Lower Churchill Project

DESIGN CRITERIA - CIVIL

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1 GENERAL

This document presents the design criteria and available data to be used for the civil works related to the Muskrat Falls facility of the Lower Churchill Project. The Generating station is located on the lower reaches of the Churchill River, approximately 35 km west of the Town of Happy Valley – Goose Bay and it comprises the Intake, Powerhouse, Spillway, RCC dams and the Transition gravity dams.

The project data presented in this document is based on the MF 1340 technical report and will be consistent with the document entitled "Lower Churchill Project – Basis of Design" and applicable Standards, codes and regulations.

The document will be revised periodically during the course of the design to reflect the final layouts and the information available from the manufacturers and suppliers of materials and equipments.

2 CONVENTIONS

All dimensions and material properties are expressed with the International System of Units (SI).

3 DESIGN PARAMETERS

3.1 ENVIRONMENTAL DATA

The following climatic data is obtained from the National Building Code of Canada (2010) for the Town of Happy Valley – Goose Bay.

Design Temperature [(1)]	January		July 2.5%	
	2.5% °C	1% °C	Dry °C	Wet °C
	-31	**-32**	**27**	**19**
Snow Load (1/50 yrs), kPa	S_s S_r		5.3 0.4	
Hourly Wind Pressures, kPa	1/10 yrs 1/50 yrs		0.33 0.42	
Seismic Data for Buildings (2% in 50 yrs)	S_a (0.2) S_a (0.5) S_a (1.0) S_a (2.0) PGA		0.13 0.091 0.057 0.020 0.045	
Degree Days Below 18°C			6670	
15 Min Rainfall, mm			18	
One Day Rainfall, (1/50 yrs), mm			80	
Annual Rainfall, mm			575	
Annual Total Precipitation, mm			960	
Driving Rain Wind Pressures (1:5 yrs), Pa			160	

(1) Note: the 1% and 2.5% used for the design conditions represent percentiles of the cumulative frequency distribution of hourly temperature.

3.2 EARTHQUAKE

The peak ground acceleration values are obtained from the Mean-Hazard Ground Motion for the Gull and Muskrat Falls Dam sites on the Lower Churchill River (Ref: document No. 722850-GI1170-40ER-0001-00 – Technical report GI1170 – Seismicity Analysis - Part two, July 2008).

The stability of the retaining works is verified by pseudo-static analyses, using sustained values of ground acceleration. The Maximum Design Earthquake (MDE) is based on an annual probability of being exceeded of 1/10 000 years. The Operating Basis Earthquake (OBE) is based on an annual probability of being exceeded of 1/200, or 2% g, whichever is larger.

Peak Horizontal Ground Acceleration $PGA_H^{MDE} = 0.094g$

$PGA_H^{OBE} = 0.02g$

3.3 ROCK FOUNDATION

Bedrock at the Muskrat Falls site generally consists of hard, unweathered gneissis type rocks with mafic and pegmatite stringers. Rock quality, as determined by Rock Quality Designation (RQD) values is mainly good to excellent but zones of poor quality, fractured rock occur locally due to shear zones and other structural elements.

3.4 HYDRAULIC DATA

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The water levels observed during the construction and the long-term operation downstream of the powerhouse, tailrace channel, and downstream of the overall dam and spillway are defined in Section 4.2 of "Design Criteria - Hydraulic " and in section 8.3 of the document entitled "Transient Analysis".

The relevant water levels are as follows:

3.4.1 Headwater Levels

Maximum flood level at PMF:	45.1	m
Upstream water level (1:1000) :	40.0	m
Maximum operating level (FSL) :	39.0	m
Minimum operating level (LSL) :	38.5	m
Maximum water level at diversion design flow :	25.0	m

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The reservoir level 40.0 m corresponds to 1:1000yr summer flood condition with two gates unavailable at the spillway and a shutdown of the powerhouse (loss of power transmission line for a short period).

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3.4.2 Water Levels at Tailrace

For the design of the powerhouse, the static water level and transient results to be considered at the tailrace are as follow:

	Discharge (m^3/s)	Water level (m)	Transient condition Min level (m)	Transient condition Max level (m)
Minimum discharge under open water conditions	534	1.30	-1.25	3.29
Minimum discharge under winter conditions	534	2.34	-	-
Max. capacity in winter	2,660	6.65	-	8.21
Q 50 yr (open flow)	6540	6.44	4.47	7.89
Q 1000 yr	8,120	7.21	5.30	8.75
PMF without Gull Island	25,060	12.61	11.04	13.82

4 CODES AND STANDARDS

The design, fabrication and construction of the structures shall conform to the following codes and standards.

	National Building Code 2010 (NBC)	
	SNC-Lavalin Procedures	
Concrete and RCC	ASTM C31	Standard Practice for Making and Curing Concrete Test Specimens in the Field
	ASTM C39	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
	ASTM C70	Standard Test Method for Surface Moisture in Fine Aggregate
	ASTM C94	Specification for Ready-Mixed Concrete
	ASTM C109	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-inch or 50-mm Cube Specimens)
	ASTM C114	Standard Test Methods for Chemical Analysis of Hydraulic Cement
	ASTM C117	Standard Test Method for Materials Finer than 0.075 mm (No. 200) Sieve in Mineral Aggregates by Washing

	ASTM C127	Standard Test Method for Density, Relative Density (Specific Gravity) and Absorption of Coarse Aggregate
	ASTM C128	Standard Test Method for Density, Relative Density (Specific Gravity) and Absorption of Fine Aggregate
	ASTM C136	Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates
	ASTM C143	Standard Test Method for Slump of Hydraulic Cement Concrete
	ASTM C150	Standard Specification for Portland Cement
	ASTM C186	Standard Test Method for Heat of Hydration of Hydraulic Cement
	ASTM C231	Standard Test for Air Content of Freshly Mixed Concrete by the Pressure Method
	ASTM C260/260M-10a	Standard Specification for Air-Entraining Admixtures for Concrete
	ASTM C441-05	Standard Test Method for Effectiveness of Pozzolans or Ground Blast Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction
	ASTM C494/C494M-11	Standard Specification for Chemical Admixtures for Concrete
	ASTM C566	Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying
	ASTM C618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
	ASTM C1017/C1017M-07	Chemical Admixture for Use in Producing Flowing Concrete
	ASTM C1064	Standard Test Method for Temperature of Freshly Mixed Concrete
	ASTM C1078	Test Methods for Determining the Cement Content of Freshly Mixed Concrete
	ASTM C1170/C1170M-08	Standard Method for Determining Consistency and Density of Roller-Compacted Concrete Using a Vibrating Table

	ASTM C1435	Molding Roller-Compacted Concrete in Cylinder Mold using Vibrating Hammer
	ASTM D4318	Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils
	BS EN 11350-3 2009	Testing Fresh Concrete Part 3 – Vebe Test
	CAN/CSA-A23.1-09/A23.2-09	Concrete Materials and Methods of Concrete Construction / Test Methods and Standard Practices for Concrete
	CAN/CSA-A23.3-04 (R2010)	Design of Concrete Structures
	CAN/CSA-A3000-08	Cementations Materials Compendium (Consists of A3001, A3002, A3003, A3004 and A3005)
	ACI 207.1R-05	Guide to Mass Concrete
	ACI 207.2R-07	Report on Thermal and Volume Change Effects on Cracking of Mass Concrete.
	ACI 207.5R99	Roller Compacted Mass Concrete
	ACI 224R-01	Control of Cracking in Concrete Structures
	ACI 302.1R-04	Guide for Concrete Floor and Slab Construction
	ACI 350.4R-04	Design Considerations for Environmental Engineering Concrete Structures
Reinforcing Steel	CAN/CSA-G30.18-09	Carbon-Steel Bars for Concrete Reinforcement
	CAN/CSA-G30.15-M92	Welded Deformed Steel Wire Fabric for Concrete
	ASTM A185/A185M-07	Steel Welded Wire Reinforcement, Plain, for Concrete
	ASTM A497/A497M-07	Steel Welded Wire Reinforcement, Deformed, for Concrete
	ASTM A615/A615M-09b	Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
Masonry	CSA-S304.1-04 (R2010)	Design of Masonry Structures
	CSA-A165 SERIES-04 (R2009)	Standards on Concrete Masonry Units (Consists of A165.1, A165.2 and A165.3)
	CSA-A179-04 (R2010)	Mortar and Grout for Unit Masonry
	CSA-A370-04 (R2009)	Connectors for Masonry
	CSA-A371-04 (R2009)	Masonry Construction for Buildings

	ASTM D1751-04(2008)	Preformed Expansion Joint Filler for Concrete Paving and Structural Construction (Nonextruding and Resilient Bituminous Types)
Joint Filler	ASTM D1752-04a(2008)	Preformed Sponge Rubber Cork and Recycled PVC Expansion Joint Fillers for Concrete Paving and Structural Construction
Waterstop	CAN/CGSB-41-GP-35M (1983)	Polyvinyl Chloride Waterstop
Joint Sealer	CAN/CGSB-37.16-M89 (withdrawn)	Filled, Cutback Asphalt for Dampproofing and Waterproofing
Steel Structures	CSA-S16-09	Design of Steel Structures
	CAN/CSA-G40.20/G40.21-04 (R2009)	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel
	ASTM A992/A992M-11	Structural Steel Shapes
	ASTM A500/A500M-10a	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
Bridge	CAN/CSA-S6-06	Canadian Highway Bridge Design Code
Cold-Formed Steel	CSA S136-07	North American Specification for the Design of Cold-Formed Steel Structural Members
Anchor Rods	ASTM F1554-07a	Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength
Bolts	ASTM A325-10	Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
Nuts	ASTM A490-10	Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength
	ASTM A563-07a	Carbon and Alloy Steel Nuts
Washers	ASTM F436-10	Hardened Steel Washers
Steel Deck	ASTM A653/A653M-10	Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy- Coated (Galvannealed) by the Hot-Dip Process
Shear Studs	ASTM A108-07	Steel Bar, Carbon and Alloy, Cold-Finished
Crane Rails	ASTM A759-10	Carbon Steel Crane Rails
Embedded Metal	ASTM A53/A53M-10	Pipe, Steel, Black and Hot-Dipped, Zinc Coated, Welded and Seamless

	ASTM A444	Standard Specification for Sheet Steel, Zinc-Coated (Galvanized) by the Hot-Dip Process for Culverts and Underdrains
Welding	CSA-W48-06	Filler Metals and Allied Materials for Metal Arc Welding
	CSA-W59-03 (R2008)	Welded Steel Construction (Metal Arc Welding)
	CSA-W47.1-09	Certification of Companies for Fusion Welding of Steel
	CSA-W178.1-08	Certification of Welding Inspection Organizations
Galvanization	ASTM A123/A123M-09	Zinc (Hot Dip Galvanized) Coatings on Iron and Steel Products
	ASTM A153/A153-09	Standard Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware
Formwork	ACI 347-04	Guide to Formwork for Concrete
	CSA 269.3-M92 (R2008)	Concrete Formwork
Wood	CAN/CSA-O86-09	Engineering Design in Wood
	CAN/CSA-O80 Series-08	Wood Preservation
	ASTM F1667-11	Driven Fasteners: Nails, Spikes and Staples
Paint	SSPC	Specifications for Surface Preparation and Coatings by the Society for Protective Coatings

5 DESIGN GUIDES

The design is done according to the following guides:

- Cement Association of Canada - Concrete Design Handbook;
- CISC-ICCA Handbook of Steel Construction, Tenth Edition;
- Dam Safety Guidelines, Canadian Dam Association, 2007;
- PTI Post-Tensioning Manual;
- AISE Technical Report No 13, Guide for the Design and Construction of Mill Buildings, June 1997;
- Formwork for Concrete (M.K. Huro);

- Design of Gravity Dams, US Bureau of Reclamation;
- FERC, Engineering Guidelines for the Evaluation of Hydropower Projects;
- ASCE 1989, Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments;
- Earthquake Spectra and Design (N.M. Newmarket W.J. Hall/1982);
- Hydrodynamic Pressures on Dams due to Horizontal Earthquakes (Zangar, C.N.; 1953). Proceedings Society on Experimental Stress Analysis, 10, 93-102;
- Hydrodynamic Pressures on Dams during Earthquakes (Zangar, C.N.; 1952). Engineering monograph No 11, U.S. Bureau of Reclamation;
- CIGB, bulletin 27, Considérations sur le calcul sismique de barrages;
- CIGB, bulletin 72, Choix des paramètres sismiques pour les grands barrages. Recommendations;
- Design of Hydraulic Structures (V.T. Chow);
- Design of Small Dams (U.S. Bureau of Reclamation);
- Reinforced Steel Institute of Canada - Manual of Recommended Standards;
- Canadian Geotechnical Design Manual (Canadian Geotechnical Institute);
- Standards for Design Review of Existing Concrete Gravity Dams, Dam Safety (DS-STD-03);
- Water Pressures on Dams during Earthquakes (Westergaard, H.M.; 1933). Transactions ASCE, 98: 413-433;
- Water Power (Jacobsen, 1974). ASCE vol. 2, chapter D;
- US Army Corps of Engineers, Retaining and Flood Walls, EM-1110-2-2502, 1989.

