

aDB ENGINEERING

A DINGLEY BOETTCHER COMPANY

Muskrat Falls Generation Project
 Unit #2, Pour D2ESB-03, Draft Tube Formwork
 Collapse Investigation Report

Prepared for:
 Nalcor Energy Lower Churchill Project

Project No.160450

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ADB STRUCTURAL ENGINEERING INC.

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 Permit No. as issued by PEGNL 0454
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EXECUTIVE SUMMARY

Nalcor Energy is currently executing the construction of the Muskrat Falls Hydroelectric Generating Facility, and has retained A.D.B. Structural Engineering Inc. (aDB) as a third-party engineer to investigate the falsework collapse that occurred on May 29, 2016. The investigation discussed herein pertains to the collapse of formwork during construction of the draft tube at Unit 2 of the powerhouse.

The Muskrat Falls Project entails construction of two hydroelectric generating stations on the lower Churchill River. The two sites, Muskrat Falls and Gull Island (Phase One and Phase Two respectively), have a combined capacity of 3,000 megawatts (MW).

aDB was on-site at the Muskrat Falls Project commencing in June 2016 to investigate the collapse. During the site visit, aDB met with Nalcor Energy LCP, Astaldi, ILF, and SNC Lavalin to discuss the events that led to the collapse.

The findings of this report suggest that one of the following occurred:

- (i) The shoring system was not designed properly
- (ii) Wood integrity of the formwork was compromised
- (iii) The shoring system was not installed correctly
- (iv) The shoring system fabrication was inadequate
- (v) A combination of these aforementioned factors

This report recommends the implementation of a temporary structure risk management and courageous safety leadership programs, the protection of wood structures against weathering, rigorous design reviews prior to construction, and rigorous daily checks of structures during construction and prior to loading.



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1 INTRODUCTION

Nalcor Energy is currently executing the construction of the Muskrat Falls Hydroelectric Generating Facility, and has retained A.D.B. Structural Engineering Inc. (aDB) as a third-party engineer to investigate the draft tube formwork collapse that occurred on May 29, 2016. aDB was engaged by the Manager of Civil Coordination for Nalcor Energy Lower Churchill Project (LCP). The Muskrat Falls project is located on the lower Churchill River, approximately 30 km west of Happy Valley-Goose Bay in Labrador.

The purpose of this report is to investigate the incident, to determine the contributing factors that led to the falsework collapse, and to recommend steps to prevent this type of incident from recurring. This report is based on observations by aDB on several visits to the Muskrat Falls project in June 2016.

2 BACKGROUND INFORMATION

The Muskrat Falls Project entails construction of two hydroelectric generating stations on the lower Churchill River. The two sites, Muskrat Falls and Gull Island (Phase One and Phase Two respectively), have a combined capacity of 3,000 megawatts (MW).

Phase one entails constructing the Muskrat Falls facility, in addition to over 1,600 km of transmission lines across Newfoundland and Labrador (NL). The project is part of Nalcor's commitment to sustainability and climate change mitigation in NL.

Construction of the Muskrat Falls Generating project commenced in 2013. The facility consists of a spillway, two dams, and a powerhouse (Figure 1). First power from the generation is expected during Q3 2019, with full project handover by Q2-Q3 2020.

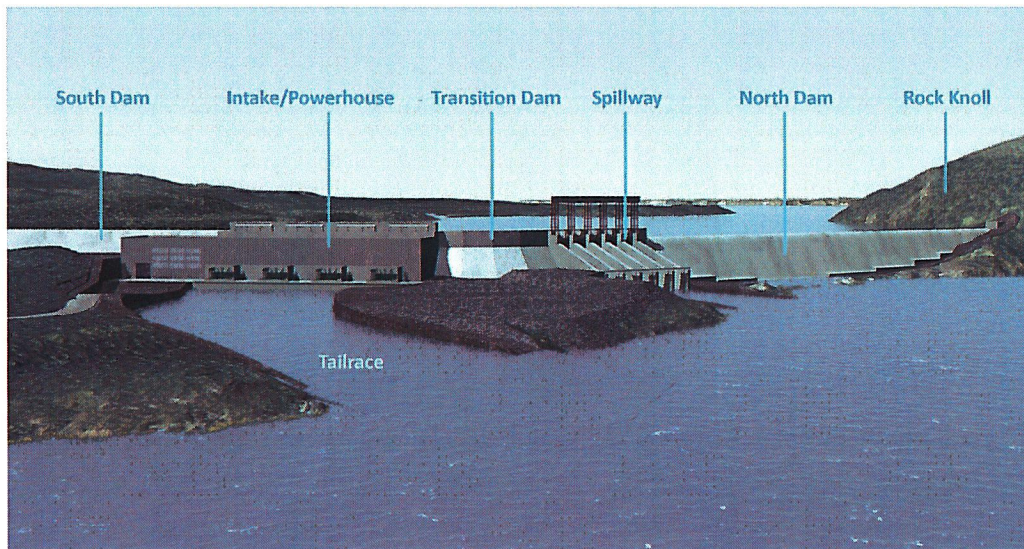


Figure 1: Muskrat Falls Generating Project

The investigation discussed herein pertains to the collapse of formwork during construction of the draft tube at Unit 2 of the powerhouse that occurred on May 29, 2016 (Figure 2).

Several workers including concrete finishers, labourers, and a foreman were involved in the incident. The incident occurred during a concrete pour of the third lift on the draft tube of Unit 2.

The partial lift above the formwork, covering the South East portion of the form (lift D2ESB-03), has an approximate volume of 530m³. The collapse happened near the end of the pour, with only 4m³ remaining in the concrete pour. The pour commenced at 10:00AM on May 29th, 2016. The formwork collapse happened at approximately 11:55PM the same day.

The collapse was significant in that it damaged all lumber shoring towers directly underneath this section of concrete. The workers in the area were finishing the concrete on the gallery's floor when the falsework collapse. The collapse resulted in the workers falling directly into the freshly poured concrete where one worker was fully submerged.



Figure 2: Unit 2 draft tube formwork collapse - view of SE corner of unit

aDB was engaged by LCP to investigate the draft tube formwork collapse. The investigative team consisting of Sean Dingley, P.Eng and Mathieu Légaré, P.Eng, has previous experience with draft tube construction, and formwork collapse investigations.

2.1 STAKEHOLDERS

The stakeholders in the project are listed in Figure 3.

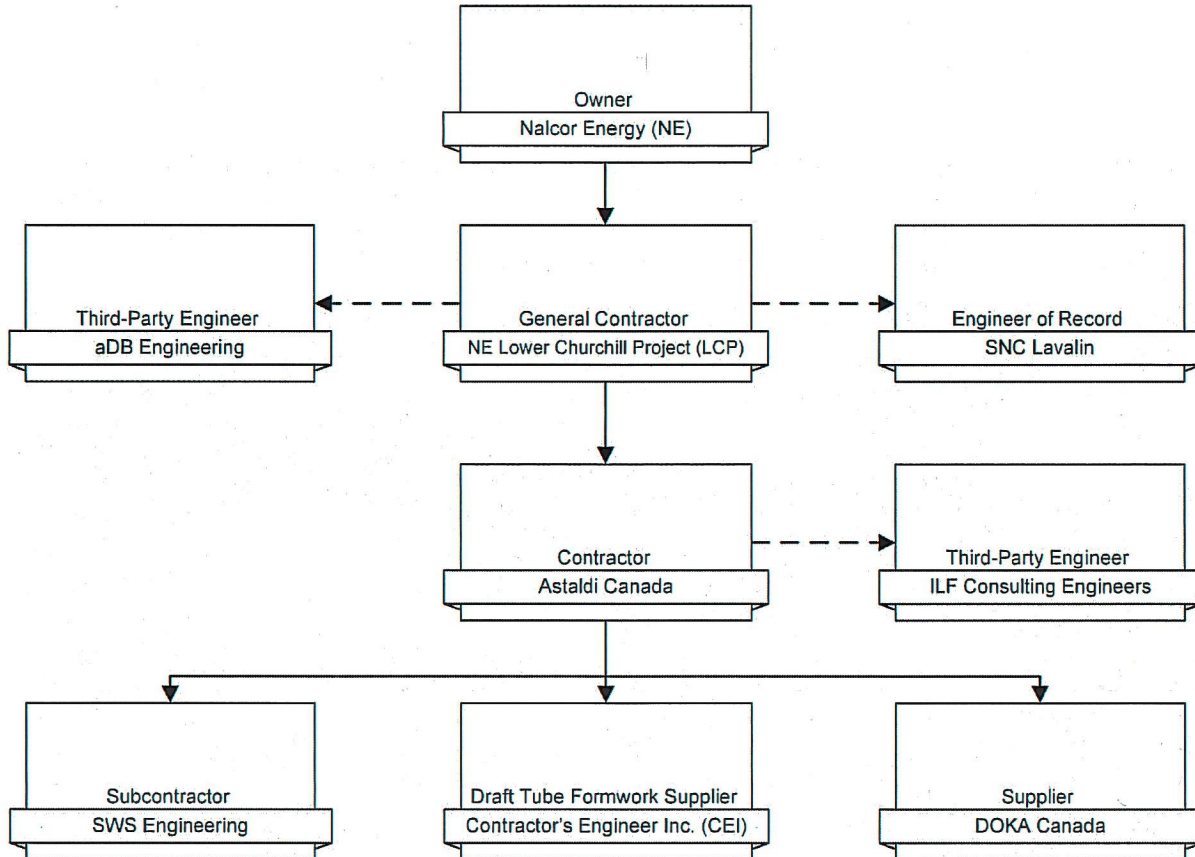


Figure 3: Project organizational chart

2.1.1 Owner

Nalcor Energy, a Crown corporation, is the Newfoundland and Labrador provincial energy company responsible for the sales and development of electrical generation capacity. The Lower Churchill Project is one of Nalcor’s development projects, and includes the Muskrat Falls Project.

2.1.2 General Contractor

Nalcor Energy LCP is the General Contractor responsible for the construction of the Muskrat Falls Project.

2.1.3 Contractor

Astaldi Canada Inc. is the Contractor for the construction of the powerhouse and spillway for the Muskrat Falls Project.

2.1.4 Draft Tube Formwork Supplier

Contractor's Engineer Inc. (CEI) is a custom design and formwork supplier, based in Neodesha, Kansas. Astaldi Canada purchased four sets of draft tube formwork from CEI for the Muskrat Falls Project powerhouse construction.



2.1.5 Subcontractor to the Contractor

SWS Engineering is a consulting firm that is subcontracted by Astaldi Canada. They produce construction packages for Astaldi.

2.1.6 Contractor's Supplier

DOKA Canada is a supplier and designer of formwork for Astaldi Canada at the Muskrat Falls Project. They design and supply formwork for all structures at Muskrat Falls with the exception of the draft tube.

