam A is located on the east side of Stephens Lake approximately 35 km east of the Keeyask study area and Thompson A is located on the north side of Paint Lake Provincial Park approximately 177 km south-west of Keeyask study area. Gillam A is considered to be representative of the Keeyask climate while the analysis of Thompson A is included for comparative purposes.

The average annual temperature in this study area for the period of 1971 to 2000 is approximately −4.2°C and −3.2ºC for Gillam A and Thompson A respectively. The annual average precipitation is 499 mm at Gillam A and 517 mm at Thompson A. Rainfall accounts for approximately 63% of the total annual precipitation while snowfall accounts for 37% at Gillam A and 67% rainfall and 33% snowfall at Thompson A. Average wind speed in the study area ranges from 14.0 km/hr to 17.8 km/hr at Gillam A and 9.5 km/h to 13.2 km/h at Thompson A. On an annual basis, the predominant wind direction is from the west.

A trend analysis was performed on temperature, precipitation and wind data for the Gillam A and Thompson A using the adjusted historical Canadian Climate Data. Adjusted historical data are specifically designed for climate change analysis over Canada as well as for performing trend analysis on climate variables. The Mann-Kendall test for trend against randomness (Wang et al., 2001 and Zhang et al., 2000) was carried out for the time series of monthly mean precipitation, minimum temperature, maximum temperature, average temperature and mean monthly wind speeds to determine the statistical significance of each trend.

Statistically significant upward seasonal trends were observed for minimum, maximum and mean temperatures at Gillam A. Thompson A indicates significant upward trends for mean and maximum temperature in winter. In comparison to Gillam A, Thompson A exhibited generally larger upward trends for the winter season. A significant downward trend was observed for annual precipitation at Thompson A while Gillam A did not indicate any significant precipitation trends. For mean wind speed, no significant trends were found at Gillam A, however, significant upward trends were found at Thompson A.

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1 INTRODUCTION

In accordance with the requirements for the Keeyask Generating Station Project (Keeyask) Environmental Impact Studies, this memo aims to characterize the existing climate of the Keeyask study area (in terms of **precipitation**, temperature, and wind) and to identify any climate trends. This information will assist in determining the potential effects of climate on the project itself. The information presented needs to be taken in perspective as it is understood that historical climate will not necessarily be representative of future climate due to **climate change**.

1.1 STUDY AREA

Within the Keeyask site there are two meteorological stations that maintain a comprehensive long-term record; Gillam Airport, (Gillam A) with Environment Canada ID#:5061001 and Thompson Airport, (Thompson A) with Environment Canada ID#:5062922. Gillam A is located on the east side of Stephens Lake approximately 35 km east of the Keeyask study area, and Thompson A is located on the north side of Paint Lake Provincial Park approximately 177 km south-west of Keeyask study area (Figure 1). The climate data obtained from the Gillam A meteorological station has been relied upon as indicative of the long-term conditions found within the Keeyask study area, while Thompson A has been used for comparative purposes.

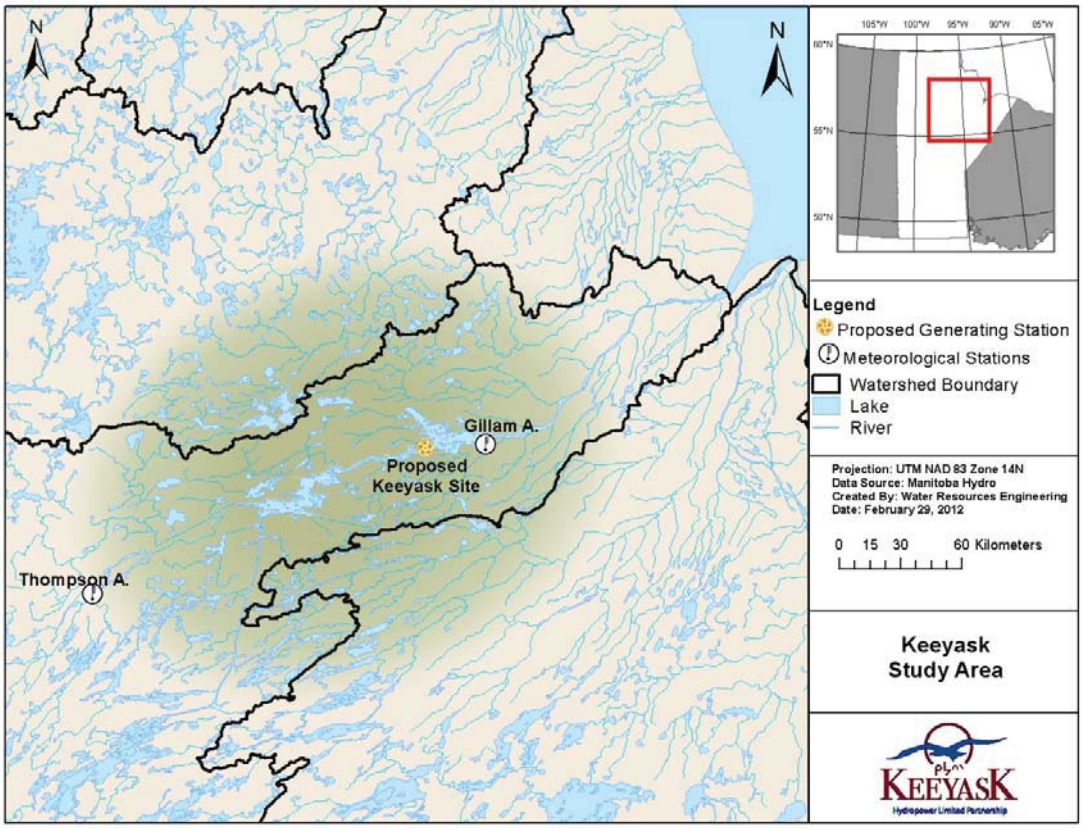


Figure 1: Study Location

2 EXISTING ENVIRONMENT

2.1 ENVIRONMENTAL INFLUENCES

The Keeyask study area is located within a colder subdivision of the High Boreal Eco-climatic Region in Manitoba (Smith et al., 1998). Keeyask is also located within the sub-arctic climate zone which is characterized by long, usually very cold winters, and short, cool to mild summers (Smith et al., 1998).

2.2 CLIMATE NORMALS

2.2.1 APPROACH AND METHODOLOGY

Climate normals from Environment Canada are used to describe the climatic characteristics of the study area. Climate Normals are arithmetic calculations based on weather observations at a given location over a specified time period. The World Meteorological Organization (WMO) describes normals as averages of climatological data computed for periods of 30 consecutive years. The WMO considers thirty years as a

sufficient amount of time to eliminate year-to-year variations (i.e. natural climate variability) and has set the current thirty year period to 1971-2000.

Climate normals representing averages (i.e. temperature and wind data) were calculated by applying the '3/5' rule that states if more than 3 consecutive daily values are missing or more than 5 daily values (in total) in a given month are missing, the monthly mean should not be computed and the year-month mean should be considered missing (Environment Canada, 2005). For normals representing totals (i.e. precipitation data), an individual month must be 100% complete (Environment Canada, 2005). The climate normals in this study were calculated by Environment Canada for the period of 1971-2000.

2010 Homogenized temperature and adjusted precipitation data over the 1971-2000 period was used to evaluate if any changes to instrumentation, station relocations and changes to observing practices were present during this period (Environment Canada, 2010). If significant changes exist then adjustments to the climate normals for the 1971-2000 period would be required. Homogenized and adjusted data was obtained from Environment Canada.

Growing degree days are a measure of heat accumulation typically used to predict the growth of vegetation or the life cycle of insects. Growing degree days are calculated by averaging the daily maximum and minimum temperatures and then subtracting a threshold base temperature. The base temperature is defined as the minimum temperature the organism requires for growth. Since not all organisms respond to temperature equally, the base temperature is species dependent. Typically a base temperature of 5°C or 10°C is used. An average growing degree days for the 1971-2000 period was calculated using both base temperatures.

The frost-free season is the period normally free of sub-freezing temperatures. **Frost-free days** are calculated as the number of consecutive days where the minimum temperature is above 0°C. In other words it is the period from the last frost in spring to the first frost in autumn. The measure of frost-free days are typically used to determine the length of the growing season for vegetation.

2.2.2 TEMPERATURE

2.2.2.1 NORMALS AND EXTREMES

The average annual temperature in this study area is approximately −4.2°C and −3.2ºC for Gillam A and Thompson A respectively. Table 1 along with Figure 2 and Figure 3 show the temperature climate normals for the Gillam A and Thompson A meteorological stations. An examination of extreme events indicates that isolated temperature events occur outside of the established normals. For example, the most pronounced extreme

maximum and extreme minimum recordings at Gillam A were +36.8°C in June of 2002 and -46.1°C in January 1975. Thompson A recorded extreme values of +37.4°C in June 1995 and −48.9°C in January 1968.

- For the winter months of December, January and February average daily temperatures for the two stations range from -25.8°C to -20.4°C. The average daily minimum temperatures range between -30.5°C and -27.0°C and daily maximums between -21.0°C and -13.7°C. The most extreme minimum daily temperature recorded, -48.9ºC, occurred at Thompson A on January 8, 1968. The most extreme maximum daily temperature recorded, +8.2ºC, occurred at Thompson A on February 28, 1993.

- For the spring months of March, April and May average daily temperatures for the two stations range from -15.1°C and 6.5°C. The average daily minimum temperatures range between -21.9°C to -0.4°C and daily maximums between -8.2°C to +13.4°C. The most extreme minimum daily temperature recorded, -48.3°C, occurred at Thompson A on March 1, 1972. The most extreme maximum daily temperature recorded, +32.6°C, occurred at Thompson A on May 27, 1986.

- For the summer months of June, July and August average daily temperatures for the two stations range from +11.4°C and +15.8°C. The average daily minimum temperatures range between +5.0°C to +9.2°C and daily maximums between +17.8°C and +22.7°C. The most extreme minimum daily temperature recorded, -10.2°C, occurred at Gillam A. on June 2, 1977. The most extreme maximum daily temperature recorded, +37.4°C, occurred at Thompson A on June 18, 1995.

- For the autumn months of September, October and November average daily temperatures for the two stations range from -12.1°C and +7.2°C. The average daily minimum temperatures range between -16.6°C to +2.6°C and daily maximums between -8.1°C to +12.8°C. The most extreme minimum daily temperature recorded, -41.1°C, occurred at Thompson A, on November 28, 1985. The most extreme maximum daily temperature recorded, +32.2°C, occurred at Thompson A on September 6, 1967.

The winter freeze-up (or winter condition) begins when the cumulated air temperatures (also called the freezing index) start to decrease. Winter conditions correspond, approximately, to the period when average daily temperatures remain below 0°C. In the study area, winter conditions are considered to exist during the five-month period of November through March. Correspondingly, summer conditions are recognized as the four-month period of June through September, when the mean daily minimum temperatures exceed 0°C. April, May and October comprise short intermediary periods between the dominant seasonal conditions of winter and summer. April, May and October

are characterized by mean daily maximum temperatures above 0°C, and mean daily minimum temperatures below 0°C.

Table 1: Temperature Normals (1971-2000) and Extremes at Gillam A and Thompson A

Sta.	Temp (°C)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yr
Gillam A	Avg	-25.8	-22.0	-15.1	-4.7	4.4	11.4	15.3	13.9	7.0	-0.4	-12.1	-22.5	-4.2
	Std Dev	3.5	3.7	3.3	3.0	2.2	1.8	1.5	1.7	1.8	2.1	3.4	3.9	1.2
	Max	-21.0	-16.3	-8.2	1.7	10.5	17.8	21.4	19.6	11.4	3.1	-8.1	-17.8	1.2
	Min	-30.5	-27.7	-21.9	-11.1	-1.8	5.0	9.2	8.2	2.6	-4.0	-16.1	-27.1	-9.6
	Extr Max	2.9	4.6	19.0	28.7	32.4	**36.8**	35.2	35.1	31.0	22.4	9.5	2.6	
	Date (yyyy/dd)	2003/ 07	1996/ 22	2000/ 14	1980/ 30	1986/ 27	**2002/ 29**	2007/ 24	1991/ 11	1996/ 01	1984/ 10	1978/ 01	1999/ 25	
	Extr Min	**-46.1**	-45.0	-42.6	-32.2	-22.8	-10.2	-1.7	-1.7	-9.1	-26.9	-39.4	-45.1	
	Date (yyyy/dd)	**1975/ 21**	1972/ 16+	1995/ 08	1977/ 06	1983/ 15	1977/ 02	1972/ 02	1979/ 23	1981/ 29	1997/ 30	1989/ 24	2004/ 24	
Thompson A	Avg	-24.9	-20.4	-12.9	-2.2	6.5	12.6	15.8	14.1	7.2	0.0	-12.0	-22	-3.2
	Std Dev	3.6	3.9	3.2	2.8	2.1	1.6	1.4	1.9	1.6	2.0	3.7	4.0	1.1
	Max	-19.4	-13.7	-5.2	4.9	13.4	19.7	22.7	21.0	12.8	4.3	-7.3	-16.7	3.0
	Min	-30.5	-27.0	-20.5	-9.2	-0.4	5.5	8.9	7.2	1.5	-4.3	-16.6	-27.2	-9.4
	Extr Max	8.1	8.2	19.1	29.4	32.6	**37.4**	35.9	34.6	32.2	24.6	13.4	5.0	
	Date (yyyy/dd)	1993/ 30	1993/ 28	1995/ 07	1980/ 30	1986/ 27	**1995/ 18**	1989/ 22	1991/ 11	1967/ 06	1984/ 09	1978/ 02	1988/ 01	
	Extr Min	**-48.9**	-47.8	-48.3	-34.4	-18.3	-5.6	-1.1	-3.5	-11.1	-27.1	-41.1	-47.6	
	Date (yyyy/dd)	**1968/ 08**	1974/ 01+	1972/ 01	1967/ 01	1969/ 19	2000/ 06	1968/ 05	1977/ 24	1974/ 28	1997/ 30	1985/ 28	1990/ 20	

Dates denoted with a "+" indicate that the extreme has occurred multiple times. The date shown corresponds to the first occurrence.

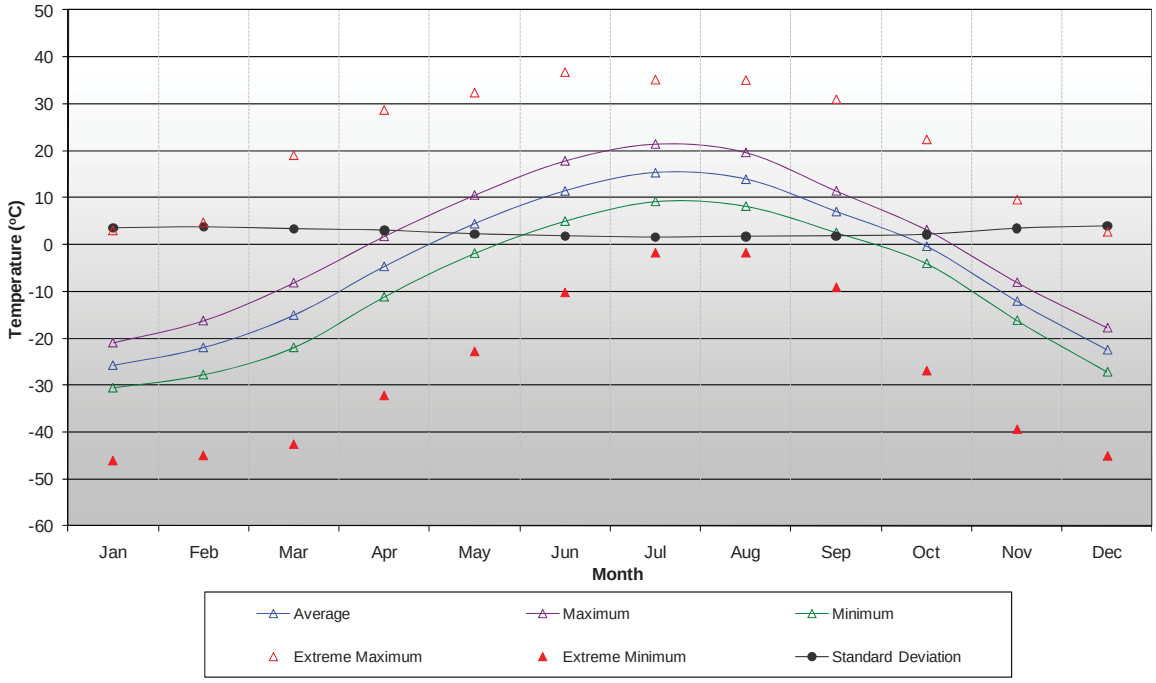


Figure 2: Temperature Characteristics for Gillam A, Manitoba (1971-2000)

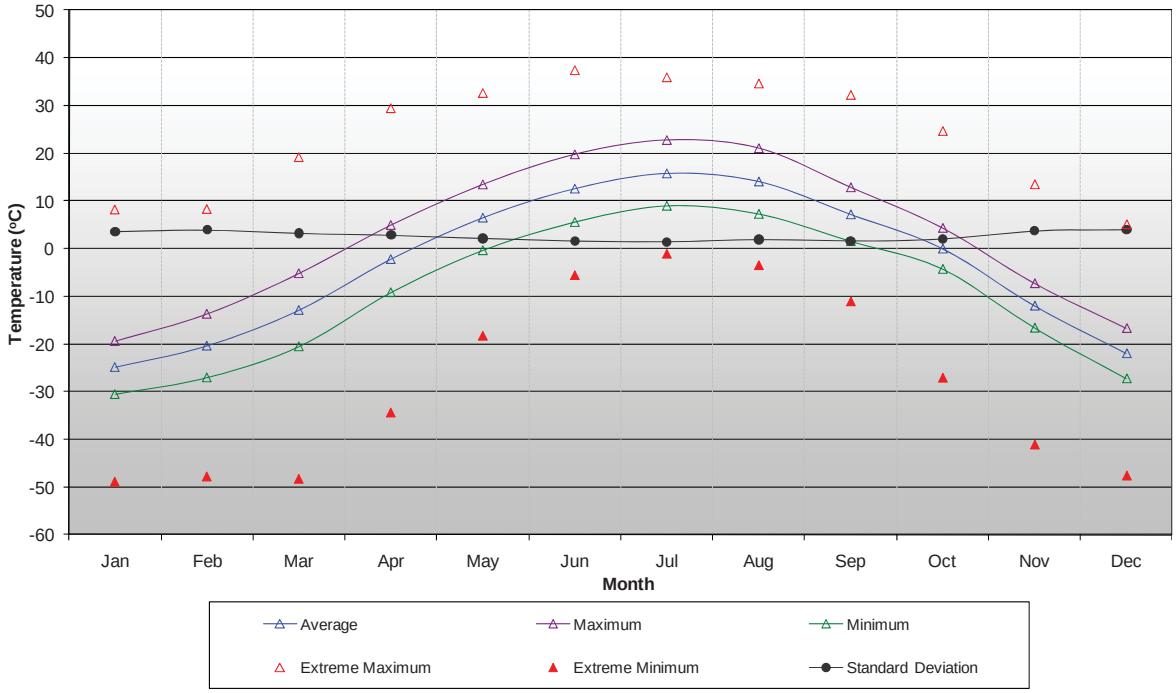


Figure 3: Temperature Characteristics for Thompson A, Manitoba (1971-2000)

2.2.2.2 FROST-FREE DAYS

The average of frost-free days at Gillam A is 91.9 days and 70.0 days at Thompson A for the period of 1971-2000. These values fall into the frost-free range reported for the northern forest zone of Canada which is between 60 to 110 days.

2.2.2.3 GROWING DEGREE DAYS

The total accumulated growing degree days with a 5°C threshold base temperature range from 969.6 at Gillam A to 1058.9 at Thompson A (Table 2). Using a 10°C base temperature the accumulated growing degree days range from 428.6 at Gillam A. and 476.8 at Thompson A. The higher total growing degree days at Thompson A reflect the warmer conditions experienced at this location.

Table 2: Growing Degree Days at Gillam A and Thompson A

Station	Base Temp	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yr
Gillam A	5°C	0.0	0.0	0.1	8.2	66.9	198.5	320.0	277.2	87.9	10.6	0.0	0.0	969.6
	10°C	0.0	0.0	0.0	1.7	20.5	86.3	167.8	130.3	20.7	1.3	0.0	0.0	428.6
Thompson A	5°C	0.0	0.0	0.2	12.3	95.3	230.2	334.9	282.3	90.7	12.8	0.2	0.0	1058.9
	10°C	0.0	0.0	0.0	1.7	30.8	102.8	180.8	137.4	22.1	1.2	0.0	0.0	476.8

2.2.2.4 HOMOGENIZED AVERAGE TEMPERATURE

A comparison between the homogenized average temperature to the non-homogenized record for the 1971-2000 time period indicates that no adjustments were made to the data record.

2.2.3 PRECIPITATION

2.2.3.1 NORMALS AND EXTREMES

Precipitation normals for the study area, including rainfall and snowfall records along with the extreme daily events of precipitation, rainfall, and snowfall are presented in Table 3 and Figures 4 and 5.

Average annual precipitation in the study area is approximately 499 mm at Gillam A and 517 mm at Thompson A. The precipitation over the months of November through April is mainly in the form of snowfall. Almost all months of the year record precipitation in the form of snowfall except for the month of July and August at Gillam A and July at Thompson A. At Gillam A rainfall accounts for approximately 63% of the total annual precipitation while snowfall accounts for 37% while at Thompson A total precipitation is comprised of 67% rainfall and 33% snowfall.

The maximum daily precipitation, falling in the form of rain, was 64.4 mm in the month of July (2000) in Gillam A and 75.3 mm at Thompson A in the month of August (1997). The maximum daily snowfall was 36.6 cm in May, 1988 at Gillam A and 45.7 cm in October, 1970 at Thompson A.

The average snow depth for both stations is greatest in the months of January to March; ranging from 41 cm to 56 cm (Table 3).

Table 3: Precipitation Characteristics (1971-2000) at Gillam A and Thompson A, Manitoba

Sta.	Precipitation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yr
Gillam A	Rainfall (mm)	0.0	0.0	0.5	3.8	29.8	51.2	81.8	77.2	48.7	19.2	1.2	0.1	313.5
	Snowfall (cm)	23.4	27.5	27.0	23.4	16.6	3.0	0.0	0.0	7.1	24.2	43.9	32.5	228.6
	Total Precip (mm)	17.5	21.2	20.3	23.2	44.2	53.9	81.8	77.2	55.0	40.9	37.5	26.7	499.4
	Avg Snow Depth (cm)	43.0	52.0	56.0	29.0	3.0	0.0	0.0	0.0	0.0	2.0	15.0	30.0	19.0
	Extr Rainfall (mm)	0.2	0.2	6.4	14.5	46.0	48.7	64.4	52.1	36.8	27.0	9.1	1.0	
	Date (yyyy/dd)	1987 / 08+	1981 / 19+	2004 / 09	1976 / 29	1988 / 31	1980 / 24	2000 / 08	1975 / 25	1999 / 18	1997 / 09	1974 / 08	1987 / 09	
	Extr Snowfall (cm)	18.8	29.0	24.0	32.0	36.6	9.0	0.0	0.5	16.6	21.6	27.0	18.2	
	Date (yyyy/dd)	1976 / 18	1999 / 11	1983 / 07	1973 / 21	1988 / 03	1992 / 04	1971 / 01+	1976 / 27	1992 / 21	1994 / 23	1983 / 24	1990 / 05	
	Extr Precip (mm)	17.5	23.8	18.6	34.3	46.0	48.7	64.4	52.1	36.8	30.6	21.6	17.2	
	Date (yyyy/dd)	1976 / 18	1999 / 11	1983 / 07	1973 / 21	1988 / 31	1980 / 24	2000 / 08	1975 / 25	1999 / 18	1997 / 09	2004 / 20	1990 / 05	
	Extr Snow Depth (cm)	102.0	107.0	102.0	89.0	56.0	8.0	0.0	0.0	15.0	55.0	66.0	97.0	
	Date (yyyy/dd)	1972 / 01+	1972 / 12+	1972 / 01+	1981 / 08	1981 / 01	1985 / 01	1971 / 01+	1971 / 01+	1984 / 23	1996 / 23	1971 / 26+	1971 / 21+	
Thompson A	Rainfall (mm)	0.1	0.2	0.8	6.2	33.0	67.9	86.1	73.7	58.5	19.9	1.6	0.2	348.2
	Snowfall (cm)	21.1	18.0	21.6	20.8	12.0	1.4	0.0	0.1	3.9	22.1	35.0	30.2	186.2
	Total Precip (mm)	18.2	15.9	20.6	26.0	44.4	69.4	86.1	73.9	62.4	41.4	32.8	26.3	517.4
	Avg Snow Depth (cm)	41.0	45.0	42.0	15.0	1.0	0.0	0.0	0.0	0.0	2.0	15.0	30.0	16.0
	Extr Rainfall (mm)	0.8	1.8	20.6	22.6	34.3	46.4	52.6	75.3	62.0	31.0	13.7	1.8	
	Date (yyyy/dd)	1987 / 04	1975 / 24	1968 / 27	1989 / 23	1986 / 12	1985 / 08	1981 /23	1997 / 25	1980 / 15	1984 / 17	1974 / 08	1967 / 18	
	Extr Snowfall (cm)	14.5	13.0	26.4	26.0	38.8	16.5	0.0	3.8	13.3	45.7	26.2	21.6	
	Date (yyyy/dd)	1968 / 25	1982 / 21+	1996 / 16	1991 / 06	1988 / 03	1969 / 11	1967 / 01+	1976 / 27	1982 / 29	1970 / 06	1990 / 22	1996 / 12	
	Extr Precip (mm)	14.5	11.4	22.8	24.8	34.3	46.4	52.6	75.3	62.0	45.0	28.3	15.8	
	Date (yyyy/dd)	1968 / 25	1982 / 21	1996 / 16	1991 / 30	1986 / 12	1985 / 08	1981 / 23	1997 / 25	1980 / 15	1970 / 06	1990 / 02	1986 / 02+	
	Extr Snow Depth (cm)	91.0	91.0	84.0	91.0	46.0	13.0	0.0	0.0	13.0	61.0	74.0	79.0	
	Date (yyyy/dd)	1968 / 29+	1968 / 01+	1982 / 16	1967 / 17+	1967 / 01+	1969 / 12+	1967 / 01+	1967 / 01+	1972 / 23	1970 / 12+	1971 / 24+	1971 / 16+	