6 MATERIALS PARAMETERS

The properties of the materials used in the design of the Powerhouse, Spillway, the gravity dams (concrete and RCC) and the related works are:

Properties of Materials	
Material Description	**Properties (SI Units)**
Concrete and RCC	
Mass concrete	γ = 2 400 kg/m^3
Reinforced concrete	γ = 2 450 kg/m^3
Roller compacted concrete (RCC)	γ = 2 350 kg/m^3
Compression strength of concrete at 28 days for General use cement type GU and 91 days for Low heat of hydration cement LH-M	f_c' = 25, 30 and 35 MPa for structural concrete and 15 and 20 MPa for fill concrete.
Compression strength of RCC at 90 days for blended cement (cement and fly ash)	f_c' = 16 MPa.
Compression strength of bedding/facing concrete at 28 days for blended cement (cement and fly ash)	f_c' = 20 MPa
Cement for mass concrete	Low Heat Portland cement type LH-M with low alkalies (less than 0.6 %)
Cement for reinforced concrete	Portland type GU with low alkalies (less than 0.6%)
Fly ash for RCC	ASTM C 618, Class F or equivalent
Concrete Aggregates	
Coarse aggegate	Excavated Rock (Granitic gneiss)
Maximum size of coarse aggregates	40 m for Primary concrete m (according to location and use) 20 mm for Secondary concrete and members less than 300 mm thick 40 mm for RCC
Water	γ = 1000 kg/m^3
Masonry Mortar	Type S
Masonry Grout Mix	

Properties of Materials	
Material Description	**Properties (SI Units)**
Type of cement Compression strength at 28 days Water / cement ratio (W/C)	Portland type GU $f_c^{'}$ = 15 Mpa From 0.45 to 0.55
Injection Grout Type of cement Compression strength at 28 days Mix: • Anchors • Voids filling • Contact • Consolidation • Curtain	Portland cement type HE or GU $f_c^{'}$ = 35 MPa Water / cement ratio: W/C = 0.38 to 0.42 W/C = 0.4 to 0.7 + superplasticizer by weight W/C = 0.4 to 0.7 + superplasticizer by weight W/C = 0.4 to 0.7 + superplasticizer by weight W/C = 0.4 to 0.7 + superplasticizer by weight
Waterstop	PVC type for fresh concrete and Mastic or Hydrophilic against existing concrete or other material
Reinforcement • Mass density • Regular grade • Weldable grade • Mechanical splice	γ = 7 850 kg/m^3 Grade 400R Grade 400W Mechanical splices are used where lapped splices are not feasible.
Steel Structure Mass density Rolled W and WT sections Built-up sections for crane girders Steel exposed to weather Rolled C, MC, S, L sections Hollow Structural Sections • Square and rectangular • Circular Checkered plates	γ = 7 850 kg/m^3 Fy = 350 MPa Grade 350 WT, Category 2 Grade 350WT, Category 4 Grade 300W Grade 350W, Class H ASTM A36 and Grade 300W

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Properties of Materials	
Material Description	**Properties (SI Units)**
Rolled plates Steel deck	Grade 350 W (unless noted otherwise) Grade 250 W
Bolts	ASTM A325 and ASTM A490
Anchor Rods (Anchor bolts)	F_y = 248 MPa F_y = 400 MPa (rebars) F_y = 380 MPa (ASTM F1551)
Rock Characteristics Type of rock Mass density Allowable compressive stress	 Granitic gneiss ρ_r = 2 700 kg/m³ σ_r = 3 000 kPa
Rock Anchors Allowable cohesion: concrete / rock Allowable shear stress in rock	 c = 0.7 MPa τ = 200 kPa
Friction Coefficients Concrete / Rockfill Concrete / Granular material • Concrete / Silt	 μ = 0.40 μ = 0.40 μ =0.35
Moist Backfill Granular Material Moraine Rockfill	 γ = 20.4 kN/m^3 γ = 22 kN/m^3 γ = 19.5 kN/m^3
Saturated Backfill Granular Material Moraine Rockfill	 γ = 21.5 kN/m^3 γ = 22.7kN/m^3 γ = 21.7 kN/m^3
Submerged Backfill Granular Material Moraine Rockfill	 γ = 11.5 kN/m^3 γ = 12.7 kN/m^3 γ = 11.7 kN/m^3
Sediments	γ = 11.4 kN/m^3

Properties of Materials	
Material Description	**Properties (SI Units)**
At-Rest Horizontal Earth Pressure Coeff. Granular Material Moraine Rockfill	 $K_o = 0.5$ $K_o = 0.5$ $K_o = 0.5$
Horizontal Earth Pressure due to Compaction and Surcharges	As per US Army Corps of Engineers, EM-1110-2-2502 or Canadian Geotechnical Design Manual (see design guides)

7 DESIGN LOADS AND LOAD FACTORS

7.1 GENERAL

This section lists the design loads to be applied on the structures, presents the methodology to be used, defines the scope of stability analysis and the design criteria to be met for the Powerhouse, Spillway, RCC Dams and the Transition Gravity Dams.

These structures will be designed to resist the following loads:

- Dead loads;
- Occupancy loads;
- Equipment;
- Environmental loads;
- Water pressure;
- Uplift;
- Ice;
- Lateral earth and sediment pressure;
- Earthquake; and
- Thermal / Volume changes.

7.2 DEAD LOADS

The dead load considered in the design is the weight of the structure including the walls, floors, partitions, roofs and all other permanent construction and fixed equipment. The approximate unit densities of materials commonly used in construction are given in Section 6.

For the stability analysis, only the dead load of the concrete, steel superstructure and water are considered.

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7.3 OCCUPANCY LOADS

The occupancy loads are live loads that are defined by floor or area use. Live loads include the weights of vehicles, cranes and structural components that are subject to movement during the construction stage or maintenance operation. These loads shall not be considered to act simultaneously with the uniformly distributed load. The floors should be investigated for the effects of any concentrated load, minus the uniform load, over the area occupied.

Description		Design loads
Powerhouse	Generator Floor between grid lines 8 and 25	
	• Uniformly distributed load	75 kPa, or
	• Concentrated or wheel load	as per Truck CL625 wheel load , see Figure A1
	• Cover of generator pit	4 kPa
	Mezzanine 1 and 2	10 kPa
	Dewatering gallery	5 kPa or 2 kPa plus 10 kN concentrated load on 1.0 m x 1.0 m surface
	Other floors	15 kPa
	Passages, toilets, stairs and catwalks	5 kPa
South Service Bay	Generator floor between grid lines 1 and 7	
	• Uniformly distributed load	75 kPa, or
	• Concentrated load	300 kN concentrated load on surfaces of 0.2 m x 0.2 m for concrete slabs and 0.3 m x 0.3 m for covers; at 2 m c/c
	• Truck + Trailer	see figure A2
	• Rotor assembly stand	To be located on slab-on-grade
	Mezzanine 1 and 2	10 kPa
	Sump pit platform	5 kPa + Pumps weight
	Other floors	15 kPa
	Passages, toilets, stairs and catwalks	5 kPa
North Service Bay	Uniformly distributed load	75 kPa, or
	Concentrated load	300 kN on surfaces of 0.3 m x 0.3 m at 2 m c/c or
	Truck + Trailer	see figure A2

Tailrace Deck	Uniformly distributed load	75 kPa or
	CL-625 truck	see figure A1 or
	Off-Highway Haul truck	see figure A3 or
	Multi-line truck	see figure A4 or
	Mobile crane "TADANO" Model GR-800XL-1	see figure A5
	• Assumed self-weight including counterweigh	49.9 t
	• Assumed maximum lifted weight	16 t
	• Maximum factored load acting on one out-rigger multiplied by a live load factor of 1.05; 1.05 x (49.9t + 16t)	69.2 t
	• The out-riggers placed on 1.5 m x 1.5 m pad	
Intake Deck	Uniformly distributed load	75 kPa or
	CL-625 truck	see figure A1 or
	Off-Highway Haul truck	see figure A3 or
	Gantry crane/ Trash Cleaning System	see figure A6
	• Assumed self-weight including counterweigh	240 t
	• Assumed gantry crane hoist capacity	40 t
	• Maximum factored load on one leg of gantry while lifting a stoplog is based on the weight of the gantry equally distributed to two legs on one side of the machine, and the stoplog jammed while lifting in which case the hoist is limited to	214.2 t

	2.1 times its capacity, multiplied by a live load factor; 1.05 x (0.50 x 240t + 2.1 x 40t)	
	• Maximum factored load during operation of the trash cleaning system during trash removal operations is based on the weight of the machine plus 5t lifted load acting on one leg of the gantry multiplied by a live load factor of 1.05; 1.05 x (240t + 5t)	257.2 t
	• Maximum factored load during movement of the gantry while transporting a stoplog is based on 85% of the weight of the gantry and stoplog distributed equally to two legs of the gantry on one side of the gantry multiplied by a live load factor of 1.5 and a dynamic load factor of 1.3; 1.5 x 42.5% x (240t + 25t)x1.3	220 t
	Mobile crane American 9530 Carrier	see figure A7
	• Assumed self-weight including counterweigh	144 t
	• Assumed maximum lifted weight	40 t
	• Maximum factored load acting on one out-rigger multiplied by a live load factor of 1.05; 1.05 x (144t + 40t)	193.2 t
	• The out-riggers placed on mass concrete	

7.4 MECHANICAL / ELECTRICAL EQUIPMENT

In the case of the equipment loads on floor, the occupancy loads given in section 7.3 are typically sufficient. However, a confirmation would be made to ensure that any equipment load does not exceed the occupancy provision. The floors will be investigated for the effects of any concentrated load minus the uniform load over the area occupied.

In addition to the gravity loads, the thrust, tangential and radial loads from equipment will be considered in the structural design. The estimated main equipment loads for the powerhouse and the spillway are provided in the following table but must be confirmed through the vendors' data.

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Equipment	Estimated loads	Unit
Turbines		
Type	(4) Kaplan	units
Rated discharge, per unit	640	m^3/s
Runner diameter	8.8	m
Runner mass	260	tonnes
Weight of rotating parts	1045	tonnes
Total thrust bearing load (normal operation, FSL)	2778	tonnes
Weight of stayring	165	tonnes
Generator		
Generator pit	18 m x 18 m	m
Rotor diameter	13.918	m
Rotor mass	630	tonnes
Vertical force per sole plate (Normal / Max)	205/295	kN
Tangential force per sole plate (nominal)	250	kN
Tangential force per sole plate (short-circuit/faulty sync.)	1540/21220	kN
Transformers		
Number of GSU transformers	4	units
Shipping mass, each	175	tonnes
Installed mass, each	242	tonnes
Generator circuit breaker	12	tonnes
Excitation transformer	8	tonnes
Excitation cubicle	8	tonnes

Auxiliary transformer (4) (each)	7	tonnes
Oil volume: conservator and radiators	40,000	litres
Oil volume (GSU)	60,000	litres
Draft Tube Gate		
Weight	30	tonnes
Hoist capacity	35	tonnes
Gated Spillway / Diversion Passage		
Height of submerged gate	11.0	m
Height of surface gate	24.37	m
Weight of submerged gate	147	tonnes
Weight of surface gate	178	tonnes
Upstream stoplog weight (each section) (8)	18.6	tonnes
Downstream stoplog weight (each section) (4)	8	tonnes
Powerhouse Cranes		
Number	2	
Maximum lifting capacity (two cranes in tandem)	2 x 380	tonnes
Maximum lifting capacity during construction stage	200	tonnes
Auxiliary crane capacity	25	tonnes
Clearance to hook from generator floor	18.0	m
Crane runway elevation	34.45	m
Number of crane wheels (each side)	8	
Maximum crane wheel load (subject to review)	574	kN
Longitudinal force (ratio of total wheel load)	10%	
Lateral force (ratio of total lifted load + trolley weight)	20%	
Allowable vertical deflection of crane runway girder due to maximum wheel load without impact	1/900	Ratio of span
Allowable horizontal deflection of crane runway girder due to crane lateral load	1/600	Ratio of span
Bumper force (subject to review)	247	kN
Powerhouse Elevator		
Interior shaft dimensions	3.3m x 3.0m	m

7.5 ENVIRONMENTAL LOADS

The environmental loads include the snow, rain and wind loads, and are evaluated according to the methods outlined in the National Building Code (2010) given in Section 3.1. Environmental conditions are assumed equivalent to the published data for Happy Valley – Goose Bay.

7.6 WATER PRESSURE

Water pressure is assumed to act normal to the contact surface and vary linearly according to the depth. Varying water depths would be considered for the different load cases as outlined in later sections.

7.7 UPLIFT

The Powerhouse and RCC dams will include a drain system to reduce the uplift force at the rock foundation. For normal loading cases, a drainage efficiency of 67% (i.e. uplift reduction) at the drain line is assumed.

For the uncracked condition, the uplift is assumed to vary linearly from upstream water pressure to the reduced uplift at the drain line, and then again linearly from this reduced uplift to the tailwater pressure.

If the crack does not exceed the drain line, the uplift is assumed to vary linearly from the upstream water pressure at the crack tip to a reduced uplift at the drain line (calculated using the recommendations of the Canadian Dam Safety Guidelines), and then again linearly from this reduced uplift to the tailwater level. If the crack exceeds the drain line, the uplift is assumed to vary linearly from the upstream water pressure at the crack tip to the tailwater level.

An unusual case is also considered which assumes a drainage efficiency of 0% of the drainage system. For the uncracked condition, the uplift is assumed to vary linearly from upstream water pressure to the tailwater pressure.

The spillway will not include drainage of the foundation and therefore the uplift pressure is assumed to vary linearly from the upstream water pressure at the base of the structure at the upstream end to the tailwater pressure at the base of the structure at the downstream end. In the case of a cracked section, the uplift pressure is considered equal to the upstream reservoir head over the total crack length. Beyond the crack tip, the uplift is assumed to vary linearly from the upstream water pressure at the crack tip to the tailwater level.

For seismic cases, the uplift pressure remains at its pre-earthquake state, since it is assumed that the oscillatory nature of earthquake does not modify the uplift.

7.8 ICE PRESSURE

For concrete dams located in areas with severe winters, the thermally-driven static ice loads used in the stability analysis are commonly assumed to be 150 kN/m (10 kips/ft) for concrete dams. This value of ice load will be used for Muskrat Falls. The ice load is normally considered to act at 300 mm (1 foot) below the water level. The ice load on the vertical gates and stoplogs is taken as half of that on the concrete surfaces. The lateral force of ice confined between the piers is assumed to be 185 kN/m.

Recent research studies (CEATI, 2003) have shown that ice thermal loading alone may be less than the traditional 150 kN/m value, but when combined with certain fluctuations in water level may also produce higher total ice loads. As an unusual ice loading case, a value of 225 kN/m is considered in the stability analysis of Muskrat Falls structures.

7.9 LATERAL EARTH AND SAND PRESSURES

The lateral earth pressure acting on the civil works is calculated using the at-rest horizontal earth pressure coefficients given in Section 6. The K_o coefficient is calculated using the internal friction angle of the retained material. Terrain slopes and vertical live loads (surcharge) are considered when applicable.

7.10 INERTIAL EARTHQUAKE FORCES RESULTING FROM GROUND ACCELERATION

The simultaneous earthquake load effect is taken into account by considering 100% of the sustained force acting in one direction combined with 33% of the sustained force acting in the other direction, in which case the ratio of horizontal to vertical loading would be 100% SG_{AH}^{MDE} : 33% SG_{AV}^{MDE} and 33% SG_{AH}^{MDE} : 100% SG_{AV}^{MDE}.