2.1.7 Contractor's Third-Party Engineer

ILF Consulting Engineers is a third-party engineer hired by Astaldi after the draft tube formwork collapsed.

2.1.8 Structural Engineer of Record

SNC Lavalin is the Structural Engineer-of-Record of concrete structures for LCP, and is responsible for the design of the permanent structures.

2.1.9 General Contractor's Third-Party Engineer

A.D.B. Structural Engineering Inc. is the General Contractor's (LCP) third-party-engineer, and is responsible for investigating the falsework collapse.

2.2 DOCUMENTS PROVIDED BY LCP

The documents provided by LCP for review are listed below.

2.2.1 Draft Tube Formwork Drawings

Draft tube formwork drawings contain the fabrication and erection details for the draft tube formwork, as prepared by CEI. The fabrication drawings provided to aDB were not stamped by a Professional Engineer. The drawings, which would then be used during erection of the draft tube structure, were stamped for the province of Newfoundland and Labrador by Yi Ping Liu with Astaldi Canada Inc.'s permit to practice. The drawings were also stamped by David Kramer (CEI) for the state of Kansas.

2.2.2 Draft Tube Formwork Calculations

The draft tube formwork calculations contain the detailed calculations relevant to the draft tube form, and were prepared by CEI and stamped for use in the state of Kansas by David Kramer, CEI owner. These calculations were stamped for the province of Newfoundland and Labrador by Yi Ping Liu using Astaldi Canada Inc.'s permit to practice.

2.2.3 Formwork Checklist for D2ESB-03

The formwork checklist is a quality control document produced by Astaldi prior to pouring a concrete lift. The formwork portion of this checklist consists of one page that was dated May 28th, 2016.

2.2.4 Structural Drawings

Concrete structural drawings were produced by SNC-Lavalin.

**2.2.5 Lift Drawings**

Work package drawings produced by SWS Engineering.

2.2.6 Pictures

Before and after collapse pictures taken by Nalcor Energy.

2.2.7 Witness Statements

The written statements from the workers involved in the collapse including labourers, a foreman, superintendent, and the on-site medic.

2.2.8 Schedule

A schedule that detailed as-built concrete pours including start and finish dates.

2.2.9 LCP Visit of CEI Fabrication Shop

LCP visited the CEI Fabrication Shop on November 18th, 2014, and produced a report of its findings.

2.2.10 Daily Construction Report

A daily construction report is produced by on-site LCP monitors that keep track of the construction works.

3 TECHNICAL BACKGROUND

3.1 COMPONENTS OF A HYDROELECTRIC GENERATING FACILITY

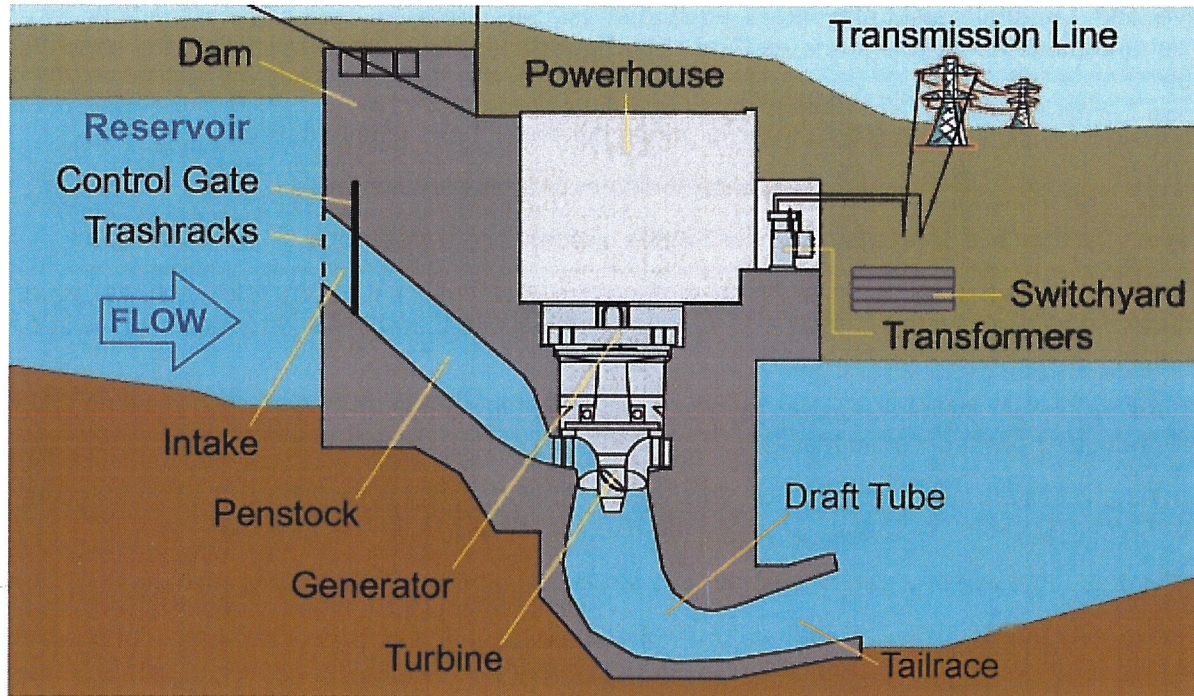


Figure 4: Cross-section of the powerhouse with draft tube below the turbine

3.2 DEFINITIONS

The definitions below are relevant to this investigative report and were sourced from CSA S269.1-16 Falsework and Formwork.

Falsework: any temporary structure used to support a permanent structure while it is not self-supporting.

Form: the mould or members in direct contact with freshly placed concrete while it is setting and gaining sufficient strength to be self-supporting.

Form face: the panel material that creates the contact surface with the freshly placed concrete providing the final shape, form, or finish.

Form tie: a tensile unit adapted to holding concrete forms secure against lateral pressure of unhardened concrete.

Formwork: the total system of support for freshly placed concrete, including the mould or sheathing, supporting members, hardware, and necessary bracing, but excluding the falsework.

Frame: the principal prefabricated structural unit in a scaffold or shore tower



Joist: a horizontal flexural member, a group of which supports the sheathing or decking, intended to be loaded on its narrow face, and usually spans horizontally between, and is supported by or upon ledgers or beams.

Lift: the height of one concrete pour

Live load: the total weight of workers, equipment buggies, vibrators, and all other loads that will exist and move about due to the method of placement, levelling, and screeding of the concrete pour.

Material load: load due to stored material (rebar bundles, stacks of shoring frames, etc.)

Mould: a shaped cavity used to give a definite form or shape to concrete

Tower: a composite vertical structure of frames, braces, and accessories.

Sheathing: material which is in direct contact with surfaces of the concrete such as wood, plywood, metal, or synthetic sheets or various combinations thereof. Also known as sheeting or lagging.

Shore: a vertical inclined support member designed to support the weight of the formwork, concrete, and construction loads.

Shoring: a system of vertical or inclined supports for forms; it may be of wood or metal posts, scaffold-type frames, or various patented members or other systems of falsework.

Soffit: the underside of a part or member of a structure, such as a beam, arch, etc.

Stud: a flexural member for vertical formwork, a group of which supports the sheathing, and usually spans between, and is support by walers.

Tower: a composite vertical structure of frames, braces, and accessories.

Waler: a member, horizontal or vertical, which transfers loads from the form to the form-tie system, form-bracing system, or both.



4 SCOPE OF WORK

The scope of work for this investigative report includes:

- aDB site visits to Muskrat Falls Project commencing in June 2016
- Visually assessing the factors that could have contributed to the formwork collapse
- Discussions on-site with the stakeholders related to the formwork collapse
- Design review and provision of a professional opinion as to whether or not the design had any inherent flaws that could have contributed to the formwork collapse
- Recommendations made to LCP to prevent the incident from recurring, based on aDB's professional judgement and observations made on-site

5 INCIDENT DESCRIPTION

On May 29, 2016, a crew of workers that included labourers, carpenters, and foremen began preparing for the D2ESB-03 concrete pour, which is the south east portion of lift 3. This lift had a volume of approximately 530 m³. The pour started at 10:00 AM, and the collapse occurred at approximately 11:55 PM, with 4 m³ of concrete remaining to be poured. At the time of collapse, at least five workers were finishing the concrete in the area where the formwork collapsed. Five workers fell into the draft tube cavity when the formwork collapsed, and at least one worker was submerged by the freshly poured concrete, which was still liquid at the time.

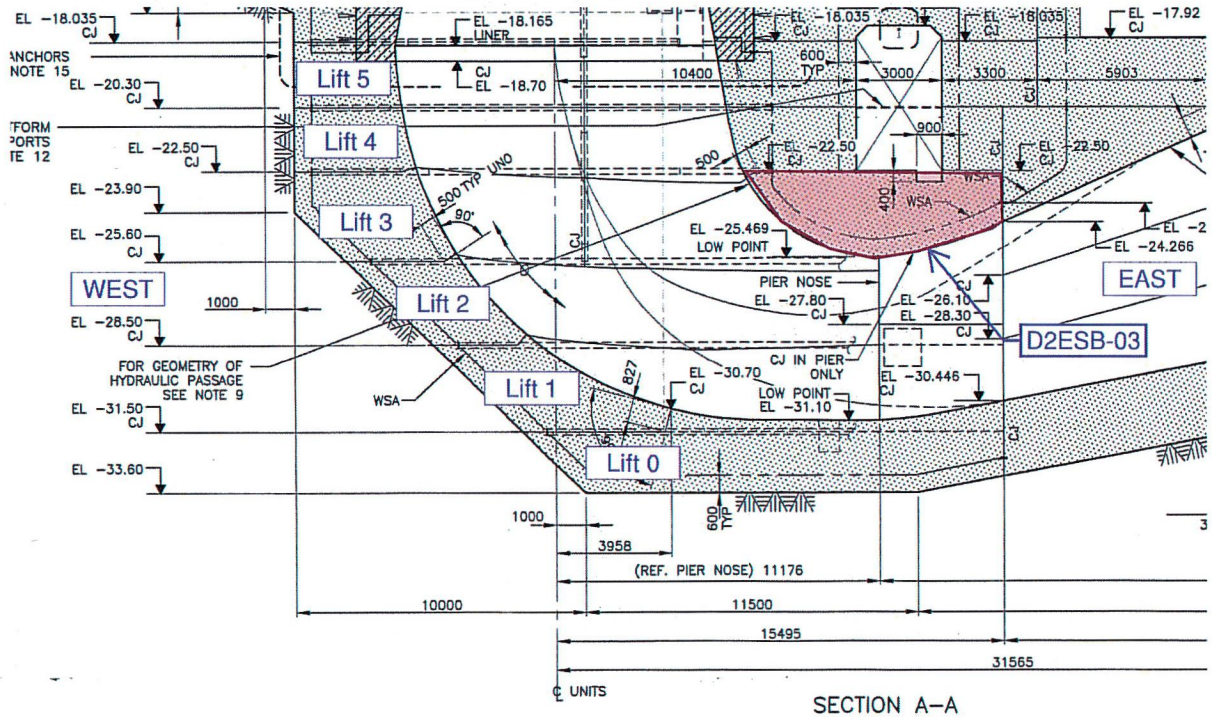


Figure 5: Typical cross-section of the draft tube

The typical cross-section of the draft tube illustrated in Figure 5 is from drawing MFA-SN-CD-3310-CV-SE-0002-01 rev C3. The drawing is titled "ALL UNITS – SECTION A-A, AT CENTERLINE OF UNIT, CONCRETE".