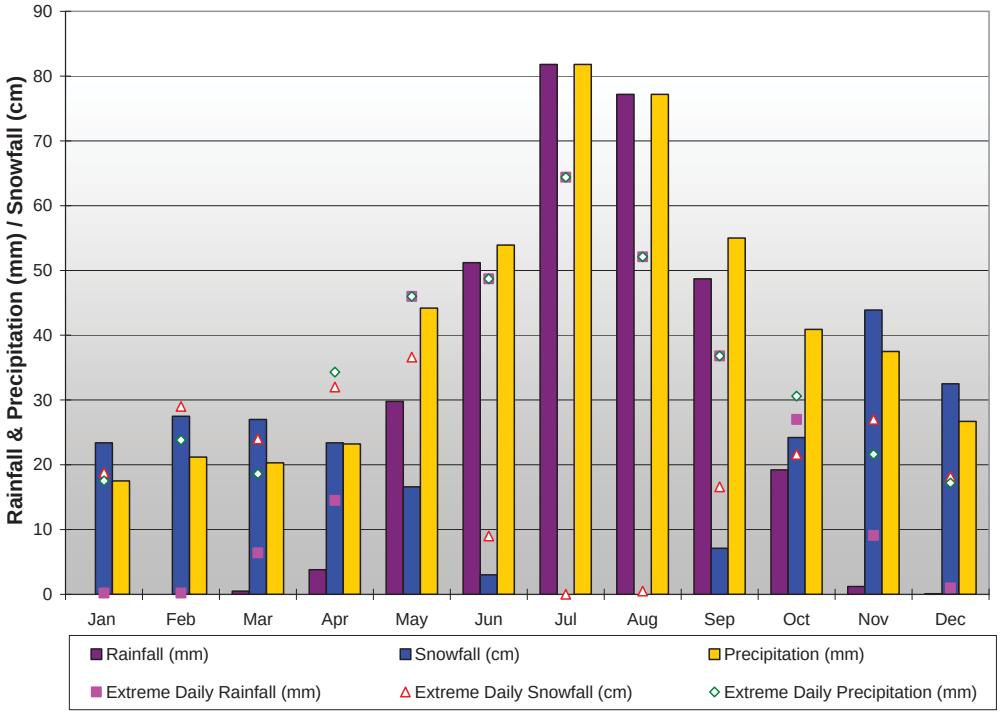


Figure 4: Precipitation Characteristics for Gillam A, Manitoba (1971-2000)

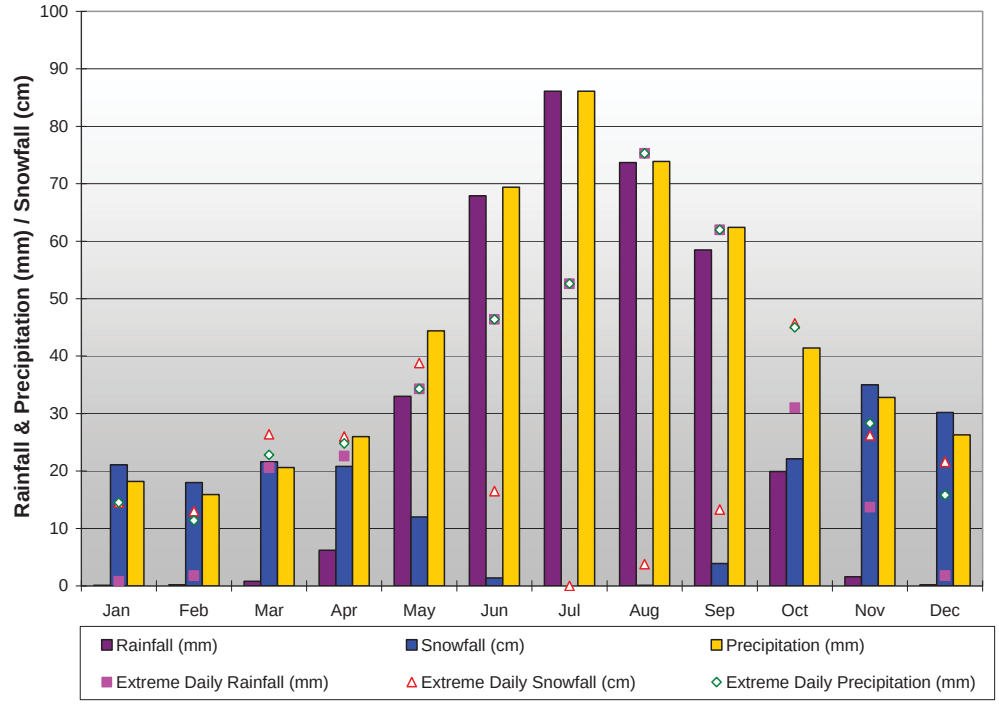


Figure 5: Precipitation Characteristics for Thompson A, Manitoba (1971-2000)

Note: Caution should be taken when comparing total rainfall (mm), snowfall (cm) and precipitation (mm) in Table 3 since they are not reported with the same units of measure.

To properly convert a snowfall measurement from cm into a liquid equivalent in mm, the density of the snow must be determined.

2.2.3.2 ADJUSTED AVERAGE PRECIPITATION

In general, the adjustment to the 1971-2000 precipitation record is small compared to the yearly total (Figure 6). The largest adjustment was made at Gillam A for November (3%). The largest adjustments were made in the months where snow is the dominant form of precipitation since snow measurements are prone to measurement error due to wind speed and snow density.

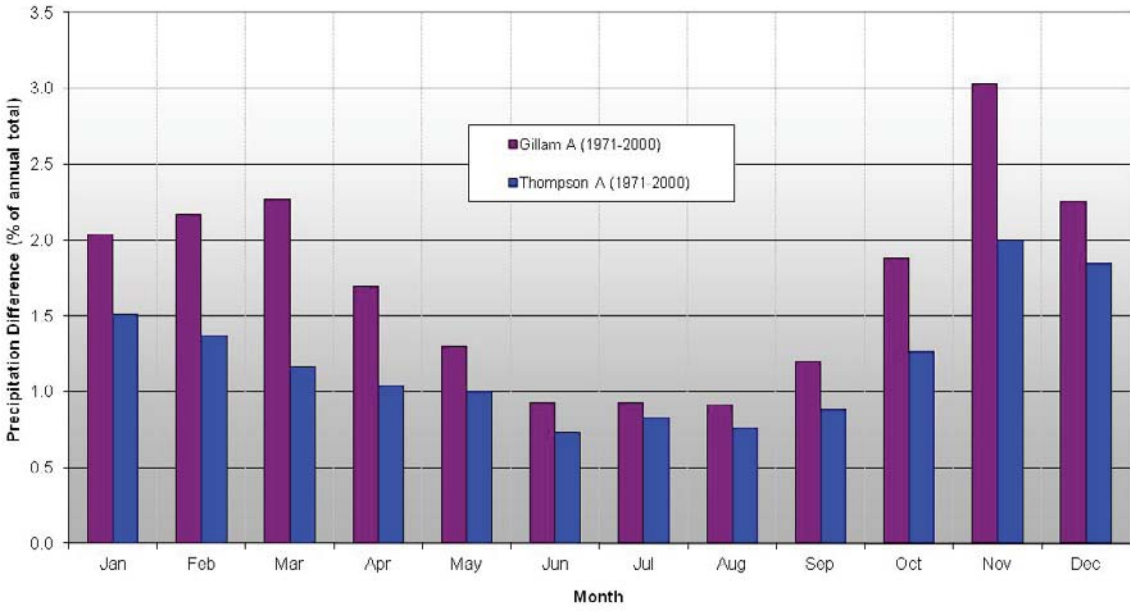


Figure 6: Monthly Total Precipitation Adjustments as a Percentage of Yearly Total Precipitation

2.2.4 WIND

2.2.4.1 WIND NORMALS AND EXTREMES

Table 4, Figures 7, 8, 9 and 10 show the wind normals and the wind roses for hourly wind speed in the study area, respectively. The wind roses show the frequency of winds blowing from the predominant directions. The length of each "spoke" around the circle is related to the frequency that the wind blows from a particular direction per unit time. Each concentric circle represents a different frequency, emanating from zero at the center to higher frequencies at the outer circles.

Average monthly wind speed in the study area ranges from 14.0 km/hr to 17.8 km/hr with an annual predominant wind direction of west at Gillam A and 9.5 km/h to 13.2 km/h with predominant direction of west at Thompson A.

Winter months (December, January, and February) are predominately comprised of the lowest average wind speeds at approximately 14.0 km/hr to 14.8 km/hr with a most frequent direction of west at Gillam A and 9.5 km/h to 10.6 km/h with a most frequent direction of west at Thompson A.

Spring (March, April and May) has average speeds slightly higher than winter ranging from 14.0 km/hr to 15.4 km/hr with a most frequent wind direction of north-east at Gillam A and from 11.6 km/h to 13.2 km/h with most frequent direction of north-east at Thompson A.

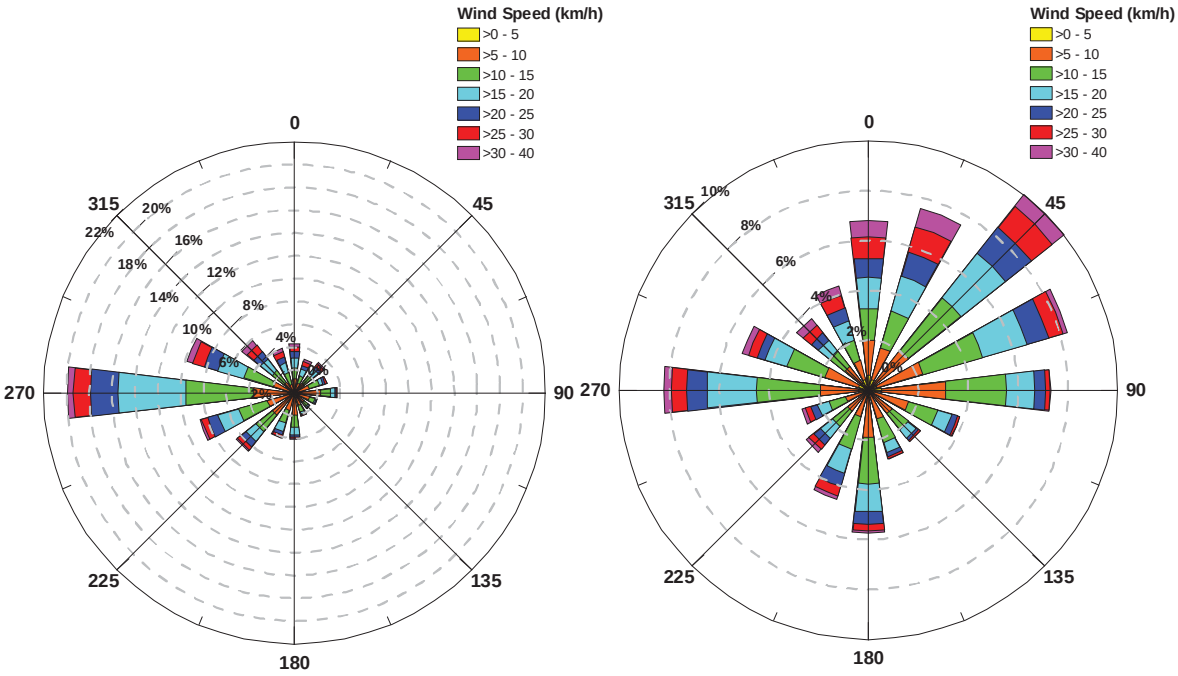
Summer (June, July and August) experiences average wind speeds that range from 15.1 km/h to 15.8 km/h with a most frequent direction of north-east at Gillam A and from 10.7 km/h to 12.2 km/h with a most frequent direction of west at Thompson A.

Autumn (September, October and November) wind speeds range from 16.4 km/h to 17.8 km/h with a most frequent direction of north-west at Gillam A and from 10.5 km/h to 11.5 km/h with a most frequent direction of west at Thompson A.

Overall, Gillam A experiences higher average wind speeds compared to Thompson A. The maximum hourly wind speed recorded was 83 km/hr in September (1981) at Gillam A while the maximum hourly average wind speed at Thompson A was 61 km/hr in September (1969).

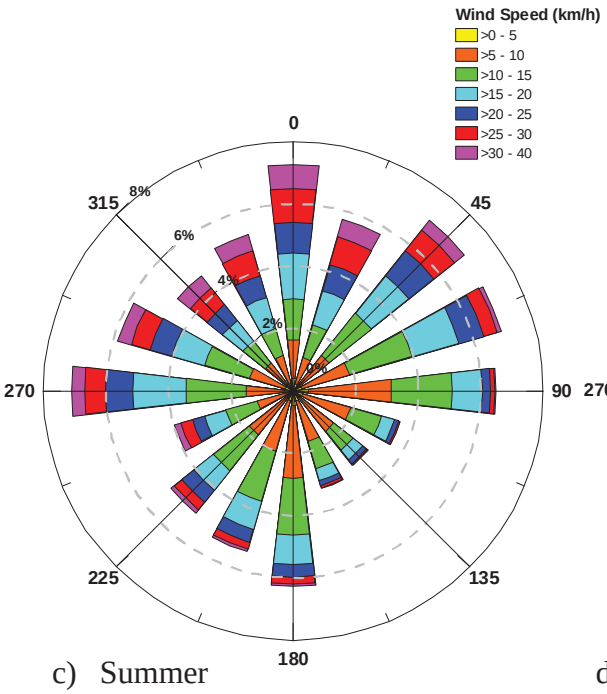
Table 4: Wind Normals for Gillam A and Thompson A, Manitoba (1971-2000)

Sta.	Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Yr
Gillam A	Speed (km/h)	14.8	14	14	15.1	15.4	15.8	15.1	15.7	17.4	17.8	16.4	14	15.5
	Most Freq Dir[n]	W	W	NE	NE	NE	NE	NE	NW	NW	NW	NW	W	NE
	Extr Max Hrly Speed (km/h)	67	65	76	61	67	67	74	70	83	63	64	65	
	Date (yyyy/dd)	1989/ 27	1982/ 21	1982/ 13	1987/ 28	1993/ 10	1993/ 26	1991/ 22+	1997/ 05	1981/ 27	1997/ 13	1971/ 06	1999/ 25	
	Extr Max Gust Speed (km/h)	89	78	91	83	100	96	107	96	91	81	74	96	
	Date (yyyy/dd)	1989/ 27	1982/ 21	1990/ 01	1987/ 28	1993/ 09	1987/ 13	1991/ 22	1986/ 19	1981/ 26+	1986/ 10+	1979/ 01+	1999/ 25	
Thompson A	Speed (km/h)	10.3	10.6	11.6	13.1	13.2	12.2	11	10.7	11.5	11.4	10.5	9.5	11.3
	Most Freq Dir[n]	W	W	NE	NE	NE	NE	W	W	W	W	W	W	W
	Extr Max Hrly Speed (km/h)	48	54	48	52	59	48	50	56	61	58	52	56	
	Date (yyyy/dd)	1984/ 24+	1988/ 27	1984/ 10	1985/ 20	1993/ 10	1967/ 02+	1968/ 13	1986/ 19+	1969/ 04	1967/ 01	1983/ 05	1999/ 25	
	Extr Max Gust Speed (km/h)	81	74	83	87	93	130	105	93	82	80	80	95	
	Date (yyyy/dd)	1989/ 27	1988/ 22+	1990/ 01	2001/ 25	1994/ 07	1975/ 28	1967/ 20	1999/ 27	2000/ 10	1988/ 09	1969/ 27	1999/ 25	

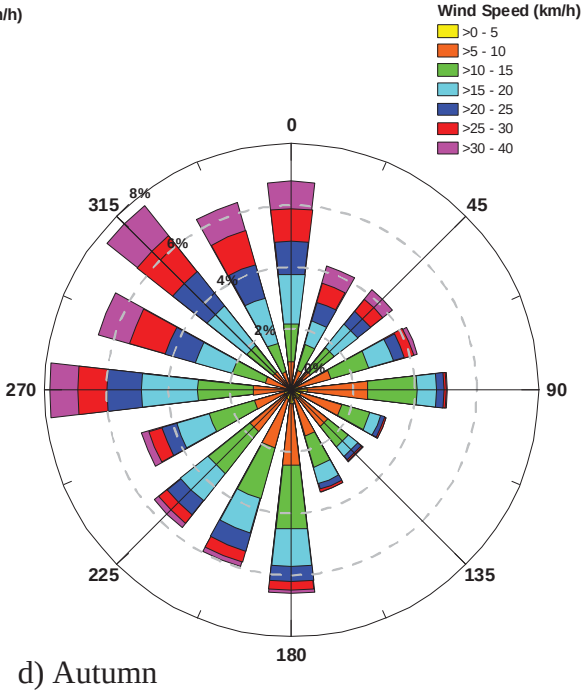


a) Winter

b) Spring



c) Summer



d) Autumn

Figure 7: Gillam A Seasonal Wind Rose for Hourly Wind Speed (1971-2000)

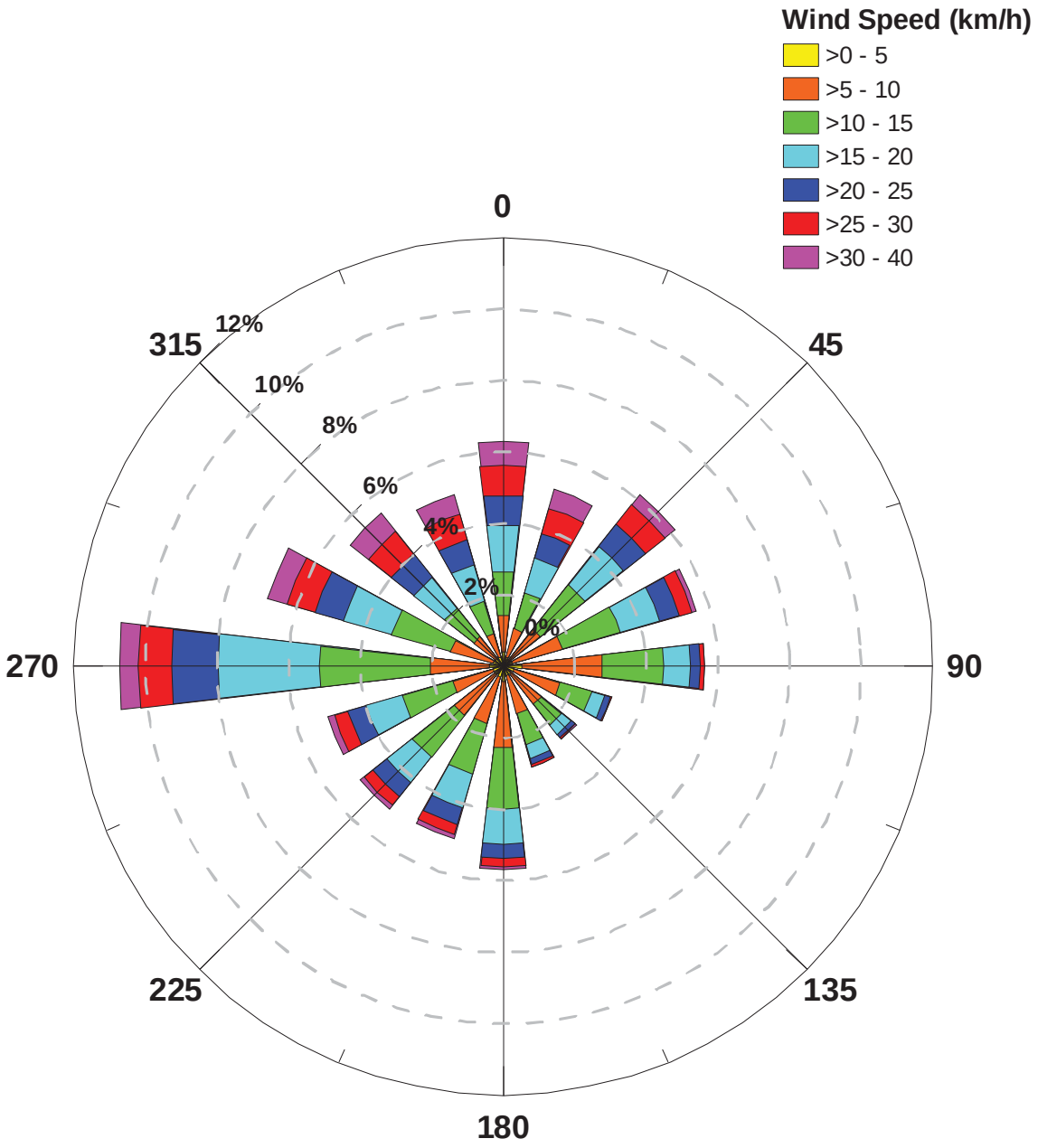


Figure 8: Annual Wind Rose for Hourly Wind Speed at Gillam A, Manitoba (1971-2000)

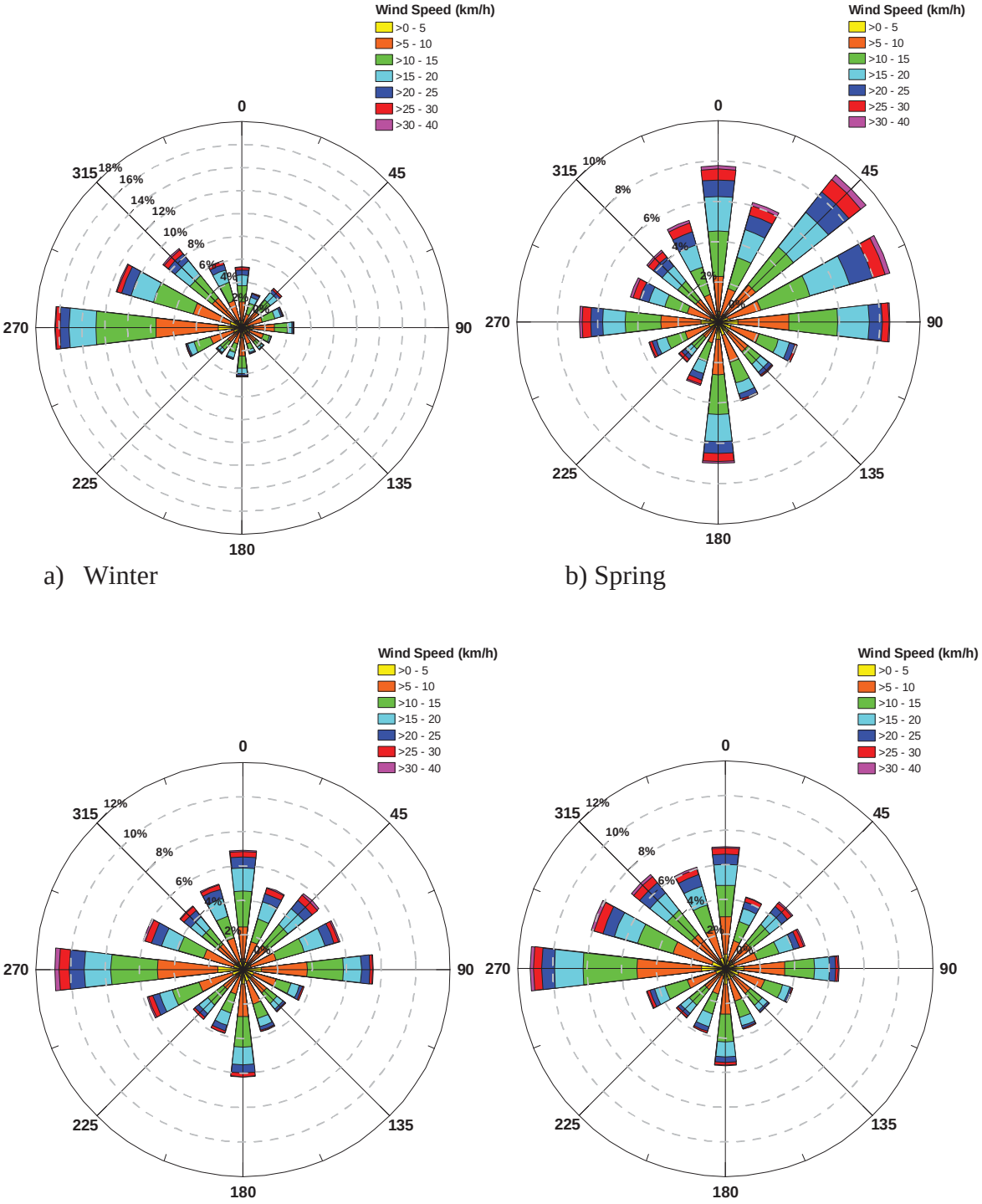


Figure 9: Thompson A Seasonal Wind Rose for Hourly Wind (1971-2000)

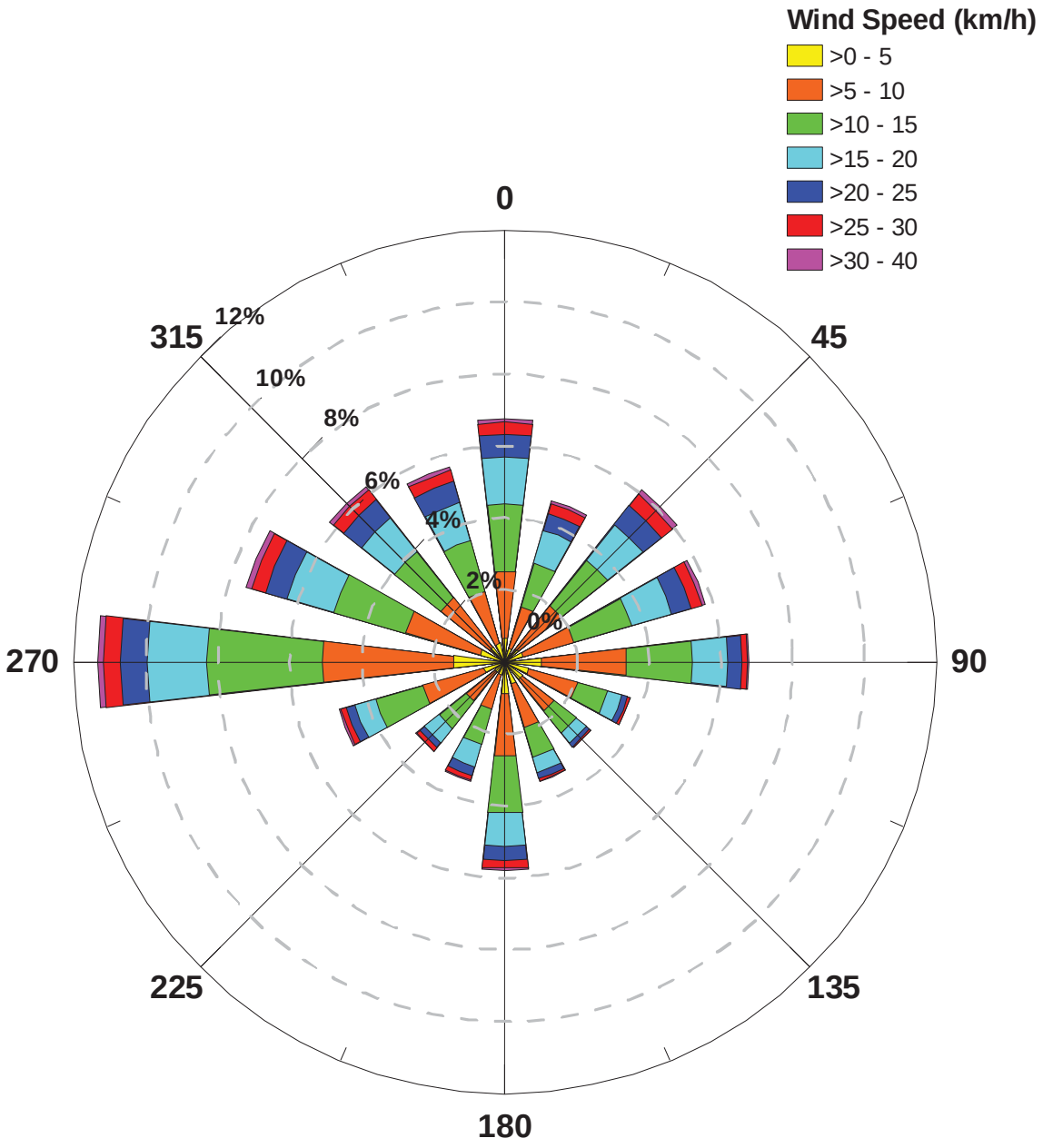


Figure 10: Annual Wind Rose for Hourly Wind Speed at Thompson A, Manitoba (1971-2000)

2.3 HISTORICAL CLIMATE TRENDS

2.3.1 DATA AND METHODOLOGY

A trend analysis of historical climate data for the Gillam A and Thompson A stations was performed on temperature, precipitation and wind data. Homogenized data is specifically designed for climate change analysis over Canada as well as for performing trend analysis on climate variables. Homogenized and adjusted data sets were produced and provided by Environment Canada (Environment Canada, 2010). **Heterogeneities** are accounted for due to changes in instrumentation, station relocation and changes to measurement techniques that may be present in the data during the period of record (Vincent et al., 2002 and Mekis et al., 2011). The trend analysis considers each month separately in addition to the annual and seasonal trends. Seasons are divided into three month periods: Winter (December, January and February), spring (March, April and May), summer (June, July and August) and autumn (September, October and November).