The structures shall resist the simultaneous action of vertical and horizontal inertial forces applied at the center of gravity of the element. These forces are proportional to the mass and the sustained ground acceleration values SG_{AH}^{MDE} and SG_{AV}^{MDE} shown in following. The steel superstructure is designed using the pseudo-static method for earthquake, as prescribed by the National Building Code.

Peak Horizontal Ground Acceleration	$PGA_H^{MDE} = 0.094$ g
	$PGA_H^{OBE} = 0.02$ g
Peak Vertical Ground Acceleration	PGA_V^{MDE} taken as ⅔ $PGA_H^{MDE} = 0.063$ g
	PGA_V^{OBE} taken as ⅔ $PGA_H^{OBE} = 0.013$ g
Sustained Horizontal Ground Acceleration	SGA_H^{MDE} taken as ⅔ $PGA_H^{MDE} = 0.063$ g
	SGA_H^{OBE} taken as ⅔ $PGA_H^{OBE} = 0.013$ g
Sustained Vertical Ground Acceleration	SGA_V^{MDE} taken as ⅔ $PGA_V^{MDE} = 0.042$ g
	SGA_V^{OBE} taken as ⅔ $PGA_V^{OBE} = 0.009$ g

7.11 HYDRODYNAMIC FORCE INDUCED BY RESERVOIR ACTION

The hydrodynamic forces acting on the water retaining structures are calculated according to the equations of Zangar or Westergaard.

7.12 THERMAL / VOLUME CHANGES

The thermal loads are based on minimum and maximum design temperatures given in Section 3.1. The recommendations of ACI contained in "Report on Thermal and Volume Change, Effects on Cracking of Mass Concrete, 207.2 R-07" are used to control the volume change associated with hydration heat of mass concrete and shrinkage. Additional forces from the thermal expansion and contraction forces from equipment supports, in particular the generating equipment, will be addressed.

7.13 LOAD FACTORS

The structural members and connections are designed such that the «Factored resistance ≥ effect of factored loads».

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Unless otherwise indicated in this document, the following load factors are considered for the design of the structures.

- Dead load 1.25
- Hydrostatic load – normal maximum operating water level: 1.25
- Hydrostatic load – unusual case: 1.10
- Hydrostatic load – extreme case: 1.05
- Ice load: 1.50
- Short circuit load applied on the foundation of the stator: 1.10
- Backfill lateral loads: 1.50
- Counteracting load: 0.90
- Water hammer (at distributor axis): 1.15
- Impact factor for moving vehicle : 1.30
- Bridge design load factors as per CSA-S6

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7.14 IMPORTANCE FACTORS (I)

The main structures are designed for a post-disaster building category. The Importance Factors (I) used in the design, as per CNB 2010, are indicated in the following table:

	Ultimate Limit Sates	Serviceability Limit Sates
Snow, I_S	1.25	0.90
Wind, I_W	1.25	0.75
Earthquake, I_E	1.5	-

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7.15 LOAD COMBINATIONS

Unless otherwise indicated in section 9 “Design criteria specific to structures” of this document, the loads combinations used for the Ultimate Limit States (ULS) are as per the National Building Code of Canada (2010).

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8 CONCRETE COVER, DEFLECTION AND CRACK CONTROL

Unless otherwise indicated, the minimum concrete cover, i.e. the distance from the concrete surface to the nearest surface of reinforcement, shall be as follows:

- Concrete placed against rock: 75 mm
- Concrete faces covered by backfill, rock fill, etc.: 75 mm
- Exterior exposed faces: 75 mm
- Faces exposed to reservoir water and hydraulic passage: 75 mm
- Faces at inter-units contraction joint: 75 mm
- Interior faces of walls and columns: 50 mm
- Interior faces of beams and slabs: 40 mm

The immediate deflection due to specified live load shall be in accordance with CSA A23.3 and less than (l_n/360), except for the spillway which are limited to (l_n/600).

For serviceability limit state, the crack width resulting from service load combinations must be minimized to prevent leakage and corrosion of reinforcement. The crack control parameter, Z, as per A23.3 clause 10.6.1, shall not exceed:

- 21,000 N/mm for surfaces exposed to reservoir water including hydraulic passage;
- 25,000 N/mm for exterior exposure and surfaces against rock;
- 30,000 N/mm for interior exposure.

9 DESIGN CRITERIA SPECIFIC TO STRUCTURES

9.1 INTAKE AND POWERHOUSE

9.1.1 General

The Intake and Powerhouse consists of a concrete substructure and a steel superstructure. The concrete structure extends from the draft tube bottom at El. -34.1m to the generator floor at El. 45.50 m. Above the level of the generator floor to the roof, the building is a structural steel braced frame with a conventional insulated roof supported on steel deck. The overall height of the intake-powerhouse structure is about 86.6 m. The width in the direction of flow is approximately 78.4 m.

The concrete substructure of intake and the Powerhouse includes:

- Intake;
- Draft tube and draft tube cone;
- Drainage pit;
- Semi-spiral case;
- Turbine floor;
- Generator housing pit;
- Generator floor (composite steel-concrete floor);
- Tailrace deck (composite steel-concrete floor);
- South service bay (slab-on-grade);

- North service bay (slab-on-grade).

The steel superstructure includes:

- Steel frame for the main building;
- Crane runway girders for cranes and supporting columns;
- Girts for the exterior walls;
- Steel structure for the mezzanine floors;
- Steel floor and roof deck;
- Roof trusses and purlins.

9.1.2 Analysis Methods

For the structural analysis of the Powerhouse and the Intake, the preliminary design of the majority of elements are analysed with 2D models. The concrete slabs and walls are analyzed by plate method, using Advanced Design America, STAAD-Pro or design tables.

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For the intake and powerhouse concrete, it is necessary to perform the design with a more detailed method using Finite Element. The FE analysis will be carried out using a general-purpose finite element 3D software Abaqus and some programs tools for the pre and postprocessing of results. The thermal changes will be taken into consideration in the analysis.

The Powerhouse concrete substructure will be assumed to have an interstitial water pressure at the rock-concrete interface and between the inter-unit contraction joints - when such pressure does not reduce the combined loads on the element.

For the steel superstructure, the analysis will be performed with Advanced Design America software. The superstructure is divided in two distinct parts namely the powerhouse building and the mezzanines. It is designed for dead and live loads, snow and wind loads as well as seismic loading. The mezzanine floors may either composed of concrete slab over steel deck designed as composite construction or concrete slab analyzed as plates using steel deck as formwork only.

9.1.3 Drainage Gallery

A drainage gallery is provided along the upstream side of the Intake. This gallery is provided for the installation of drains and grouting works of the rock foundation.

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9.1.4 Dewatering Gallery

A dewatering gallery is provided along the downstream side of the powerhouse. This gallery gives access to the draft tube cone from a door located at El. -16.87 m. This floor also gives access to the inspection gallery located at El. -10.40 m. The steel beams used as a base for the runner maintenance platform are permanently stored at the ceiling level of this gallery so that they can be rolled into place when needed. All floor drainage water collected in the dewatering gallery trench flows to an oil/water interceptor, located in the sump pit, to trap oil contaminants

9.1.5 Intake Structure

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The water intake is a reinforced concrete structure built integrally with the powerhouse. The intake hydraulic passage for each generating unit is divided into three 6.50 m wide water passages.

The intake hoist building is located directly above the intake gate shaft. Removable roof covers above the intake hoist building allow for the installation and maintenance of the intake gates and the hoists. A trashrack cleaning system is provided and would operate on rail tracks mounted on concrete curbs/walkways on the intake deck.

The intake deck at El. 45.50 m connects the south transition dam to the centre transition dam, spillway and the north RCC dam. Moving concentrated loads such as mobile cranes and trucks handling equipment parts is used in the design of the intake deck including covers.   The vehicle loads used for the design are defined in section 7.3. Where mobile cranes or gantry crane (trash cleaning system) are in use, the design should include outrigger loads applied on mass concrete.

The concrete around the hydraulic passage at the Intake is subjected to the following loads:

- Self-weight of concrete and equipment;
- Live loads on the Intake deck;
- Mobile crane on the Intake deck;
- Uplift pressure;
- Hydrostatic pressure acting on the submerged concrete faces within the hydraulic passage (internal hydrostatic pressure);
- Hydrostatic pressure acting on the submerged concrete of the inter-unit contraction joints (external hydrostatic pressure);
- Hydrostatic pressure acting on the bulkhead gates or the head gates, if applicable;
- Thermal expansion of the ice sheet;
- Earthquake induced inertial and hydrodynamic forces;
- Thermal effects.

For the structural design of the Intake, the following conditions are analyzed:

- Normal - Hydraulic passage full with the reservoir at the full supply water level (FSL);
- Temporary - Hydraulic passage dewatered and external water pressure;
- Temporary - Hydraulic passage full with the reservoir at the full supply water level (FSL) and water-hammer calculated from 15% of static water head in the semi-spiral case, gradually reducing to zero at the face of the Intake;
- Unusual - Hydraulic passage full with the headwater level (1:1000);
- Extreme - Hydraulic passage full with the headwater level at the probable maximum flood (PMF).

9.1.6 Spiral Case

The concrete around the spiral case is subjected to the following loads:

- Self weight of concrete and equipment;
- Live loads on the floor directly above the spiral case;
- Loads transmitted by the stator sole plates;

- Loads transmitted by the lower bracket sole plates;
- Loads transmitted by the stay ring anchors;
- Loads transmitted by the generator brakes sole plates;
- Internal hydraulic pressure;
- External hydraulic pressure acting on the submerged concrete of the inter-unit contraction joints (external hydrostatic pressure);
- Thermal effects.

For the design of the spiral case, the following conditions are analyzed:

- Normal - Hydraulic passage full with the reservoir at the full supply water level (FSL);
- Temporary - Hydraulic passage dewatered and external water pressure acting on the submerged concrete of the inter-unit contraction joints;
- Temporary - Hydraulic passage full with the reservoir at the full supply water level (FSL) and water-hammer of 15% of static water head in the semi-spiral case;
- Unusual - Hydraulic passage full with the reservoir at headwater level (1:1000);
- Extreme - Hydraulic passage full with the headwater level at the probable maximum flood (PMF).

9.1.7 Draft Tube

The concrete around the draft tube is subjected to the following loads:

- Self weight of concrete;
- Loads on the floor directly above the draft tube;
- External hydraulic pressure;
- Uplift;
- Internal hydraulic pressure.

For the design of the draft tube, the following conditions are analyzed:

- Normal - Draft tube full and normal tailrace water level;

- Temporary – Draft tube dewatered and external water pressure acting on the submerged concrete of the inter-unit contraction joints.

9.1.8 South Service Bay

The South service bay is located at the south end of the powerhouse. In order to accelerate the turbine/generator installation work schedule, the service bay floor at El. 15.5m surface is designed sufficiently large to accommodate the assembly of two generator rotors and one runner simultaneously. After construction and assembly of the turbine/generator units, part of the service bay area will be occupied by the warehouse, as well as mechanical and electrical workshops. The unit control room, along with terminal room and communication room, is located on the first mezzanine above the service bay area and provides a view of the entire generator floor.

The south service is designed for the live loads defined in section 7.3. The main part of the south service floor consists of slab-on-grade structure. The rotor assembly stand will be located on slab-on-grade portion of south service bay.

9.1.9 North Service Bay

The North service bay is located at the north side of the powerhouse. During the construction period of the powerhouse, it will be used mainly as an unloading and laydown area for the civil contractor. It is necessary for an efficient planning of work for the civil and T&G contractors.

After the completion of civil work, part of the north service bay floor will be occupied by mechanical auxiliary equipment rooms (fire pumps, service pumps, water treatment room, compressor room and oil storage room).

The North service bay floor is designed for the live loads defined in section 7.3 and vehicles during the construction stage or maintenance operation.

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9.1.10 Tailrace Deck

The Tailrace deck is a composite steel-concrete floor located on the downstream side of the powerhouse. Four GSU transformers, along with a spare transformer, are located on the tailrace deck at El. 15.50 m. The transformers are connected to the switchyard using aerial conductors. Each transformer has prefabricated firewall enclosure and self contained oil retaining basin.

The Tailrace deck is designed for the occupancy loads defined in section 7.3. The vehicle and mobile crane loads shall be considered to act simultaneously with the GSU transformers in place. Under these conditions, the loading used for the design should include the weight of the heaviest pieces of equipment, such as the complete transformer plus the weight of the truck or mobile crane

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9.1.11 Steel Superstructure

9.1.11.1 General

The steel superstructure is composed of rigid frames with a main span of 30.20 m. It also includes a second span of 6.15 m to the intake concrete structure, adjacent to the main span, which is required for the 2 mezzanines. The steel structure is completed by secondary columns as well as by a series of roof purlins and girts. It supports all roof loads, including the loads of the different mechanical and electrical components (lights and ventilating units). The mezzanines are made-up of a steel structure with a 200 mm thick concrete slab on steel deck.

The downstream steel columns of permanent superstructure are extended to the turbine floor at El. 6.50m and to support composite steel-concrete floor at El. 15.50m.

9.1.11.2 Structural system at construction stage

During the construction of the powerhouse, the permanent steel superstructure is used as shelter for winter protection. The stability of the building during that period is provided by temporary bracing without considering any lateral support from the composite steel-concrete floor at El. 15.50 m.

The design shall include all forces due to construction sequence and temporary works of the steel superstructure. The erection sequence assumed for the steel superstructure is shown on the drawings with the maximum crane loads.

For the temporary construction stage, the steel superstructure is designed for the following conditions:

- One main crane for maximum lifting capacity of 200 tonnes;
- Auxiliary crane for maximum lifting capacity of 50 tonnes.

The erection sequence assumed for the steel superstructure is shown on the drawings with the maximum crane loads. The wind pressure considered during the construction stage is for a period of 10 years of recurrence.

9.1.11.3 Structural system for long-term operation

During operation, the stability of the building in the longitudinal direction is provided by vertical cross bracings which are located under the crane girders and on the centerline of the building columns.

In the upstream-downstream direction, the stability of the building between grid lines 6 and 24 is provided by horizontal supports to the intake massive concrete. These supports, located at the roof level and at the first mezzanine, are designed to allow free vertical and longitudinal movement between the structural steel and the massive concrete wall of the intake.