The worker that was submerged by concrete was able to remove himself from the concrete with the help of a nearby coworker. The rest of the workers were able to walk away from the scene of the collapse. The workers were treated for minor injuries. The incident scene was then frozen, quarantined, and investigated.



6 SITE OBSERVATIONS

aDB was on-site at the Muskrat Falls Project on June 15, 2016 to investigate the collapse. During the site visit, aDB met with Nalcor Energy LCP, Astaldi, ILF, and SNC Lavalin to discuss the events that led up to the collapse.

aDB visited all four draft tube areas: Unit 1, Unit 2, Unit 3, and Unit 4. The main area of interest, Unit 2 draft tube area, could only be viewed from the perimeter because the area was inaccessible for safety reasons during the aDB's site visit in June 2016. The northern section of the Unit 2 draft tube was open for aDB to visit in October 2016.

Given that the failed formwork was buried under hardened concrete, the neighbouring areas were visited (including the northern section of Unit 2, and the remaining draft tubes) to look for factors that could have contributed to the collapse.

For each of the following subsections, additional pictures can be found in the appendix.

6.1 UNIT 2 – JUNE 2016 SITE VISIT

Approximately 500 m³ of freshly poured concrete covered the collapsed formwork. As such, direct observations to determine the condition of the formwork at the Unit 2 draft tube could not be made.

The catastrophic formwork failure covered a large area. Six shoring towers at Row B and Row C that supported the load of concrete were completely destroyed.

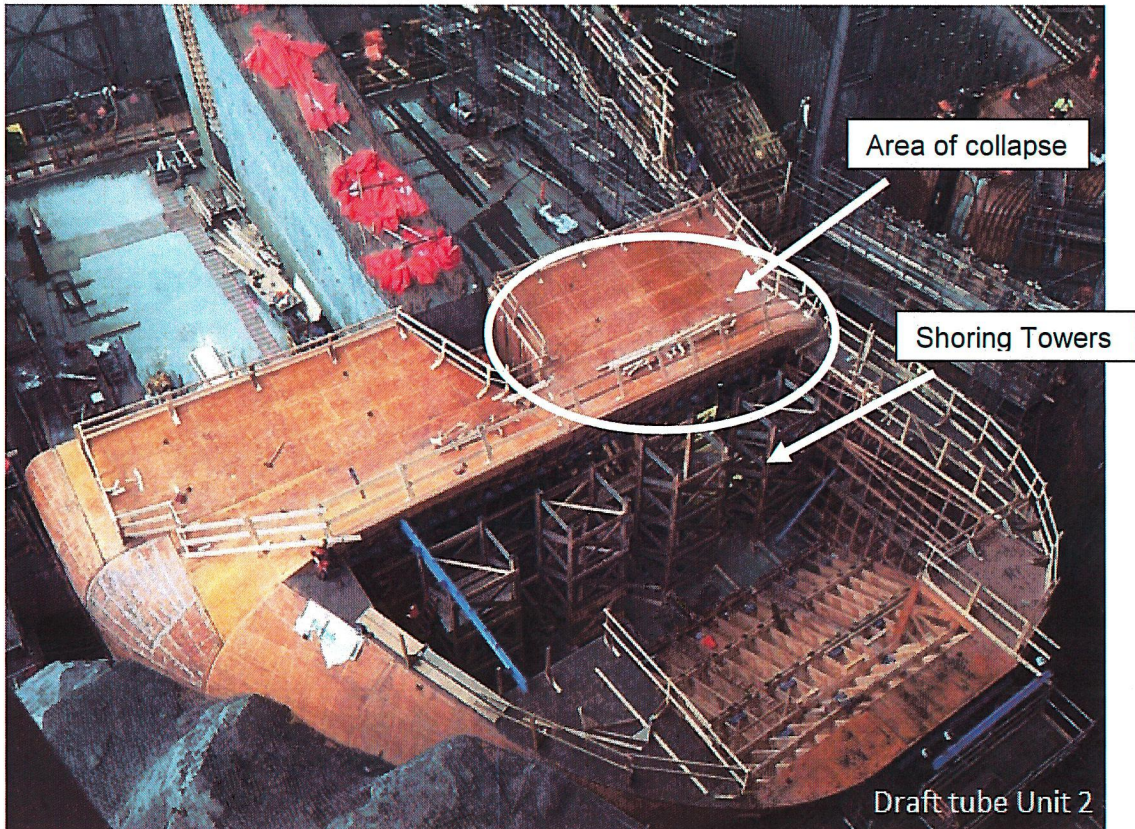
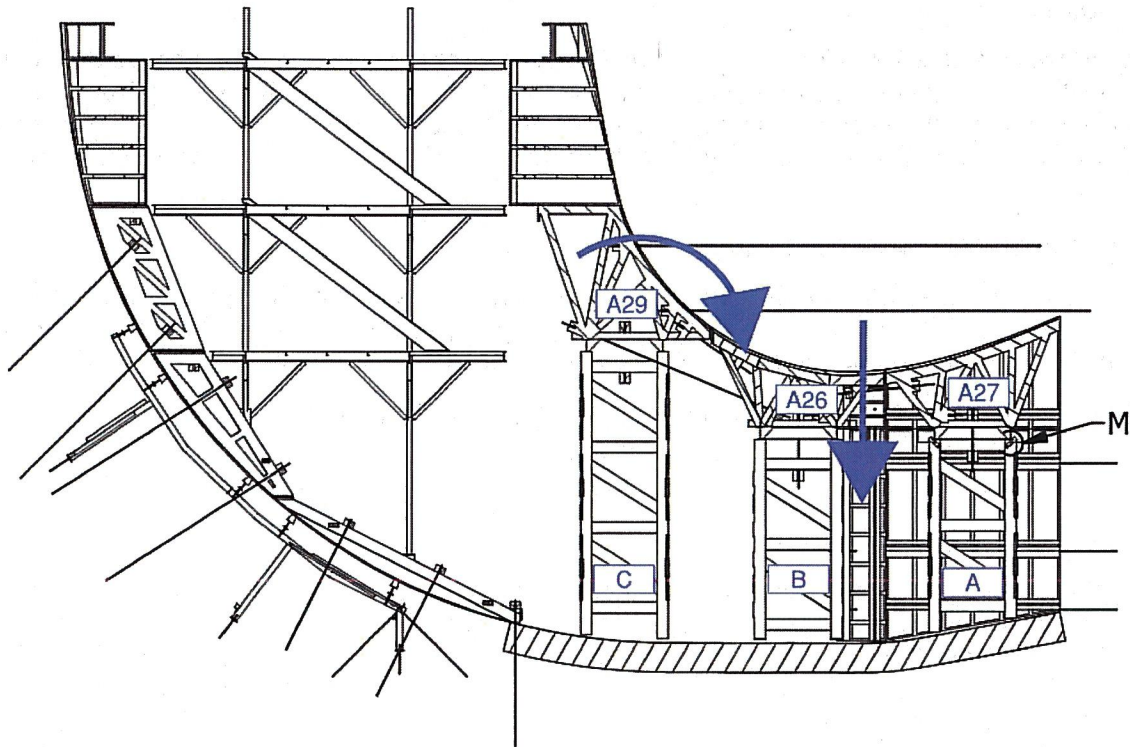


Figure 6: Unit 2 draft tube on October 10, 2015

Once the formwork collapsed, the freshly poured concrete flowed downstream into the Unit 2 draft tube southern outlet. As a result, the bottom of the four downstream shoring tower rows that were constructed with DOKA supplied formwork and shoring were severely bent and damaged. Once the concrete settled, it covered the lower portion of nearby shoring tower legs. During the site visit, no evidence of destabilizing sideways movement of the draft tube formwork was found as the failure appeared to be vertical in nature.

Concrete poured downwards and in between shoring tower rows A and B (Figure 7), which suggests issues with the shoring towers. The side panels (draft tube panel A29 and the north built-in-place panel) were pulled over the fallen concrete by the tie-rods that held lateral concrete pressure.



SECTION D-D

Figure 7: Section view of the draft tube formwork with assumed collapse sequence

Figure 7 illustrates drawing MFA-AT-SD-3310-CS-D04-0003-02 rev C1. The drawing is titled "DRAFT TUBE ELBOW – WOOD FORMWORKS – GENERAL DRAFT TUBE FORMSYSTEM VIEW".

6.2 UNIT 2 – AUGUST 2016 AND OCTOBER 2016 SITE VISIT

The area south of Unit 2's draft tube was cleared for access by August 2016. A site visit was conducted by aDB in August 2016 and October 2016.

6.2.1 Unit 2 – Lumber Weathering

During the site visit, a clearly defined line marking a flooding elevation was observed on the towers and the formwork approximately three feet from the ground. Fungi and decayed wood was also evident on the tower. Observations made at Unit 2 correlated to observations made at Unit 1's draft tube formwork (Section 6.3).

6.2.2 Unit 2 – Downstream Anchors

The downstream anchors used to laterally stabilize the draft tube from the lateral concrete pressure were discussed during the June 2016 site visit as a potential cause of the collapse. On October 18th, 2016 aDB accessed a scaffold that was built for concrete remediation at the draft tube and tailrace interface. From the platform, the tie rod anchors that held the draft tube formwork laterally (East-West direction) were assessed. It is impossible to confirm if the anchors were installed as designed. Refer to aDB report 'DT2 Downstream Anchor Observations' dated October 27th, 2016 for further details.

6.3 UNIT 1

The formwork and falsework in the Unit 1 draft tube was already loaded by draft tube lifts 3 and 4 when the Unit 2 draft tube formwork collapsed. On June 15, 2016, aDB was walked through this area to study the falsework in detail. The following sections outline the observations made in the Unit 1 area. These observations can be correlated to the Unit 2 draft tube formwork collapse.