- For temperature, a regression model was used to remove the known heterogeneities in the station data due to changes to instrument, station relocations and changes to observing practices (Vincent et al., 2002). For Gillam A, a step was identified and corrected in the minimum temperature series in 1970 due to the joining of two station observations.

- With respect to precipitation data, adjustments were applied separately for rain and snow data. For rain data, corrections were applied for wind undercatch, evaporation and gauge specific wetting losses. Snow density corrections were applied to snow ruler measurements. Heterogeneities from station relocation were carefully minimized and trace precipitation were properly accounted (Mekis and Vincent, 2011). For the Gillam A record, data from two stations were combined to generate an adjusted time series from 1943-2010, however, a gap in the record is observed from 1962-1966.

- Heterogeneities in the wind speed data due to the changes in anemometer height and type, location and instrumentation were corrected (Wan et al., 2009). A change point was detected and adjusted due to instrumentation changes in 1979 for Gillam A and in 1970 for Thompson A.

In this study, a computer program implementing the **Mann-Kendall test** for trend against randomness (Wang et al., 2001 and Zhang et al., 2000) was used to analyze the time series of minimum temperature, maximum temperature, mean temperature, monthly precipitation and average wind speed values to determine both the magnitude and **statistical significance** of each trend. The effects of **serial correlation** were also taken into account in the trend analysis program. For temperature data, a record period of at

least 42 years was used (1943-2008 for Gillam A and 1967-2008 for Thompson A). Since several years of precipitation data is missing at Gillam A, only data from 1967-2010 was examined. Precipitation trend analyses considered data that spans a minimum of 39 years (1967-2009 for Gillam A and 1968-2006 for Thompson A). For wind, a record period of at least 38 years was used (1971-2008 for Gillam A and 1971-2008 for Thompson A). Date ranges used in the trend analyses are identified in Table 5, Table 6, Table 7, Table 8 and Table 9. Differences in date ranges exist due to data availability and reliability concerns.

All trends were assessed for statistical significance at the 5% level using a critical standard normal **Z-statistic** value of 1.96. A two-sided test for trend compared this critical Z-value to those Z-values obtained from the historical climate data, with trends being accepted on the condition that $|Z| \geq Z_{cr}$. A positive value of Z indicates an increasing trend, whereas a negative value identifies a decreasing trend. The following sections will report on significant trends only (bolded in tables).

Trend analyses were performed on the homogenized temperature, adjusted precipitation and homogenized wind speed data for the entire period of record available. The objective of the trend analyses was to identify trends in the record that were due to a climate change signal.

2.3.2 TEMPERATURE TRENDS

Temperature trends for Gillam A and Thompson A are summarized in Table 5 and Table 6 respectively. Trends are reported in degree Celsius per decade (ºC/dec.).

Annually: Significant upward trends were detected for mean and maximum temperature at both Gillam A and Thompson A. In addition, significant upward trend in minimum temperature was detected at Thompson A.

Seasonally: Significant upward trends were detected for minimum and mean temperature in winter, spring and summer for Gillam A. Spring and summer also shows significant upward trends for maximum temperature at Gillam A. Thompson A indicates significant upward trends for mean and maximum temperature in winter. No significant trends were detected at either location for autumn.

Monthly: Significant upward trends were detected in January for the two locations and April, May and June at Gillam A.

Table 5: Summary of Temperature Trends at Gillam A

	Period	Gillam A					
		T_{min}		T_{mean}		T_{max}	
		°C/dec.	Z-S	°C/dec.	Z-S	°C/dec.	Z-S
January	1943-2008	0.50	**2.19**	0.46	**2.26**	0.42	1.91
February	1943-2008	0.31	1.42	0.28	1.32	0.26	1.15
March	1943-2008	0.25	0.88	0.18	0.88	0.25	1.12
April	1943-2008	0.36	1.56	0.43	**1.96**	0.43	**2.13**
May	1943-2008	0.33	**2.09**	0.33	**2.15**	0.40	**2.00**
June	1943-2008	0.27	**2.24**	0.32	**2.69**	0.39	**2.77**
July	1943-2008	0.10	1.20	0.14	1.52	0.11	0.92
August	1943-2008	0.10	1.12	0.11	1.10	0.14	1.12
September	1943-2008	0.03	0.54	0.12	1.06	0.20	1.35
October	1943-2008	-0.15	-0.92	-0.10	-0.44	0.06	0.41
November	1943-2008	0.03	0.16	0.08	0.45	0.15	0.82
December	1943-2008	0.16	0.41	0.08	0.42	0.00	0.08
Annual	1943-2008	0.14	1.70	0.16	**2.39**	0.17	**2.46**
Winter	1944-2008	0.40	**2.37**	0.25	**2.13**	0.22	1.72
Spring	1943-2008	0.31	**2.15**	0.40	**2.56**	0.41	**2.73**
Summer	1943-2008	0.17	**2.34**	0.21	**2.33**	0.24	**2.14**
Autumn	1943-2008	-0.01	-0.08	0.00	0.07	0.10	0.79

°C/dec. = degree Celsius per decade; Z-S = Z-Statistics; Bold Z-Statistics indicate significant trend

Table 6: Summary of Temperature Trends at Thompson A

	Period	Thompson A					
		T_{min}		T_{mean}		T_{max}	
		°C/dec.	Z-S	°C/dec.	Z-S	°C/dec.	Z-S
January	1967-2008	0.95	1.91	0.97	**2.15**	1.03	**2.23**
February	1967-2008	0.70	1.14	0.52	1.02	0.50	0.86
March	1967-2008	0.67	1.28	0.57	1.07	0.35	0.77
April	1967-2008	0.01	0.02	0.01	0.01	0.23	0.55
May	1967-2008	0.02	0.06	0.00	0.03	0.00	-0.03
June	1967-2008	-0.06	-0.24	0.11	0.39	0.33	1.22
July	1967-2008	0.26	1.22	0.24	1.13	0.27	1.24
August	1967-2008	0.39	1.86	0.31	1.41	0.28	0.79
September	1967-2008	0.12	0.46	0.27	1.28	0.44	1.34
October	1967-2008	0.19	0.92	0.29	1.32	0.44	1.54
November	1967-2008	0.06	0.17	0.11	0.28	0.10	0.18
December	1967-2008	0.40	0.73	0.39	0.75	0.60	1.12
Annual	1967-2008	0.26	**1.99**	0.33	**2.22**	0.33	**2.36**
Winter	1968-2008	0.70	1.91	0.75	**2.11**	0.64	**2.33**
Spring	1967-2008	0.21	0.55	0.13	0.42	0.13	0.39
Summer	1967-2008	0.20	1.40	0.30	1.80	0.38	1.90
Autumn	1967-2008	0.08	0.41	0.13	0.81	0.25	1.25

°C/dec. = degree Celsius per decade; Z-S = Z-Statistics; Bold Z-Statistics indicate significant trend

2.3.2.1 PRECIPITATION TRENDS

Precipitation trends for Gillam A and Thompson A can be found in Table 7 and Table 8 respectively. Trends are reported in millimeters per year (mm/year). A significant downward trend in precipitation was detected annually for Thompson A. No other significant trends were identified for precipitation at Gillam A and Thompson A.

Table 7: Summary of Precipitation Trends at Gillam A

	Gillam A		
	Period 1	mm/year	Z-S
January	1967-2010	0.07	0.72
February	1967-2010	-0.04	-0.13
March	1967-2010	0.12	0.50
April	1967-2010	-0.34	-1.72
May	1967-2010	-0.42	-1.43
June	1967-2010	-0.31	-0.88
July	1967-2010	0.04	0.05
August	1967-2009	-0.31	-0.61
September	1967-2010	0.16	0.29
October	1967-2010	0.03	0.04
November	1967-2010	0.29	0.90
December	1967-2010	-0.32	-1.38
Annual	1967-2009	-0.57	-0.39
Winter	1967-2010	-0.26	-0.80
Spring	1967-2010	-0.41	-0.78
Summer	1967-2009	-0.10	-0.17
Autumn	1967-2010	0.28	0.34

Z-S = Z-Statistics; Bold Z-Statistics indicate significant trend

Table 8: Summary of Precipitation Trends at Thompson A

	Thompson A		
	Period	mm/year	Z-S
January	1968-2007	0.09	0.48
February	1968-2007	0.13	0.63
March	1968-2007	0.14	0.73
April	1967-2007	-0.09	-0.31
May	1967-2007	0.25	0.64
June	1967-2007	-0.32	-0.94
July	1967-2007	-0.30	-0.52
August	1967-2007	-0.43	-0.81
September	1967-2007	-0.01	-0.01
October	1967-2007	-0.69	-1.76
November	1967-2007	-0.11	-0.38
December	1967-2006	-0.47	-1.93
Annual	1968-2006	-3.13	**-2.14**
Winter	1968-2007	-0.25	-0.92
Spring	1968-2007	0.37	0.59
Summer	1967-2007	-1.16	-1.34
Autumn	1967-2007	-0.58	-0.93

Z-S = Z-Statistics; Bold Z-Statistics indicate significant trend

2.3.2.2 WIND TRENDS

Mean wind speed trends for Gillam A and Thompson A can be found in Table 9. Trends are reported in kilometers per hour per decade (km/h/dec.).

Significant upward trends in mean wind speed were observed at Thompson A on an annual basis as well as in the winter season. February, March and August indicated upward trends on a monthly basis.

No significant trends were detected at Gillam A.

Table 9: Summary of Mean Wind Speed Trends at Gillam A and Thompson A

	Gillam A			Thompson A		
	Period	km/h/dec.	Z-S	Period	km/h/dec.	Z-S
January	1971-2008	-0.89	-0.68	1971-2008	0.32	1.35
February	1971-2008	0.40	0.72	1971-2008	0.72	**2.66**
March	1971-2008	0.30	1.06	1971-2008	0.46	**2.25**
April	1971-2008	0.18	0.80	1971-2008	0.27	0.77
May	1971-2008	0.03	0.04	1971-2008	0.23	0.98
June	1971-2008	-1.09	-1.47	1971-2008	0.24	0.80
July	1971-2008	-1.68	-1.66	1971-2008	0.04	0.17
August	1971-2008	-0.69	-0.27	1971-2008	0.39	**2.20**
September	1971-2008	-1.27	-1.56	1971-2008	0.16	0.59
October	1971-2008	-1.53	-1.76	1971-2008	-0.01	-0.07
November	1971-2008	-1.14	-0.80	1971-2008	0.33	1.48
December	1971-2008	0.40	0.64	1971-2008	0.43	1.50
Annual	1971-2008	0.01	0.09	1971-2008	0.37	**2.86**
Winter	1971-2008	0.10	0.37	1971-2008	0.48	**2.41**
Spring	1971-2008	0.13	0.41	1971-2008	0.23	1.03
Summer	1971-2008	-1.00	-1.43	1971-2008	0.27	1.66
Autumn	1971-2008	-0.99	-1.57	1971-2008	0.19	0.69

Z-S = Z-Statistics; Bold Z-Statistics indicate significant trend

2.3.3 OTHER TREND STUDIES OF INTEREST

To verify the results obtained in this study, it is interesting to compare the results with similar studies. One study of particular relevance analyzed temperature and precipitation trends in Canada during the 20th century (Zhang et al., 2000). Station data was gridded using a statistical optimal interpolation technique, which is useful for data-sparse regions like northern Manitoba. The Mann-Kendall test for trend against randomness was used to detect significant trends. In the study, Zhang et al. (2000) detected the greatest warming in the spring, followed by the summer as well as annually for daily maximum temperatures in the Gillam and Thompson area.

2.3.4 REMARKS

The results obtained in this trend analysis, while insightful, are only preliminary. Further study is required to form a more comprehensive analysis. This could include analyzing data from other stations near Gillam A and Thompson A as well as investigating the data using other variables and levels of significance.

3 CONCLUSIONS AND RECOMMENDATIONS

This study characterized the existing climate and identified significant climate trends in the Keeyask study area. Two Environment Canada meteorological stations, Gillam A and Thompson A, were analyzed in this report. Gillam A is considered to be representative of the Keeyask climate while the analysis of Thompson A is included for comparative purposes.

The average annual temperature normals are approximately −4.2°C and −3.2ºC for Gillam A and Thompson A respectively. The average annual precipitation normals are 499 mm at Gillam A and 517 mm at Thompson A. Normal wind speed in the study area ranges

from 14.0 km/hr to 17.8 km/hr with predominant direction of west at Gillam A station. Normal winds speeds at the Thompson A station range from 9.5 km/h to 13.2 km/h with a predominant direction of west.

Environment Canada homogenized and adjusted data was used for analyzing temperature, precipitation and wind speed trends. The time frame used in the analysis varied between parameters and stations. Where possible, the longest record period was used. The analysis was done monthly, seasonally and annually. The following summarizes the significant trends that were detected;

- Significant upward temperature trends were detected for both Gillam A and Thompson A in minimum, mean and maximum temperature time series. However, not every month/season/annum series exhibited a significant trend.

- Significant downward trends were detected for annual precipitation at Thompson A but no significant trends were detected at Gillam A. Due to missing values in the Gillam record, the trend analysis did not use the entire homogenized precipitation record.

- Average wind speeds exhibited significant upward trends for Thompson A on an annual basis as well as in the winter and during several individual months. No significant trends for wind speed were observed at Gillam A.

It is important to note that the historical climate may not be representative of future climate conditions due to climate change. As such, the information presented in this report should be used with caution and accompanied with a clear understanding of climate change and climatic influences.

For a more complete trend analysis, it is recommended that more stations be used within the Nelson River basin to obtain a comprehensive understanding of the climate trends that may or may not impact the Keeyask project.

4 GLOSSARY

GLOSSARY

Climate change	Climate change refers to a change in the state of the climate that can be identified (e.g., by using statistical tests) by changes in the mean and/or the variability of its properties, and that persists for an extended period, typically decades or longer. Climate change may be due to natural internal processes or external forcings, or to persistent anthropogenic changes in the composition of the atmosphere or in land use.
Degree-days	Days of average temperature over a specified threshold temperature multiplied by the number of °C the daily average exceeds the threshold temperature.
Extremes	Extreme values refer to the absolute greatest value on record for a certain parameter. In the case of temperature, there is an extreme value for both the maximum and minimum temperatures observed in the record period. Extremes are accompanied by the date of occurrence. In instances where the same extreme value has occurred multiple times, the first date of occurrence is recorded.
Frost-free days	The number of consecutive days where the minimum temperature is above 0°C. In other words it is the period from the last frost in spring to the first frost in autumn.
Heterogeneities	Biases totally unrelated to changes in climate that, left uncorrected, can lead to inaccurate predictions of climate trends as part of climate change studies. For example differences in measurement techniques (i.e. instrument relocation, gauge undercatch, and trace measurements).
Homogenized data	Data that is free of variations due to non-climatic factors.
Mann-Kendall test	A nonparametric test for randomness against trend.

Maximum temperatures	Maximum monthly, seasonal and/or annual average temperatures calculated from daily maximum station records.
Mean temperatures	Mean monthly, seasonal and/or annual average temperatures calculated from daily mean station records.
Minimum temperatures	Minimum monthly, seasonal and/or annual average temperatures calculated from daily minimum station records.
Precipitation	Refers to *total* precipitation, or the sum of all the different precipitation types (i.e. rain, snow, ice pellets, drizzle, etc.).
Serial correlation	The correlation of a variable with itself over successive time intervals.
Statistical significance	A fixed probability of wrongly rejecting the null hypothesis of a statistical hypothesis test if it is in fact true.
Wind speed	The speed of motion of air in km/h, usually observed at 10 metres above the ground. It represents the average speed during the two-minute period ending at the time of observation.
Wind direction	The direction from which the wind is blowing with respect to north (360 degrees on the compass). For example, an easterly wind is blowing from the east, not toward the east. It represents the average direction during the two minute period ending at the time of observation and may be expressed to the nearest 10 degrees, or to one of the 16 points of the compass (N, NE, WNW, etc.).
Z-statistic value	A quantity calculated from a sample of data, whose value is used to decide whether or not a null hypothesis should be rejected in a particular hypothesis test.

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6 APPENDIX A - ABBREVIATIONS

ABBREVIATIONS

Avg.....Average

˚C .....Degrees Celsius

ºC/dec.…… .....Degrees Celsius per Decade

cm.....Centimeters

Dir^n…….Direction

Extr .....Extreme

Freq. .....Frequent

km/h .....Kilometers per Hour

km/h/dec.....Kilometers per Hour per Decade

Max. .....Maximum

Min. .....Minimum

mm .....Millimeters

mm/year .....Millimeters per Year

Precip .....Precipitation

Sta. .....Station

Std Dev.....Standard Deviation

Temp .....Temperature

T_{max} .....Maximum Temperature

T_{mean} .....Mean Temperature

T_{min} .....Minimum Temperature

WMO .....World Meteorological Organization

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Considerations Given To Potential Fisheries Issues During Construction

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Hatch Ltd.

Checked by _____ (I.R. Dewar) May 17, 2013
Hatch Ltd.

Approved by _____ (I.R. Dewar) May 17, 2013
Hatch Ltd.

Accepted by _____ (MARC ST.LAURENT) 2013/05/31
Manitoba Hydro

Hatch Ltd.
Winnipeg, Manitoba

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - ENVIRONMENTAL MITIGATION AND COMPENSATION MEASURES

CONSIDERATIONS GIVEN TO POTENTIAL FISHERIES ISSUES DURING CONSTRUCTION

REV 0

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1 INTRODUCTION

Notice to Readers: *Environmental Protection Plans for the Keeyask Generating Station and the South Access Road were produced after this document was developed. Where this document and the Environmental Protection Plans are inconsistent, the Environmental Protection Plans shall be used.*

This memorandum has been prepared to provide a review of various construction related concerns and issues related to the aquatic environment that were identified by the aquatic specialists, North/South Consultants, as well as possible means of mitigation. Many of the issues were originally raised during a workshop held on May 5, 2006. Subsequent to that date, there have been numerous changes to the construction schedule and to the designs of the cofferdams that were intended to mitigate, to the extent possible, concerns raised at the workshop. The issues raised at the workshop were related to the following key activities:

- Stream and river crossings during access road construction.
- Blasting operations.
- In-stream construction and removal of the structures required for river management.
- Discharges into water bodies as a result of cofferdam dewatering, concrete production and discharges from the waste water treatment plant.

Specific issues considered herein are:

- Any construction schedule deviations that may be required/possible for in-stream construction relative to the "Timing Windows" identified in the Department of Fisheries and Oceans (DFO) document DFO/2007-1329, "Manitoba In-Water Construction Timing Windows for the Protection of Fish and Fish Habitat".
- Compliance with the requirements in DFO document "Guidelines for the Use of Explosives in or Near Canadian Fisheries Waters (DFO 1998 with revised Table 1)".
- Stranding of fish during dewatering activities.
- Elevated total suspended sediment (TSS) levels as a result of construction activities.
- Deposition of deleterious substances into a water body.
- Fish habitat alteration, disruption or destruction (HADD).

It should be noted that the DFO document on in-water work timing windows provides a general guideline that would restrict the in-water work at the Keeyask site to the period of

July 16 - August 31. It does, however, indicate that site or project specific review may alter the restricted periods. The aquatics consultant has undertaken extensive studies at the site and believes that a case could be made for permissible in-water work between July 16 – September 15. Subsequent sections of this memo will review the implications of the use of the standard DFO windows and the "modified" window.

The following sections outline how each of the key activities noted above, and any corresponding guidelines or regulations related to these activities have been taken into account in designs and scheduling of the project. Where deviations from the guidelines or regulations appear to be necessary in order to successfully construct the project, available mitigation options are discussed.

2 ACCESS ROAD CONSTRUCTION – STREAM AND RIVER CROSSINGS

The proposed development of the Keeyask Generating Station will include the construction of two access roads, one on the north side of the Nelson River from Provincial Road 280 to the site and one on the south side of the river from the west end of the existing Butnau Dam to the site. There will also be a construction haul road between the generating station site and potential borrow sources located in Deposits N-5 and G-3 that will involve the crossing of a small side channel of the Nelson River. The road to the north of the project is being constructed as part of the Keeyask Infrastructure Project and is therefore not discussed further within this document.

On the alignments for the roads under consideration herein, there are 5 river and stream crossings that would be under the legislation of the *Fisheries Act and the Navigable Waters Protection Act*.

The design criteria and guidance provided by the aquatic consultants include:

1. Selection of crossing method to minimize habitat loss and alteration.
2. Avoidance of working during spawning periods.
3. Avoidance of working in-the-wet.
4. Maintenance of natural hydraulics to avoid bank scouring downstream.

The proposed south access road stream crossings are all of small streams and will be designed to utilize single or double corrugated steel culverts with the culvert size designed to accommodate a 1:50 year flood frequency to maintain natural hydraulic conditions. As it is known that these are fish bearing streams other than for the crossing of the former channel of the Butnau River, the crossings will also be designed to accommodate fish passage by using culvert flow velocity criteria, as defined by the

Department of Fisheries and Oceans and Manitoba Natural Resources guidelines. The crossing of the former channel of the Butnau River, just to the downstream of the Butnau Dam, is not a fish bearing stream. The only flow through the culvert below the road embankment is seepage from the dam and the culvert is elevated and has a V notch weir on it to monitor the seepage quantities.

All of the crossings will be designed with a layer of rock riprap for inlet and outlet protection against scour. Erosion protection measures will be outlined in the construction drawings and specifications and will be based on the "Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat" [Ref 1] and "Manual of Erosion and Sedimentation Control During Highway Construction" [Ref 2]/

The construction of all the south access road crossings is currently proposed to be undertaken in the winter months to avoid spring spawning other than for the Butnau River crossing, and working in-the-wet.

Deposits N-5 and G-3 have been identified as two primary sources of material for the construction of the Keeyask GS. Erosion in recent years has created new channels in the Nelson River to the northeast of the Keeyask project site. These new channels were only recently mapped. The haul road access to Deposit N-5 and G-3 now requires the construction of two stream crossings. These crossings will consist of rock fill causeways complete with through culverts.

The design of the causeways is based on the Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat and the Guidelines for Mine Haul Road Design. The design is based on accommodating a CAT 777D off-highway rock wagon, which is anticipated to be the largest hauling truck that would be used on the project.

The causeways will be rockfill embankments. They will consist of a free draining rockfill groin, with a 0.6 m thick layer of road topping material placed on the crest of the causeway. The total width of the crest will be about 24.0 m. The side slopes are assumed to be 1.5H:1V.

Two 1.0 m and one 1.5 m diameter culverts will be installed in the causeway to Deposit N-5 to allow fish passage. The culverts will be installed at different elevations to allow fish passage when the water level fluctuates. Due to the temporary nature of the causeways, the culverts will not be designed to mimic natural riverbed environment.

Culverts will not be installed in the causeway to Deposit G-3.

3 BLASTING OPERATIONS

It is known that the project requires a large amount of rock to be blasted from the area. As these activities will be taking place near the water, the aquatic consultants identified the potential to kill or injure fish and fish spawn and reported that the blasting activities fall under the legislation of the *Fishery Act*.