The wind columns of 28m height, which are located on grid lines 1 and 28, will be supported at mid height by horizontal wind trusses. Between grid lines 1 and 7, the horizontal forces are transmitted to the vertical cross bracing on grid line 1 and to the upstream wall on grid line 6.

The maximum wheel loads of main cranes used for the design of the steel superstructure are shown on the drawings and in Appendix A (Figures A8 and A9).

In addition to the vertical and horizontal forces from cranes and wind, the steel superstructure is designed for the forces from conductors and guard wires, which are attached to the building columns.

9.1.11.4 Load factors for crane loads

The Powerhouse is equipped with two main bridge cranes. The lifting of the rotor as well as the lifting of the runner, shaft and head cover assembly will require the use of the two bridge cranes in tandem in combination with the main follower beam and the two auxiliary lifting beams.

Each bridge crane comprises two main trolleys and one auxiliary trolley.The lifting test certification of the bridge cranes will follow a very strict procedure or specification defined by mechanical discipline and coordinated with civil discipline for the steel superstructure.

The maximum crane loads acting on the steel superstructure occurs when the two cranes work in tandem and lifting the rotor. The total weight of the rotor is well known and the lifting operation of the rotor, which is not very frequent, is done at very slow speed and following a well defined procedure. Therefore, maximum wheel loads from the two bridges cranes working in tandem occur only several times during the life time of the structural steel building.

The load factors to be used for the design of the steel superstructure are:

- Self weight of the cranes : 1.25
- Weight of lifted equipment (rotor, runner, shaft and head cover assembly) : 1.25
- Self weight of lifting beam: 1.25
- Horizontal, lateral and longitudinal, crane breaking force: 1.25
- Impact load factor: 1.25

These load factors may be slightly different from the value proposed by National Building Code of Canada (NBC) where a load factor of 1.5 plus an impact of 25% are proposed for forces coming from cranes (live loads).

For steel superstructure design, the common practice in the industry for major power plants is to use a load factor of 1.25 for the maximum lifting capacity of two cranes working in tandem plus an impact of 25%. These factors are considered appropriate for heavy loads and light duty cranes and where the maximum lifting capacity has a big impact on the steel tonnage and costs of the steel superstructure.

9.2 SPILLWAY

9.2.1 General

The Spillway is a concrete structure which will include one temporary and two vehicle bridges, the gates and stoplogs, and the steel hoisting structure. The design of the gates, stoplogs, and hoisting structure are addressed in other design criteria (Hydro-mechanical).

1 The spillway concrete structure is approximately 68 m long, 75 m wide and 40 m high with five (5) water passages. The structure is located as close as possible to the south of the existing Churchill River shoreline but at sufficient distance to allow construction of the North Transition Dam and the longitudinal Riverside Cofferdam in dry conditions.

All five spillway passages will be operated during both the diversion and the operation phases. During the two (2) winters of the diversion phase, the reservoir level will be raised to El. 25.0 m. Subsequently, the reservoir will be impounded to El. 39.0m in preparation for the powerhouse commissioning and the operation phase.

The different design conditions and water levels at the spillway during the construction period are addressed in other design criteria (Design criteria-Hydraulic).

The spillway discharge channel linking the spillway to the Churchill River is largely excavated in rock with depth varying from 4 to 10 m. The channel has a concrete slab-on-grade and concrete side walls over its full length for erosion protection.

9.2.2 Analysis Method

For the structural analysis of the spillway concrete structure, the majority of elements are analysed with 3D models using STAAD-Pro software and designed by conventional methods. All the volume variation due to drying, shrinkage or thermal changes will be taken in consideration in the analysis.

For the bridge structures, the analysis will be performed using the simplified methods outlined in the Canadian Highway Bridge Design Code (CSA-S6), except that large vehicle loads may be verified using a software program called SECAN which utilizes the semi-continuum method of analysis, one of the approved advanced methods provided for in the bridge code.

9.2.3 Design and Loads

9.2.3.1 Concrete Substructure

The concrete around hydraulic passages of the Spillway is subjected to the following loads:

- Self weight of concrete and equipment;
- Live loads on the structure;
- External hydraulic pressure;
- Uplift;
- Internal hydraulic pressure;
- External thermal loads.

For the design of the Spillway, the following conditions are analyzed (where ‘gate removed’ is indicated, it is assumed water is flowing through the passage):

- Normal - Gate removed with the reservoir at the full supply water level (FSL);
- Normal - Hydraulic passage dewatered with loads and external water pressure;
- Unusual - Gate removed with the reservoir at the unusual water level (1:1000);
- Unusual - Hydraulic passage dewatered with the reservoir at the unusual water level (1:1000);
- Extreme - Gate removed with the reservoir at the probable maximum flood (PMF) level.
- Normal - Construction scenario – gate removed prior to construction of the rollway, water level at construction reservoir level;

- Normal - Construction scenario – hydraulic passage dewatered prior to construction of the rollway but adjacent passage has gate removed and/or gate lowered with the FSL;
- 1 An additional load case is analyzed to determine the sensitivity of the spillway to a scenario wherein the gate becomes non-operational during a PMF. This case is not required to meet the acceptance criteria because there are back-up systems in place to ensure the gates are operational, however the results are used to highlight the importance of the backup systems.

1 *9.2.3.2 Bridge Structures*

For the design of the permanent upstream bridge structures, the following vehicles loads are considered:

Description	Design loads
Gantry Crane/ Trash Cleaning System	
• Assumed self-weight of Gantry Crane / Trash Cleaning System including counterweight:	240 t
• Assumed gantry crane hoist capacity:	40 t
• Assumed stoplog weight:	25 t
• Maximum factored load on one leg of gantry while lifting a stoplog is based on the weight of the gantry equally distributed to two legs on one side of the machine, and the stoplog jammed while lifting in which case the hoist is limited to 2.1 times its capacity, multiplied by a live load factor of 1.05; 1.05 x (0.50 x 240t + 2.1 x 40t)	214.2 t
• Factored load on the leg of the crane at the opposite end of the gantry; 1.05 x (0.50 x 240t)	126 t
• Gantry is centered on the width of the hydraulic passage during lifting of stoplogs	

• Maximum factored load during movement of the gantry while transporting a stoplog is based on 85% of the weight of the gantry and stoplog distributed equally to two legs of the gantry on one side of the gantry multiplied by a live load factor of 1.5 and a dynamic load factor of 1.3; 1.5 x 42.5% x (240t + 25t)x1.3 • Gantry may be positioned anywhere along the length of the bridge during transport of the stoplog. This load may also be combined with a tractor-trailer positioned anywhere on the bridge where it does not interfere with the operation of the gantry. The trailer will be assumed to hold a stoplog, and the truck + trailer + stoplog weights will be multiplied only by a dead load factor of 1.25, no dynamic factor will be used because the truck will be assumed to be parked	220 t
• Maximum factored load during operation of the trash cleaning system during trash removal operations is based on the weight of the machine plus 5 t lifted load acting on one leg of the gantry multiplied by a live load factor of 1.05 • Gantry may be positioned anywhere along the length of the bridge during trash cleaning operations.	257 t
Assumed empty tractor-trailer (unfactored)	
• Front axle:	54 kN
• Mid tandem axles combined:	172 kN
• Rear tandem axles combined:	154 kN
Mobile crane (200 metric tonnes)	
• Assumed self-weight of 200t American 9530 crane complete with counterweight:	145 t
• Assumed maximum lifted weight by the crane:	40 t
• Maximum load on one outrigger is based on the crane self-weight plus the lifted weight supported on one outrigger, multiplied by a live load factor of 1.05: 1.05 x (145t + 40t)	194.3 t

• The crane outrigger may be positioned anywhere along the length of the bridge (Note that the dimensions restrict the outrigger from being placed on the innermost girders which are not designed for the outrigger directly above them)	
Grove RT700E crane • Assumed self-weight of crane complete with outrigger: • Assumed maximum lifted weight by the 50 t crane: • Maximum load on the outrigger: 1.05 x (40.8t + 10t) • The crane outrigger may be positioned anywhere on the bridge	40.8 t 10 t 53.3 t
Design truck from CSA S6-06 • CL-625 positioned anywhere on the bridge.	

For the design of the permanent downstream bridge structures, the following vehicles loads are considered:

Mobile crane (200 metric tonnes) • Assumed self-weight of 200t American 9530 crane complete with counterweight: • Assumed maximum lifted weight by the 200 t crane without the use of outriggers: • Maximum load on one group of tires at one corner of the crane is based on the crane self-weight plus the lifted weight multiplied by a live load factor of 1.05: 1.05 x (145 t + 15t) • The crane may be positioned anywhere along the length of the bridge	145 t 15 t 168 t
Grove RT700E crane • Assumed self-weight of 50 t crane complete with outrigger: • Assumed maximum lifted weight by the 50 t crane: • Maximum load on the outrigger: 1.05 x (40.8t + 10t) • The crane outrigger may be positioned anywhere on the bridge	40.8 t 10 t 53.3 t

Design truck from CSA S6-06
• CL-625 positioned anywhere on the bridge.

For the design of the temporary downstream bridge structures, the following vehicles loads are considered:

Mobile crane (200 metric tonnes)	
• Assumed self-weight of 200t American 9530 crane complete with counterweight:	145 t
• Assumed maximum lifted weight by the 200t crane without the use of outriggers:	15 t
• Maximum load on one group of tires at one corner of the crane is based on the crane self-weight plus the lifted weight multiplied by a live load factor of 1.05: 1.05 x (145t + 15t)	168 t
• The 200t crane may be positioned anywhere along the length of the bridge	
Loaded Caterpillar 773e off-road haul truck	
Design truck from CSA S6-06	
• CL-625 positioned anywhere on the bridge.	

9.3 RCC DAMS

9.3.1 General

The following section presents the main design consideration for the RCC dams. It includes general comments on such details as:

- Type of RCC mix;
- Drainage galleries and their locations;
- Vertical joints and their spacing;
- Type of facing;

- Drains;
- Rollway surface;
- Layer details;
- Instrumentation.

9.3.2 RCC Mix Design

In general, RCC mixes are classified in three categories:

- Low cementitious mix (60 to 120-130 kg/m^3);
- Medium cementitious mix (120-130 to 180-190 kg/m^3); and
- High cementitious mix (180-190 to 250 kg/m^3).

Given the relatively short height of the dam (i.e. maximum of 35 meters), a medium cementitious mix design is considered for the RCC dams. The mix will contain sufficient moisture content to include the necessary additives, including the air-entraining agent. In turn, this mix will provide the seepage control required at the interface of the RCC layers.

The main parameters and criteria for the RCC mix are summarized below:

- Compressive strength of 16 MPa at 90 days, 20MPa at 365 days;
- Cementitious content 150 to 190 kg/m^3;
- Cement and fly ash content ratio 40%/60% with increasing and decreasing increments of 10 kg/m^3 (see detailed mix program);
- Maximum aggregate size - 40 mm;
- Target Vebe time - 20 to 25 seconds;
- Air content 8%;
- Set retarder agent 12 to 18 hours.

In accordance with accepted practice for RCC dams, an RCC trial mix program will be required to confirm the RCC mix design. Given the impact of site specific construction materials, this is an important element of RCC dams.

The details of the proposed RCC trial mix program will be presented in a separate technical note. These mixes are to be prepared well in advance of the dam construction in order to allow for a proper review and analysis of the results, and to possibly incorporate modifications to the mix design.

9.3.3 Galleries

It is understood that grouting works will be required for the foundation. Hence, a gallery will be included in the design to provide access for the grouting works. In addition, this gallery will allow for the installation of foundation drains, internal drains for the RCC and instrumentations.

To facilitate drainage by gravity, the lowest section of the gallery will be set at elevation 8.16 m which is approximately 1.0 m above the tailwater level for the 1:1000 yr flood and provides sufficient flood protection for the gallery while keeping it as low as possible. The drainage gallery will be located 5 m from the upstream face and RCC is placed between the gallery and the upstream face

The gallery will incorporate a trench along the upstream side where the drain holes will be provided. A small collector drain may be considered along the downstream face with shallow transverse drains to direct any seepage to the main drain located upstream.

9.3.4 Vertical Joints

Vertical joints will be provided at every 25 m to 30 m. Based on experience, this spacing should be sufficient for the control of cracking for a medium cementitious mix. The final spacing will be determined based on the total cementitious content and the fly ash percentage.

PVC waterstops will be provided at the vertical joints. These will be located approximately 0.3 m from the dam face and will be embedded in either a facing concrete or a grout enriched RCC.

For the concrete crest and rollway, the vertical joints would be located at the RCC joints and in the middle of the RCC monoliths (i.e. 12.5 m or 15 m spacing).

9.3.5 Facing Detail

Given the exposure of the upstream face of the dam to freeze-thaw conditions and erosion, a facing detail will be required. This facing detail may be provided with either a facing mix (i.e. conventional concrete) or a grout enriched RCC.

The facing concrete will have a compressive strength of 20 MPa at 28 days.

Two methods are currently used for enriching RCC (GEVR and GE-RCC). The GE-RCC method, whereby the grout is placed on the RCC after it has been spread, is considered more appropriate. The trial placement section and site tests will determine which method is most efficient.

Given that a 2 m bedding application is considered along the upstream edge of the layers for enhancing seepage control, the same mix is considered for the facing application. This application has been successfully used on several RCC projects in which SNC-Lavalin has participated.

9.3.6 Drains

Foundation drains will be provided. The diameter, spacing and depth will be confirmed after the necessary geotechnical field investigations have been carried out. However, 75 mm diameter holes with a spacing of 3 m would be considered.

Internal drains are a standard detail for RCC dams. Given the large number of joints usually spaced at 300 mm apart, these drains are essential to drain the RCC mass in the event seepage migrates into the joints.

The provision of drain holes is considered less critical when the sloped layer method is used to place the RCC. However, considering that this measure reduces uplift pressure and will facilitate the drainage of seepage, drain holes are considered.

Also, consideration could be given to using inclined internal drain holes. These inclined drain holes have the advantage of intercepting potential cracks in the dam as well as crossing the monolith joints.

9.3.7 Rollway Surface

A reinforced concrete rollway surface is currently considered. The concrete would have a compressive strength of 25 MPa at 28 days with steel reinforcement.

Dowels would be provided along the RCC steps to properly attach the rollway rebar mat to the RCC mass. The rollway thickness would be approximately 1 m.