6.3.1 Unit 1 – Maintenance Issues

The formwork in the Unit 1 draft tube displayed evidence of exposure to high relative humidity, rain, and snow. The bottom of the formwork also displays evidence of having been submerged in water for a prolonged period of time. The evidence observed includes:

- A clear water line mark approximately 3 feet above the ground
- Ice built up between the ribs (observed in June)
- Wood appeared to be decayed with fungus and mushrooms growing on the lumber

6.3.2 Unit 1 - Tower Buckling

Noticeable S-shaped buckling was evident in the built-up posts of the shoring frames. The steel beam cantilevered fulcrum posts displayed the most noticeable buckling. These posts will take slightly more load than the other posts of the falsework system (about 5% more load). The buckling observed was consistently in the North-South plan (the plan parallel to the smallest dimension of the shoring leg). During a subsequent site visit on October 18, 2016, the buckling of the post was even more noticeable, indicating that the load had increased since the first visit, or that material properties of the wood had diminished over time.

6.3.3 Unit 1 – Compression Failure

Compression failure is evident on some built-up columns at the interface of the shoring leg and the steel beam. The lumber used for the construction of the built-up columns have a depth of 235mm. The W250x25 steel beams sitting on top of the shoring columns have a flange width of 102mm. Per the Design Review section (Section 8), there are no steel plates between the two elements to spread the load equally through all the vertical wood fibers. The loaded wood fibers below the beam were sheared off from the unloaded wood fibers due to the concrete load.

6.4 UNIT 3

The Unit 3 draft tube formwork was still under construction when Unit 2's draft tube formwork collapsed. In June 2016, it was noted that the first level panels were installed along a few shoring towers. No concrete had been poured on any of the panels, and the towers were placed in their respective positions.

6.4.1 Unit 3 – Lumber Weathering

The towers displayed evidence of severe weathering. The lumber planks were dark grey / black in colour due to the weathering. This is indicative of the towers having been damaged before installation. It is also an indication that the wood structures were not well protected in storage between fabrication and installation.

A picture taken by Nalcor during Unit 2's draft tube formwork installation in Fall 2015 displays the same noticeable dark grey / black colour on the shoring towers (refer to Section 7).



6.4.2 Unit 3 – Gaps in Built-up Tower Leg Joints

Noticeable gaps in the built-up shoring tower columns at the lumber butt joints were observed. The gaps in the built-up legs were identified as a fabrication quality issue post-collapse. The brace configuration built into the legs do not facilitate for lumber expansion or shrinking as the wood moisture content changes, which does not mitigate built-up column deformation. Assuming wood moisture content of the tower lumber has increased since fabrication, the internal lamination with the braces will have expanded and pulled the lumber of the other lamination apart. Refer to Section 8.5.3 for details.

6.4.3 Unit 3 - Fabrication Workmanship

There is evidence that the lumber was damaged during fabrication by improperly handling the wood saw. There is evidence of damaged planks being used in fabrication of the built-up towers. The damaged planks were then assembled to make the built-up tower legs. No quality control programs, at either the fabrication shop or on-site, identified the defects before erection of the towers in the Unit 3 draft tube.

6.5 UNIT 4

Unit 4 was fully assembled when the formwork collapse at Unit 2 occurred. The first draft tube pour was on November 28th, 2015. Level 2 concrete lifts D4ESA-02 and D4ENA-02 were poured around the draft tube formwork on April 22nd, 2016. Levels 3 and 4 concrete lifts were poured around the upstream side of the draft tube formwork on May 9th and 19th, 2016. Rebar was being installed for the third downstream level. As most of the draft tube formwork surfaces are covered and the remaining surface of the form are loaded with rebar, no major deficiencies would remain to be fixed for any of the upcoming pours.

6.5.1 Unit 4 – Installation Workmanship

6.5.1.1 Misalignment of Beam above Tower

CEI's design requires kickers between the tower bottoms. The kickers consist of 4x6 planks wedged between the towers and the adjacent formwork to lock the towers in place in the correct alignment. The kickers were already installed when aDB visited the site in June 2016. Kickers are typically installed towards the end of tower installation, and requires a significant amount of work to be completed. The aforementioned provides an indication that the contractor had completed the tower installation.

There is clear evidence that the towers were installed out of alignment with the steel beams they were supposed to support. Some beams are not sitting in the middle of the tower leg. In one instance, the steel beam was sitting on half of the leg.

6.5.1.2 Insufficient Shims

A wooden tower's height cannot be adjusted once the towers are set. If a tower column is too short to accommodate the height required to support the steel beam, shims are used to fill the gap. Shims would allow the load from the steel beam to spread uniformly across the column's section.

As noted in Section 8.6 (Tower Installation Detail), the CEI design does not detail any shims at the top of the tower to spread the bearing load or to adjust the tower height. The design allows for leg length adjustment by the bottom shim/grout detail only.



There is evidence that the leg height was not perfectly adjusted and that top leg shims were required. Some of the shims were missing, leaving a gap between the steel beam and the leg. In other cases, shims were installed but the material chosen and the installation itself were not sufficient to transfer the bearing load adequately to the legs.

6.5.1.3 Insufficient Brace Nailing

The wood tower components (legs and braces) are pre-built by CEI. Before being set in place, the different components need to be fastened together. The tower assembly consists of nailing the east and west side braces to the legs. As discussed by the different parties involved during the visit, the nailing pattern appeared insufficient. CEI designs described the nailing pattern to be "10-8d nails, each splice plate".

The site nailing of the braces appears to be in compliance with the CEI design.

6.5.1.4 Poor Handling during Formwork Adjustment

Observations on-site indicated noticeable damage to Row B's North tower indicating that readjustment was completed without the required precautions after the module was installed. The readjustment resulted in deformation to the wood structure. It would be expected that damage of this nature would be addressed, either right away, or before adding the rebar load to the shoring.

Typically, when erecting the draft tube formwork, the shoring towers are put in place and then the modules above are installed. The towers have to be installed correctly so the modules above bear properly on them. If, after the module installation, the crew determines that some of the tower legs do not line up perfectly, the tower has to be re-aligned below the steel beam. This operation is not straight forward because the module's weight prohibits the relocation of the towers.

6.5.2 Unit 4 – Fabrication Workmanship

6.5.2.1 Uneven Lumber at the top of the tower

Observations on-site indicated tower legs with laminations that were not flush at the top of the tower. This observation was discussed during the visit as a fabrication issue. It is unlikely that the tower was shipped from the fabrication shop in this uneven condition. As discussed in previous sections, it is most likely the result of changes to wood moisture content (per Section 8.5.3).

6.5.2.2 Splice Location in Built-up Column

The longest tower legs were assembled using built-up planks that were spliced together. The splicing rules from CSA S269.1-16 Falsework and Formwork standard apply in this case (Section 8.5.1). Splices were found close to each other between the built-up laminations.

Per the CSA standard, the tower legs were built inadequately.

6.6 GENERAL OBSERVATIONS OF DRAFT TUBE FORMWORK MATERIAL

6.6.1 Identification of Lumber

Per Clause 5.2.1.1 of CSA086-09 standard – Engineering Design in Wood, lumber used in the construction of structures shall be identified by a grade stamp. Each stamp must identify the



grading organism, the saw mill number, the wood species, the quality of the lumber and the moisture content.

Site observations indicated very few stamps on the formwork and shoring, which raises questions about the wood quality. ILF Consulting took wood samples of the collapsed structure to further analyze the wood, which should reveal the nature and properties of the lumber. At the time of writing this information has not been provided to aDB.

6.6.2 Grade of Lumber

Per the Southern Pine Inspection Bureau, 2x10 size No. 1 southern pine lumber standards dictate that the maximum wood knot size shall be 2.5" in diameter at edge and 3.25" in diameter at the centerline of the lumber.

According to the standard for this size of southern pine lumber, the lumber found on site appears to be in compliance with the specifications.

6.7 WITNESS STATEMENTS

Witness statements were provided by 14 workers. Of the 14 workers, at least five were above the draft tube formwork at the time of the collapse. Those five workers fell into the collapsed area.

A common theme that ties the witness statements together is that the collapse occurred rapidly. Terms used to describe the incident include:

"Everything went extremely fast, we all went down in seconds."

"All of a sudden, the form gave out."

"Heard a pop, then crashing sounds, and was sucked into a big hole."

Workers heard cracks or pops followed by a rapid fall into the collapsed area. Several workers commented on the loud noises they heard during the collapse.

7 COMMENTARY ON PROJECT TIMELINES

The objective of this section is to comment on the length of time elapsed between formwork construction to the time the formwork was placed on-site. In our opinion, the time elapsed is an important consideration because the formwork material appears to be exposed to the elements during this time without adequate protection.

The engineered drawings were stamped by CEI and Astaldi in October 2014. All formwork modules for the Unit 1 draft tube were ready to ship in November 2014, which coincided with Nalcor Energy LCP’s visit to the CEI fabrication shop.

In November 2014, Unit 2’s formwork modules were in production and were planned to be shipped to site in December 2014. Unit 1 and Unit 2 modules were installed beginning in September 2015. The installation of the draft tube formwork for Unit 2 on-site was completed in March 2016. The first concrete lift pour for Unit 2 was in October 2015 and second level pour was in April 2016. The collapse happened in May 2016, more than one year after the formwork fabrication.

In our opinion, from the time the formwork modules arrived on-site, to the time the formwork modules were installed and concrete was poured around the formwork, the modules were exposed to the elements and were not protected adequately.

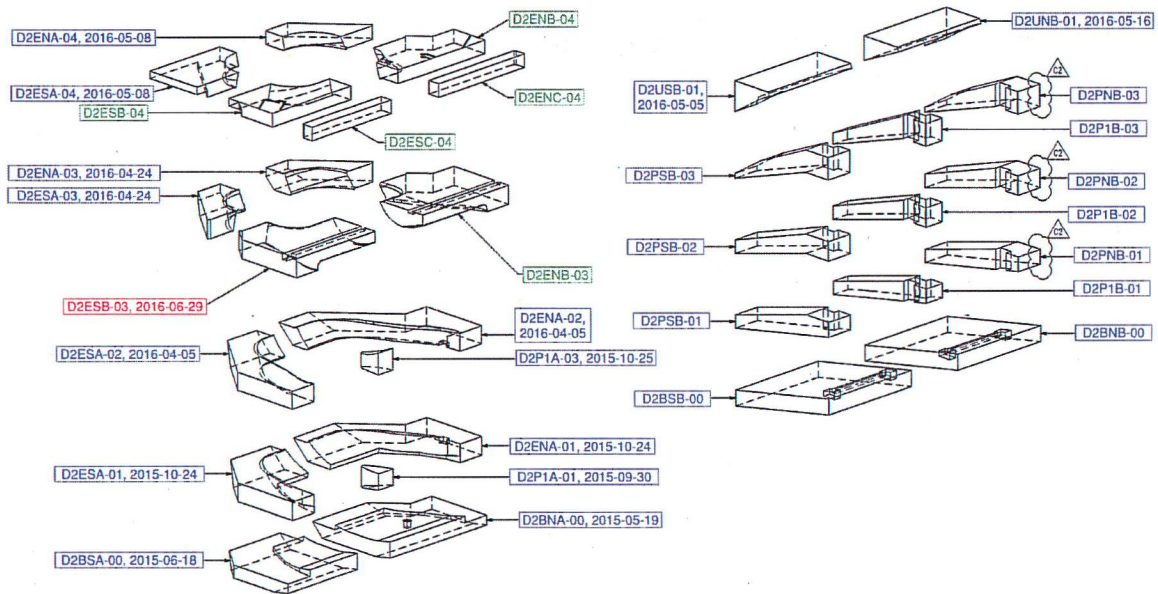


Figure 8: Exploded view of Unit 2 with scheduled pour dates

Figure 8 illustrates drawing number MFA-AT-SD-3312-CV-D99-0002-01 rev C2. The drawing is titled “DRAFT TUBE UNIT 2 - CIVIL - POUR CODING SYSTEM”.