The design criteria and guidance received from the aquatic consultants include:

- Adhering to "Guidelines for the Use of Explosives in or Near Canadian Fisheries Waters (DFO 1998, with revised version of Table 1)".

The above mentioned Guidelines will be referenced in the Stage I Cofferdam Contract Specifications, as well as in the General Civil Contract Specifications and the contractors will be required to work within the prescribed limitations. It is a general practice to achieve the results stipulated in this guideline through proper design of the drilling patterns for the blasts and through the use of sufficient delays within any blast. Tables 1 and 2 below, which are presented in the above guideline, were developed from general equations for determining the setback distance necessary for confined explosives to meet guideline criteria with respect to fish habitat and spawning habitat, respectively. The equations are presented in Appendix 2 of the guideline.

For example, the current estimate and schedule assumes 20 kg of dynamite per hole in the Tailrace channel area. Therefore, with use of a delay of over 25 ms between every 5 holes, the blast would be equal to the 100 kg noted above for Table 1. The setback distance required from the centre of detonation to fish habitat would be 50.3 m.

During the period of egg incubation in a spawning bed, the restrictions are more stringent, as outlined in Table 2 of the Guidelines. These guidelines are designed to ensure that the peak particle velocity in spawning beds does not exceed the 13 mm sec -1 criteria. These criteria would also be adhered to.

From Table 2, which applies to fish spawning habitat, using 25 kg of dynamite for each explosive charge, the setback requirement is 75.5 m. Thus, if the spawning areas are within 75.5 m of the blast area during the period of egg incubation, the dynamite amounts per delay will have to be limited accordingly, which in turn may result in an increase to the costs and schedule compared to the current assumptions. Using 20 kg of dynamite per hole (as per the current estimate), the setback requirement is calculated to be 67.5 m. Currently it has been assumed that no spawning areas will be within the blast zone setbacks for the purposes of the schedule and cost estimate.

Table 1: Setback distance (m) from centre of detonation of a confined explosive to fish habitat to achieve 100 kPa guideline criteria for various substrates

Substrate Type	Weight of Explosive Charge (kg)							
	0.5	**1**	**2**	**5**	**10**	**25**	**50**	**100**
Rock	3.6	5.0	7.1	11.0	15.9	25.0	35.6	50.3
Frozen Soil	3.3	4.7	6.5	10.4	14.7	23.2	32.9	46.5
Ice	3.0	4.2	5.9	9.3	13.2	20.9	29.5	41.8
Saturated Soil	3.0	4.2	5.9	9.3	13.2	20.9	29.5	41.8
Unsaturated Soil	2.0	2.9	4.1	6.5	9.2	14.5	20.5	29.0

Table 2: Setback distance (m) from centre of detonation of a confined explosive to spawning habitat to achieve 13 mm sec-1 guideline criteria for all types of substrate.

	Weight of Explosive Charge (kg)						
	0.5	**1**	**5**	**10**	**25**	**50**	**100**
Setback distance (m)	10.7	15.1	33.7	47.8	75.5	106.7	150.9

The only instance where the criteria for use during the egg incubation period would be problematic is where the setback distance limits the amount of explosive that can be used to an amount that results in an unproductive blast or a drill pattern that is too dense to be reasonable. This is an issue for those blasts within the excavations that are located closest to the cofferdams and thus closest to a potential spawning area. As the excavations for the Powerhouse, Spillway and Tailrace Channel are all scheduled to have blasting activities occurring during the egg incubation period for the fall spawning fish (see preliminary schedule periods listed below), there may be some difficulty in achieving the criteria listed in Table 2 in some instances, and almost assuredly in the case of the Tailrace Channel.

The environmental team has suggested monitoring of the pressures to establish a site specific relationship and then to adaptively manage the blasting operations.

Figure 1, Principal Structures and Construction Facilities General Arrangement, Figure 2, Summary Construction Schedule, and Figure 3 (Sheets 1 to 3), Stage IV Studies Axis GR-4, Construction Sequence, are attached to the memorandum to assist the reader in gaining an understanding of the issues and the discussion in the text that follows.

Monitoring will be undertaken to assess compliance with the guideline values and an adaptive management plan will be implemented, if required.

The anticipated schedule dates for bedrock blasting activities in those areas in proximity to the river are as listed below.

Quarry Q-7 (for rockfill for Stage I cofferdam construction):

- mid July to mid October, 2014.
- early July to end of September, 2015.

Powerhouse and Spillway (including channels within confines of Stage I Cofferdams):

- mid July 2015 to mid April, 2016.

Tailrace Channel (within confines of the summer level cofferdam):

- late September to early December, 2018.
- last week of July, 2019.

4 CONSTRUCTION AND REMOVAL OF STRUCTURES FOR RIVER MANAGEMENT

The management of the river to allow for construction of the permanent generating station structures "in-the-dry" was identified during the May 2006 workshop as having the potential for major impacts to fish and their life cycle. As such, the aquatic consultants identified these activities as falling under the legislation of the *Fisheries Act and the Navigable Waters Protection Act*, with the major issues/concerns being related to working in-stream within the restricted windows for spawning and egg incubation, blocking fish movement, adding sediments to the water, habitat loss, and stranding fish.

Subsequent to the May 2006 workshop, the cofferdam designs were modified, if required and where possible, to reduce the potential for increases in TSS levels during both their construction and removal phases. The schedule for the construction of the cofferdams has also been modified, to the extent deemed possible, in order to comply with the DFO guidelines. Where schedule modification was not possible, the construction timing was based on the priorities set by the aquatic consultant as to preferential time periods of the year to undertake in-stream work.

The construction or removal activities associated with the various cofferdams are discussed below, along with comments on any of the above noted issues. Potential mitigation options are discussed, where mitigation is a possibility.

4.1 QUARRY COFFERDAM CONSTRUCTION

ISSUES

The Quarry Cofferdam's construction is scheduled for July 17 - 22, 2014 as shown on the schedule and depicted in Sequence No. 1 of Figure 3. Thus, it is within the non-restricted in-stream time period. A temporary increase in TSS levels resulting from material placement will be localized. It is anticipated that there will be no significant impacts on TSS levels compared to normal background levels, as measured at a central monitoring station (K-Tu-2). It is possible, however, that there may be some pools of water left within the channel to the downstream of the cofferdam, once the flows are cut off in this channel, within which fish may become stranded.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

In regard to a localized elevation of TSS levels, refer the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

A fish salvage operation should be undertaken and a requirement for a salvage fishery can be added to the dewatering specification clause within the Stage I Cofferdams Contract.

4.2 CONSTRUCTION OF NORTH CHANNEL ROCK GROIN

ISSUES

The North Channel Rock Groin's construction is scheduled for July 28 to August 18 of 2014, as shown on the construction schedule and depicted in Sequence No. 1 of Figure 3. Thus, it is within the non-restricted in-stream time period. The main issue is the need to pre-armour the shore of the island where the groin will close against it, to prevent erosion of the overburden which in turn could lead to unacceptable increases in TSS levels. Depending on the armouring methodology employed, it may be necessary to undertake the shore armouring within the in-stream restricted period before July 15. The amouring may consist of riprap armouring along the shoreline (believed to be bedrock controlled at normal water levels), possibly in gabion baskets; or could consist of a groin projecting out from the shore in the area where final closure of the channel would occur, which would be composed of clean rockfill material, gabions, or other material that is not anticipated to increase TSS levels in the river significantly.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

It is anticipated that Mitigation will not be required, other than taking due caution during the shoreline armouring/groin construction to avoid introduction of unacceptable levels of deleterious substances and increases in TSS. As work in this area will likely be required before July 15, the area should be assessed for the presence of spawning grounds.

4.3 CONSTRUCTION OF THE NORTH CHANNEL STAGE I COFFERDAM

ISSUES

The North Channel Cofferdam is scheduled to be constructed August 18 – Sept 3, 2014 as shown on the schedule and depicted in Sequence No. 2 of Figure 3. Thus, this raises the issue of encroaching into the first few days of the fall in-stream work restricted period. It is also likely that there will be some ponds of water left on the river bed that may contain stranded fish, once the flow within the river channel has been cut off by the cofferdam. It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction or partial removal of this cofferdam.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

A fish salvage operation should be undertaken and a requirement for a salvage fishery can be added to the dewatering specification clause within the Stage I Cofferdams Contract. An attempt can be made to expedite the pace of construction to complete before the DFO guidelines' fall in-stream restricted period, but this would be a very aggressive schedule. The construction should be able to be completed within the window identified by the aquatics consultant, i.e. before September 15. Refer to the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

4.4 CONSTRUCTION OF THE POWERHOUSE COFFERDAM

ISSUES

The Powerhouse Cofferdam is scheduled to be constructed July 28 – October 14, 2014 as shown on the schedule and depicted in Sequence No. 1 and 2 of Figure 3. Thus, this raises concerns in regard to construction during the fall in-stream work restricted season and the stranding of fish during the subsequent dewatering of the cofferdam enclosed area. It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction or partial removal of this cofferdam.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

The construction schedule cannot avoid in-stream work within this period without significant delay to the project's overall construction schedule and thus the addition of a very sizable cost impact. In regard the fish stranding, a fish salvage operation should be undertaken and a requirement for a salvage fishery can be added to the dewatering specification clause within the Stage I Cofferdams Contract. Refer the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

4.5 CONSTRUCTION OF THE SPILLWAY COFFERDAM AND THE CENTRAL DAM COFFERDAM

ISSUES

The Spillway Cofferdam is scheduled to be constructed July 16 – October 14, 2015, as shown on the schedule and depicted in Sequence No. 3 on Figure 3, while the Central Dam cofferdam is scheduled to be constructed August 10 – October 6, 2015. Thus, this raises concerns with regard to construction during the fall in-stream work restricted season and the stranding of fish during the subsequent dewatering of the cofferdam enclosed areas. It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction or partial removal of these cofferdams. Concerns were also expressed during the workshop in regard to habitat loss associated with the construction of the Central Dam cofferdam and subsequently by the construction of the Central Dam and the impoundment of the forebay to the upstream of these areas.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

The construction schedule cannot avoid in-stream work within this period of the fall without significant delay to the project's overall construction schedule and thus the addition of a very sizable cost impact. It was deemed preferable to delay the start of construction of these cofferdams and thus avoid the spring in-stream restricted period rather than attempt to avoid working within the fall restricted period.

In regard the fish stranding, a fish salvage operation should be undertaken and a requirement for a salvage fishery can be added to the dewatering specification clause within the General Civil Contract. It could also be specified that the Spillway cofferdam must enclose the area before mid September to help prevent spawning fish from entering the area and becoming trapped. Refer to the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

The footprint of the permanent structures has been designed to be the minimum amount necessary for their safe construction and operation. Any associated habitat loss is a required element of the project and will be accounted for in the project compensation.

4.6 REMOVAL OF PORTIONS OF THE SPILLWAY STAGE I COFFERDAM

ISSUES

The removal of the Spillway Stage I Cofferdam is scheduled to take place from July 25 – August 29, 2017, as shown on the schedule and depicted in Sequence No. 5 on Figure 3. Thus this raises the issue of potentially elevated TSS levels in the water affecting spawning habitat and fish behavior, as well as in-stream work being undertaken within the restricted period identified by DFO. The work should be able to be completed within the modified window proposed by the aquatics consultant, i.e. before September 15, 2017.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

It has been assessed that the removal of the cofferdam will have minimal impacts on TSS levels.

4.7 CONSTRUCTION OF SOUTH DAM STAGE II UPSTREAM ROCKFILL SECTION

ISSUES

The construction of the South Dam Stage II Upstream Cofferdam's rockfill section is scheduled to take place from August 30 - October 12, 2017, as shown on the schedule and depicted in Sequence No. 5 of Figures 3. Thus this raises concerns with regard to construction during the fall in-stream work restricted season and with respect to creating a blockage to fish movement. It has been assessed that construction of this structure will have minimal impact on TSS levels.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

The construction schedule for completion of the Spillway, to the point where it is ready to permit diversion flows to pass through, and the need to complete the South Dam Stage II Upstream and Downstream Cofferdams early in 2018, to permit work to commence on the South Dam, does not leave any flexibility in shifting this work to an earlier or later time period to avoid the in-stream restricted construction periods. No mitigation options are suggested. Refer to the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

4.8 CONSTRUCTION OF SOUTH DAM STAGE II UPSTREAM AND DOWNSTREAM COFFERDAMS

ISSUES

The construction of the cofferdams is scheduled to take place from May 10 - July 17, 2018 as depicted in Sequence No. 6 of Figure 3. This raises concerns with regard to construction during the spring in-stream work restricted season and the stranding of fish during the subsequent dewatering of the cofferdam enclosed area. It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction.

Although the construction schedule in Figure 2 shows the Stage II Upstream and Downstream cofferdams being constructed from August 30 – October 17, 2017 the construction of the cofferdams is likely to extend into the spring and summer of 2018 as depicted in Sequence No. 6 of Figure 3. This raises concerns with regard to construction during the spring in-stream work restricted season and the stranding of fish during the subsequent dewatering of the cofferdam enclosed area. It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

As discussed above under the South Dam Stage II Upstream Rockfill Section, there is no float in the schedule that would permit the rescheduling of these activities to a later time period and thus no mitigation options have been identified. Possible methods of excluding fish from the vicinity of the cofferdam should be reviewed.

4.9 CONSTRUCTION OF TAILRACE SUMMER LEVEL COFFERDAM

ISSUES

Construction of the Tailrace Summer Level Cofferdam is scheduled for July 16 – September 14, 2018, as shown on the schedule and depicted in Sequence No. 7 of Figure 3, with repairs, following winter overtopping of the cofferdam, being scheduled for June 1 – June 28, 2018, as shown on the schedule and depicted in Sequence No. 8 of Figure 3. This raises an issue with construction during the fall spawning period identified in the DFO guidelines, as well as undertaking repairs during the spring spawning period. The fall construction should, however, be completed within the permissible window proposed by the aquatics consultant, i.e., by September 15.

In addition, concerns were raised in regard to habitat loss and stranding of fish during the dewatering of the cofferdam enclosed area.

It has been assessed that there are unlikely to be any significant increases in TSS levels as a result of the construction or partial removal of this cofferdam.

POSSIBLE MITIGATION / ENGINEERING ASSESSMENT

The start of construction has been scheduled for after the end of the spring spawning in-stream work restriction period. This results in the unavoidable completion of construction falling within the DFO guidelines fall spawning season. Additionally, the completion of the cofferdam in mid September pushes the blasting associated with the tailrace channel into the fall spawners' egg incubation period. Possible methods of preventing the fish from entering the area should be investigated.

In order to complete the tailrace excavation and cofferdam removal in time for first power in November, 2019, it is unavoidable that the repairs to any damage sustained by the cofferdam during the winter be undertaken in the spring in-stream restricted period. It is believed that this repair work is unlikely to require a great deal of work having to be undertaken below the water level and, depending upon the ice conditions that year, efforts could be made to undertake the work in early May. Possible methods of preventing fish from entering the area should be investigated.

In regard to fish stranding, a fish salvage operation should be undertaken in both years when the enclosed area is dewatered and a requirement for a salvage fishery can be added to the dewatering specification clause within the General Civil Contract.

Refer to the Keeyask Generating Station In-Stream Sediment Management Plan for potential measures to mitigate TSS.

The development of the proposed Keeyask Generating Station (Keeyask GS) will result in an increase in upstream water level in the zone of hydraulic influence, as well as lower water levels within the area located immediately downstream of the Spillway. As suggested by North/South Consultants Inc. (NSC), these water level changes could potentially impact existing sturgeon spawning habitat within and near to Gull Rapids. To compensate for this loss of habitat it has been proposed that a new sturgeon spawning area downstream of the Keeyask GS be developed.

5 DISCHARGE OF EFFLUENT OR WATER FROM CONSTRUCTION AREAS TO THE NELSON RIVER

The following sections discuss the various sources of fluids that will be discharged into the river and the measures that will be taken to ensure that they do not produce detrimental effects on the aquatic environment.

5.1 WASTE WATER TREATMENT PLANT EFFLUENT

This section has been superseded by EIS Project Description Supporting Volume, Section 3.3.1.2.

5.2 CONCRETE PROCESSING PLANT EFFLUENT

This section has been superseded by EIS Project Description Supporting Volume, Section 3.3.1.3.

5.3 COFFERDAM DEWATERING AND CONTINUED PUMPING

This section has been superseded by EIS Project Description Supporting Volume, Section 3.3.3.

5.4 CONTAINMENT FILLS

This section has been superseded by the Keeyask GS Environmental Protection Plan.

6 CONCLUSIONS

The potential exists for some construction activities to leave an impact on the environment. It is the responsibility of all parties involved to identify potential impacts, assess alternatives to minimize impacts, implement measures to mitigate impacts and as a final step, compensate for impacts that are unavoidable. By identifying the potential issues and recommending mitigation requirements, the Aquatic Consultants have aided the Engineering Consultants to modify the timing of construction activities in the schedule and reduce impacts on fish and their life processes.

REFERENCES

1. Department of Fisheries and Oceans and Manitoba Natural Resources 1996 – "Manitoba Stream Crossing Guidelines for the Protection of Fish and Fish Habitat", May 1996.
2. Manitoba Transportation and Government Services 1995 – "Manual of Erosion and Sedimentation Control During Highway Construction".
3. Department of Fisheries and Oceans 1993 – "Land Development Guidelines for the Protection of Aquatic Habitat", Second Printing, September 1993.
4. Manitoba Transportation and Government Services 2004 – "Grading and Surfacing Specifications".

FIGURES

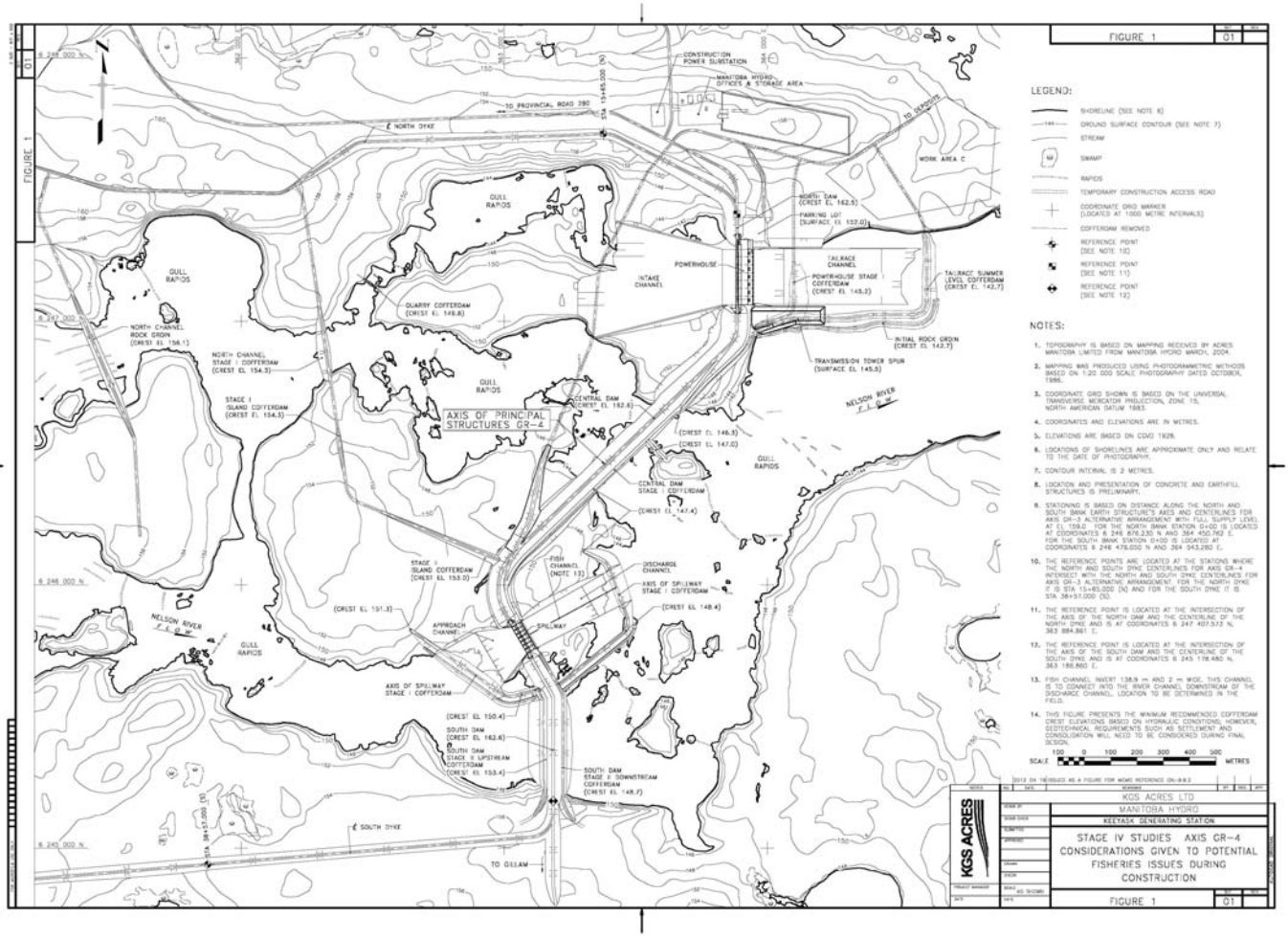
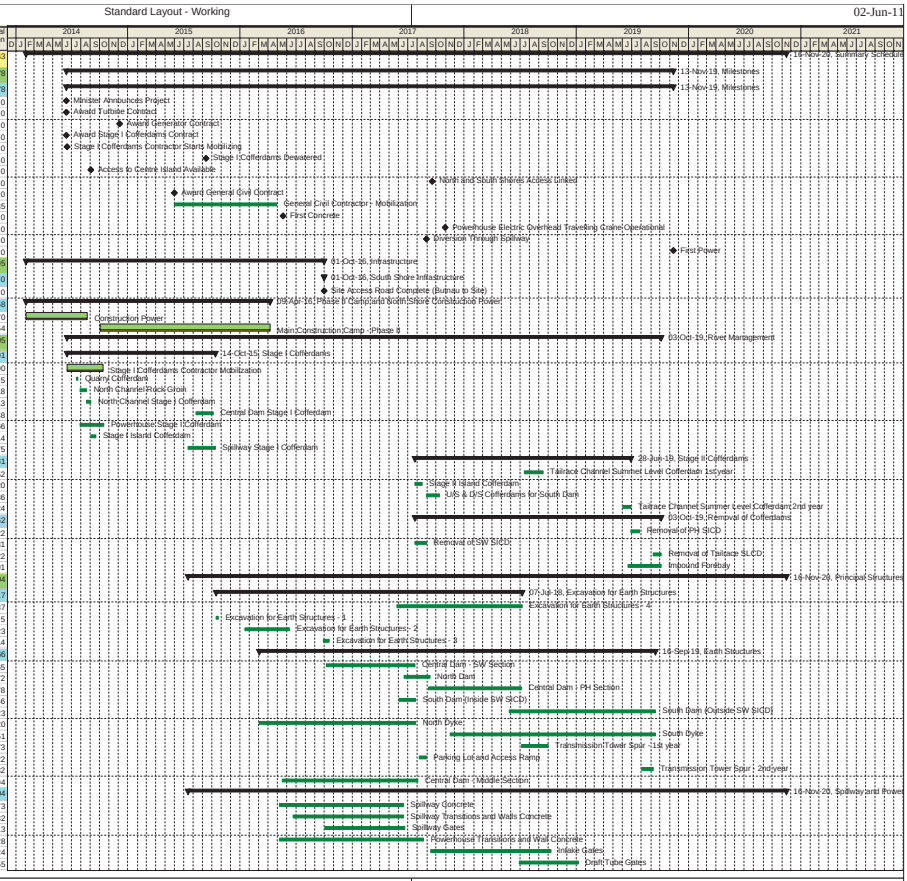