9.3.8 Layers Detail

RCC layers are generally placed in horizontal layers 300 mm thick. Occasionally layers up to 400 mm or slightly more are placed. Placing the RCC in horizontal layers 300 mm thick will probably result in many cold joints and the need to apply a bedding mix quite frequently, which will hamper the progress of the work.

An RCC placement method using sloped layers (1:10 to 1:15 slope) considerably enhances the quality of bond. These bands of sloped layers are approximately 2.5 m to 3 m in height. This method was specifically developed in China to improve the quality of bond and reduce the number of cold joints.

Given the long and relatively narrow RCC dams, this placement method is best suited for the project.

To improve seepage control, a 2 m band of bedding mix would be provided along the upstream face of the dam. The same concrete mix that is used for the facing application would also be used for the bedding application. This technique was successfully used on several previous SNC-Lavalin projects. Placing this narrow 2 m band of bedding mix will not affect the progress of the RCC placement.

9.3.9 Instrumentation

The RCC dams are relatively low with a maximum height of 35 m. Consequently, and in accordance with accepted practice, the instrumentation that will be required will be fairly conventional.

The following instrumentation is considered for the time being for the RCC dams:

- Vibrating wire piezometers;

- Open standpipe piezometers;
- Crest monuments;
- Joint meters; and
- Flow weirs.

Preliminary quantities for the instrumentation are presented in the following table.

Instrument	Preliminary quantities	Comments
Vibrating wire piezometer	10	To be installed in PVC conduits.
Open stand pipe piezometer	10	
Crest monuments	30	Located at monolith joints.
Joint meters	12	Spread out over length of dam.
Flow weir (V-notch type)	7	Spread out as required

Approximately 50 percent of the piezometers will be of the vibrating wire type. These will be installed inside PVC conduits which will also act as stand pipe piezometers. In case of failure of the vibrating wire piezometers, the holes will still be used as stand pipe piezometers. The remaining 50 percent of the piezometers will be only of the stand pipe type. These are considered more reliable over the long term.

1

9.4 TRANSITION DAMS

9.4.1 General

The transition dams are a CVC gravity dam type founded on bedrock. The crest of the transition dams is set at level 45.5 m in order to coincide with the Intake-Powerhouse crest level. Each transition dam is divided into a few blocks and has a foundation drainage curtain over its full length. Double PVC waterstops plus one hydrophilic or bentonite waterstop are provided at contraction joints.

9.4.2 Centre Transition Dam

The Centre transition dam is conventional concrete dam approximately 30 m high with a crest width of 10.8 m located between the powerhouse and the spillway. The dam is approximately 65 m long and includes a drainage gallery to reduce the uplift pressures. The drainage gallery will be accessible from the powerhouse via stair no. 7 in the powerhouse and water collecting in the drainage gallery is drained to the tailrace by gravity.

The centre transition dam includes an elevated deck (platform) to support the spillway electrical building, which houses the generator and MCC's.

9.4.3 North Transition Dam

North Transition Dam is conventional concrete dam approximately 38 m high and 15 m long located between the gated spillway and the North RCC Dam. The crest is 9.3 m wide at El. 45.5 m. The access to the North RCC dam drainage gallery will be through the north transition dam gallery.

9.4.4 South Transition Dam

The South transition dam is conventional concrete dam approximately 20 m high and 50 m long with retaining wall at south end. The surface in contact with South rockfill dam is sloped in two directions; 1H:7V in the vertical plane and 5° in the horizontal plane to ensure positive contact between the concrete and the rockfill dam core.

A minimum horizontal clearance is kept between the downstream toe of the dam and the excavation for the powerhouse to ensure there is approximately a 45° to 50° vertical angle between the toe of the powerhouse excavation and the downstream toe of the dam. The drainage gallery will be accessible from the powerhouse via stair no. 1 in the powerhouse and water collecting in the drainage gallery is drained to the tailrace by gravity.

9.4.5 Instrumentation

The Transition dams are relatively low with a maximum height of 38 m. Consequently, and in accordance with accepted practice, the instrumentation that will be required will be fairly conventional. The following instrumentation is considered for the transition dams:

- Piezometers;

- Crest monuments;
- Flow weirs.

9.4.6 Skin Reinforcement

Reinforcing steel will be provided around the drainage gallery and at the crest for all Transition dams. Design parameters for lift heights and the use of skin reinforcement for the upstream and downstream face of the dam are addressed in separate technical note.

1

9.5 SEPARATION WALL

The separation wall is a conventional concrete dam approximately 100 m long located on the south side of the spillway channel. It will form the divide between the spillway and the powerhouse excavation.

The separation wall is built to a minimum elevation of 26.0 m at the upstream portion, with an intermediate portion at El. 35 m and the downstream portion near the centre transition dam up to El. 40 m. The three elevations are required for hydraulic reasons to separate the flow between the gated spillway and intake.

The separation wall structure will also act as support of a temporary bridge crossing the diversion channel to the riverside cofferdam, giving access to the north upstream cofferdam and the north spur.

For the design of the separation wall, unbalanced ice load and a differential water level during construction and operation are considered.

10 STABILITY ANALYSIS

10.1 GENERAL

The approaches used to evaluate the stability of retaining structures are those recommended by the following recognized authorities:

- Canadian Dam Association (CDA), Dam Safety Guidelines, 2007;

- US Army Corps of Engineers (USACE): Engineering Manual EM1110-2-2200, Gravity Dam Design, 1995;
- Federal Energy Regulatory Commission (FERC): Engineering Guidelines for the Evaluation of Hydropower Projects, Gravity Dams, 2002; and
- US Army Corps of Engineers (USACE): Engineering Manual 1110-2-2502, Retaining and Flood Walls, 1989.

According to these procedures, the stress distribution was obtained using the rigid body assumption and a trapezoidal distribution of the reaction. Moreover, the uplift is considered as an active force and therefore is included in the calculation of the reaction so that the reaction and the stresses computed include the effect of the uplift.

For pseudo-static seismic loading cases (method of the seismic coefficient), the uplift diagram is considered constant and equal to the diagram prevailing before the earthquake. Any formation or increase in length of crack during the earthquake does not affect the uplift pressure diagram as in other loading cases.

The stability analysis verifies that the structures meet, under each established load case, the following criteria:

- Overturning (position of resultant);
- Sliding;
- Buoyancy;
- Allowable compressive stress (rock and concrete).

10.2 OVERTURNING

The overturning stability is calculated by applying all the vertical and horizontal loads defined by each loading condition and calculating the sum of moments caused by these forces relative to the toe of the structure. The acceptance criteria are:

- Normal case: A structure is considered stable when the analysis shows no tension at upstream or downstream end. This criteria is met when the resultant force is within the median third of the base (100% of the base in compression);
- Unusual case: The resultant force is in the median half of the base (75% of the base in compression);
- Extreme case: The resultant force is within the base so that allowable stresses are respected.

10.3 SLIDING

The sliding stability is verified by using the following equations. The calculated safety factors must be greater or equal to the acceptance criteria given in Section 10.6.

The sliding stability factor of safety for horizontal plane is determined using the following equation:

$$SF = \frac{\sum V \tan ø + cA}{\sum H}$$

Where:

SF	=	Safety Factor
ΣV	=	Sum of vertical forces including uplift;
ΣH	=	Sum of horizontal forces;
c	=	Cohesion;
A	=	Area of surface in compression;
tan ø	=	Friction coefficient;

10.4 BUOYANCY

The safety factor against buoyancy is calculated with the following equation.

$$SF = \frac{W}{U}$$

Where:

W = Sum of vertical forces applied downward (kN);

U = Uplift (kN).

10.5 ALLOWABLE STRESSES

The compressive rock (or concrete) stresses are calculated by taking into account the state of cracked or uncracked sections. It is assumed that there is no effective tensile stress (i.e. bond) between the concrete structure and the rock foundation.

A crack is considered to occur if the analysis indicates that the entire base is not in compression. If the analyzed section cracks, full reservoir pressure acts over the cracked length. From the downstream end of the crack, uplift decreases linearly in the uncracked section until it reaches the drain or the downstream toe (See also Section 7.6). The length of the crack is determined through an iterative process until the overturning analysis indicates that the remaining portion of the base of the structure is in compression.

10.6 ACCEPTANCE CRITERIA

For the purpose of evaluating the stability, combinations of loads are categorized by nature of their likelihood of occurrence.

- Group N: Load combinations during normal operating conditions or planned construction;
- Group U: Load combinations during unusual conditions;
- Group E: Load combinations during extreme conditions.

The safety factors and acceptance criteria for stability analysis are given in the following table.

Stability Criteria and Safety Factors

Loading Case	Resultant Location	SF Sliding†		SF Uplift	Compressive Stress on Rock	Concrete Stress
		Friction only	Friction and cohesion			
Normal	Middle third	1.5	3.0*	1.2	≤ allowable stress	0.3 $f_c^{'}$
Unusual	Middle half	1.3	2.0*	1.1	≤ allowable stress	0.5 $f_c^{'}$
Extreme	Within the base	1.1	1.3*	1.05	≤ 1.33 allow. stress	0.5 $f_c^{'}$
Post-Earthquake	Within the base	1.1	1.5*	1.1	≤ allowable stress	0.5 $f_c^{'}$

† If the sliding surface is considered unbonded, only friction is relied on to ensure the stability of the structure

* Factors are based on limited or no test data.

$f_c^{'}$ = compressive strength of concrete or RCC.

The acceptable factors of safety should be consistent with the adopted shear strength values and load combinations. The proposed acceptance criteria for the sliding safety factor will be based on the peak shear strength parameters and using the same approach as SNC-Lavalin practice which requires that safety factors against sliding be checked for both shear-friction conditions: with and without considering cohesion. By adopting this approach, we ensure that the contribution from the friction component in the global safety factor is not marginal compared to cohesion component, i.e. the cohesion is not relied upon excessively to provide the resistance (see Technical Memo entitled "Sliding Safety Analysis of Gravity Dams and Common Practice", MFA-SN-CD-2300-CV-RP-0002-01).

10.7 SHEAR STRENGTH PARAMETERS

The basic criteria for the safety assessment against sliding are the ratio between the driving forces and the resisting forces (available shear strength) along the considered sliding surface. The stability assessment against sliding has to be carried out considering the potential sliding surfaces in the gravity retaining structure body, at the structure-foundation interface and in the foundation.

In general, the available shear strength is expressed by a Mohr-Coulomb criterion and consists of the frictional and the cohesion component. The shear strength parameters to be used as a basis for the stability assessment against sliding of concrete gravity structures at Muskrat Falls will depend on the following parameters:

- Properties of the rock foundation and concrete;
- Roughness of the excavated surface;
- Angle of friction between the two materials;
- Stress distribution along the contact surface.

The following shear strength parameters are used for the stability analysis of the Powerhouse, Spillway and the Transition gravity dams (see Technical Memo entitled "Sliding Safety Analysis of Gravity Dams and Common Practice", MFA-SN-CD-2300-CV-RP-0002-01).

Interface	Peak Shear Parameters	
	Ø (degree)	Cohesion (kPa)
Lift joints	50	250
Concrete -Rock	50	300
Rock discontinuities	51	150

Ø = angle of friction at the interface

The following shear strength parameters are used for the stability analysis of RCC dams.

Interface	Peak shear parameters	
	Ø (degree)	Cohesion (kPa)
RCC lift joints	43	250
RCC / Rock interface	45	300
Rock discontinuities	51	150

10.8 LOAD COMBINATION FOR STABILITY ANALYSIS

10.8.1 Powerhouse – Load Combinations

The stability of the Powerhouse is evaluated for the following basic load combinations:

Comb. Name	General Description	Dead Load	U/S (m)	D/S (m)	Water Passage	Drain Efficiency	Ice (kN/m)	OBE	MDE
N1	Unit in operation, with ice	✓	39.0	6.65	Full	67%	150	-	-
N2	Partial dewatering for runner inspection, with ice	✓	39.0	6.65	Empty from head gates to base of draft tube cone	67%	150	-	-
N2a	Partial dewatering for runner inspection, with ice	✓	39.0	2.34	Empty from head gates to base of draft tube cone	67%	150	-	-
N2b	Partial dewatering for runner inspection	✓	39.0	1.30	Empty from head gates to base of draft tube cone	67%	-		
N3	Full dewatering of hydraulic passage for inspection, with ice	✓	39.0	6.65	Empty from bulkhead gates to draft tube gates	67%	150	-	-
N3a	Full dewatering of hydraulic passage for inspection	✓	39.0	2.34	Empty from bulkhead gates to draft tube gates	67%	150	-	-
N3b	Full dewatering of hydraulic passage for inspection, with ice	✓	39.0	1.30	Empty from bulkhead gates to draft tube gates	67%	-	-	-
U1	Partial dewatering for runner insp. With 0% drain eff. + ice	✓	39.0	6.65	Empty from head gates to base of draft tube cone	0%	150	-	-
U1a	Partial dewatering for runner insp. With 0% drain eff.	✓	39.0	2.34	Empty from head gates to base of draft tube cone	0%	150	-	-
U1b	Partial dewatering for runner insp. With 0% drain eff. + ice	✓	39.0	1.30	Empty from head gates to base of draft tube cone	0%	-	-	-

Comb. Name	General Description	Dead Load	U/S (m)	D/S (m)	Water Passage	Drain Efficiency	Ice (kN/m)	OBE	MDE
U2	Full dewatering of hydraulic passage for insp., with ice	✓	39.0	6.65	Empty from bulkhead gates to draft tube gates	50%	150	-	-
U2a	Full dewatering of hydraulic passage for insp.	✓	39.0	2.34	Empty from bulkhead gates to draft tube gates	50%	150	-	-
U2b	Full dewatering of hydraulic passage for insp., with ice	✓	39.0	1.30	Empty from bulkhead gates to draft tube gates	50%	-	-	-
U3	Partial dewatering for runner inspection, without ice	✓	40.0	7.21	Empty from bulkhead gates to base of draft tube cone	67%	-	-	-
U4	Post-earthquake with unit in operation, with ice	✓	39.0	6.65	Full	67% with cracked section of worst E comb.	150	-	-
U5	Unit in operation with unusual ice	✓	39.0	6.65	Full	67%	225	-	-
U6	Partial dewatering of hydraulic passage for insp with unusual ice	✓	39.0	6.65	Empty from bulkhead gates to base of draft tube cone	67%	225	-	-
E1(1)	PMF Conditions	✓	45.1	12.61	Full	67%	-	-	
E2	Horizontal MDE with Partially dewatered unit	✓	39.0	6.44	Empty from head gates to base of draft tube cone	67%	-	-	1H:1/3V
E2a	Vertical MDE with Partially dewatered unit	✓	39.0	6.44	Empty from head gates to base of draft tube cone	67%	-	-	1/3H:1V
E3	Horizontal OBE with part. dewatered unit + ice	✓	39.0	6.65	Empty from head gates to base of draft tube cone	67%	150	1H:1/3V	-
E3a	Vertical OBE with part. dewatered unit + ice	✓	39.0	6.65	Empty from head gates to base of draft tube cone	67%	150	1/3H:1V	-

(1) An additional load case is analyzed to determine the sensitivity of the powerhouse to a scenario wherein the hydraulic passage is fully dewatered during a PMF. It is not a mandatory load combination to be considered for the stability assessment of the powerhouse.