It is unusual for this type of formwork to be fabricated several years before utilisation. Since the wooden structure was fabricated, stored and used over an extended duration, it would be expected that the material would be protected from the elements throughout its life cycle. Protection from elements includes keeping the moisture away from the lumber through covers and heating.

Pictures of work advancement are included in the Appendix.



8 DESIGN REVIEW

The draft tubes are all identical in design. The shoring towers at Unit 1 were analyzed as they were the only towers that had loads placed on them. The towers indicated signs of overloading, and observations of the aftermath of the collapse provide clues which leads the authors of this report to suggest that the formwork failure originated at the towers. The soffit panels did not indicate signs of overloading. As such, analysis was focused on the Unit 1 draft tube towers.

8.1 DESIGN STANDARDS

The CEI erection drawings do not make any reference to the design standards used in preparation of the drawing. Expected design standards for this type of project in Newfoundland and Labrador, when designed in 2014, would include:

Table 1: List of applicable standards

| Standard | Title |
|--------------------------------|---|
| CSA S269.1 - 1975 | Falsework for Construction Purposes |
| CSA S269.3 - M92 | Concrete Formwork |
| CSA 086 - 09 | Engineering Design in Wood |
| CSA S16 - 09 | Design of Steel Structure |
| Occupational Health and Safety | Newfoundland & Labrador Occupational Health & Safety Regulation |

Analysis conducted for this report is based on the standards listed in Table 1. Canadian standards are considered to be conservative when compared to American standards for material strengths of American southern pine species. American standards are also referenced in this report.

CEI calculations refer to the following American standards and literature:

Table 2: Relevant American Standards and Literature

| Standard | Title |
|-----------------|---|
| M. K. Hurd | Formwork for Concrete 6 th Edition |
| APA | The Engineered Wood Association |
| AISC | Steel Construction Manual 9 th Edition |
| NDS 2005 | American Wood Council, Manual for Engineered Wood Construction |
| IBC 2006 | International Code Council, International Building Code |
| CBC 2007 | California Building Code |
| ASCE 7-05 | American Society of Civil Engineers, Minimum Design Loads for Building and Other Structures |
| ASCE 7-02 | American Society of Civil Engineers, Minimum Design Loads for Building and Other Structures |

8.2 DESIGN PRESSURE

The general notes on the erection drawings call for the following design pressures: "Maximum Applied Concrete Pressure 1,526 psf" and "Maximum Applied Re-Shore Pressure 2,544 psf".

Structural drawings provided by SNC indicate that lift 3 above the draft tube has the largest concrete thickness at approximately 9.74 ft. The design pressures for this analysis were derived by taking into account the thickness of lift 3. This concrete thickness represents approximately 1,461 psf of concrete. Concrete density is assumed to be 150 lbs/ft³.

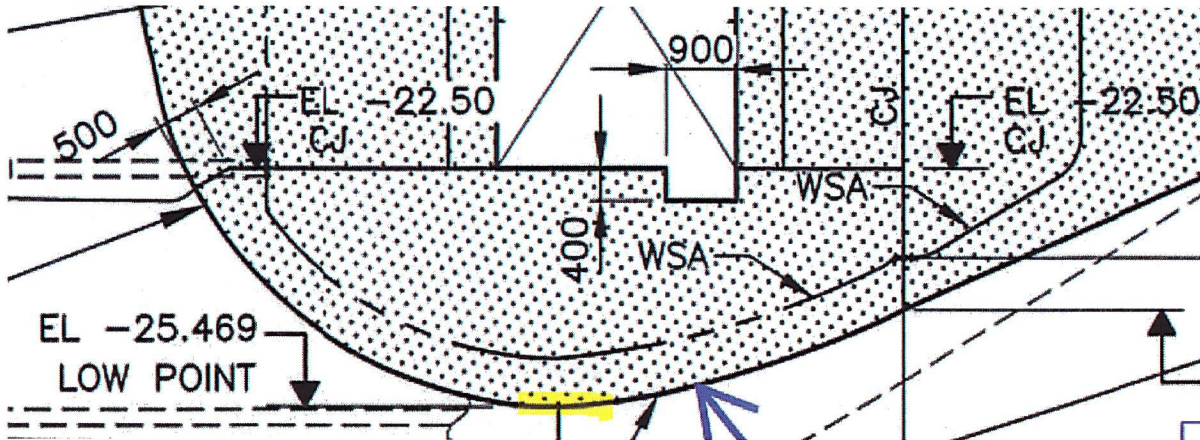


Figure 9: Design concrete pressure based on lift 3 thickness

The design pressures for the tower leg capacity calculation consists of the combined load of concrete, formwork, and access loads. Access load, per CSA standards, is 40 psf. Formwork load, per CEI drawings for panel weights, is 22 psf. For lift 3, this design pressure is therefore 1,523 psf, which is approximately the maximum permissible applied concrete pressure per CEI's general notes in the erection drawing.

8.3 TOWER CAPACITY AND CEI CALCULATION

Given the design pressure of 1,523 psf, and the steel beam configuration (which overhangs at each end), the shoring tower column should have a minimum allowable capacity of 57,700 lbs for placing the concrete for lift 3.

CEI used the 2005 NDS American standard to calculate the tower capacity. However, the CEI calculations omit the built-up characteristics of the shoring legs, which are considered by incorporating a Kf factor of 0.6 for a nailed built-up column. Per the CEI calculation, a Kf factor of 1.0 was used, which is inappropriate for this application. With a Kf of 1.0, and the leg unbraced length at 6 feet, CEI calculated an allowable capacity of 78,000 lbs. Assuming the correct Kf factor of 0.6, and using the as-built unbrace length of 5.5 ft., the shoring allowable capacity is 48,200 lbs. These calculations assume the usage of the 2005 NDS American standard.

If the same calculation is completed using the Canadian standard, the allowable capacity of the four 2x10 built-up legs is 40,500 lbs.

Per the American and the Canadian codes, the compressive strength of the shoring tower legs is insufficient.



8.4 TOWER BRACE CAPACITY

The tower braces are constructed with 2x6 and 2x10 lumber. Per CSA standards, the braces have to be designed to laterally withstand 2% of the vertical load. The actual brace loads in this tower configuration are 1,400 lbs for the 2x6 braces and 2,800 lbs for the 2x10 braces. Using the brace lengths shown on the drawing, the 2x6 braces allowable capacity is 1,385lbs and the 2x10 braces allowable capacity is 2,325lbs as per CSA standards.

The same lumber capacities with the American standard are respectively 1,775lbs and 2,977lbs.

The nailing requirement is respectively 7 and 14, 4-inch common nails at each end of the braces.

Per CSA code, the compressive strength of the wood braces is inadequate. However, the compressive strength of the wood braces is adequate per American requirements.

8.5 TOWER FABRICATION DETAILS

8.5.1 Nailing and Splicing Details

CEI tower fabrication drawings specify the lumber size, lumber grade, tower dimension and bracing configuration. Assembly methods such as nail size/length and spacing are not mentioned in the specifications, however. The specifications for splicing of lumber in the built-up column, per drawing note 6, indicate "Splice as necessary".

Per CSA standards, a built-up column of the size indicated by the drawings requires a minimum of two rows of 6-inch long nails along the length of the member. The rows are required to be a maximum of six inches apart and the nails of the same row are required to be a maximum of nine inches apart. Adjacent nails of the same row are required to be driven from opposite sides of the column.

Butt splices are required when the tower leg length is longer than the lumber length. The overall splice length should be a minimum of four feet long. The distance between individual splices of adjacent lamination must be at least half the overall splice length.

The information highlighted above is critical for the built-up column fabrication. In our opinion, this information should have been included in the fabrication documents.

8.5.2 Brace Configuration

The brace configuration illustrated on the tower fabrication drawing integrates the braces within the built-up column laminations. This is made possible by cutting a plank of one of the middle laminations to introduce the brace into the column.

This configuration is in contradiction with the built-up column splicing specifications identified above.