FIGURE 1
LEGEND:
SHORELINE (SEE NOTE 6)
GROUND SURFACE CONTOUR (SEE NOTE 7)
STREAM
SWAMP
RAPIDS
TEMPORARY CONSTRUCTION ACCESS ROAD
COORDINATE GRID MARKER (LOCATED AT 1000 METRE INTERVALS)
COFFERDAM REMOVED
REFERENCE POINT (SEE NOTE 10)
REFERENCE POINT (SEE NOTE 11)
REFERENCE POINT (SEE NOTE 12)
NOTES:
1. TOPOGRAPHY IS BASED ON MAPPING RECEIVED BY ACRES MANITOBA LIMITED FROM MANITOBA HYDRO MARCH, 2004.
2. MAPPING WAS PRODUCED USING PHOTOGRAMMETRIC METHODS BASED ON 1:20 000 SCALE PHOTOGRAPHY DATED OCTOBER, 1986.
3. COORDINATE GRID SHOWN IS BASED ON THE UNIVERSAL, TRANSVERSE MERCATOR PROJECTION, ZONE 15, NORTH AMERICAN DATUM 1983.
4. COORDINATES AND ELEVATIONS ARE IN METRES.
5. ELEVATIONS ARE BASED ON CGVD 1928.
6. LOCATIONS OF SHORELINES ARE APPROXIMATE ONLY AND RELATE TO THE DATE OF PHOTOGRAPHY.
7. CONTOUR INTERVAL IS 2 METRES.
8. LOCATION AND PRESENTATION OF CONCRETE AND EARTHFILL STRUCTURES IS PRELIMINARY.
9. STATIONING IS BASED ON DISTANCE ALONG THE NORTH AND SOUTH BANK EARTH STRUCTURE'S AXES AND CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT WITH FULL SUPPLY LEVEL AT EL. 159.0. FOR THE NORTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 876.230 N AND 364 450.762 E. FOR THE SOUTH BANK STATION 0+00 IS LOCATED AT COORDINATES 6 246 476.000 N AND 364 943.280 E.
10. THE REFERENCE POINTS ARE LOCATED AT THE STATIONS WHERE THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–4 INTERSECT WITH THE NORTH AND SOUTH DYKE CENTERLINES FOR AXIS GR–3 ALTERNATIVE ARRANGEMENT. FOR THE NORTH DYKE IT IS STA 15+45.000 (N) AND FOR THE SOUTH DYKE IT IS STA 38+57.000 (S).
11. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE NORTH DAM AND THE CENTERLINE OF THE NORTH DYKE AND IS AT COORDINATES 6 247 407.573 N, 363 884.861 E.
12. THE REFERENCE POINT IS LOCATED AT THE INTERSECTION OF THE AXIS OF THE SOUTH DAM AND THE CENTERLINE OF THE SOUTH DYKE AND IS AT COORDINATES 6 245 178.480 N, 363 186.860 E.
13. FISH CHANNEL INVERT 138.9 m AND 2 m WIDE. THIS CHANNEL IS TO CONNECT INTO THE RIVER CHANNEL DOWNSTREAM OF THE DISCHARGE CHANNEL. LOCATION TO BE DETERMINED IN THE FIELD.
14. THIS FIGURE PRESENTS THE MINIMUM RECOMMENDED COFFERDAM CREST ELEVATIONS BASED ON HYDRAULIC CONDITIONS. HOWEVER, GEOTECHNICAL REQUIREMENTS SUCH AS SETTLEMENT AND CONSOLIDATION WILL NEED TO BE CONSIDERED DURING FINAL DESIGN.
SCALE 100 0 100 200 300 400 500 METRES
CONSTRUCTION POWER SUBSTATION
MANITOBA HYDRO OFFICES & STORAGE AREA
TO PROVINCIAL ROAD 280
TO THOMPSON
WORK AREA C
NORTH DAM (CREST EL. 162.5)
PARKING LOT (SURFACE EL. 157.0)
GULL RAPIDS
POWERHOUSE
INTAKE CHANNEL
TAILRACE CHANNEL
POWERHOUSE STAGE I COFFERDAM (CREST EL. 145.2)
TAILRACE SUMMER LEVEL COFFERDAM (CREST EL. 142.7)
QUARRY COFFERDAM (CREST EL. 146.8)
NORTH CHANNEL ROCK GROIN (CREST EL. 156.1)
NORTH CHANNEL STAGE I COFFERDAM (CREST EL. 154.3)
STAGE I ISLAND COFFERDAM (CREST EL. 154.3)
INITIAL ROCK GROIN (CREST EL. 142.7)
TRANSMISSION TOWER SPUR (SURFACE EL. 145.5)
NELSON RIVER
AXIS OF PRINCIPAL STRUCTURES GR–4
CENTRAL DAM (CREST EL. 152.6)
(CREST EL. 146.3)
(CREST EL. 147.0)
CENTRAL DAM STAGE I COFFERDAM
(CREST EL. 147.4)
STAGE II ISLAND COFFERDAM (CREST EL. 153.0)
FISH CHANNEL (NOTE 13)
DISCHARGE CHANNEL
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL. 148.4)
(CREST EL. 151.3)
APPROACH CHANNEL
SPILLWAY
AXIS OF SPILLWAY STAGE I COFFERDAM
(CREST EL. 150.4)
SOUTH DAM (CREST EL. 162.6)
SOUTH DAM STAGE II UPSTREAM COFFERDAM (CREST EL. 153.4)
SOUTH DAM STAGE II DOWNSTREAM COFFERDAM (CREST EL. 148.7)
℄ SOUTH DYKE
TO GILLAM
6 243 000 N
6 244 000 N
6 245 000 N
6 246 000 N
6 247 000 N
KGS ACRES LTD
MANITOBA HYDRO
KEETASK GENERATING STATION
STAGE IV STUDIES AXIS GR–4 CONSIDERATIONS GIVEN TO POTENTIAL FISHERIES ISSUES DURING CONSTRUCTION
FIGURE 1
01

Summary Schedule | Standard Layout - Working | 02-Jun-11

Activity ID	Activity Type	Activity Name	Start	Finish	Original Duration
Summary Schedule			03-Feb-14	16-Nov-20	2043
Milestones			15-Jun-14	13-Nov-19	1978
Milestones			15-Jun-14	13-Nov-19	1978
S-03	Finish Milestone	Minister Announces Project		15-Jun-14	0
S-04	Finish Milestone	Award Turbine Contract		15-Jun-14	0
S-05	Finish Milestone	Award Generator Contract		05-Dec-14	0
S-08	Finish Milestone	Award Stage I Cofferdams Contract		15-Jun-14	0
S-09	Finish Milestone	Stage I Cofferdams Contractor Starts Mobilizing		16-Jun-14	0
S-10	Finish Milestone	Stage I Cofferdams Dewatered		12-Sep-15	0
S-11	Finish Milestone	Access to Centre Island Available		03-Sep-14	0
S-12	Finish Milestone	North and South Shores Access Linked		19-Sep-17	0
S-13	Finish Milestone	Award General Civil Contract		31-May-15	0
S-14	Level of Effort	General Civil Contractor - Mobilization	01-Jun-15	30-Apr-16	335
S-15	Finish Milestone	First Concrete		21-May-16	0
S-16	Finish Milestone	Powerhouse Electric Overhead Travelling Crane Operational		31-Oct-17	0
S-17	Finish Milestone	Diversion Through Spillway		29-Aug-17	0
S-18	Finish Milestone	First Power		13-Nov-19	0
Infrastructure			03-Feb-14	01-Oct-16	805
South Shore Infrastructure			01-Oct-16	01-Oct-16	0
S-23	Finish Milestone	Site Access Road Complete (Butnau to Site)		01-Oct-16	0
Phase II Camp and North Shore Construction Power			03-Feb-14	09-Apr-16	658
S-104	Task Dependent	Construction Power	03-Feb-14	21-Aug-14	170
S-24	Task Dependent	Main Construction Camp - Phase II	02-Oct-14	09-Apr-16	454
River Management			16-Jun-14	03-Oct-19	1595
Stage I Cofferdams			16-Jun-14	14-Oct-15	401
S-32	Task Dependent	Stage I Cofferdams Contractor Mobilization	16-Jun-14	11-Oct-14	100
S-34	Level of Effort	Quarry Cofferdam	17-Jul-14	22-Jul-14	5
S-37	Level of Effort	North Channel Rock Groin	28-Jul-14	18-Aug-14	18
S-38	Level of Effort	North Channel Stage I Cofferdam	18-Aug-14	03-Sep-14	13
S-39	Level of Effort	Central Dam Stage I Cofferdam	10-Aug-15	06-Oct-15	48
S-40	Level of Effort	Powerhouse Stage I Cofferdam	28-Jul-14	14-Oct-14	66
S-41	Level of Effort	Stage I Island Cofferdam	03-Sep-14	18-Sep-14	14
S-42	Level of Effort	Spillway Stage I Cofferdam	16-Jul-15	14-Oct-15	75
Stage II Cofferdams			25-Jul-17	28-Jun-19	581
S-44	Level of Effort	Tailrace Channel Summer Level Cofferdam 1st year	16-Jul-18	14-Sep-18	52
S-45	Level of Effort	Stage II Island Cofferdam	25-Jul-17	16-Aug-17	20
S-46	Level of Effort	U/S & D/S Cofferdams for South Dam	30-Aug-17	12-Oct-17	36
S-54	Level of Effort	Tailrace Channel Summer Level Cofferdam 2nd year	01-Jun-19	28-Jun-19	24
Removal of Cofferdams			25-Jul-17	03-Oct-19	662
S-48	Level of Effort	Removal of PH SICD	29-Jun-19	25-Jul-19	22
S-50	Level of Effort	Removal of SW SICD	25-Jul-17	29-Aug-17	31
S-52	Level of Effort	Removal of Tailrace SLCD	09-Sep-19	03-Oct-19	22
S-53	Level of Effort	Impound Forebay	18-Jun-19	03-Oct-19	91
Principal Structures			16-Jul-15	16-Nov-20	1604
Excavation for Earth Structures			16-Oct-15	07-Jul-18	817
S-106	Level of Effort	Excavation for Earth Structures - 4	26-May-17	07-Jul-18	337
S-56	Level of Effort	Excavation for Earth Structures - 1	16-Oct-15	21-Oct-15	5
S-66	Level of Effort	Excavation for Earth Structures - 2	18-Jan-16	10-Jun-16	123
S-96	Level of Effort	Excavation for Earth Structures - 3	30-Sep-16	17-Oct-16	14
Earth Structures			04-Mar-16	16-Sep-19	1066
S-108	Level of Effort	Central Dam - SW Section	08-Oct-16	24-Jul-17	65
S-57	Level of Effort	North Dam	19-Jun-17	12-Sep-17	72
S-58	Level of Effort	Central Dam - PH Section	06-Sep-17	06-Jul-18	78
S-60	Level of Effort	South Dam (Inside SW SICD)	02-Jun-17	27-Jul-17	46
S-61	Level of Effort	South Dam (Outside SW SICD)	28-May-18	16-Sep-19	223
S-62	Level of Effort	North Dyke	04-Mar-16	25-Jul-17	420
S-63	Level of Effort	South Dyke	16-Nov-17	16-Sep-19	551
S-64	Level of Effort	Transmission Tower Spur - 1st year	07-Jul-18	01-Oct-18	73
S-65	Level of Effort	Parking Lot and Access Ramp	07-Aug-17	31-Aug-17	22
S-94	Level of Effort	Transmission Tower Spur - 2nd year	01-Aug-19	07-Sep-19	32
S-98	Level of Effort	Central Dam - Middle Section	18-May-16	03-Aug-17	194
Spillway and Powerhouse			16-Jul-15	16-Nov-20	1604
S-68	Level of Effort	Spillway Concrete	09-May-16	17-Jun-17	173
S-71	Level of Effort	Spillway Transitions and Walls Concrete	23-Jun-16	15-Jun-17	132
S-72	Level of Effort	Spillway Gates	04-Oct-16	21-Jun-17	213
S-74	Level of Effort	Powerhouse Transitions and Wall Concrete	09-May-16	21-Aug-17	228
S-75	Level of Effort	Intake Gates	13-Sep-17	10-Oct-18	324
S-76	Level of Effort	Draft Tube Gates	29-Jun-18	07-Jan-19	155



Remaining Level of Effort | Primary Baseline | Remaining Work
Actual Level of Effort | Actual Work | Critical Remaining Work

Page 1 of 2

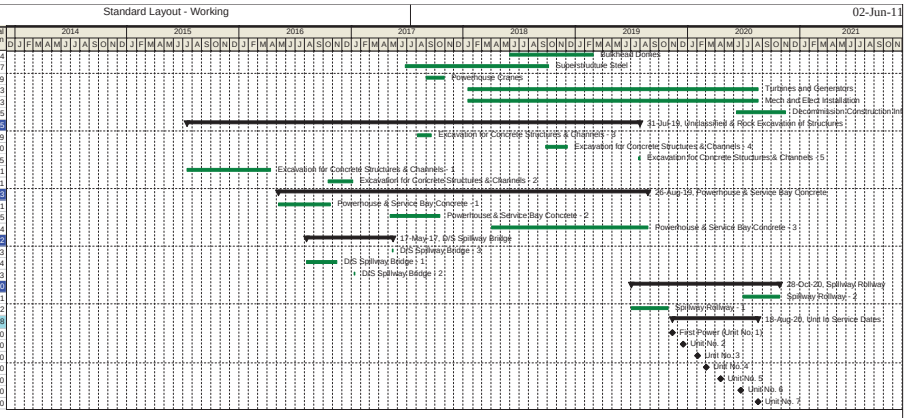
TASK filter: All Activities

(c) Primavera Systems, Inc.

Figure 2 (Sheet 1 of 2)

Summary Schedule | Standard Layout - Working | 02-Jun-11

Activity ID	Activity Type	Activity Name	Start	Finish	Original Duration
S-77	Level of Effort	Bulkhead Domes	01-Jun-18	01-Mar-19	224
S-78	Level of Effort	Superstructure Steel	27-Jun-17	06-Oct-18	387
S-79	Level of Effort	Powerhouse Cranes	02-Sep-17	31-Oct-17	49
S-80	Level of Effort	Turbines and Generators	16-Jan-18	18-Aug-20	783
S-81	Level of Effort	Mech and Elect Installation	16-Jan-18	18-Aug-20	783
S-82	Level of Effort	Decommission Construction Infrastructure	06-Jun-20	16-Nov-20	135
Unclassified & Rock Excavation of Structures			16-Jul-15	31-Jul-19	1215
S-107	Level of Effort	Excavation for Concrete Structures & Channels - 3	04-Aug-17	19-Sep-17	39
S-117	Level of Effort	Excavation for Concrete Structures & Channels - 4	27-Sep-18	06-Dec-18	60
S-127	Level of Effort	Excavation for Concrete Structures & Channels - 5	26-Jul-19	31-Jul-19	5
S-67	Level of Effort	Excavation for Concrete Structures & Channels - 1	16-Jul-15	12-Apr-16	221
S-97	Level of Effort	Excavation for Concrete Structures & Channels - 2	17-Oct-16	06-Jan-17	61
Powerhouse & Service Bay Concrete			09-May-16	26-Aug-19	993
S-73	Level of Effort	Powerhouse & Service Bay Concrete - 1	09-May-16	24-Oct-16	141
S-83	Level of Effort	Powerhouse & Service Bay Concrete - 2	08-May-17	16-Oct-17	135
S-93	Level of Effort	Powerhouse & Service Bay Concrete - 3	03-Apr-18	26-Aug-19	424
D/S Spillway Bridge			08-Aug-16	17-May-17	232
S-109	Level of Effort	D/S Spillway Bridge - 3	15-May-17	17-May-17	3
S-69	Level of Effort	D/S Spillway Bridge - 1	08-Aug-16	15-Nov-16	84
S-99	Level of Effort	D/S Spillway Bridge - 2	10-Jan-17	12-Jan-17	3
Spillway Rollway			02-Jul-19	28-Oct-20	400
S-100	Level of Effort	Spillway Rollway - 2	30-Jun-20	28-Oct-20	101
S-70	Level of Effort	Spillway Rollway - 1	02-Jul-19	30-Oct-19	102
Unit In Service Dates			13-Nov-19	18-Aug-20	278
S-84	Finish Milestone	First Power (Unit No. 1)		13-Nov-19	0
S-85	Finish Milestone	Unit No. 2		18-Dec-19	0
S-86	Finish Milestone	Unit No. 3		03-Feb-20	0
S-87	Finish Milestone	Unit No. 4		03-Mar-20	0
S-88	Finish Milestone	Unit No. 5		18-Apr-20	0
S-89	Finish Milestone	Unit No. 6		22-Jun-20	0
S-90	Finish Milestone	Unit No. 7		18-Aug-20	0

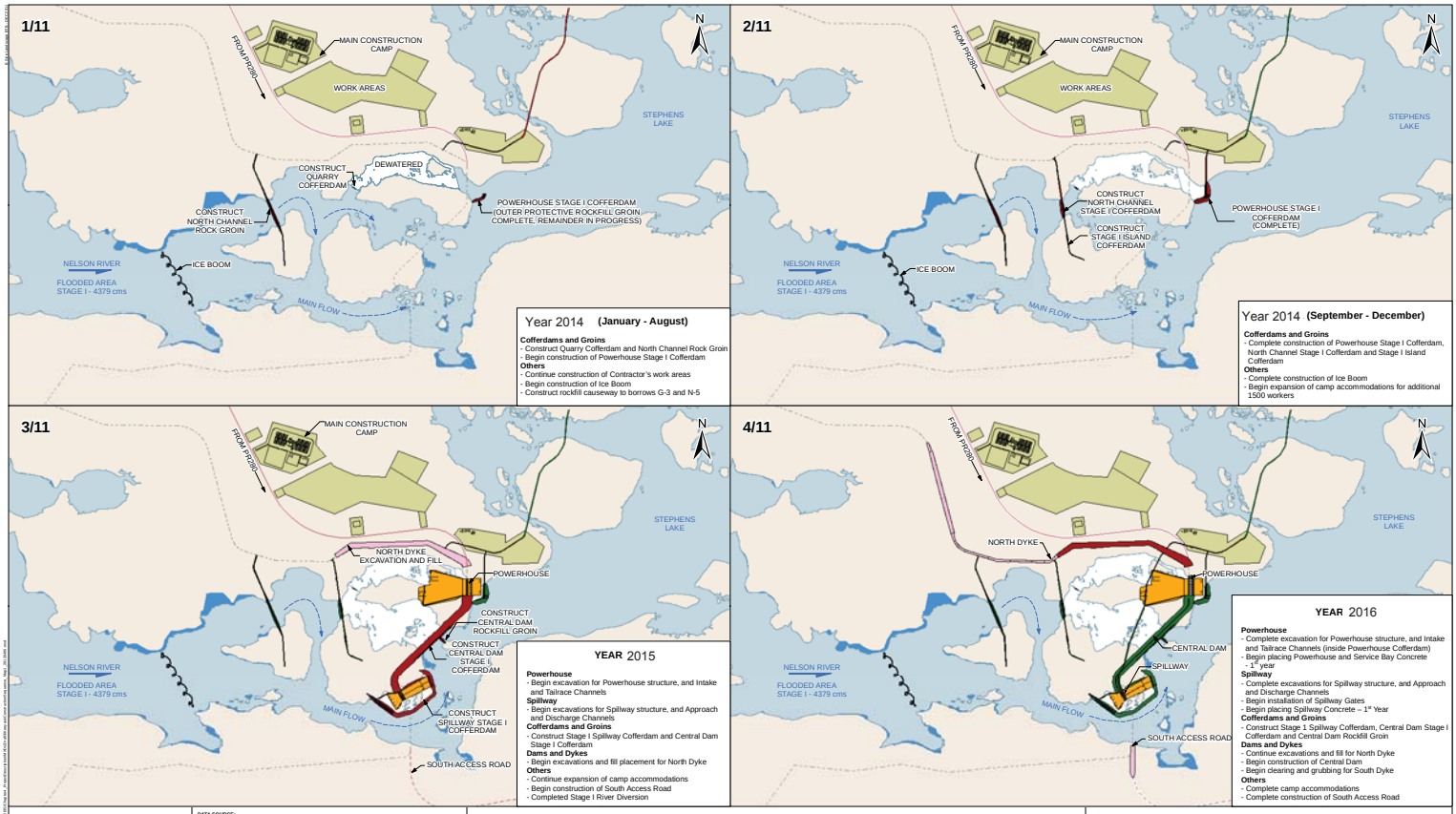


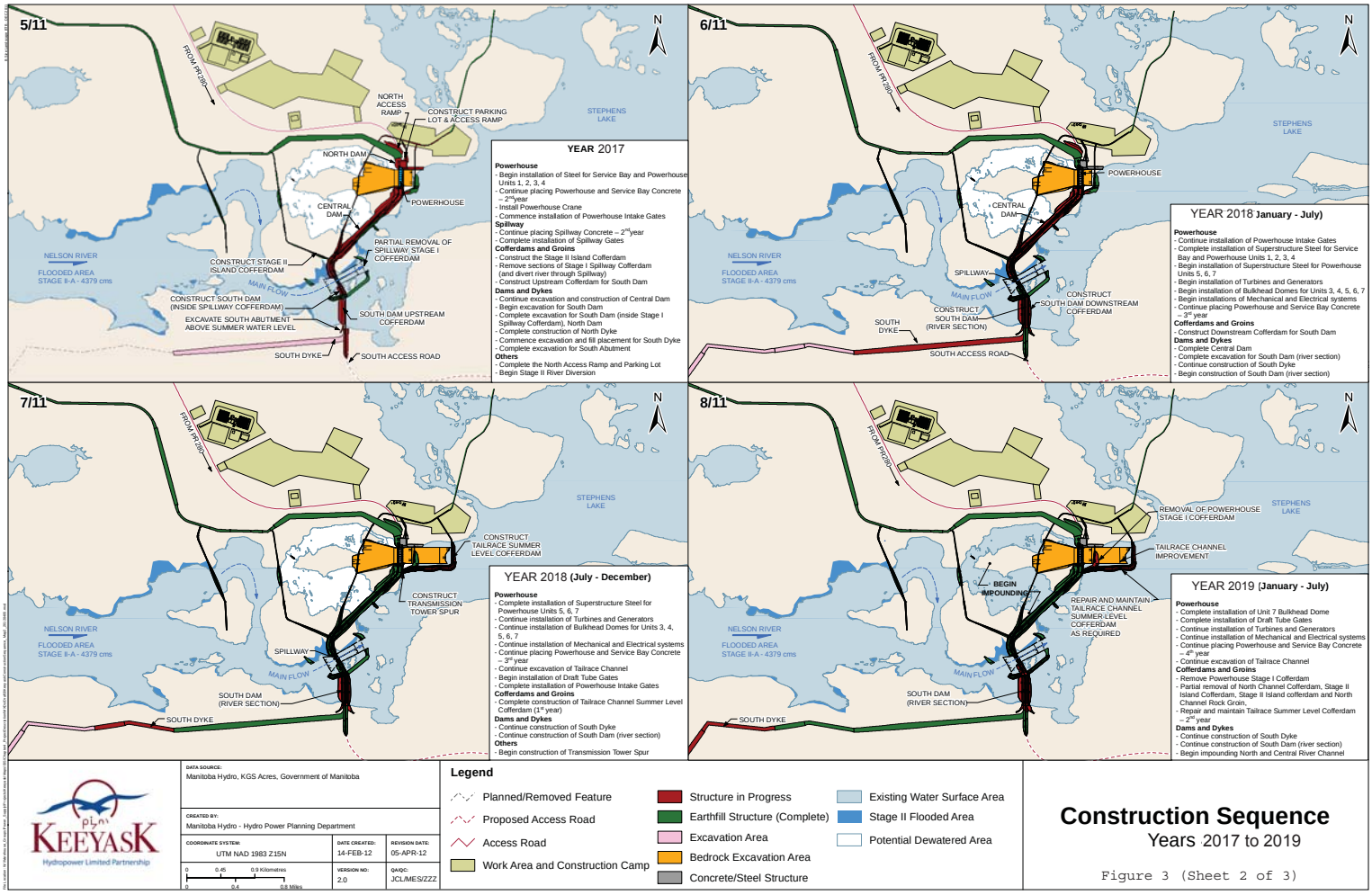
Remaining Level of Effort | Primary Baseline | Remaining Work
Actual Level of Effort | Actual Work | Critical Remaining Work

TASK filter: All Activities

(c) Primavera Systems, Inc.

Figure 2 (Sheet 2 of 2)


1/11
MAIN CONSTRUCTION CAMP
FROM PR280
WORK AREAS
STEPHENS LAKE
CONSTRUCT QUARRY COFFERDAM
DEWATERED
CONSTRUCT NORTH CHANNEL ROCK GROIN
POWERHOUSE STAGE I COFFERDAM (OUTER PROTECTIVE ROCKFILL GROIN COMPLETE, REMAINDER IN PROGRESS)
NELSON RIVER
ICE BOOM
FLOODED AREA STAGE I - 4379 cms
MAIN FLOW
N
Year 2014 (January - August)
Cofferdams and Groins
- Construct Quarry Cofferdam and North Channel Rock Groin
- Begin construction of Powerhouse Stage I Cofferdam
Others
- Continue construction of Contractor's work areas
- Begin construction of Ice Boom
- Construct rockfill causeway to borrows G-3 and N-5
2/11
MAIN CONSTRUCTION CAMP
FROM PR280
WORK AREAS
STEPHENS LAKE
CONSTRUCT NORTH CHANNEL STAGE I COFFERDAM
CONSTRUCT STAGE I ISLAND COFFERDAM
POWERHOUSE STAGE I COFFERDAM (COMPLETE)
NELSON RIVER
ICE BOOM
FLOODED AREA STAGE I - 4379 cms
MAIN FLOW
N
Year 2014 (September - December)
Cofferdams and Groins
- Complete construction of Powerhouse Stage I Cofferdam, North Channel Stage I Cofferdam and Stage I Island Cofferdam
Others
- Complete construction of Ice Boom
- Begin expansion of camp accommodations for additional 1500 workers
3/11
MAIN CONSTRUCTION CAMP
FROM PR280
STEPHENS LAKE
NORTH DYKE EXCAVATION AND FILL
POWERHOUSE
CONSTRUCT CENTRAL DAM ROCKFILL GROIN
CONSTRUCT CENTRAL DAM STAGE II COFFERDAM
NELSON RIVER
FLOODED AREA STAGE I - 4379 cms
MAIN FLOW
CONSTRUCT SPILLWAY STAGE I COFFERDAM
SOUTH ACCESS ROAD
N
YEAR 2015
Powerhouse
- Begin excavation for Powerhouse structure, and Intake and Tailrace Channels
Spillway
- Begin excavations for Spillway structure, and Approach and Discharge Channels
Cofferdams and Groins
- Construct Stage I Spillway Cofferdam and Central Dam Stage I Cofferdam
Dams and Dykes
- Begin excavations and fill placement for North Dyke
Others
- Continue expansion of camp accommodations
- Begin construction of South Access Road
- Completed Stage I River Diversion
4/11
FROM PR280
STEPHENS LAKE
NORTH DYKE
POWERHOUSE
CENTRAL DAM
SPILLWAY
NELSON RIVER
FLOODED AREA STAGE I - 4379 cms
MAIN FLOW
SOUTH ACCESS ROAD
N
YEAR 2016
Powerhouse
- Complete excavation for Powerhouse structure, and Intake and Tailrace Channels (inside Powerhouse Cofferdam)
- Begin placing Powerhouse and Service Bay Concrete - 1st year
Spillway
- Complete excavations for Spillway structure, and Approach and Discharge Channels
- Begin installation of Spillway Gates
- Begin placing Spillway Concrete – 1st Year
Cofferdams and Groins
- Construct Stage 1 Spillway Cofferdam, Central Dam Stage I Cofferdam and Central Dam Rockfill Groin
Dams and Dykes
- Continue excavations and fill for North Dyke
- Begin construction of Central Dam
- Begin clearing and grubbing for South Dyke
Others
- Complete camp accommodations
- Complete construction of South Access Road
KEEYASK
Hydropower Limited Partnership
DATA SOURCE:
Manitoba Hydro, KGS Acres, Government of Manitoba
CREATED BY:
Manitoba Hydro - Hydro Power Planning Department
COORDINATE SYSTEM:
UTM NAD 1983 Z15N
DATE CREATED:
14-FEB-12
REVISION DATE:
05-APR-12
VERSION NO:
2.0
QA/QC:
JCL/MES/ZZZ
0 0.45 0.9 Kilometres
0 0.4 0.8 Miles
Legend
Planned/Removed Feature
Proposed Access Road
Access Road
Work Area and Construction Camp
Structure in Progress
Earthfill Structure (Complete)
Excavation Area
Bedrock Excavation Area
Concrete/Steel Structure
Existing Water Surface Area
Stage I Flooded Area
Potential Dewatered Area
Construction Sequence
Years 2014 to 2016
Figure 3 (Sheet 1 of 3)