U/S: Upstream Water Level — D/S: Downstream Water Level

OBE: Operating Basis Earthquake — MDE: Maximum Design Earthquake

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10.8.2 Spillway – Load Combinations

The stability of the Spillway is evaluated for the following basic load combinations:

Comb. Name	General Description	Dead Load	U/S (m)	D/S (m)	Water Passage	Drain Efficiency	Ice (kN/m)	OBE	MDE
N1	Permanent stoplogs in position U/S and D/S, with ice, 1:50 levels	✓	39.0	6.44	Empty D/S of stop log	N/A	150	-	-
N2	Gate in position, with ice, 1:50 levels	✓	39.0	6.44	Empty D/S of gate	N/A	150	-	-
N3	Gate lifted, 1:50 levels	✓	39.0	6.44	Water profile from hydraulic analysis	N/A	-	-	-
N4	Construction case, winter – gate in position, no rollway, with ice, 1:20 levels	✓	25.0	8.0	Tailwater level D/S of gate	N/A	150	-	-
N5	Construction case, winter – gate in position, no rollway, with ice, minimum levels	✓	25.0	4.1	Empty D/S of gate	N/A	150	-	-
N6	Construction case, fall 2016 – temporary stoplogs in position, 1:20 levels after impoundment	✓	39.0	4.46	No water between stoplogs	N/A	150	-	-
U1	Gates in position with ice, 1:50 levels	✓	39.0	6.44	Tailwater D/S of gate	N/A	225	-	-
U2	Gates in position, 1:1000 levels	✓	40.0	7.21	Tailwater D/S of gate	N/A	-	-	-
E1(1)	PMF Conditions	✓	45.1	12.61	Water profile from hydraulic analysis	N/A	-	-	
E2	Horizontal MDE with gates in position	✓	39.0	6.44	Empty D/S of gate	N/A	-	-	1H:1/3V
E2a	Vertical MDE with gates in position	✓	39.0	6.44	Empty D/S of gate	N/A	-	-	1/3H:1V
E3	Horizontal OBE with gates in position + ice	✓	39.0	6.44	Empty D/S of gate	N/A	150	1H:1/3V	-
E3a	Vertical OBE with gates in position + ice	✓	39.0	6.44	Empty D/S of gate	N/A	150	1/3H:1V	-
E4	Post-earthquake with gates in position, with ice	✓	39.0	6.44	Empty D/S of gate	N/A	150	-	-

N/A = Not Applicable

1

(1) An additional load case is analyzed to determine the sensitivity of the spillway to a scenario wherein the gate becomes non-operational during a PMF. It is not a mandatory load combination to be considered for the stability assessment of the spillway because there are back-up systems in place to ensure the gates are operational.

10.8.3 RCC and Transition Gravity Dams – Load Combinations

The stability of the RCC dams, including Transition dams, is evaluated for the following basic load combinations:

	North RCC dam		South RCC and Transition gravity dams	
Usual Loading Cases	N1	Upstream water level: 39.0 m Downstream water level (1:50 yr): 6.44 m Ice load – 150 kN/m Drain efficiency of 67%	N1	Upstream water level: 39.0 m Downstream water level: rock level Ice load – 150 kN/m Drain efficiency of 67%
Unusual Loading Cases	U1	Upstream water level: 39.0 m Downstream water level : 6.44 m Ice load – 225 kN/m Drain efficiency of 67%	U1	Upstream water level: 39.0 m Downstream water level: rock level Ice load – 225 kN/m Drain efficiency of 67%
	U2	Upstream water level: 39.0 m Downstream water level : 6.44 m Ice load – 150 kN/m Drain efficiency of 0 %	U2	Upstream water level: 39.0 m Downstream water level: rock level Ice load – 150 kN/m Drain efficiency of 0 %
	U3	Upstream water level: 40.0 m Downstream water level: 7.21 m No ice load Drain efficiency of 67 %	U3	Upstream water level: 40.0 m Downstream water level: rock level No ice load Drain efficiency of 67 %
Extreme Loading Cases	E1	Upstream water level: 45.1 m Downstream water level : 6.44 m No ice load Drain efficiency of 67%	E1	Upstream water level: 45.1 m Downstream water level: rock level No ice load Drain efficiency of 67%
	E2a	Upstream water level: 45.1 m Downstream water level: 12.61 m PMF without GI No ice load Drain efficiency of 67%	E2a	N/A
1	E2b(1)	Upstream water level: 45.1 m Downstream water level: 12.61 m No ice load Drain efficiency of 50%	E2b	Upstream water level: 45.1 m Downstream water level: rock level except 12.61 m for North Transition Dam. No ice load Drain efficiency of 50%
	E3	Upstream water level: 39.0 m Downstream water level : 6.44 m No ice load Full sustained horizontal ground acceleration (1:10 000) One third of sustained vertical ground	E3	Upstream water level: 39.0 m Downstream water level: rock level No ice load Full sustained horizontal ground acceleration (1:10 000) One third of sustained vertical

		North RCC dam		South RCC and Transition gravity dams
		acceleration (1:10 000) Drain efficiency of 67%		ground acceleration (1:10 000) Drain efficiency of 67%
	E4	Upstream water level: 39.0 m Downstream water level: Rock level No ice load Full sustained vertical ground acceleration (1:10 000) One third of sustained horizontal ground acceleration (1:10 000) Drain efficiency of 67%	E4	Upstream water level: 39.0 m Downstream water level: rock level No ice load Full sustained vertical ground acceleration (1:10 000) One third of sustained horizontal ground acceleration (1:10 000) Drain efficiency of 67%
	E5	Upstream water level: 39.0 m Downstream water level: 6.44 m Ice load – 150 kN/m Drain efficiency of 67% Full sustained horizontal ground acceleration (1:200) One third of sustained vertical ground acceleration (1:200)	E5	Upstream water level: 39.0 m Downstream water level: rock level Ice load – 150 kN/m Drain efficiency of 67% Full sustained horizontal ground acceleration (1:200) One third of sustained vertical ground acceleration (1:200)
	E6(1)	Upstream water level: 39.0 m Downstream water level: 6.44 m Ice load – 225 kN/m Drain efficiency 0%	E6	Upstream water level: 39.0 m Downstream water level: rock level Ice load – 225 kN/m Drain efficiency 0%
	E7(1)	Upstream water level: 40.0 m (1:1000) Downstream water level: 7.21 m No ice load Drain efficiency of 0%	E7	Upstream water level: 40.0 m (1:1000) Downstream water level: rock level No ice load Drain efficiency of 0%
Post-Earthquake	PE1	Upstream water level: 39.0 m Downstream water level: 6.44 m No ice load No cohesion Regular uplift	PE1	Upstream water level: 39.0 m Downstream water level: rock level No ice load No cohesion Regular uplift

(1) It is not a mandatory load combination to be considered for the stability assessment of the dams. It is considered for sensitivity analysis on impact of drain efficiency.

10.8.4 Separation Wall – Load Combinations

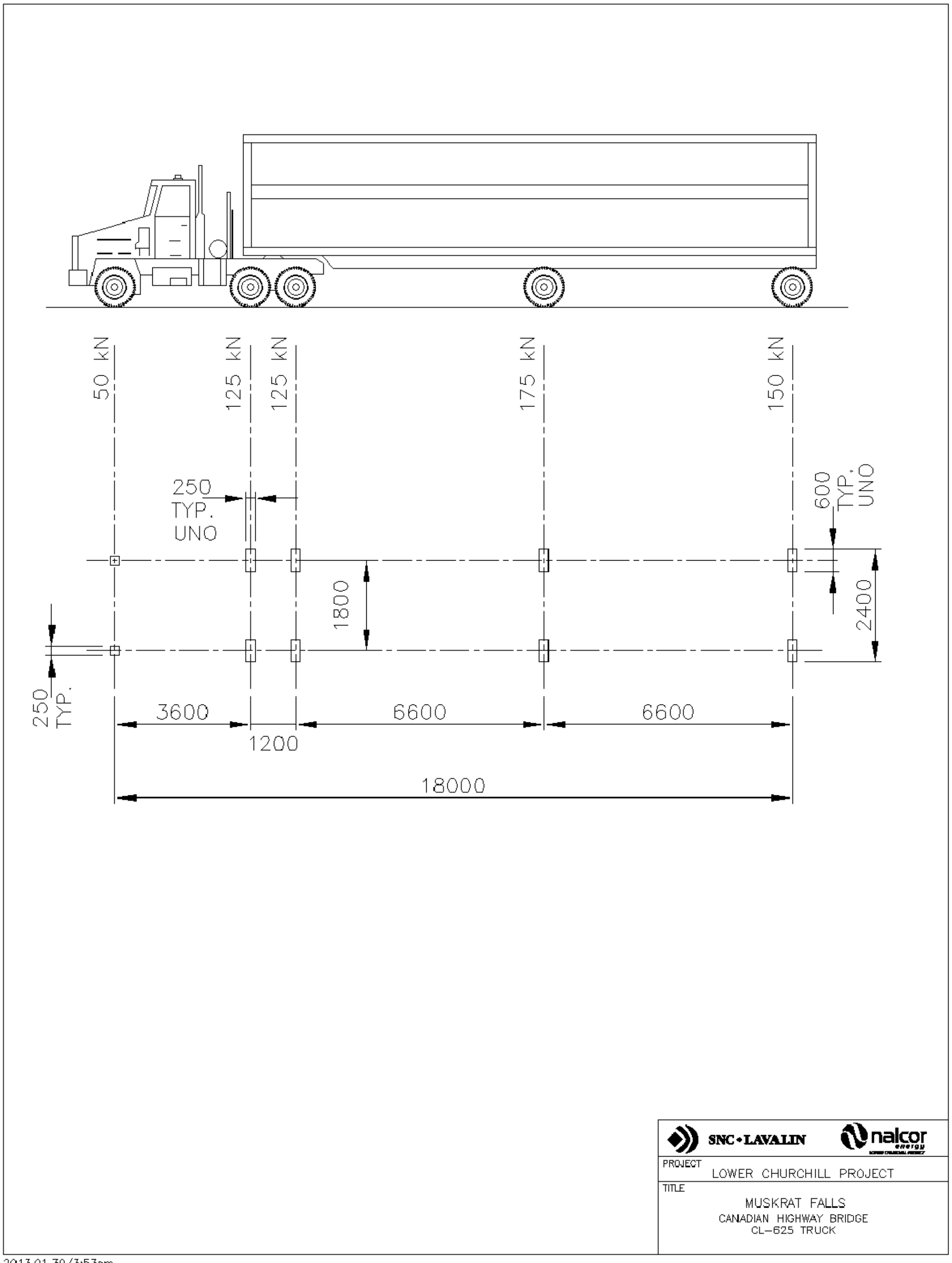
The stability of the separation wall is evaluated for the following basic load combinations:

Usual Loading Cases	N1	Upstream water level: 39.0 m Downstream water level: 37.77 Ice load : 150 kN/m (Load is reversible depending if the flow is directed through the powerhouse or the spillway)
	N2	No Water Backfill lateral earth pressure, backfill at El 28.00 Bridge Action
	N3	Water level in spillway channel during diversion: 25.0 m Water level in Intake approach channel: dry No Ice
Unusual Loading Cases	U1	Upstream water level: 39.0 m Downstream water level: 37.77 Ice load – 225 kN/m (Load is reversible depending if the flow is directed through the powerhouse or the spillway)
Extreme Loading Case	E1	Upstream water level: 39.0 m Downstream water level: 37.77 No ice Full sustained horizontal ground acceleration (1:10 000) One third of sustained vertical ground acceleration (1:10 000) (Load is reversible depending if the flow is directed through the powerhouse or the spillway)
	E2	No Water Backfill lateral earth pressure, backfill at El 28.00 Bridge Action Full sustained horizontal ground acceleration (1:10 000) One third of sustained vertical ground acceleration (1:10 000)