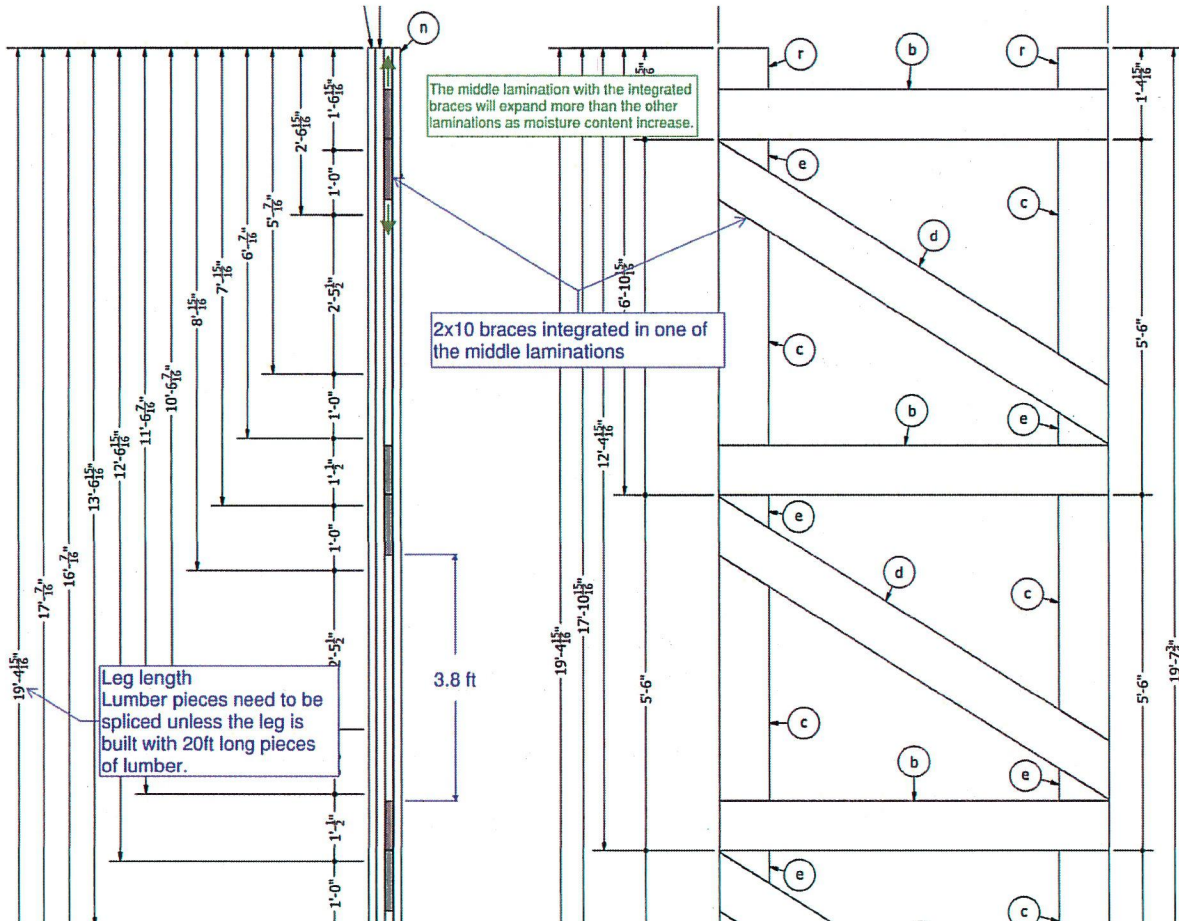


Figure 10: Shoring tower braces assembly detail

Figure 10 is from CEI drawing "W-41b, rev A", titled "Shoring Tower C41"

8.5.3 Wood Moisture vs. Wood Expansion

Changes in wood moisture content have a minor effect on lumber length and a significant effect on lumber thickness and depth. Wood structures need to be designed to accommodate this relative difference of change in dimension (section dimensions vs length dimensions) unless they are fabricated and used over a very short period of time (e.g. over the course of six months).

The tower legs were built with four laminations. Three of the four 2x10 built-up columns were built using length-wise planks. Relatively similar elongation would be experienced by the length-wise planks if moisture content of the wood increases; the elongation effect on length would be minor.

On the other hand, the lamination with the diagonal braces would experience significant elongation upon wood moisture content increases (Figure 11). This change in lumber dimension given changing wood moisture content has the potential to significantly compromise structural integrity if it is not accounted for in the design. Distortion in the tower legs was observed on-site, and has the potential to compromise the structural integrity of the tower legs.

In our opinion, the tower legs cannot be expected to retain their design dimensions over the course of more than six months while exposed to the elements. Wood moisture content fluctuations would result in distortion of the tower legs.

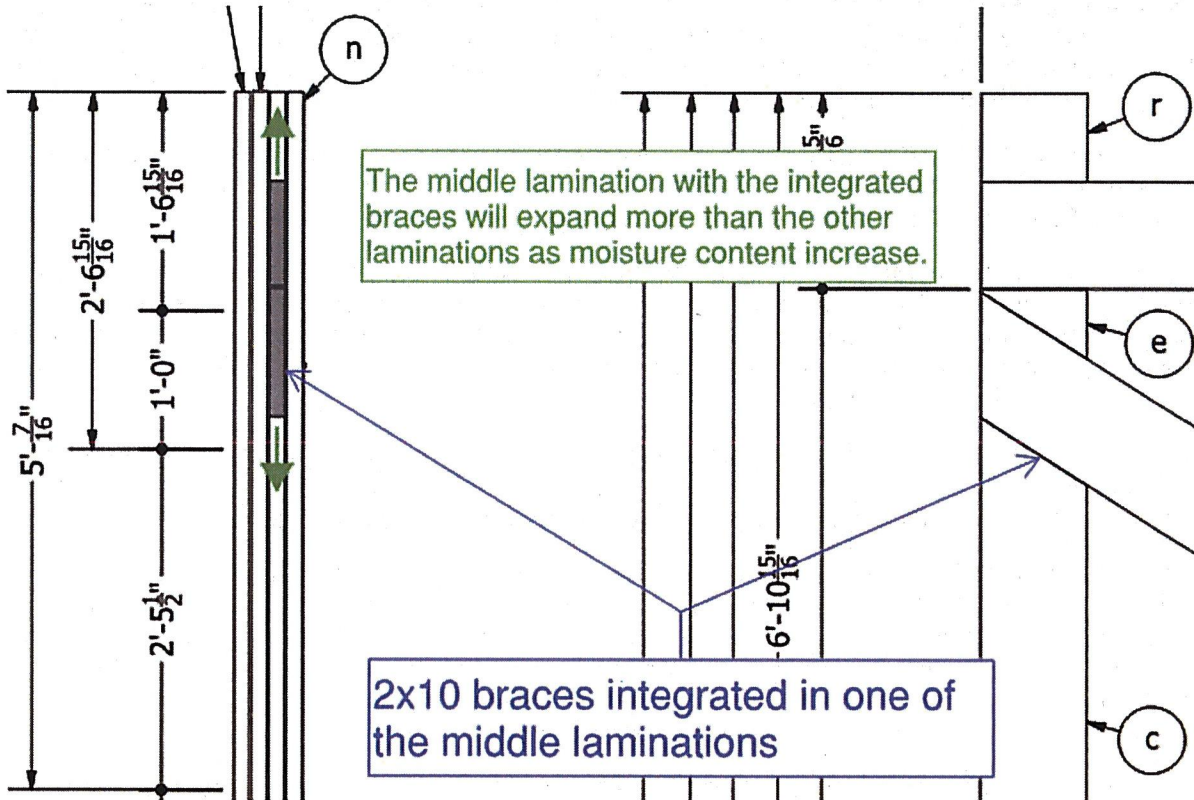


Figure 11: Shoring tower braces assembly detail, drawing "W-41b, Rev A – Shoring Tower C41"

8.6 TOWER INSTALLATION DETAIL

The details for the tower installation identify the wedges, the grout pad, and the anchors required on the concrete floor for the tower to sit on. The installation details are necessary to adequately transfer load from the legs to the ground, and to adjust the tower height. The drawing package provides no specific details with regards to how the steel beam (between the tower and soffit panels) should be sitting on top of the tower. The lumber used for the construction of the built-up legs has a depth of 235mm. The W250x25 steel beams sitting on top of the shoring legs have a flange width of 102mm. Without a steel plate to fully support the top of the tower leg, or adequate shims for adjustment between the beam and the top of the posts, the load from the beam is transferred to only a small section of the column. As such, there are no means for leg height adjustment.

In our opinion, a steel plate for load distribution of the full surface of the leg and steel shim for leg height adjustment at the interface of the beams and tower legs would be required to transfer the load uniformly across the leg section and to account for leg height adjustments.

9 DISCUSSION

9.1 LACK OF MAINTENANCE ON WOOD STRUCTURES

Wood will decay and fungus will grow on wood surfaces if certain conditions are met. The key condition for wood to decay and fungus to grow is high wood moisture content. The Canadian Wood Council recommends limiting the wood moisture content to 20% or less (weight of water in wood over weight of wood itself).

Where the lumber is protected from rain and ground water, the wood moisture content is a function of the average ambient air relative humidity. A 20% wood moisture content can be expected if the relative humidity is above 85% for an extended period of time.

In the event that lumber is in contact with ground water, the moisture content cannot be controlled. Fungus will grow at a slower pace on submerged lumber due to the absence of oxygen. The wood just above the water level will be saturated with water, thus facilitating fungus growth at a rapid pace. The submerged part of the lumber will also experience rapid fungus growth as the water drains away.

The analysis described in this report assumes the towers were fabricated with sound lumber in Kansas. Hence, it is our opinion that the lumber used in the fabrication of the formwork was exposed to the elements in between the transportation, storage and/or utilisation process.

The formwork was built in the summer of 2014, installed at Unit 2 in the fall of 2015, and lifts 3 and 4 of the Unit 2 draft tube were poured during the spring of 2016. Hence, it would be expected that precautions would be taken between fabrication of the form and concrete pours to preserve the wood structures. Examples of such precautions would be adequate formwork covers that protect from the elements, minimal heating inside the formwork and aforementioned covers, and constant removal (i.e. pumping) of all incoming water.

Per our observations, the wood structures were not protected adequately against the elements. There were no indications of covers, heating, or pumping, which resulted in increased wood moisture content, which led to material degradation.

9.2 TOWER BUCKLING

Shoring towers are comprised of four built-up columns. Each built-up column consists of 4-ply 2x10 lumber. The shoring towers in this project are used to hold the soffit panel modules (see the A29 panel in Figure 7 for an example). Shoring leg buckling is an expected failure mode for shoring towers, where the tower integrity fails before reaching the material compression limit. Observations at the Unit 1 draft tube indicate that the shoring tower built-up columns experienced buckling prior to the lumber reaching its yield strength. Evidence of material buckling includes the existence of s-shaped buckling on some of the tower's built-up columns.

Per CEI shoring and reshore capacity, the shoring towers were designed with the intention of supporting lift 3 and lift 4.

Buckling is expected to occur when the built-up column load reaches its ultimate capacity. The s-shaped buckling of the posts at the Unit 1 draft tube appears to have occurred after lift 3 was poured. If the posts had buckled during the lift 3, it is likely that the form would have collapsed. There are three possible reasons for the built-up columns to buckle after initial loading:

1. After pouring lift 3, the loads on the built-up columns were at near capacity. Additional load would have manifested from the deflection of lift 3 as lift 4 was being poured.



However, the period of time between pouring lift 3 and lift 4 allowed lift 3 to cure. As such, lift 3 was able to support its own weight and the weight of lift 4. This would have allowed the tower to buckle without total collapse.

2. Additional internal stresses in each built-up column could originate from an increase in wood moisture content (refer to Section 8.5.3). A 5% change in wood moisture content has a 1% effect on the lumber width and depth. In this case, due to the design with which the braces were integrated in the shoring built-up column, the 1% effect on the brace depth could result in built-up column growth of 10mm. A constrained built-up column extension of this scale would result in an increased built-up column load of about 24,000 lbs (or 40% of the design load). Once lift 3 was poured, the two ends of the built-up column would have been constrained, leaving no space for material elongation, thus, increased internal stresses on each built-up column.
3. As wood moisture content increases, lumber loses its resistance capacity. CSA standards specify a reduction in resistance capacity for wet service condition ($K_{sc}=0.91$). After pouring the Unit 1 draft tube (level 3), the legs were fully loaded and near the ultimate capacity. Evidence indicates that wood was exposed to the elements and that the ground water was allowed to pool at the bottom of the draft tube. Hence, the resistance capacity of the tower legs decreased over the exposure period, which ultimately led to tower buckling.