KEEYASK Hydropower Limited Partnership

DATA SOURCE: Manitoba Hydro, KGS Acres, Government of Manitoba

CREATED BY: Manitoba Hydro - Hydro Power Planning Department

COORDINATE SYSTEM:	DATE CREATED:	REVISION DATE:
UTM NAD 1983 Z15N	14-FEB-12	05-APR-12
0 0.45 0.9 Kilometres / 0 0.4 0.8 Miles	VERSION NO: 2.0	QA/QC: JCL/MES/ZZZ

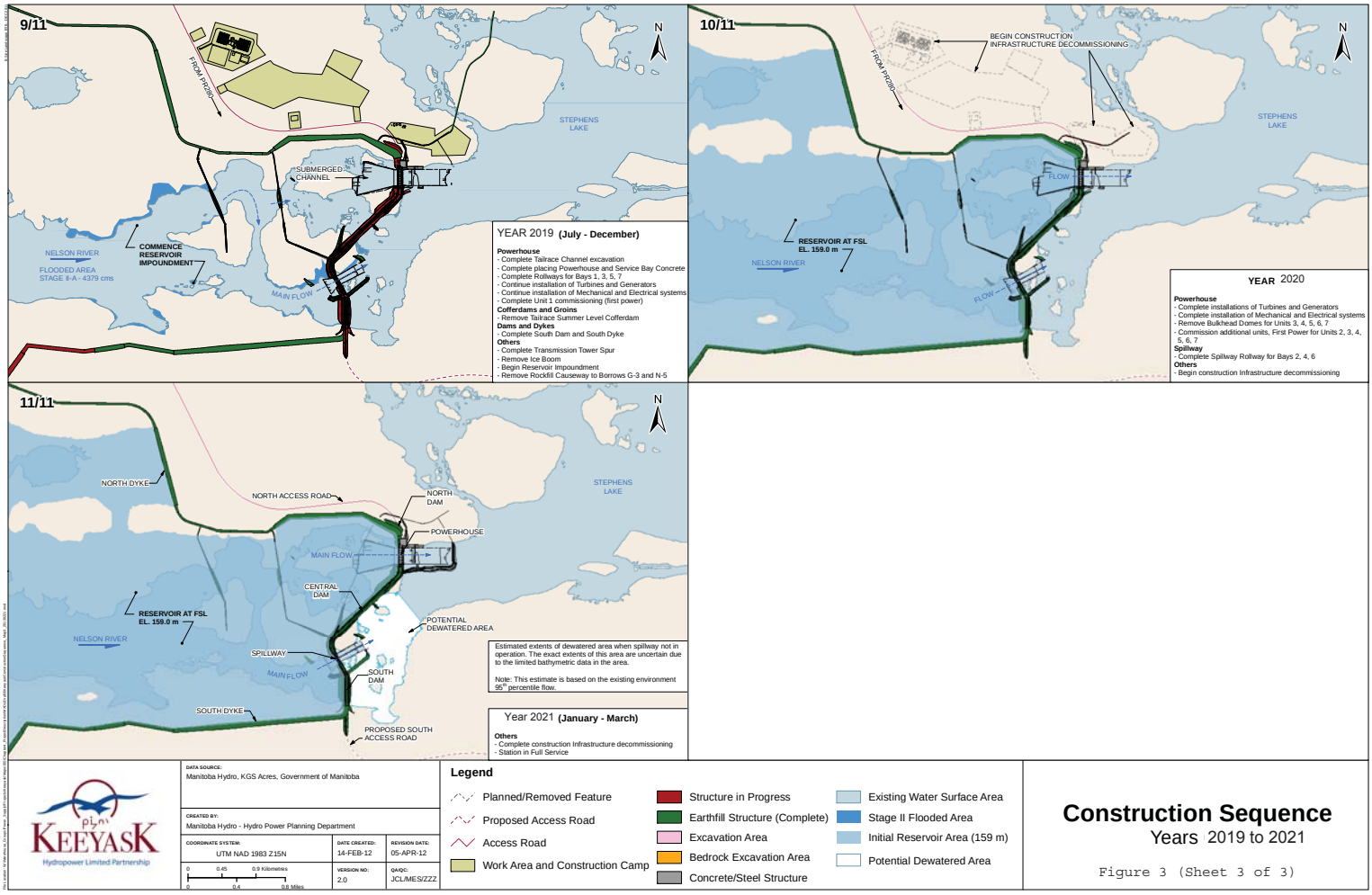
Legend

- Planned/Removed Feature
- Proposed Access Road
- Access Road
- Work Area and Construction Camp
- Structure in Progress
- Earthfill Structure (Complete)
- Excavation Area
- Bedrock Excavation Area
- Concrete/Steel Structure
- Existing Water Surface Area
- Stage II Flooded Area
- Potential Dewatered Area

Construction Sequence

Years 2017 to 2019

Figure 3 (Sheet 2 of 3)



KEEYASK Hydropower Limited Partnership

DATA SOURCE: Manitoba Hydro, KGS Acres, Government of Manitoba

CREATED BY: Manitoba Hydro - Hydro Power Planning Department

COORDINATE SYSTEM:	DATE CREATED:	REVISION DATE:
UTM NAD 1983 Z15N	14-FEB-12	05-APR-12
0 0.45 0.9 Kilometres / 0 0.4 0.8 Miles	VERSION NO: 2.0	QA/QC: JCL/MES/ZZZ

Legend

- Planned/Removed Feature
- Proposed Access Road
- Access Road
- Work Area and Construction Camp
- Structure in Progress
- Earthfill Structure (Complete)
- Excavation Area
- Bedrock Excavation Area
- Concrete/Steel Structure
- Existing Water Surface Area
- Stage II Flooded Area
- Initial Reservoir Area (159 m)
- Potential Dewatered Area

Construction Sequence

Years 2019 to 2021

Figure 3 (Sheet 3 of 3)

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-9.8.3

Rev. 1, March 21, 2012

Preliminary Review of Building Islands for Bird Habitat

Manitoba Hydro File 00195-11150-0004_02

Prepared by (P. PANTEL) March 21, 2012
KGS Acres Ltd.

Checked by (J. ROGERS) March 21, 2012
KGS Acres Ltd.

Approved by (I. R. Dewar) March 21, 2012
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

KEEYASK GENERATION PROJECT

STAGE IV STUDIES - ENVIRONMENTAL MITIGATION AND COMPENSATION MEASURES

PRELIMINARY REVIEW OF BUILDING ISLANDS FOR BIRD HABITAT

REV 1

DELIVERABLE GN-9.8.3

MANITOBA HYDRO FILE: 00195-11150-0004_02

2012/03/21

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

KGS ACRES

Hydropower Limited Partnership

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Table 2 Cost Estimate of Bird Habitat Options

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Figure 3 Velocities in Tailrace for Flow of 3850 m^3/s and Kettle Forebay at el 141.1 m

1 INTRODUCTION

The site of the proposed Keeyask Generating Station (GS) is approximately 730 km north of Winnipeg, at the head of Stephens Lake on the Nelson River. The proposed Axis GR-4 for the structures crosses the Nelson River at the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam. A site location plan is shown in Figure 1.

This memorandum provides a preliminary review of the conceptual designs for providing nesting habitats for the birds at the Keeyask GS site. It also discusses the location of the borrow sources and the schedule for constructing the habitat.

A detailed design with a review of flow velocities, ice loading, and detailed volume calculations, stability and foundation bearing capacity analyses was not developed in this study. Detailed studies will be undertaken during final design should any of the concepts discussed herein be included in the project's finalized arrangement.

2 BACKGROUND

Manitoba Hydro has retained the services of Stantec Inc. (Stantec), formerly TetrES Consultants Inc. Stantec's role on the Keeyask GS studies is to conduct the terrestrial environmental impact studies. As part of their study, Stantec indicated that construction of the Keeyask GS will likely have an impact on the nesting colony of gulls at Gull Rapids.

Stantec estimated that there are currently about 1,000 nesting pairs of gulls at Gull Rapids, and that the majority of the birds nest on some of the islands in the south channel of Gull Rapids, downstream of the Principal Structures. It is anticipated that an adequate number of islands may develop in the south channel once it dries up after construction to provide adequate nesting habitat; however the suitability and the quantity will not be known until after the project is completed. Based on the current engineering design of the Principal Structures, the location of the candidate islands that may be formed in the south channel of Gull Rapids are shown in Figure 1. The uncertainty of the suitability of islands (post-construction) is based on a number of factors, one of which is the limited bathymetric information available within the area in question.

It was originally proposed that the course of action would be to implement a post-project monitoring program for a three-year period to determine if the downstream islands would provide adequate gull nesting habitat. However, it was believed that should the

post-project monitoring indicate that there is insufficient habitat; three options may be considered for the creation of nesting habitat:

1. Construction of an artificial island either within Stephens Lake or within the forebay of the Keeyask GS (constructed in-the-dry prior to impoundment).
2. Conversion of one of the small barges used for aquatic habitat rehabilitation into an artificial floating island.
3. Leaving a remnant portion of the Tailrace Summer Level Cofferdam to form an artificial island along the south side of the Tailrace Channel.

3 DESIGN OPTIONS

3.1 OPTION 1 - ARTIFICIAL ISLAND CONSTRUCTION

Stantec indicated that if an island were to be constructed, the preferred location would be upstream of the Keeyask GS as it would be in close proximity to the existing habitat. As such, the construction of an artificial island within Stephens Lake has not been given further consideration in this memorandum. Furthermore, after some discussion between Stantec, KGS Acres and Manitoba Hydro (MH), it was deemed to be more practical to construct the island within the Keeyask forebay area prior to impoundment and therefore the island would be in place prior to the Post-Project Monitoring.

There are two potential locations for the forebay area island, shown as Alternatives 1 and 2 on Figure 1. Both locations would involve the construction of a permanent island with an exposed surface area of about 0.4 ha above the Full Supply Level (FSL). The width of the island would be about 40 m and the length would be about 100 m. Should Stantec indicate that the birds may be attracted to longer elongated nesting areas, then the width and the length may be adjusted accordingly.

The conceptual design details of the alternatives are shown in Figure 2. The body of the island would be clay or impervious fill, sourced immediately adjacent to the site location. The perimeter of the island would be protected by erosion using a coarse filter of Class 4 material, Class 5 riprap bedding, and Class 6 riprap. A 0.5 m layer of Class 2 fill would be used as a granular cap on top of the island to create a hard, relatively smooth surface that is suitable habitat for birds in the Gull Rapids area. Since the island would be a permanent structure, the materials used for construction would have to meet certain gradational requirements as outlined in the Geotechnical Design Criteria [Ref 1]. A summary of the estimated fill quantities that would be required for the construction of Option 1, Alternatives 1 and 2 is shown in Table 1.

The crest elevation of the island would be set at el 160.75 m and the perimeter of the island would be set at el 160.5 m to prevent ponding. The island would be about 1.5 m above the FSL of el 159 m. At the site for Alternative 1, the island would have a height of about 5.5 m above the original ground surface. At the site for Alternative 2, the island would have a height of about 1.5 m above the original ground surface. It is envisioned that some large cobbles and small boulders could be placed on top of the crest to provide some topographical relief and shelter to simulate the existing nesting islands.

Conventional construction techniques are generally applicable to this structure. Construction will be in-the-dry with material placed in lifts and compacted.

The current hydraulic design suggests that the water flow in this area can be improved by lowering the North Rock Groin and other forebay hydraulic improvements. These hydraulic improvements may be required regardless of the construction of artificial islands at either the Alternative 1 or Alternative 2 sites.

Alternative 1

Alternative 1 is located near the southern limit of the freeboard/road section of the North Channel Rock Groin, as shown on Figure 1. The topography at the site varies slightly from el 154.5 m to el 155.5 m, but the bulk of the site is at el 155 m, which is 4 m below the FSL of el 159.0 m. Constructing the island at this site would have a negligible impact on the flows toward the Powerhouse or Spillway structures.

Construction would commence in the summer prior to First Power following the receding of spring flows. Construction would be completed prior to impounding the forebay and prior to the full lowering of the North Channel Rock Groin.

Alternative 1 can be accessed across the crest of the North Channel Rockfill Groin. Some of the excavated materials from the lowering of the North Channel Rock Groin may possibly be used for the construction of Alternative 1.

Alternative 2

A second site (Alternative 2) was considered along the south shoreline, directly to the south of the large island, as shown on Figure 1. This site was identified because it is a topographic high. The 2010 LiDAR maps show contour elevations varying from about el 158.5 m up to el 159.5 m. This places the base of the ground surface between 0.5 m below, to 0.5 m above the FSL (el 159.0 m). Constructing an artificial island at this site would minimize the fill quantities and place the island in an area where it would have a minimal influence on the flows towards the Powerhouse and Spillway structures.

The site for Alternative 2 could be accessed prior to river closure by using the South Access Road and the South Dyke alignment or via the Stage II South Cofferdams following Stage II River Closure. For this alternative, construction could begin as soon as access to the site is available, and would be completed prior to forebay impoundment.

3.2 OPTION 2 – ARTIFICIAL FLOATING ISLAND

Several barges and tugboats may be purchased by MH for some of the mitigation measures currently under consideration for the Keeyask GS. It is assumed for Option 2 that one of the barges that is purchased is converted into an artificial floating island for use as a bird nesting area. This "barge island" would be portable, and would removed from the river in late fall and re-installed in early spring. A tugboat used for other mitigation measures, is assumed to be used for the removal and installation of the barge. One advantage of this option is that the barge could be positioned at different locations each year in an effort to identify areas that will have the greatest effectiveness to attract birds.

It is envisioned that the barge would be placed downstream of the Tailrace Channel, with possible locations indicated on Figure 1. The barge is 15 m long and 4 m wide. The barge will be capped with either waste concrete or shotcrete to form a hard surface suitable for nesting habitat.

The barge would be anchored at its four corners using a chain of suitable strength; the other end of each of the four main anchor chains would attach to chains with their ends embedded into concrete anchors. Each concrete anchor would be of suitable size to be moved by a tugboat, while the combined anchor point would be able to immobilize the barge. Anchors would be placed fore and aft to stabilize the vessel. Each anchor would be equipped with a solar-powered LED anchor light with a magnetic mount and switch mounted on a buoy. A permanent hazard beacon would be attached to the barge to notify boaters of the barge location.

The barge would not be installed along commonly navigated routes. Appropriate signage would be placed in the area to warn boaters and others of the use of the barge as a bird habitat so as to not interfere with the vessel's intended operation. A photo of a representative tugboat is shown in Appendix A.

To prevent damage of the barge due to winter ice effects, it is assumed that the barge would be removed each winter prior to ice formation and reinstalled following melting of the ice each spring. Each anchor chain would be attached to the barge by a clevis allowing the barge to be disconnected from the anchors to allow for ease of relocation. It is assumed that the anchors and anchor chains would be left on the lake bottom over the winter.

Installation and removal of the barge would require use of divers to connect and disconnect the chains from the concrete anchors.

It is envisioned that the barge would be stored at and mobilized from a barge landing in the vicinity of the downstream boat-launch. Alternatively, it could remain afloat at a location where ice action is not likely to damage it.

Development of Option 2 would take place following impoundment when the habitat rehabilitation works are complete.

3.3 OPTION 3 – TAILRACE SUMMER LEVEL COFFERDAM REMNANT ISLAND

It was proposed that consideration be given to leaving a portion of the Tailrace Summer Level Cofferdam in place to serve as permanent bird island habitat. A portion of the cofferdam was selected, as shown in Figure 1, to study the effect of a cofferdam remnant remaining downstream of the Powerhouse on the economics of the project. The remnant would be located at least 100 m east of the Transmission Tower Spur to minimize impacts on hydraulic outflow from the Tailrace Channel.

The ends of the remnant island would be treated with a granular filter, and the perimeter would be protected from erosion using riprap armour. The crest would be capped in such a way to promote bird nesting habitat by using materials with rough jagged edges. This area could be used as a dump site for waste concrete to achieve the desired shapes and elevations.

A hydraulic review of this option was undertaken. The review concluded that the remnant island would adversely affect Powerhouse tailwater levels by disrupting flow patterns downstream of the Transmission Tower Spur. Figure 3 shows the flow from the Powerhouse Tailrace Channel expanding to the south downstream of the Transmission Tower Spur; the presence of a remnant island would interrupt these flow patterns.

Figure 3 also shows the flow expanding from the Powerhouse Channel along the south shoreline to the southeast of the Transmission Tower Spur. Whitefish spawning habitat is planned along this shoreline, and the presence of a remnant island would negatively affect the flow to this area.

For these reasons, this option is not being explored further at this time and a cost estimate of this option was not developed. Should there be an interest in further exploring this option in the final design stage of the project, it is recommended that a numerical model study be undertaken to quantify the impacts of the proposed island.

[redacted]

5 SUMMARY AND RECOMMENDATIONS

This memorandum provides a preliminary review of three conceptual designs for providing bird nesting habitats at the Keeyask GS site:

1. Construction of an artificial island either within Stephens Lake or within the forebay of the Keeyask GS (constructed in-the-dry prior to impoundment).
2. Conversion of one of the small barges used for aquatic habitat rehabilitation into an artificial floating island.
3. Leaving a remnant portion of the Tailrace Summer Level Cofferdam to form an artificial island along the south side of the Tailrace Channel.

[redacted]

If the construction of bird island habitat upstream in the forebay (i.e., Option 1) is included in the project's finalized arrangement, detailed studies reviewing flow velocities, ice loading, and detailed volume calculations, stability and foundation bearing capacity analyses should be developed.

REFERENCES

1. KGS Acres Ltd. (2010), "Keeyask Generating Station, Stage IV Studies, Geotechnical Design Criteria", GN-4.1.3, Manitoba Hydro File 00195-06310-0002_01.
2. KGS Acres Ltd. (2009), "Keeyask Generating Station, Stage IV Studies, Basis for the 2007 Direct Cost Estimate", GN-7.1, Manitoba Hydro File 00195-04220-0057_00.

TABLES

Table 1
Keeyask Generating Station
Stage IV Studies, Axis GR-4
Fill Requirements – Construction of Artificial Islands

Alternative	Fill Class[1]				
	Impervious Fill (m^3)	Granular	Processed Granular	Processed Rockfill	Common Rockfill
		Class 2 Sand Fill (m^3)	Class 4 Granular (m^3)	Class 5 Riprap Bedding (m^3)	Class 6 Rockfill (m^3)
Alternative 1	21,100	2,200	4,700	11,700	9,100
Sources	Immediately adjacent to site	Deposit G-1 or Deposit G-3	SCP[2]	SCP[2]	SCC[3]
Alternative 2	4,000	2,100	1,900	2,700	5,200
Sources	Immediately adjacent to site	Deposit G-1 or Deposit G-3	Quarry Q-1	Quarry Q-1	Quarry Q-1

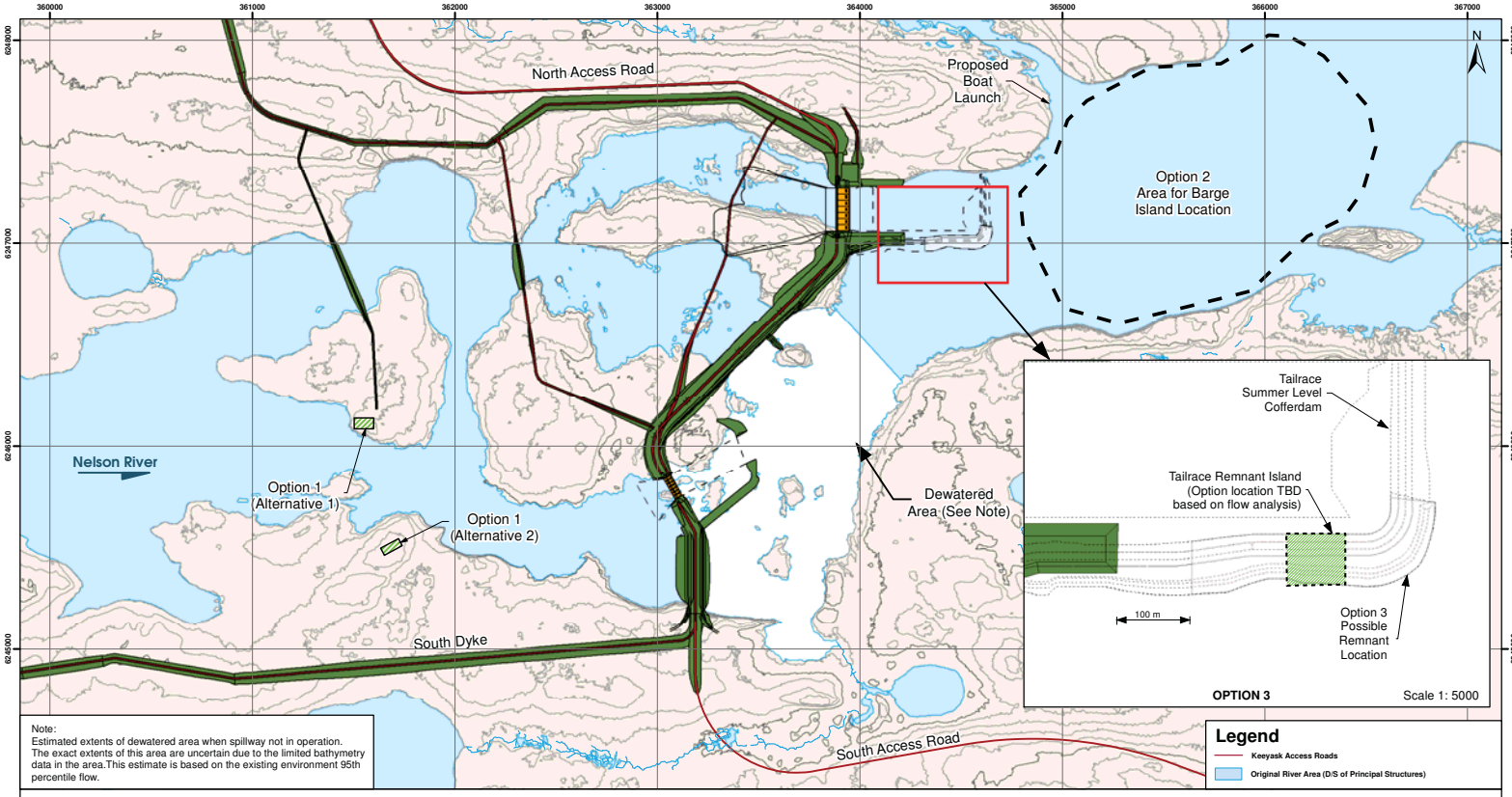
Notes:

(1) Based on the neat-line bid item volume take-offs.

(2) SCP - Stockpiled at Crusher Plant.

(3) SCC - Stockpiled at Central Cofferdam Area.

FIGURES



0 0.25 0.5 1 Kilometers

0 0.25 0.5 1 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Existing Gull environment shoreline, infrastructure, contours and dewatered polygon provided by Manitoba Hydro, 2011.
2. Habitat data provided by KGS Acres, 2011.

Contours may not necessarily conform to shorelines depicted, due to differing scales.

FOR GENERAL REFERENCE ONLY

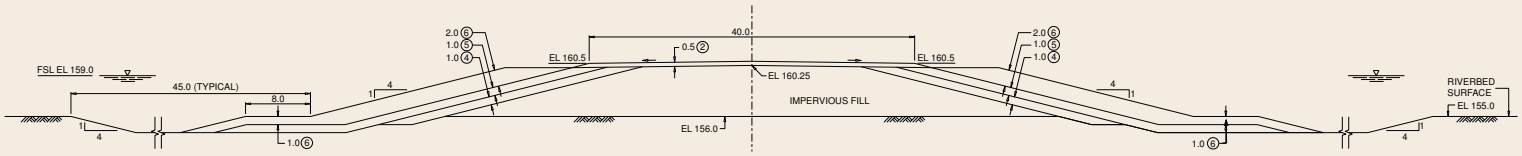
KEEYASK STAGE IV STUDIES AXIS GR-4 BIRD ISLAND HABITAT OVERVIEW OF OPTIONS

Figure 1

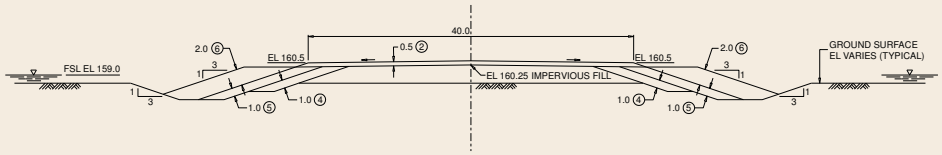
Created By: KGS ACRES

File Location: P:\P\10008\16\shared\deliver\GR4\GN_9_8\GN_9_8_3\rev_1\CAD\GIS

Date Created: January 05, 2012



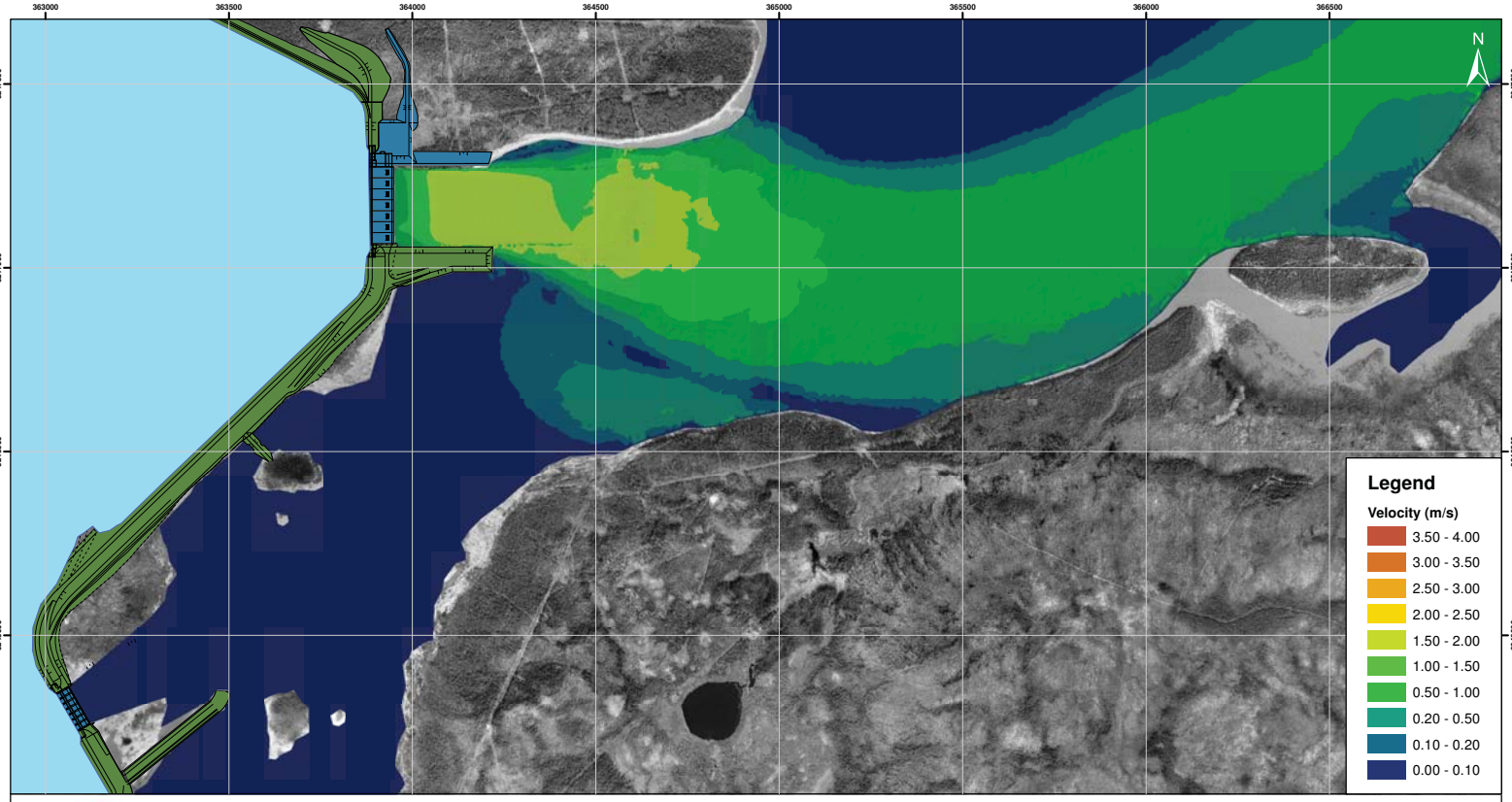
OPTION 1 - ALTERNATIVE 1
N.T.S



OPTION 1 - ALTERNATIVE 2
N.T.S.