1

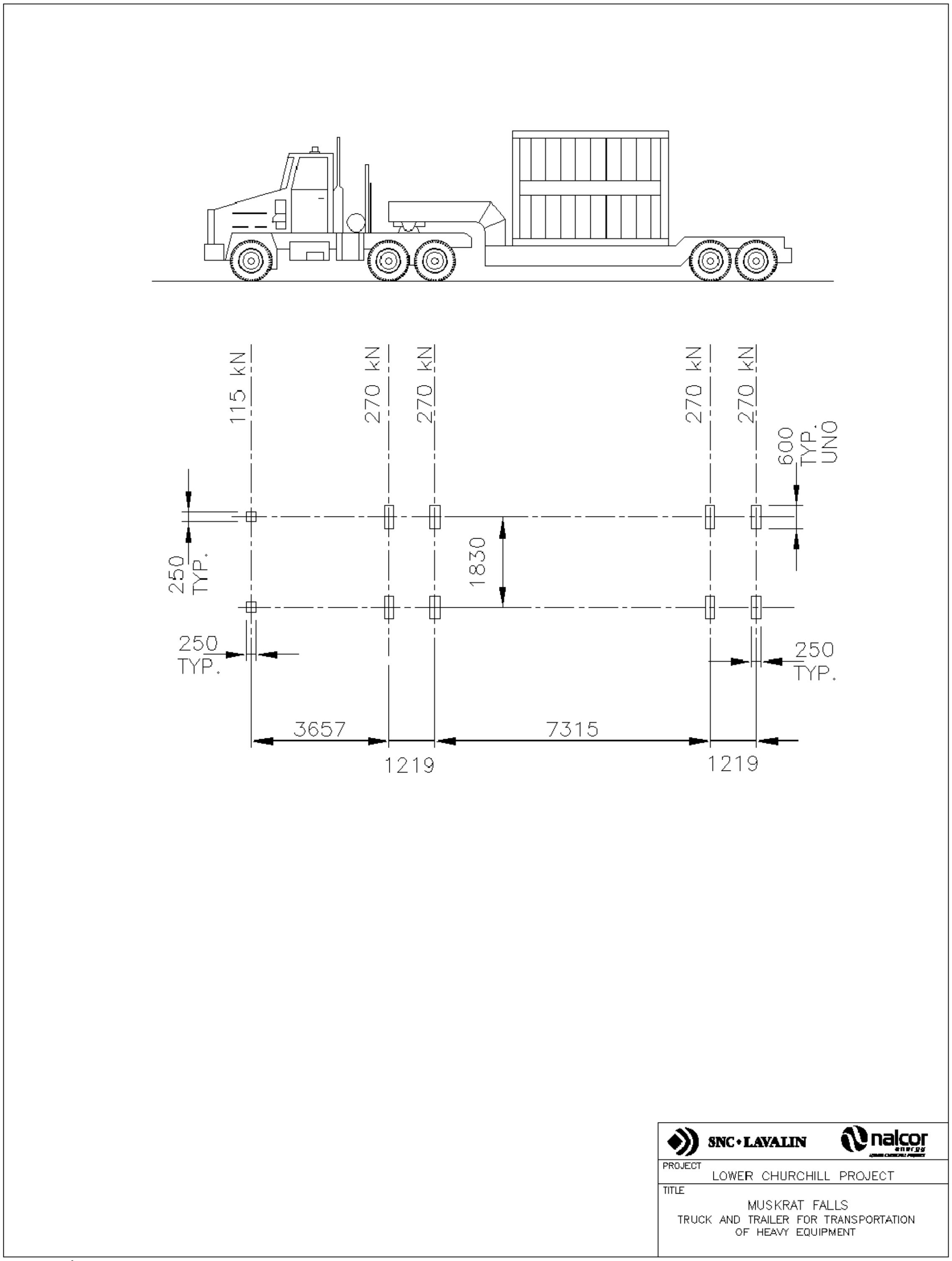
APPENDIX A

VEHICLE AND CRANE LOADS



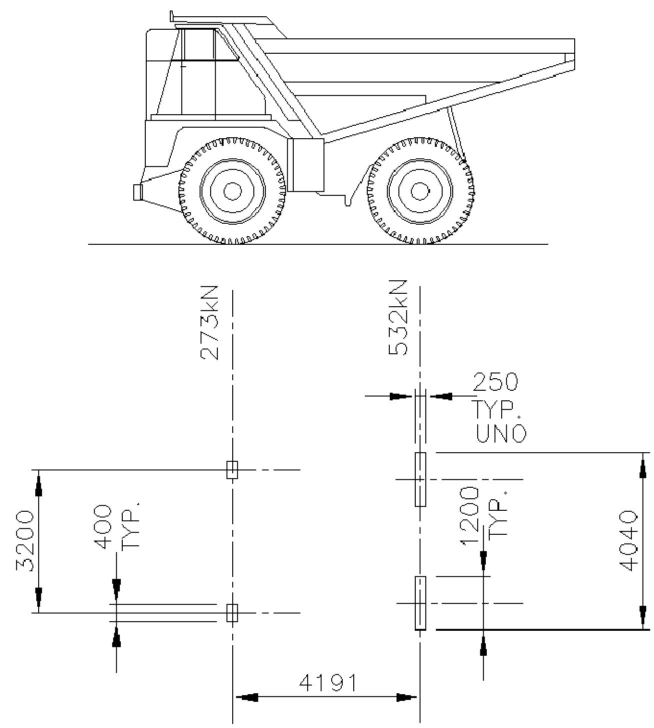
50 kN
125 kN
125 kN
175 kN
150 kN
250
TYP.
UNO
600
TYP.
UNO
1800
2400
250
TYP.
3600
1200
6600
6600
18000
SNC·LAVALIN
nalcor energy
PROJECT
LOWER CHURCHILL PROJECT
TITLE
MUSKRAT FALLS
CANADIAN HIGHWAY BRIDGE
CL-625 TRUCK
2013.01.30/3:53pm