9.3 CEI CALCULATIONS

Wood lumber strength properties from both Canadian and American standards are defined by destructive testing of full size lumber planks. Published properties represent the lower 5th percentile of the test result. As such, the standard ensures that less than 5% of the planks are weaker than the published properties.

The percentile-based projection distribution for wood strength is wide as compared to steel materials. For instance; the 95th percentile could be twice as strong as the 5th percentile. Although it is inappropriate to under-design wood structures, they are in general stronger than the calculated values due to the inherent nature of developing the published properties.

This could be a reason as to why the draft tube at Unit 1 did not collapse although the built-up column design was inadequate for the design load.

9.4 GAPS IN JOINTS OF BUILT-UP TOWER LEGS

The gaps in the joints of the built-up tower are not related to poor fabrication workmanship but to poor design that did not take into consideration the behavior of the wood upon changes in moisture content. In addition, the storage methods used on-site for the wood structure failed to prevent the moisture content from increasing.

9.5 INSPECTION

The formwork checklist for pour D2ESB-03 was filled out and reviewed prior to the pour. The checklist was signed by an Astaldi foreman, a field engineer, a quality controller, and a superintendent on May 28, 2016. It was also signed by a Nalcor representative on May 29, 2016.

The formwork checklist contains 14 listed items to be inspected prior to placement of concrete. The draft tube formwork was not explicitly on the checklist, however. Two items on the checklist



were relevant to the structural integrity of the formwork: Item 8 - "Formwork and Falsework [to] conform with approved shop drawing" and Item 14 - "Doka checklist completed" (which is not applicable as there is no DOKA formwork involved in this pour.

Newfoundland & Labrador OH&S regulation Item 385 states: "Immediately before the placement of concrete or other loading, an employer shall ensure that the concrete formwork and falsework is inspected by a qualified person."

It is questionable as to whether the intent and spirit of the OH&S regulation was met by filling and signing the formwork checklist referenced above. Moreover, the formwork checklist does not identify the "qualified person" responsible for the inspection of the draft tube falsework and formwork. Given the types of temporary structures used in this project, and the level of risk involved in the construction works, aDB would expect the inspection to be completed by the formwork designer or the designer's designate. A separate certificate of conformance signed by the inspector would also be expected.

The defects identified in the previous sections should be obvious to any carpenter, whether they are an apprentice, journeyman, or a master. For instance, decayed wood and fungus growth on a wooden structure should immediately raise questions and red flags. The quality of the wood was so poor in some cases that it could be picked at with a pen. The beam misalignment above the tower leg should also have raised red flags. It should not take a quality control program or inspection of any kind to highlight such an obvious defect. The same comments apply to the formwork erection supervisors.

9.6 TAILRACE SOFFIT CONCRETE POUR

Other construction activities that may have affected the integrity of the draft tube formwork include the formwork erection and concrete pours of lifts D2USB-01 and D2UNB-01. These two pours are downstream of the draft tube and are a continuation of D2USB-03 and D2ENB-03 above the water passage. The tailrace soffit pours were completed prior to the draft tube soffit pours. To build the construction joint at the draft tube tailrace interface, DOKA designed a wall-form for the upstream side of D2USB-01 and D2UNB-01.

The upstream tailrace wall form sits on the draft tube formwork. To counteract the lateral pressure on this wall-form, DOKA designed a system of ties attached to the tailrace soffit form, below the pour. Because the ties are tied down to the soffit, as they hold the wall form in place horizontally, they also pull the wall-form down. These loads are the combined tension and shear loads. Due to the ties, the total shear load may be several times greater than the shear load produced by the weight of the form alone. Therefore, the draft tube formwork needs to withstand the vertical shear loads due to the tie downs.

The tailrace wall-form weight and tie-down combined load is approximately 2,000 lbs per linear foot all along the downstream edge of the draft tube formwork. This combined load is less than the draft tube formwork design load. The tailrace wall load is applied at the very edge of the draft tube formwork. The draft tube structure is not designed to sustain loads at its edge only. Unless special precautions are taken to rebalance the load on the draft tube, the panel with load on the edge is subject to overturning.

Although nothing has been reported regarding the possible overturning of the draft tube panels during the tailrace concrete pours, it is possible that this disrupted the integrity of the formwork prior to the pouring concrete on the draft tube level 3 form.

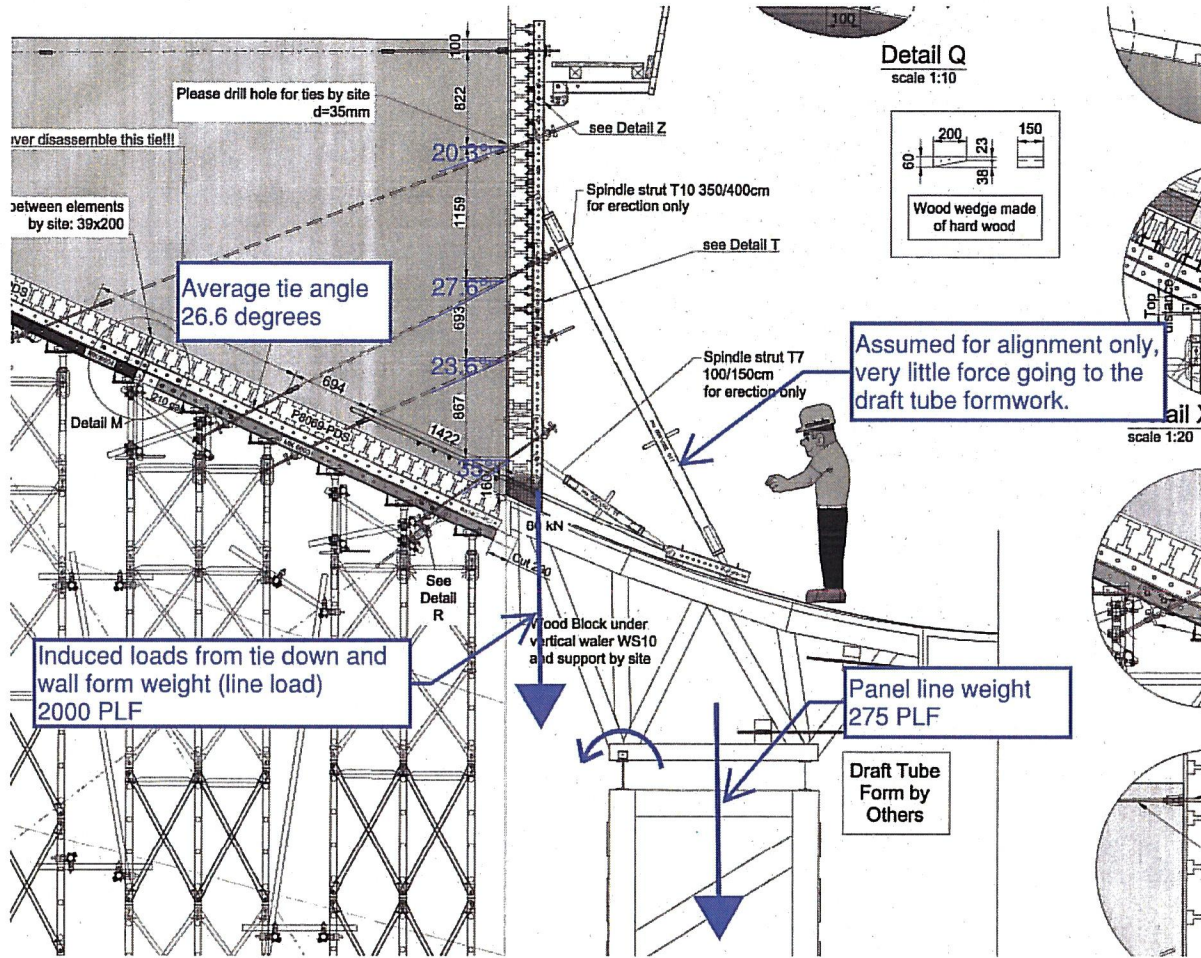


Figure 12: Tailrace wall-form sitting on the draft tube formwork

Figure 12 drawing number and title are "MFA-AT-SD-3310-CS-D04-5690-01 rev C1 - Draft Tube Slab – Units 1,2,3,4; Section 1-1 - D(1,2,3,4)USB-01".

The draft tube was designed by CEI for its own concrete load. DOKA designed the tailrace formwork with its wall-form sitting on the draft tube formwork. DOKA has a note on their drawings that indicates: "Draft Tube Form by Others".

The project interfacing of two suppliers led to a design gap between two different suppliers. The contractor should be responsible for closing the design gap by communicating with the draft tube formwork designer to ensure the new load is acceptable. aDB cannot confirm whether or not CEI was notified by Astaldi regarding the change in load. Nevertheless, additional support would be required to counteract the draft tube panel overturning.

10 RECOMMENDATIONS

10.1 RISK MANAGEMENT OF TEMPORARY STRUCTURES

CEI designed the temporary structures, and Astaldi approved those designs. Per the evidence discussed herein, the shoring towers were under-designed. Moreover, the inspection process before loading was inadequate. It is also unclear whether or not the structural adequacy of formwork was inspected by a competent person.

The failures discussed above are partially a result of the lack of a risk management of temporary structures program. In this case, the risks involved were poorly identified, the reviewer's experience was questionable, and the inspection process was unclear.

Assessment of risks related to the construction and upkeep of temporary structures should be a daily task. The stakeholders should be involved in the process of identifying the risks associated with every temporary structure. The program should identify the risks that each stakeholder would be responsible for mitigating. The experience of the designer, reviewer, and inspector should be commensurate with the level of risk associated with the types of temporary structures employed by the project. Their experience should be known and approved by an individual who is responsible for managing the risks associated with the temporary structures and also by stakeholders.

10.2 WOOD STRUCTURE PRESERVATION

The integrity of the wood was compromised by the elements given the length of time between fabrication and utilization in constructing and loading the formwork. These structures do not age well unless protected adequately from the elements. The following precautions should always be observed to preserve the wood structure for an extended period of time:

- After fabrication, protect the wood surface with water repellent products that do not compromise the wood material properties
- Protect the wood structure from rain with waterproof material immediately after fabrication. The wrapping should be completed so trapping of moisture within the wrapping is avoided.
- The wrapped structure should be stored in a well ventilated area.
- Once unwrapped and installed, the wood form should be implemented immediately, and the structure should be protected from the elements (e.g. rain and snow).
- Incoming water to the area should be continuously drained.
- The installed structure's interior should be ventilated. If the ventilation isn't sufficient to limit the ambient air humidity, the structure's interior should also be heated.
- The most efficient way to protect the wood structure would be to limit the length of its lifecycle.