KEEYASK STAGE IV STUDIES AXIS GR-4
BIRD ISLAND HABITAT OPTION 1 DETAILS

Figure 2



0 250 500 1,000 Meters

0 500 1,000 2,000 3,000 Feet

Projection: Universal Transverse Mercator Zone 15N, NAD83

Data Sources:
1. Aerial Photography provided by Manitoba Hydro, 2007.
2. Shoreline created by KGS Acres for reference only.
3. Velocity data provided by KGS Acres, 2011.

FOR GENERAL REFERENCE ONLY

KEEYASK STAGE IV STUDIES AXIS GR-4 VELOCITIES IN TAILRACE FOR FLOW OF 3850 m^3/s AND KETTLE FOREBAY AT EL 141.1 m

Figure 3

Created By: KGS ACRES

File Location: P:\P\10008\16\shared\deliver\GR4\GN_9_8\GN_9_8_3\rev_1\CAD\GIS

Date Created: January 11, 2012

APPENDIX A

Photo 1: Typical Tugboat

Source:

Stark, Joseph P. (Great Lakes Shipyard). Message to David Ranta (KGS ACRES) [Email]. "Truckable Workboat and Barges". November 19, 2009 2:28 PM.

KEEYASK GENERATION PROJECT

STAGE IV STUDIES – ENVIRONMENTAL MITIGATION AND COMPENSATION MEASURES

FIRST ORDER ASSESSMENTS OF DEEP AND SHALLOW ROCKY SHOALS ENHANCEMENTS FOR IMPROVING CONDITIONS IN THE RESERVOIR FOR FISH

REVISION 1

DELIVERABLE GN 9.8.10

MANITOBA HYDRO FILE: 00195-11100-0201_05

2013/03/22

PREPARED FOR:
HYDRO POWER PLANNING DEPARTMENT
POWER PROJECTS DEVELOPMENT DIVISION
POWER SUPPLY

PREPARED BY:

KGS ACRES

Hydropower Limited Partnership

Manitoba Hydro
Hydro Power Planning Department
Power Projects Development Division

Keeyask Generating Station
Stage IV Studies - Axis GR-4

Design Memorandum GN-9.8.10

Rev. 1, March 22, 2013

First Order Assessments of Deep and Shallow Rocky Shoals Enhancements for Improving Conditions in the Reservoir for Fish

Manitoba Hydro File 00195-11100-0201_05

Prepared by _____ for (E. Neufeld) 2013.03.22
KGS Acres Ltd.

Checked by _____ (P. PANTEC) 2013.03.22
KGS Acres Ltd.

Approved by _____ (I.R. Dewar) 2013.03.22
KGS Acres Ltd.

Accepted by _____
Manitoba Hydro

KGS Acres Ltd.
Winnipeg, Manitoba

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Figure 2 Inventory of Possible Shallow and Deep Rocky Shoal Habitat in Forebay

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Figure 3.2 Velocities in Impounded Forebay for a 95th Percentile Flow at el. 159 m, 4430 m^3/s Total Flow, 430 m^3/s Flow Through Spillway

Figure 4 Potential Shallow and Deep Rocky Shoal Habitat Locations in Forebay

Figure 5 Revised Preferred Habitat Locations

Appendix B

Figure B-1 Mineral Sediment Deposition – Year 1 After Impoundment

Figure B-2 Mineral Sediment Deposition – Year 5 After Impoundment

Figure B-3 Mineral Sediment Deposition – Year 15 After Impoundment

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Table 2 Summary of Potential Habitat Locations in Forebay (2012$)

Table 3 Ranking of Potential Habitat Locations in Forebay (Developed by North-South Consultants, Inc. [Ref 3])

Table 4 Summary of Preferred Habitat Locations in Forebay (2012$)

1 INTRODUCTION

The site of the proposed Keeyask Generating Station (Keeyask GS) is approximately 730 km north of Winnipeg at the head of Stephens Lake on the Nelson River. Stephens Lake is the reservoir for the existing Kettle GS, the upstream limit of which is Gull Rapids. The proposed GR-4 axis for the structures crosses the Nelson River at the rapids. This places the Principal Structures approximately 50 km downstream of Split Lake and 30 km west of Gillam. The access road from Gillam to the Butnau Dam on the south shore of Stephens Lake terminates 18 km to the east of the site. A site location plan is shown in Figure 1.

The purpose of this memorandum is to conduct a preliminary assessment of mitigation measures to improve conditions in the forebay for fish.

Draft 1 of this memorandum was issued to Manitoba Hydro on August 11, 2009. An additional nine sites were submitted for consideration in an email from Dave Magnusson of Manitoba Hydro to Jim Smith of KGS ACRES on November 12, 2009 [Ref 1].

Draft 2 of this memorandum was issued to Manitoba Hydro on May 26, 2010 and included an assessment of these sites with the same criteria as used in Draft 1. In Draft 2, the areas of shallow and deep rocky shoals were revised to 1,000 m^2 for all sites to conform to criteria outlined in an email from Dave Magnusson of Manitoba Hydro to Jim Smith of KGS ACRES on December 21, 2009 [Ref 2]. The reduction in area resulted in a significant cost decrease from Draft 1, which is addressed in more detail in Section 7 of this report.

Revision 0 of this memorandum incorporated the comments made by North/South Consultants, Inc. (North/South) on the Draft 2 version of the memorandum [Ref 3]. North/South identified preferred sites for habitat development as summarized in Table 3, shown in Figure 5 and discussed in Section 6. The cost estimate for developing these sites was revised in 2012 dollars as discussed in Section 7 and presented in Table 4.

The current version of this memorandum incorporates revised depth criteria for both the shallow and deep rocky shoals as defined by North/South Consultants, Inc. in their comments on Rev 0 of this document. The changes in the depth criteria required some modifications to the locations for the preferred sites, but these changes have not impacted the estimated cost for developing these sites. The footprints of the revised preferred sites illustrated in the Figures have also been revised.

This memorandum has been organized into two parts: Part One includes the analysis and cost estimate included in Draft 2 of this memorandum, and Part Two presents the analysis and cost estimate resulting from the comments received.

2 GENERAL ARRANGEMENT

The general arrangement of the proposed Keeyask GS is shown in Figure 2. The layout corresponds to that proposed for Axis GR-4, with a Full Supply Level (FSL) at el 159.0 m.

The selected arrangement consists of two long riverbank dykes on the north and south sides of the river, which are linked to the Principal Structures that span the Nelson River at Gull Rapids. The Principal Structures consist of a Powerhouse Complex constructed on the north side of the river, which will be connected to the North Dyke through a 99 m long North Dam and a short concrete transition structure. The Spillway will be located on South Island, approximately 1.5 km south of the Powerhouse. A Central Dam will connect the Spillway and the Powerhouse structures. The Spillway will be connected to the South Dam by means of a short concrete transition structure. The South Dam, approximately 565 m long, will connect to the South Dyke on the south riverbank.

3 HABITAT BACKGROUND

Construction of the Keeyask GS will result in the alteration of some areas of the existing walleye and lake whitefish spawning habitat in the reservoir. Over time, suitable habitat would likely develop in these areas as surface organic and mineral material is eroded away; however, to provide for habitat immediately after impoundment, spawning reefs will be constructed “in the dry” prior to impoundment. The creation of boulder/cobble/gravel habitat in shallow (2-4 m depth) and deep (3.5-6 m depth) water would, in addition to providing spawning habitat for walleye and lake whitefish, also provide rearing and foraging habitat. This would improve habitat diversity within the newly-formed reservoir. Where sufficient velocity exists, reefs are also being created in deep water. Criteria for spawning habitat are provided in Table 1. A photo of a constructed walleye spawning shoal is found in Appendix A.

Locations identified as being suitable based on the criteria in Table 1 were further refined according to the following criteria: whether their respective location is adjacent to known or suspected present-day spawning habitats; how likely it is to be exposed to fine particulate sedimentation post-impoundment; and whether it is a minimum distance of 3 km upstream of the proposed locations of the GS and spillway intake structures so as to minimize entrainment and downstream passage of newly hatched fish.

Figures 3.1 and 3.2 show water velocities in the post-impoundment Forebay using data provided by Manitoba Hydro. Figure 3.1 shows the 5th percentile post-project flow at the low operating level of el 158 m, while Figure 3.2 shows the 95th percentile post-project flows at the FSL of el 159 m.

PART ONE – PROPOSED SITES

4 HABITAT INVENTORY

A high level review of all potential habitats available within the post-impoundment forebay near the Keeyask GS was conducted and is shown in Figure 2. The habitat inventory subdivided the forebay into the following four areas based on water depth (relative to FSL of 159 m) and substrate type:

- Shallow habitat (< 3-4 m).
- Deep habitat (> 4 m).
- Exposed bedrock in shallow habitat (< 3-4 m).
- Exposed bedrock in deep habitat (> 4 m).

Shallow habitat areas are shown between the FSL of el 159 m and el 155 m in Figure 2. el 155 m is 3 m below the low operating level of el 158 m, slightly deeper than the ideal whitefish spawning depth of 2.5 m. It should be noted that the area between the low operating level of el 158 m and the FSL of el 159 m is not ideal for habitat creation, as the habitat may be above the water level during low operating conditions. Shallow habitat sites discussed in Section 5 and shown in Figures 4 and 5 are placed between el 155 and el 158 to ensure the habitat remains under water. Deep habitat is shown below el 155 m in Figure 2. Bedrock outcrops shown in Figure 2 were identified using historical JD Mollard airphotos and the technical memorandum "Existing Environment Mineral Erosion" [Ref 4].

5 PROPOSED SITES

The habitat inventory shown in Figure 2 and discussed in Section 4 was refined to identify areas that would benefit from enhancement activities. The identified prospective development zones included sites 1A, 1B, 1C-1, 1C-2, 1D, 1E, 2A-1, 2A-2, 2B, 2C-1 and 2C-2, as shown in Figure 4 and Table 2. The costs to develop these sites are presented in Table 2 in 2012 dollars and are discussed in more detail in Section 7. Only development zones 1B and 1E are located along the dyke lines, as the velocities along the dykes are

fairly low and habitat development along dyke lines has not traditionally produced good spawning grounds.

During the site assessment/selection process, an additional nine sites (1F, 1G, 1H, 1J, 1K, 1L, 2D, 2E and 2F) were identified as suitable for shallow and deep habitat by North/South. The selection of these sites was based on depth, velocity, substratum material, exposure, dissolved oxygen and minimum area criteria. The layout of these sites was provided to KGS ACRES from North/South as an attachment to the email in Reference 5.

The development zones identified in Figure 4 have much larger surface areas than 1,000 m^2, and indicate regions where smaller pockets of suitable habitat may be developed. The precise habitat locations will be optimized in the final design stage based on more detailed criteria, site investigations and minimization of sediment deposition based on information such as that which is provided in the figures in Appendix B.

5.1 SHALLOW HABITAT (< 3-4 M)

Eleven potential shallow nearshore habitat development zones are identified in Figure 4 as 1A through 1L. The development zone identifier/label of 1I was not used to avoid confusion. Sites 1A, 1B, 1C, 1F and 1G are accessible from the north shore, while Sites 1D, 1E, 1H, 1J, 1K and 1L are accessible from the south shore. Sites 1C, 1F and 1G are only accessible during winter months.

5.1.1 NORTH SHORE

Site 1A is located along the south shore of Deposit N-6 in the north-central portion of the forebay. It has fairly low flow velocities of up to 0.1 m/s at a 5^{th} percentile flow at the low operating level of el 158 m and up to 0.2 m/s at a 95^{th} percentile flow at the FSL of el 159 m. It does not receive significant wave action from the prevailing winds. The peat depth recorded in a borehole directly north of the site is 0.1 m and therefore the site will likely not require additional rockfill placement in order to achieve the required elevations for shallow rocky shoals or gabion basket wire placement to prevent infiltration of the organic material into the rock layer.

Site 1B is located along the North Dyke. The site would receive some wave action due to prevailing winds but has very low velocities of up to 0.1 m/s at both a 5^{th} percentile flow at the low operating level of el 158 m and a 95^{th} percentile flow at the FSL of el 159 m. Traditionally, areas along dyke lines have not produced good spawning grounds, even when appropriate enhancements are made. There is some concern that oxygen levels may be low due to anticipated high BOD demands related to vegetation and low flows in the

area. If this area is unsuitable for whitefish or walleye habitat, other species may benefit from the resulting habitat, which is expected to consist of emergent macrophytes. There may be an opportunity to enhance this area for pike spawning. Peat depths noted in boreholes across this site range from 0.4 m to 1.5 m, likely requiring additional rockfill placement to achieve the required elevations for shallow rocky shoals and gabion basket wire placement to prevent infiltration of the organic material into the rock layer. An alternative site for 1B is shown on Figure 4. This site may be more suitable for rocky shoal habitat due to a lower rate of sediment deposition based on the information presented in Figure B-1.

Sites 1C-1 and 1C-2 are located on the north shore of Caribou Island and would only be accessible by winter road during the winter months, assuming suitable ice conditions develop to permit establishment of an ice road. These sites have lower velocities of up to 0.1 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.2 m/s at a 95th percentile flow at the FSL of el 159 m. The sites would receive good wave action from the prevailing winds. Peat depths at 1C-1 and 1C-2 are unknown due to the lack of borehole investigations on Caribou Island. It is assumed that peat deposits will not pose a problem since the sites are not in a low-lying area.

Site 1F is located southwest of Caribou Island, and experiences higher flow velocities of up to 0.3 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.5 m/s at a 95th percentile flow at the FSL of el 159 m. The site would receive some wave action from the prevailing winds. Peat depths are unknown due to the lack of borehole investigations in the area and may pose a problem since the site is in a fairly flat area that may be subject to ponding. Due to its distance from the dyke line, it is assumed that Site 1F would only be developed if Site 1G was developed, requiring a winter haul road to access the site. Due to this site's proximity to a high velocity area, the location of the rocky shoal within the suggested site outline shown on Figure 4 will be selected to optimize fish access and fry distribution following hatching.

Site 1G is located west of Caribou Island. It has flow velocities of up to 0.2 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m. It would receive some wave action from the prevailing winds. Peat depths are unknown due to the lack of borehole investigations in the area and may pose a problem since the site is in a fairly flat area that may be subject to ponding. Due to its distance from the dyke line, it is assumed that Site 1G would only be developed if Site 2A is developed, requiring a winter haul road to access the site.

5.1.2 SOUTH SHORE

Site 1D is located along the south portion of Deposit S-5 in the southwest corner of the forebay and is in a region of higher wave action with velocities of up to 0.1 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.3 m/s at a 95th percentile flow at the FSL of el 159 m. Peat depths in the area range between 0 m and 0.3 m.

Site 1E is located along the South Dyke line, reducing its probability of success as traditionally, areas along dyke lines have not produced good spawning grounds. Also the prevailing flows may transport fry over the Spillway in certain years. However, this area may produce some viable spawning habitat under appropriate conditions. The site will receive some wave action from prevailing winds and has velocities of up to 0.1 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.3 m/s at a 95th percentile flow at the FSL of el 159 m. Peat depths around this site vary between 0 m and 5.05 m, which will require optimization of the final habitat location to avoid the need for additional rockfill placement to achieve the required elevations for shallow rocky shoals and gabion basket wire placement to prevent infiltration of the organic material into the rock layer. Site 1E could be reshaped to extend further northward and join Site 1L, while having less development to the southwest.

Site 1H is located south of Caribou Island within the limits of Deposit S-5. It would receive some wave action from prevailing winds. The site has velocities in its northwest corner of up to 0.6 m/s at a 5th percentile flow at the low operating level of el 158 m and over 0.7 m/s at a 95th percentile flow at the FSL of el 159 m, but velocities in the remainder of the site are not as high. The site contains regions of low peat deposition between 0 m and 0.3 m, therefore it is anticipated that additional rockfill placement will not be required to achieve the required elevations for shallow rocky shoals. Gabion basket wire placement to prevent infiltration of the organic material into the rock layer will also not be required.

Site 1J is located in the upstream end of Gull Lake and is the furthest site from the Keeyask GS. The site would receive good wave action from prevailing winds and has velocities of up to 0.2 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m. Peat depths are unknown due to a lack of borehole investigations, but it is anticipated that peat deposits will not pose a problem as the site is not in a low-lying area.

Site 1K, located in the upstream end of Gull Lake, would receive good wave action from prevailing winds and has velocities of up to 0.2 m/s at a 5th percentile flow at the low

operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m. Peat depths observed near this site are 0.2 m, and thus it is anticipated that additional rockfill placement will not be required. Gabion basket wire placement to prevent infiltration of the organic material into the rock layer will also not be required.

Site 1L is located at the north end of Deposit S-18, would receive good wave action from prevailing winds and has velocities of up to 0.4 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.5 m/s at a 95th percentile flow at the FSL of el 159 m. The site contains regions of low peat deposition of 0.1 m, thus it is anticipated that additional rockfill placement will not be required to achieve the required elevations for shallow rocky shoals. Gabion basket wire placement to prevent infiltration of the organic material into the rock layer will also not be required.

5.2 DEEP OFFSHORE HABITAT (> 4 M)

Figure 4 shows the locations of six areas where deep habitat could be created: 2A through 2F. Sites 2A and 2D are accessible from the north shore, while Sites 2B, 2C, 2E and 2F are accessible from the south shore. Sites 2A and 2D are only accessible during the winter months. Additional locations where bedrock is likely exposed in the existing river channel are shown on Figures 2 and 4. These areas would naturally provide suitable deep offshore habitat.

5.2.1 NORTH SHORE

Sites 2A-1 and 2A-2 are located in the northwest corner of the immediate forebay area, around an island that will be flooded during impoundment. The island has a shoreline of exposed bedrock which could be expanded by stripping surface vegetation and overburden, where practical. A 1 m layer of additional rock material can be dumped around the exposed bedrock as necessary. Drilling and blasting of the rock outcrops for sourcing rock is also an option. It is anticipated that this rock material would be placed during winter construction conditions. Site 2A-1 experiences velocities of up to 0.1 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.2 m/s at a 95th percentile flow at the FSL of el 159 m. Site 2A-2 experiences velocities of up to 0.2 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.3 m/s at a 95th percentile flow at the FSL of el 159 m. The sites are near potential spawning grounds at Sites 1C-1, 1C-2 and 1G.

Site 2D is located southwest of Caribou Island and may require deposition of a 1 m layer of rock material to create suitable deep habitat, as substrate conditions and depth to bedrock at the site are unknown. It is adjacent to the shallow rocky shoal site 1F,

and experiences higher velocities of up to 0.4 m/s at both a 5th percentile flow at the low operating level of el 158 m and a 95th percentile flow at the FSL of el 159 m.

5.2.2 SOUTH SHORE

Site 2B is located on the Quarry Q-8 site. It is anticipated that materials from Quarry Q-8 will not be used in the construction of the Keeyask GS, but surface material can be stripped from the quarry area to expose the underlying rock. No additional rockfill placement would be required, as the site would be suitable for deep habitat. A 1 m thick layer of stripping is assumed, as the depth to rock varies around the site with some bedrock outcrops observed. The site experiences velocities of up to 0.2 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m.

Sites 2C-1 and 2C-2 are well situated near the Spillway. Site 2C-2 is accessible from the Central Dam and Site 2C-1 is accessible from the south shore. Site 2C-1 consists of exposed bedrock and would only need placement of scattered boulders and cobbles to be suitable for deep habitat. A 1 m thick layer of rock could be placed at Site 2C-2 to create a deep rocky shoal. It is assumed that the material at the 2C-2 site can be sourced from a stockpile located adjacent to the site. However, the site is bordered by an exposed strip of bedrock which, if practical, might be enlarged by stripping surface vegetation and overburden, thus avoiding the more costly thick rock layer placement. Site 2C-1 experiences velocities of up to 0.2 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m. Site 2C-2 experiences velocities of up to 0.3 m/s at a 5th percentile flow at the low operating level of el 158 m and up to 0.4 m/s at a 95th percentile flow at the FSL of el 159 m. Depths along the shoreline in the area of 2C-2 range between 0 m and 0.4 m. These areas may be too close to shore to be effective. The development of the 2C-2 site could occur concurrently with the Shoreline Erosion Protection program for the Stage I Diversion, introducing potential cost savings.

Site 2E is located at the west end of Deposit S-5 next to the potential spawning grounds at 1D and 1H. Peat depths immediately east of the site range between 0.2 m and 0.4 m. At the northwestern end of the area, velocities reach up to 0.5 m/s at a 5th percentile flow at the low operating level of el 158 m and over 0.7 m/s at a 95th percentile flow at the FSL of el 159 m, but are lower in the remainder of the site. A 1 m thick layer of rock would be placed at the site in order to create suitable deep habitat.

Site 2F is located immediately north of Deposit S-18 next to the potential spawning grounds at 1L. Velocities at the site reach up to 0.4 m/s at a 5th percentile flow at the low

operating level of el 158 m and up to 0.5 m/s at a 95th percentile flow at the FSL of el 159 m. Peat depths in the area range between 0.1 m and 0.9 m. A 1 m thick layer of rock would be placed at the site in order to create suitable deep habitat.

PART TWO – PREFERRED SITES

Part Two of this memorandum presents the analysis and cost estimate resulting from comments made by North/South on Draft 2 of the memorandum [Refs 3 and 5]. North/South identified preferred sites for habitat development as shown in Table 3, presented in Figure 5 and discussed in Section 6. The cost estimate for developing these sites has been revised to 2012 dollars as discussed in Section 7 and presented in Table 4.

6 PREFERRED HABITAT ASSESSMENT

North/South reviewed the potential habitat locations presented in Draft 2 of this memorandum and identified preferred habitat development locations in the forebay [Ref 3]. The list of potential habitat locations was refined according to the following criteria: whether its location is adjacent to known or suspected present-day spawning habitats, how likely it is to be exposed to fine particulate sedimentation post-impoundment, and whether it is a minimum distance of 3 km upstream of the proposed locations of the GS and spillway intake structures so as to minimize entrainment and downstream passage of newly hatched fish [Ref 6].

In early 2012, North/South reviewed the potential habitat locations provided in Revision 0 of this memorandum and refined the preferred depth criteria for both shallow and deep rocky shoals. Manitoba Hydro undertook the task of analyzing and updating the locations for the preferred sites based on the revised criteria. Taking the new depth criteria into consideration a comparison was made to the previously defined preferred sites for deep and shallow rocky shoals. The revised outlines for the habitat locations are presented in Figures 4 and 5 as well as Figures B-1 to B-4 in Appendix B. Based on the comparison study the changes to the preferred sites have not resulted in any impacts on the cost estimate discussed in Section 7.

The following describes the methodology used by Manitoba Hydro to update the deep and shallow shoals. Note that the shape files include metadata which also indicates the methodology used:

- Rev 0 deep and shallow shoals were reviewed to evaluate the changes required.
- The Water Depth Post-Project 5th percentile grid developed by WRE was used in order to evaluate the updated shoals meet the new depth criteria.

- The Mineral Sediment Deposition (Post-Project Year 1) Grid developed by KGS Acres was used in order to evaluate the updated shoals meet the sedimentation criteria.
- The proposed deep and shallow shoals total land areas were maintained between revisions.
- The deep and shallow shoals width criteria specified in Rev 0 were maintained between revisions.
- The locations of the deep and shallow shoals were maintained as much as possible between revisions.
- Only the proposed deep and shallow shoals were updated based on the new criteria.

The North/South comments are summarized below and in Table 3. The preferred habitat locations are shown in Figure 5.