FIGURE-A1



2013.01.30/3:56pm

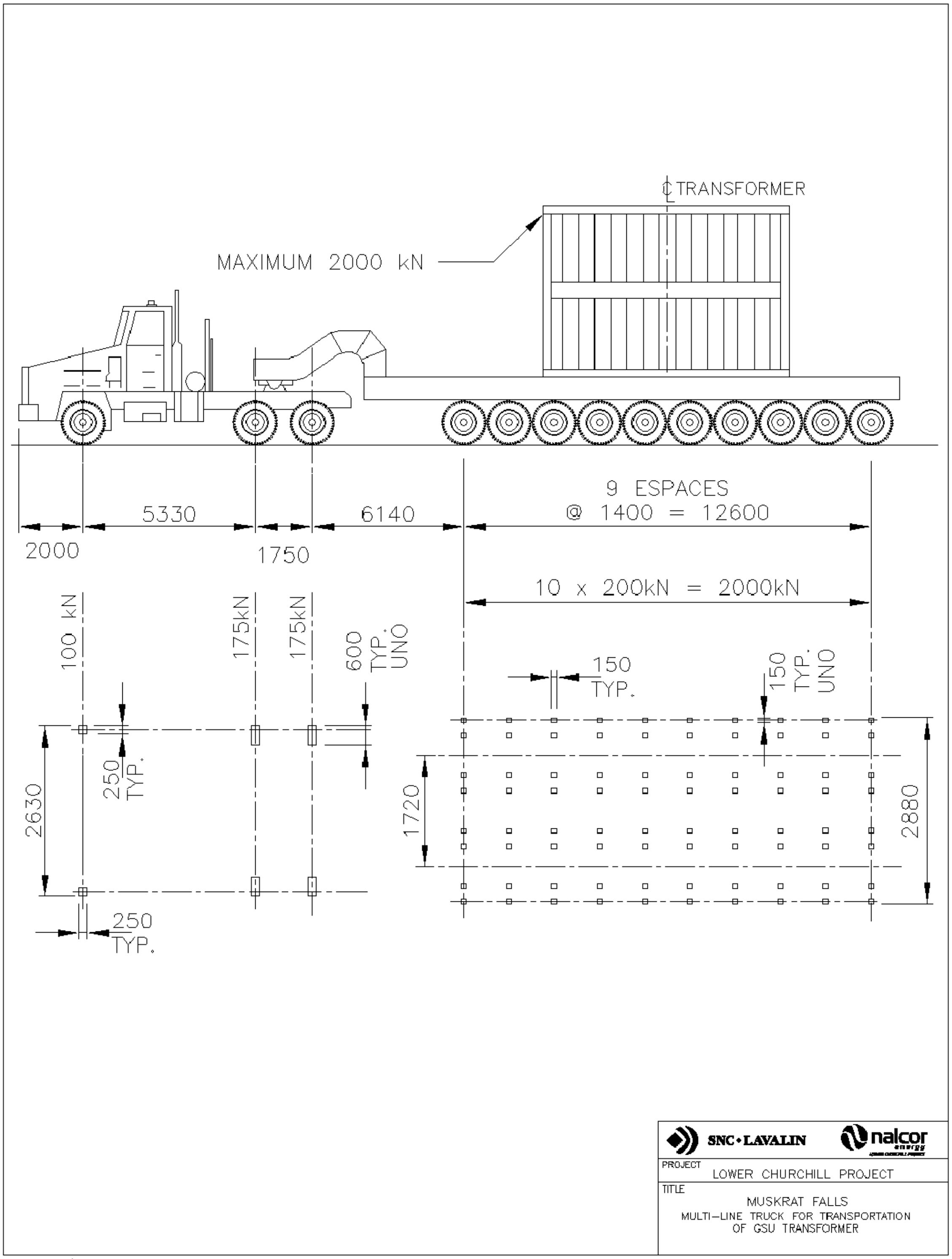
FIGURE-A2



SNC·LAVALIN	nalcor energy
PROJECT	LOWER CHURCHILL PROJECT
TITLE	MUSKRAT FALLS HAUL TRUCK EUCLID MODEL R-50

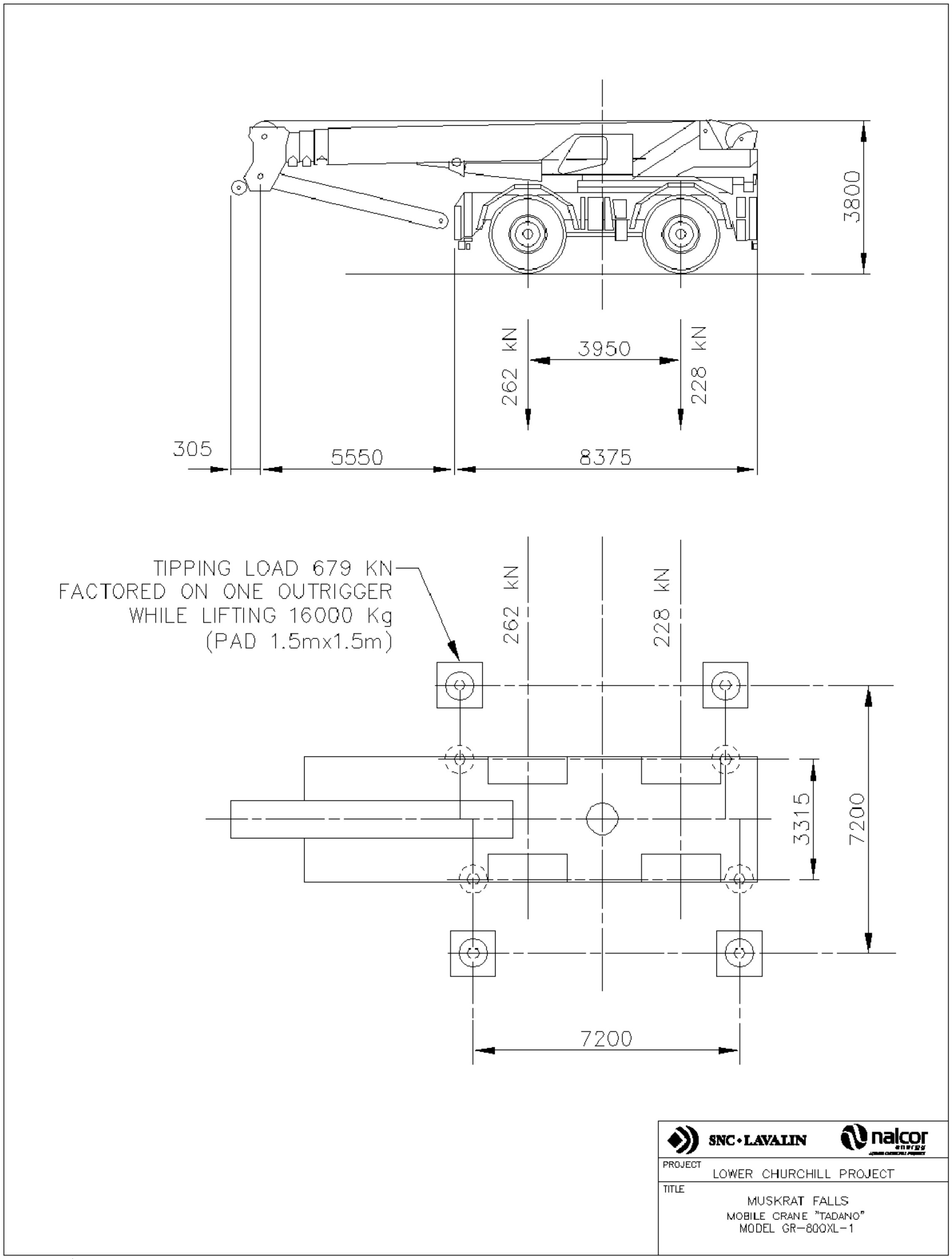
2013.01.30/3:57pm

FIGURE-A3



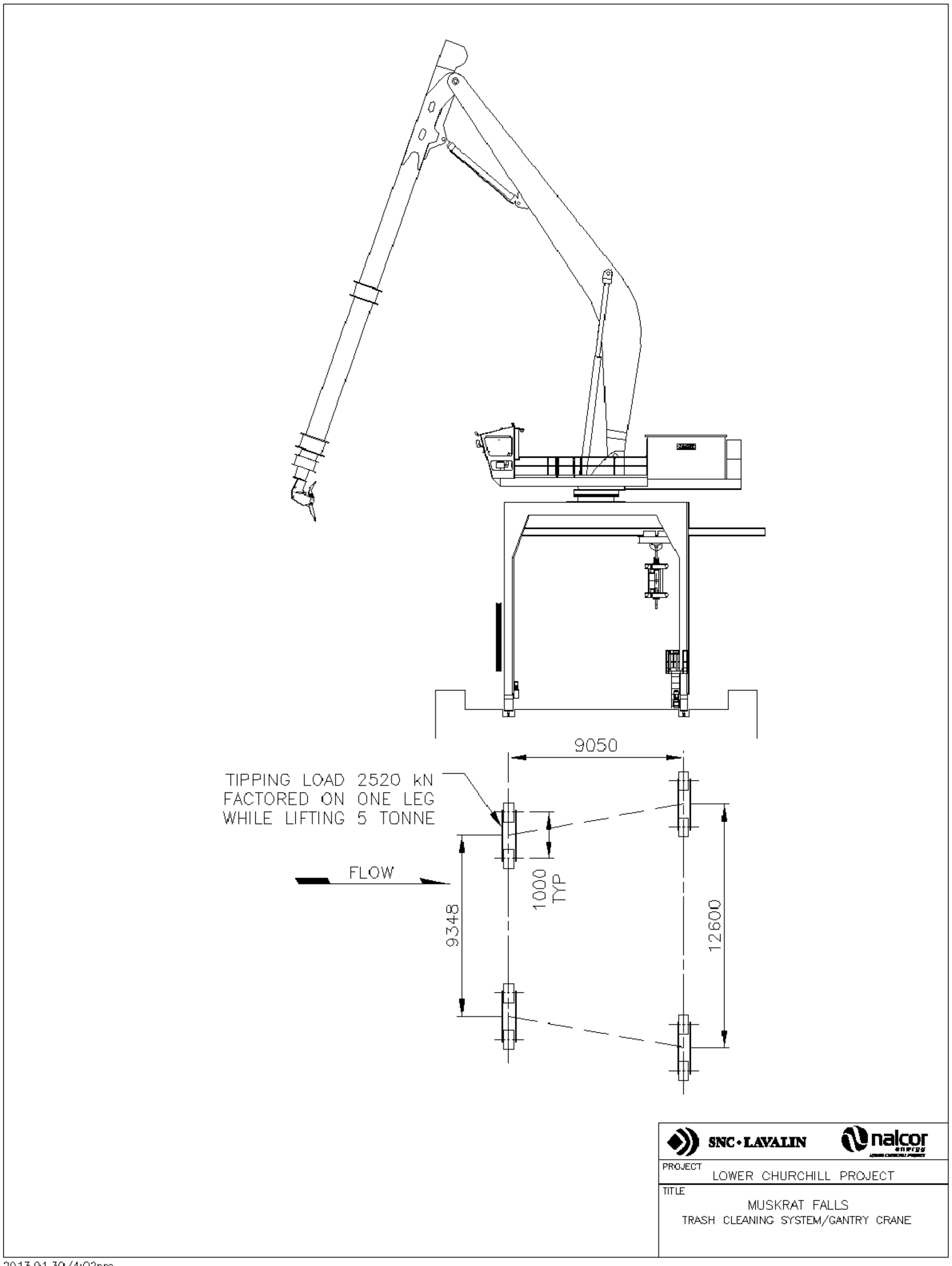
2013.01.30/4:00pm

FIGURE-A4



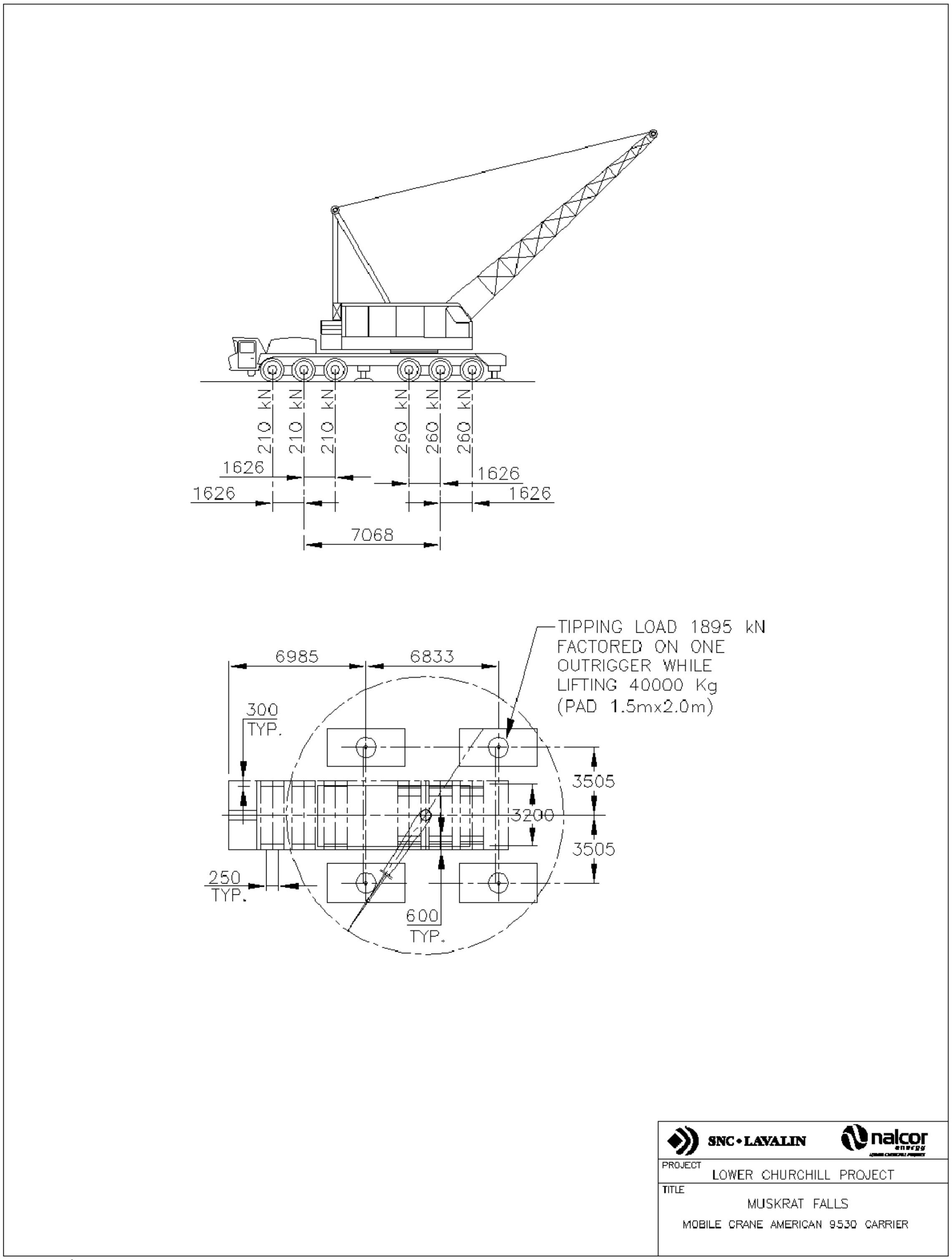
3800
262 kN
3950
228 kN
305
5550
8375
TIPPING LOAD 679 KN
FACTORED ON ONE OUTRIGGER
WHILE LIFTING 16000 Kg
(PAD 1.5mx1.5m)
262 kN
228 kN
3315
7200
7200
SNC·LAVALIN
nalcor
PROJECT
LOWER CHURCHILL PROJECT
TITLE
MUSKRAT FALLS
MOBILE CRANE "TADANO"
MODEL GR-800XL-1
2013.01.30/4:01pm

FIGURE-A5



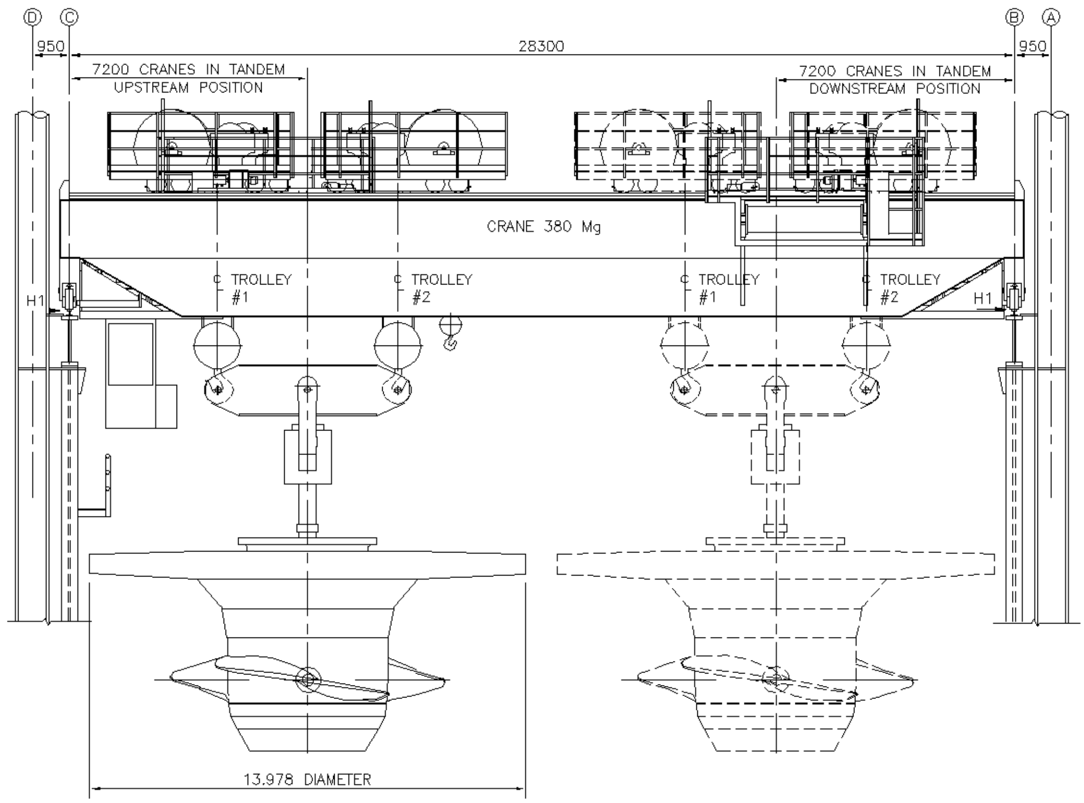
9050
TIPPING LOAD 2520 kN
FACTORED ON ONE LEG
WHILE LIFTING 5 TONNE
FLOW
9348
1000
TYP
12600
SNC·LAVALIN
nalcor
PROJECT
LOWER CHURCHILL PROJECT
TITLE
MUSKRAT FALLS
TRASH CLEANING SYSTEM/GANTRY CRANE
2013.01.30/4:02pm

FIGURE-A6



210 kN
210 kN
210 kN
260 kN
260 kN
260 kN
1626
1626
1626
1626
7068
TIPPING LOAD 1895 kN
FACTORED ON ONE
OUTRIGGER WHILE
LIFTING 40000 Kg
(PAD 1.5mx2.0m)
6985
6833
300
TYP.
3505
3200
3505
250
TYP.
600
TYP.
SNC·LAVALIN
nalcor energy
PROJECT
LOWER CHURCHILL PROJECT
TITLE
MUSKRAT FALLS
MOBILE CRANE AMERICAN 9530 CARRIER
2013.01.30/4:03pm

FIGURE-A7



- TANDEM LIFTING CAPACITY 2x380 Mg
- AUXILIARY HOOK 25 Mg
- CRANE MASS W1 = 187 Mg EACH
- TROLLEY MASS W2 = 43 Mg EACH
- LATERAL BREAKING FORCE ON EACH WHEEL H1 = FOLLOWING NBC 2010

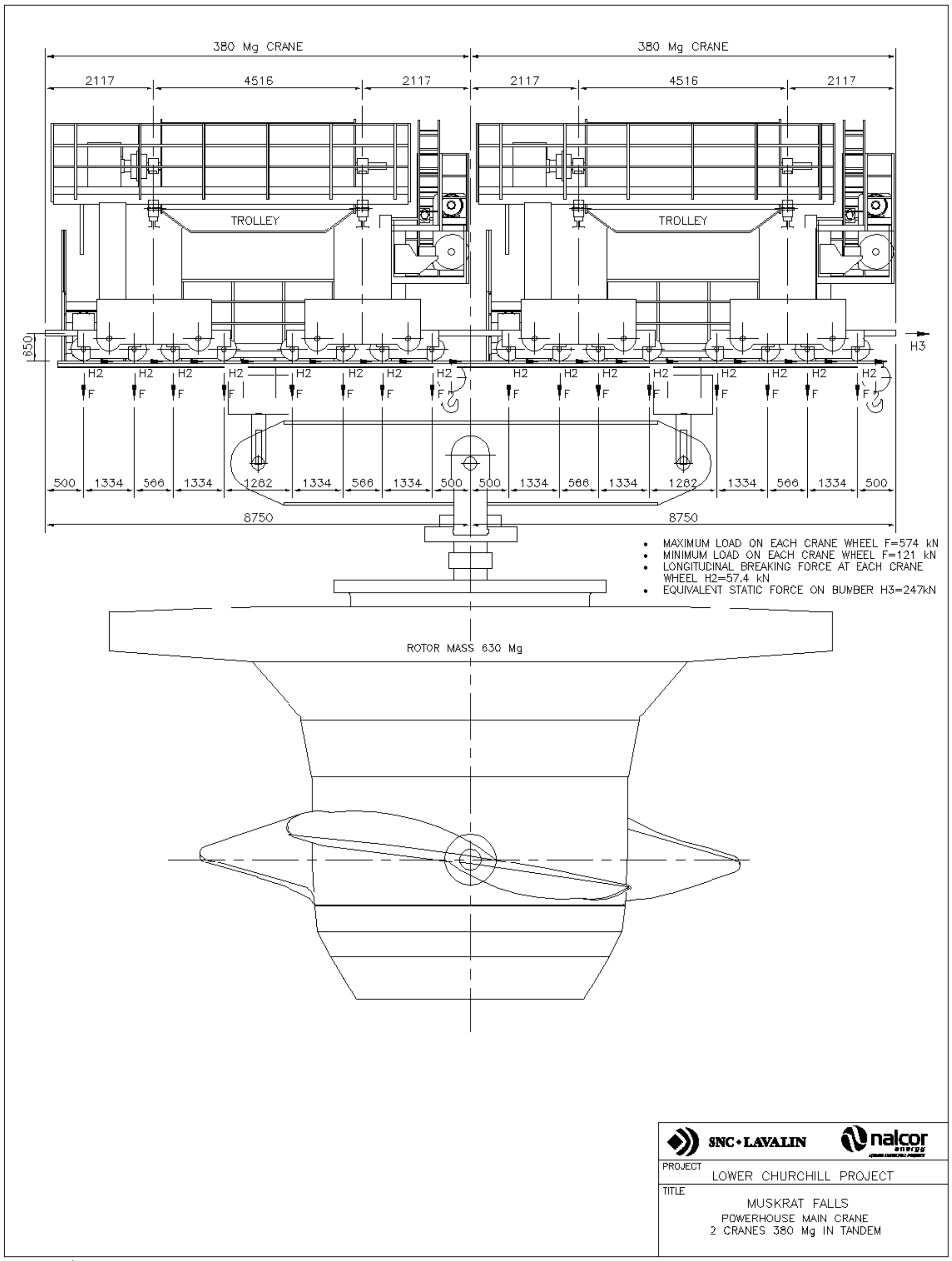
SNC·LAVALIN nalcor energy

PROJECT LOWER CHURCHILL PROJECT

TITLE MUSKRAT FALLS
POWERHOUSE MAIN CRANE 380 Mg

2013.01.30/4:04pm

FIGURE-A8



2013.01.30/4:05pm

FIGURE-A9