6.1 SHALLOW HABITAT (< 3-4 M)

Sites 1F, 1G and 1H were identified as the most suitable for development of shallow water spawning shoals and are considered to be priority habitat. Sites 1D, 1J, 1K and 1L also have suitable attributes for development of shallow water spawning shoals and are included on the list of preferred sites.

6.1.1 NORTH SHORE

Site 1A is not suitable for development because water velocities are low and exposure is not above average.

Site 1B is not suitable for development because water velocities are low, exposure is not above average and it may have higher than average sediment deposition as seen on Figure B-1.

Sites 1C-1 and 1C-2 are not suitable for development, as they have low velocities and low exposure, but less sediment deposition is predicted at these sites than at Sites 1A and 1B. These factors make development at these sites more attractive than at Sites 1A and 1B.

Site 1F is very suitable for development as it has good velocities and exposure, and is adjacent to existing known or suspected walleye spawning habitat and deep water shoal Site 2D. Figure B-1 indicates that sediment deposition may be a concern in the downstream portion of this site; therefore, development should be concentrated in the

upstream portion. It is assumed that this site would be developed in the winter using an access road across the ice in the forebay.

Site 1G is very suitable for development as it has good velocities and exposure, and is adjacent to existing known or suspected walleye spawning habitat. No concerns about sediment deposition are apparent at this site. It is assumed that this site would be developed in the winter using an access road across the ice in the forebay.

6.1.2 SOUTH SHORE

Site 1D is suitable for development as it has good velocities and exposure.

Site 1E is not suitable for development. The northeastern portion of the site has suitable attributes for development, but is closer to the Powerhouse and Spillway than other options and is therefore less attractive. The southwestern portion of the site is not suitable due to low velocities and higher sediment deposition as seen in Figure B-1.

Site 1H is very suitable for development as it has good velocities and exposure, and is adjacent to existing known or suspected walleye spawning habitat. The upstream portion of the site is adjacent to deepwater shoal Site 2E. No concerns regarding sediment deposition are apparent at this site.

Site 1J is suitable for development as it has good velocities and exposure, and is adjacent to existing known or suspected walleye spawning habitat. Although Figure B-1 indicates that sediment deposition may be unacceptably high at this site, it is still recommended for inclusion in the list of preferred sites.

Site 1K is suitable for development as it has good velocities and exposure, and is adjacent to existing known or suspected walleye spawning habitat. Although Figure B-1 indicates that sediment deposition may be unacceptably high at this site, it is still recommended for inclusion in the list of preferred sites.

Site 1L is suitable for development as it has good velocities and exposure, and is adjacent to deepwater shoal Site 2F. No concerns regarding sediment deposition are apparent at this site.

6.2 DEEP OFFSHORE HABITAT (> 4 M)

Most of the deep offshore habitat sites were assessed to be suitable for development of deep water spawning shoals. The exceptions were Sites 2B and 2C-2, as these sites are too close to the Powerhouse and Spillway.

6.2.1 NORTH SHORE

Sites 2A-1 and 2A-2 are suitable for development as they have good velocities. However, Figure B-1 indicates that sediment deposition may be a concern at these locations. For the purpose of this study it is assumed that these sites would be developed in the winter using an access road across the ice in the forebay. Alternatively, if a decision was to be made to construct the deep and shallow rocky shoals at these sites in the summer then a barge will be required for the alternative approach. We note that the cost estimate is based on developing these sites in the winter.

Site 2D is very suitable for development as it has good velocities and exposure and is adjacent to shallow shoal Site 1F and lake whitefish spawning habitat. No concerns regarding sediment deposition are apparent at this site. It is assumed that this site would be developed in the winter using an access road across the ice in the forebay.

6.2.2 SOUTH SHORE

Site 2B is not suitable for development as it is too close to the Spillway and Powerhouse. It is within the 3 km exclusion zone from the Principal Structures shown on Figure 5 and may expose post-larval fish to downstream transportation out of the reservoir.

Site 2C-1 is suitable for development, but it is located within the 3 km exclusion zone boundary from the Spillway and Powerhouse shown on Figure 5 and may expose post-larval fish to downstream transportation out of the reservoir. No concerns regarding sediment deposition are apparent at this site.

Site 2C-2 is not suitable for development as it is too close to the Spillway and Powerhouse. It is well within the 3 km exclusion zone from the Principal Structures shown on Figure 5 and may expose post-larval fish to downstream transportation out of the reservoir.

Site 2E is very suitable for development as it has good velocities and exposure and is adjacent to shallow shoal Site 1H as well as lake whitefish spawning habitat. No concerns regarding sediment deposition are apparent at this site.

Site 2F is very suitable for development as it has good velocities and exposure and is adjacent to shallow shoal Site 1L. No concerns regarding sediment deposition are apparent at this site.

7 COST ESTIMATES

[redacted]

[redacted]

8 SUMMARY

Based on comments from North/South [Ref 3], preferred shallow and deep habitat sites have been identified as discussed in Section 6 and presented in Figure 5. The preferred shallow habitat sites include Sites 1D, 1F, 1G, 1H, 1J, 1K and 1L, while the preferred deep habitat sites include Sites 2A-1, 2A-2, 2C-1, 2D, 2E and 2F.

[redacted]

Should a decision be made to further investigate any of the alternatives for fish habitat, the following suggestions are made with regard to further studies that could be undertaken to enhance the effectiveness of the mitigation measures:

- Rocky shoal habitat locations shown in Figure 5 could be optimized to minimize sediment deposition based on the sediment deposition estimates discussed in the

"Project Environment Sedimentation and Effects Assessment" memorandum [Ref 7] and presented in Figures B-1 through B-4.

- Consideration could be given to the development of more spawning areas.
- Consideration could be given to the development of a selected number of sites followed by enhancement of successful sites based on post-project monitoring.
- Consideration could be given to the benefits of enhancing sites deeper than 6 m in areas where current would remove sediment. It is possible that the existing habitat in these areas would not require enhancement.

It should be noted that creating successful spawning habitat for walleye and whitefish in lakes is among the most challenging habitat enhancement tasks. Numerous criteria are involved and results are unpredictable. Further studies to identify existing spawning areas within the Nelson River upstream of the Principal Axis would be useful, as success is more likely if enhancements are being made to an existing spawning area.

REFERENCES

REFERENCES

1) Magnusson, Dave (Manitoba Hydro), "FW: Candidate sites for creation of Shallow (walleye) and Deep (lake whitefish) spawning habitat" [Email], Message to Jim Smith (KGS ACRES), sent November 12, 2009, 9:40 PM.

2) Magnusson, Dave (Manitoba Hydro), "FW: Keeyask Compensation Options - spawning shoals" [Email], Message to Jim Smith (KGS ACRES), sent December 21, 2009, 9:26 AM.

3) North/South Consultants, Inc., "Keeyask Stage IV Engineering Design Memorandum – Quality Review Comments and Responses, Comments on Memo GN-9.8.10 Draft 2", posted to Manitoba Hydro eRoom on November 24, 2010.

4) J.D. Mollard and Associates Limited (2008), Memorandum GN-9.2.2, "Keeyask Generation Project, Stage IV Studies – Physical Environment, Existing Environment Mineral Erosion", Manitoba Hydro File 00195-11100-00153_00, March 20, 2008.

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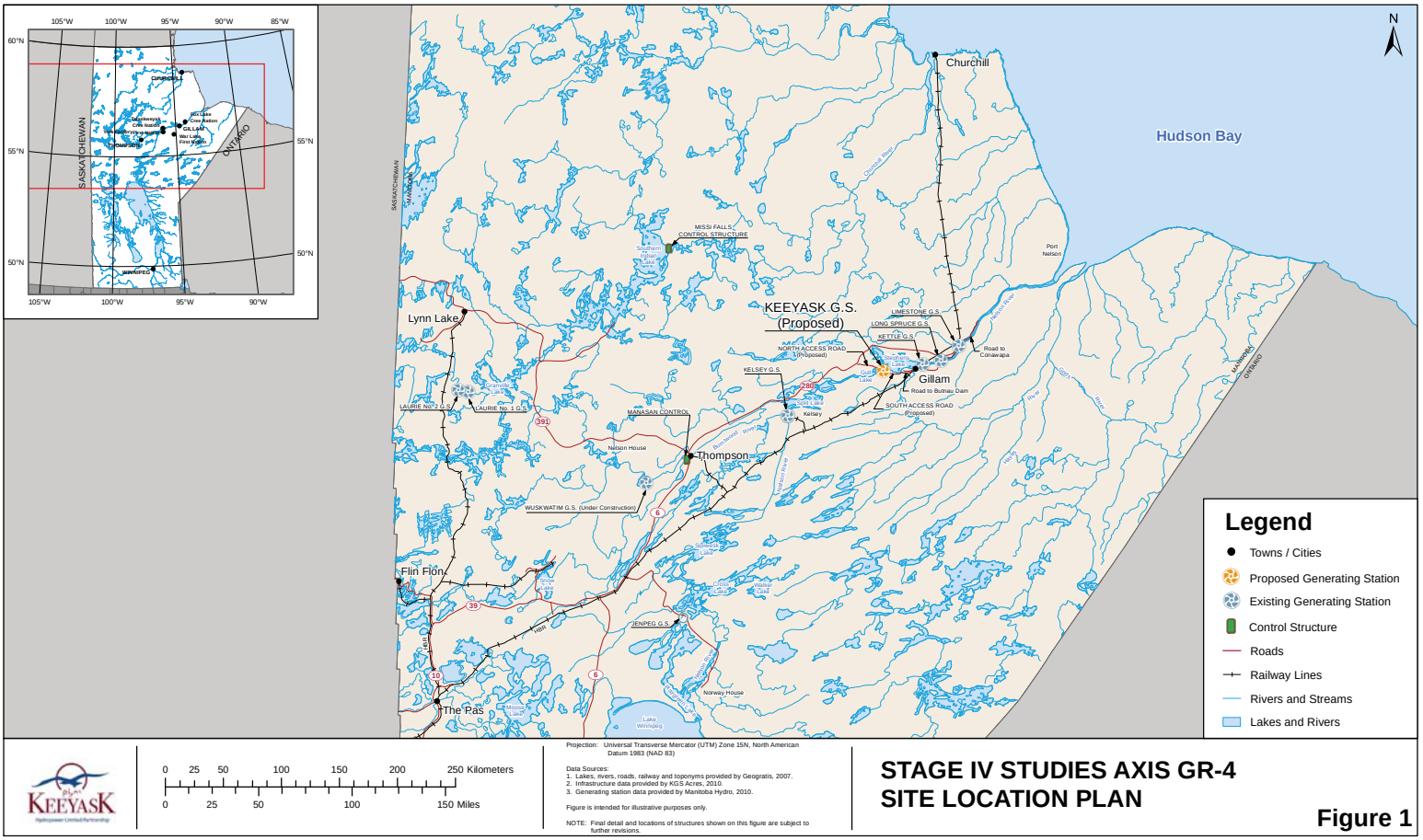
6) Schneider-Vieira, Friederike (North/South Consultants, Inc.), "FW: Keeyask Stage IV Studies - GN-9.8.10 First Order Assessment of Deep and Shallow Rocky Shoals for improving fish habitat" [Email], Message to Philippe Pantel (KGS ACRES), sent July 28, 2011, 5:28 PM.

7) KGS ACRES Ltd. (2009), Memorandum GN-9.2.9, "Keeyask Generation Project, Stage IV Studies – Physical Environment: Project Environment Sedimentation and Effects Assessment", Manitoba Hydro File # 00195-11100-0160_01.

8) Roni, Phil, et al. (2005), "Habitat rehabilitation for inland fisheries: Global review of effectiveness and guidance for rehabilitation of freshwater ecosystems," Food and Agriculture Organization of the United Nations, FAO Fisheries Technical Paper 484, Rome.

FIGURES



Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Lakes, rivers, roads, railway and toponyms provided by Geogratis, 2007.
2. Infrastructure data provided by KGS Acres, 2010.
3. Generating station data provided by Manitoba Hydro, 2010.

Figure is intended for illustrative purposes only.

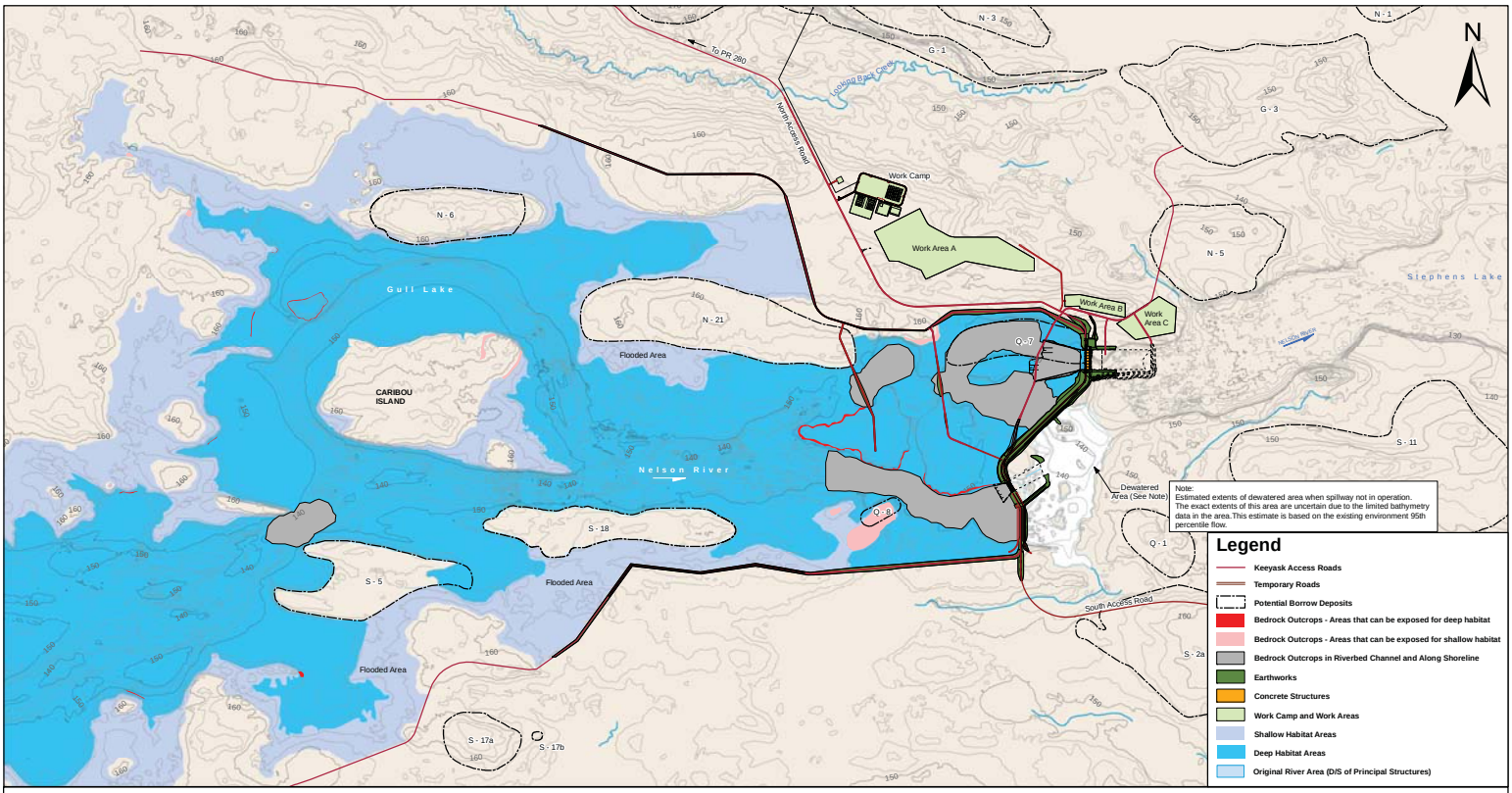
NOTE: Final detail and locations of structures shown on this figure are subject to further revisions.

STAGE IV STUDIES AXIS GR-4 SITE LOCATION PLAN

Figure 1

Created By: KGS ACRES

Date Created: March 5, 2013



0 0.5 1 2 3 Kilometers

0 0.25 0.5 1 1.5 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Dewatered area provided by Manitoba Hydro, 2011. Existing Gull environment shoreline and post project shoreline provided by Manitoba Hydro, 2010. Contours provided by Manitoba Hydro, 2005
2. Stephens Lake existing environment shoreline provided by North/South Consultants, 2009 (Only for cartographic purposes)
3. Infrastructure data provided by KGS Acres, 2010. Habitat data provided by KGS Acres, 2011

Contours may not necessarily conform to shorelines depicted, due to differing scales.

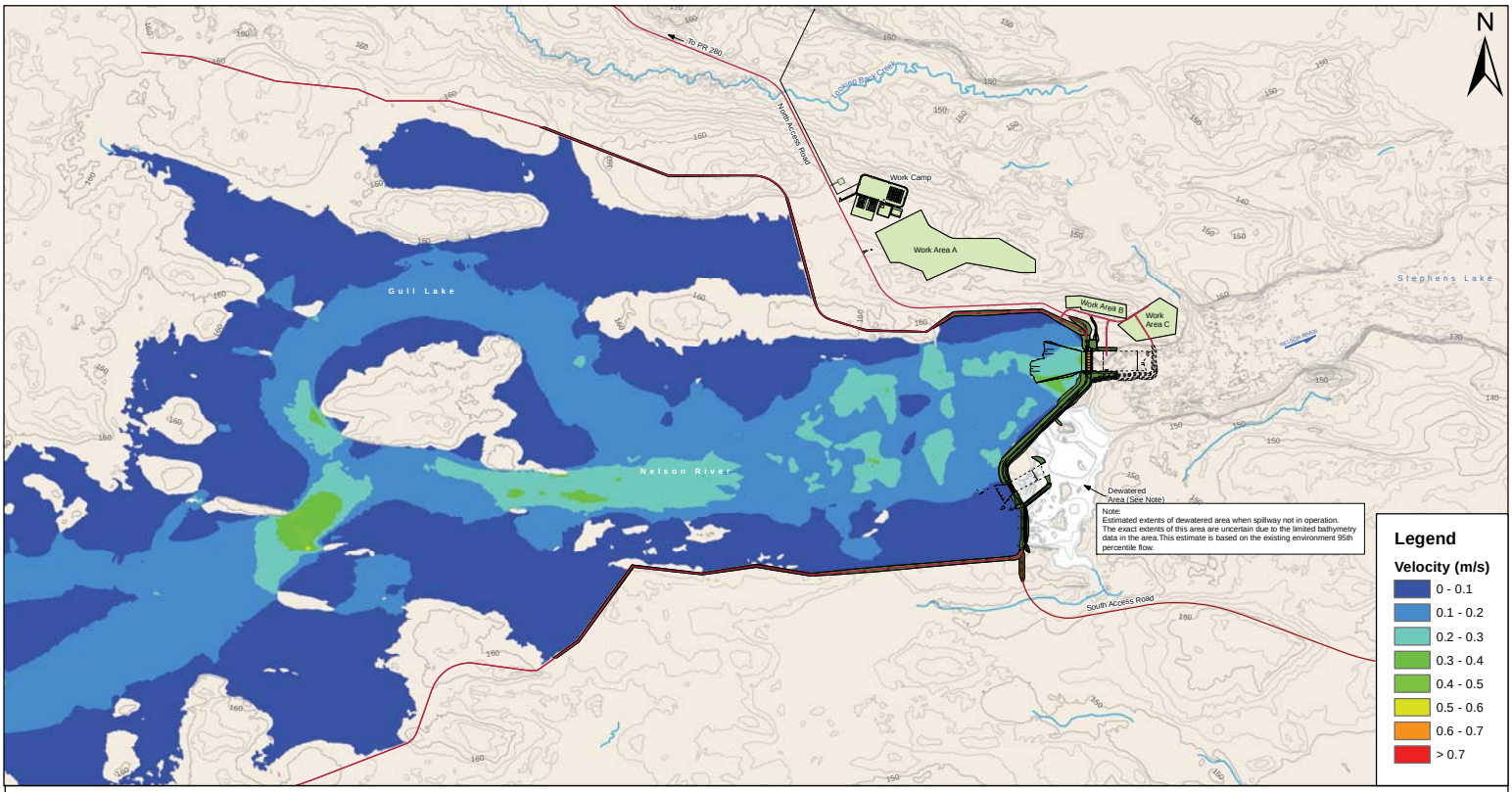
FOR GENERAL REFERENCE ONLY

KEEYASK STAGE IV STUDIES AXIS GR-4 INVENTORY OF POSSIBLE SHALLOW & DEEP ROCKY SHOAL HABITAT IN FOREBAY

Figure 2

Created By: KGS ACRES

Date Created: March 5, 2013



0 0.5 1 2 3 Kilometers

0 0.25 0.5 1 1.5 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Dewatered area and velocity data provided by Manitoba Hydro, 2011. Existing Gull environment shoreline and post project shoreline provided by Manitoba Hydro, 2010. Contours provided by Manitoba Hydro, 2005
2. Stephens Lake existing environment shoreline provided by North/South Consultants, 2009 (Only for cartographic purposes)
3. Infrastructure data provided by KGS Acres, 2010

Contours may not necessarily conform to shorelines depicted, due to differing scales
FOR GENERAL REFERENCE ONLY

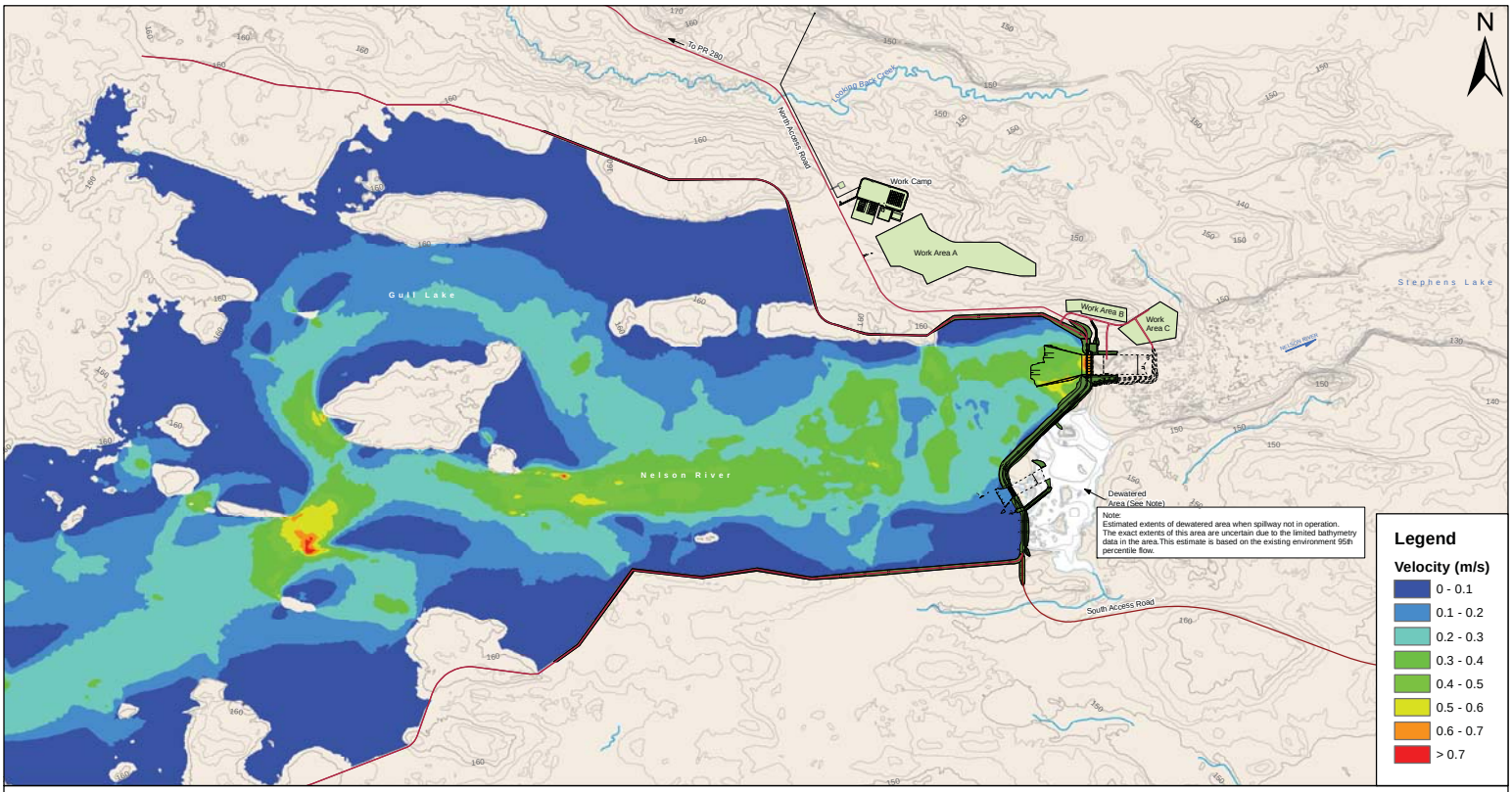
STAGE IV STUDIES AXIS GR-4 VELOCITIES IN IMPOUNDED FOREBAY FOR 5th PERCENTILE FLOW AT EL. 158 m

2170 m^3/s TOTAL FLOW

Figure 3.1

Created By: KGS ACRES

Date Created: March 5, 2013



0 0.5 1 2 3 Kilometers

0 0.25 0.5 1 1.5 2 Miles

Projection: Universal Transverse Mercator (UTM) Zone 15N, North American Datum 1983 (NAD 83)

Data Sources:
1. Dewatered area and velocity data provided by Manitoba Hydro, 2011. Existing Gull environment shoreline and post project shoreline provided by Manitoba Hydro, 2010. Contours provided by Manitoba Hydro, 2005
2. Stephens Lake existing environment shoreline provided by North/South Consultants, 2009 (Only for cartographic purposes)
3. Infrastructure data provided by KGS Acres, 2010

Contours may not necessarily conform to shorelines depicted, due to differing scales
FOR GENERAL REFERENCE ONLY

STAGE IV STUDIES AXIS GR-4 VELOCITIES IN IMPOUNDED FOREBAY FOR 95th PERCENTILE FLOW AT EL. 159 m

4430 m^3/s TOTAL FLOW - 430 m^3/s FLOW THROUGH SPILLWAY

Figure 3.2

Created By: KGS ACRES

Date Created: March 5, 2013