11 CONCLUSION

11.1 COMMENTARY ON WORK CULTURE

Per CSA S269.3 standards section 8.1.1, "Formwork shall be assembled, erected, and stripped under the supervision of a competent person". Additionally, per CSA S269.1 section 7.2.2 Supervision of Workmen indicates that "only competent supervisors experienced in the construction of temporary support structures shall supervise the erection of the falsework. It is our opinion that workmen should be adequately instructed by such supervisors on the hazards that they and others will be exposed to during the erection period and on the precautions that must be taken because of those potential hazards".

Based on observations made on-site by aDB, worksite culture seems to contradict the spirit of the CSA standards. Ideally, the crew on-site would have a clear picture of what they are building, and how they are going to build it. Moreover, the crew would also ideally have the competence to identify the difference between good and bad workmanship.

For instance, during the site visit, aDB found ice built up on the formwork which indicates that the bottom of the formwork was underwater at some point over the winter. Competent crew would have noticed this issue and flagged it as a safety and quality concern. A competent supervisor would also have had the capacity and proficiency to notice the water in the draft tube area, and identified a need to pump the water out of the construction area. The lack of action in this case indicates a lack of competency, a lack of safety leadership, a complacent workforce, or a combination of the aforementioned.

The labour crew and its direct supervision failed to assess and identify the issues discussed herein. The carpenter crew were in direct contact with the decayed lumber and did not flag the inadequate use of poor material as an issue.

In our opinion, implementing an effective safety leadership program on-site would empower crew to raise safety and quality issues so as to prevent another similar failure from recurring. Workers should have been trained to understand and be aware of quality and workmanship issues. They should be encouraged to speak up about the smallest of issues and ensure they are aware of the expected end-result of their day-to-day work. Upper management should ensure workers' competence and raise awareness of the expectation that subpar workmanship is not acceptable.

11.2 CAUSES OF THE FORMWORK COLLAPSE

The collapse destroyed all the evidence that might have allowed aDB to pin-point a single cause of the collapse. James Reason, in a book *Human Error* published in 1990, created the "Swiss cheese model" of system failure wherein an ideal system is analogous to layers of Swiss cheese. The holes in the Swiss cheese are areas where processes can fail, and each slice of cheese is a "defense" layer. If an error passes through one slice, it should be caught by the next layer of defense. Catastrophic failure occurs when an error passes through all layers of defense in a system.

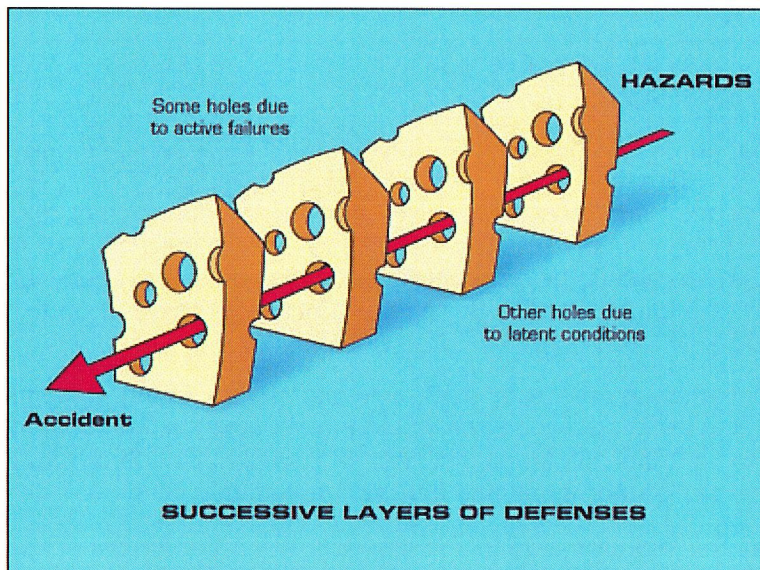


Figure 13: James Reason's "Swiss Cheese Model" per *Human Error* (1990)

There were several layers of defense prior to the formwork collapse. These layers included engineered design, design review, crew and supervisor competencies, training and diligence while performing the work (e.g. fabrication, storage, erection), and formwork inspection prior to pouring concrete. Evidence indicates that each of these layers were inadequate in preventing a catastrophic failure from occurring.

It is impossible to base the analysis discussed herein solely on the exact condition of the formwork before the incident given that it was buried in concrete. The nature of failure made it difficult to discern if the shoring towers were all there. Design analysis and the site observations were heavily relied upon to identify contributing factors that led to the collapse.

The collapse was large, quick, and not progressive in nature. The collapse was fast, indicating that each section of the shoring was at or near ultimate capacity, and when the ultimate capacity was reached in one area, and the collapse started, followed by failure of the adjacent overloaded structures.

The findings of this report suggest that one of the following occurred:

- (i) The shoring system was not designed properly
- (ii) Wood integrity of the formwork was compromised
- (iii) The shoring system was not installed correctly
- (iv) The shoring system fabrication was inadequate
- (v) A combination of these aforementioned factors

There are many potential issues discussed within this report which may have influenced the load-carrying capability of the shoring system:

- The shoring tower's capacity was under-designed.
- The tower leg lumber splices could have been inadequate.
- The formwork installation may have been deficient.
- The integrity of the wood material could have been compromised.



11.3 CLOSING REMARKS

aDB trusts that the findings of this analysis are written and delivered to your satisfaction. Utmost care was taken to ensure the analysis was completed to the highest of standards. Should you have any further questions or comments, please do not hesitate to contact the undersigned.

Mathieu Légaré, P.Eng
Construction Engineer, aDB Engineering

Sean Dingley, P.Eng
Principal, aDB Engineering



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5 INCIDENT DESCRIPTION



Unit 2 draft tube - View from the North West corner of unit, looking below

Picture from Nalcor

Taken May 22, 2016



Unit 2 draft tube - View from North West corner of unit, looking below

Picture from Nalcor

Taken May 28, 2016



Muskrat Falls Powerhouse – Four Draft Tubes

Picture from Nalcor

Taken May 29, 2016 at 15:44



Unit 2 draft tube - View from North West corner of unit, looking below

Picture from Nalcor

Taken May 30, 2016

6 SITE OBSERVATIONS

6.1 UNIT 2 - JUNE 2016 SITE VISIT



Unit 2 draft tube - View of South East corner of unit, looking below

Picture from aDB Engineering

Taken June 15, 2016

6.2 UNIT 2 - AUGUST 2016 AND OCTOBER 2016 SITE VISIT



Unit 2 - Shoring Tower weathering

Picture from aDB

Taken August 25, 2016



Unit 2 - Shoring tower rotten lumber

Picture from aDB

Taken August 25, 2016

6.3 UNIT 1

6.3.1 Unit 1 - Maintenance Issues



Unit 1 - Ice built up within ribs of panels A9

Picture from aDB

Taken June 16, 2016



Unit 1 - Fungi growth on lumber of tower B6

Picture from aDB

Taken June 16, 2016



Unit 1 - Fungi growth on lumber of tower B4

Picture from aDB

Taken June 16, 2016



Unit 1 - Decayed wood on tower B3

Picture from aDB

Taken June 16, 2016



Unit 1 - Decayed wood on tower B2

Picture from aDB

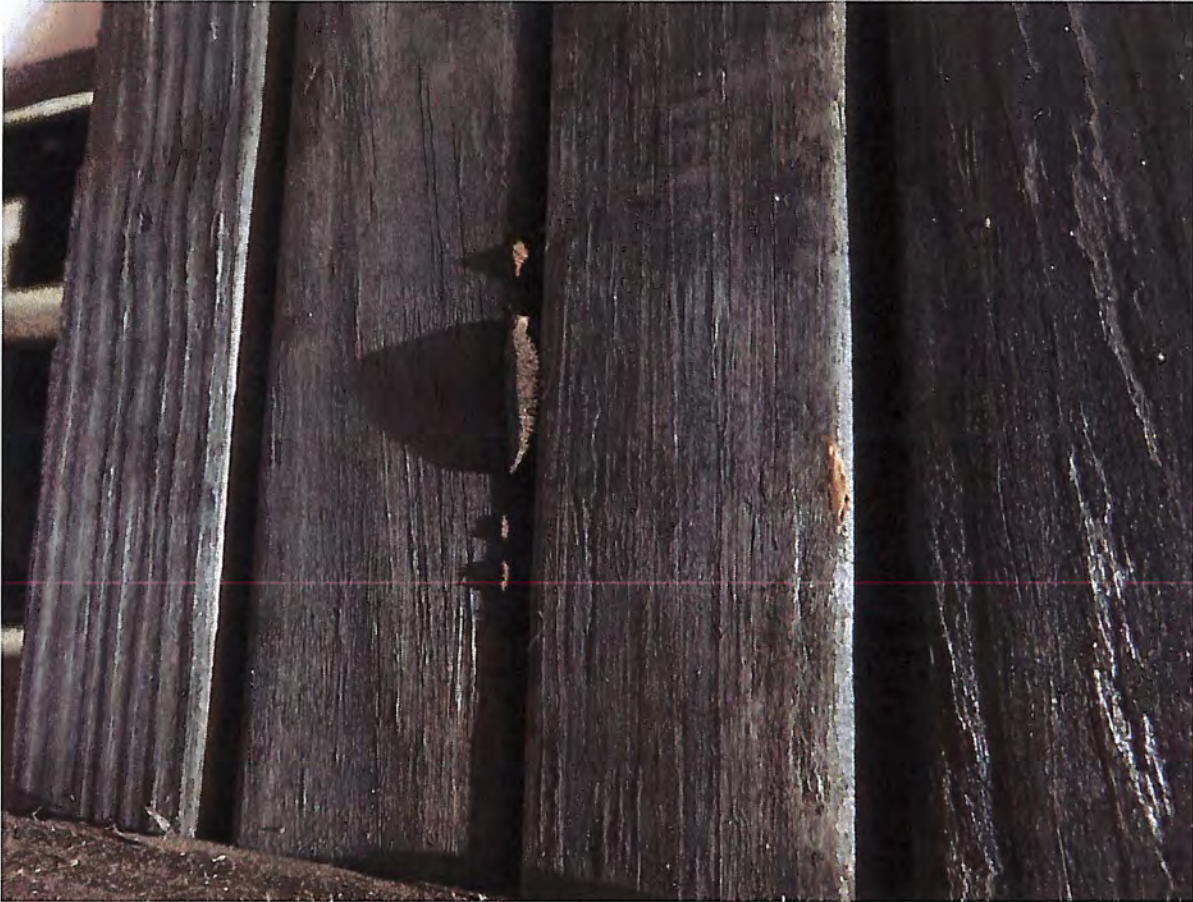
Taken June 16, 2016



Unit 1 – Mushroom growth on tower B3

Picture from aDB

Taken June 16, 2016



Unit 1 – Mushroom growth on tower B3

Picture from aDB

Taken June 16, 2016



Unit 1 - Dry rot on shoring tower

Picture from aDB

Taken June 15, 2016

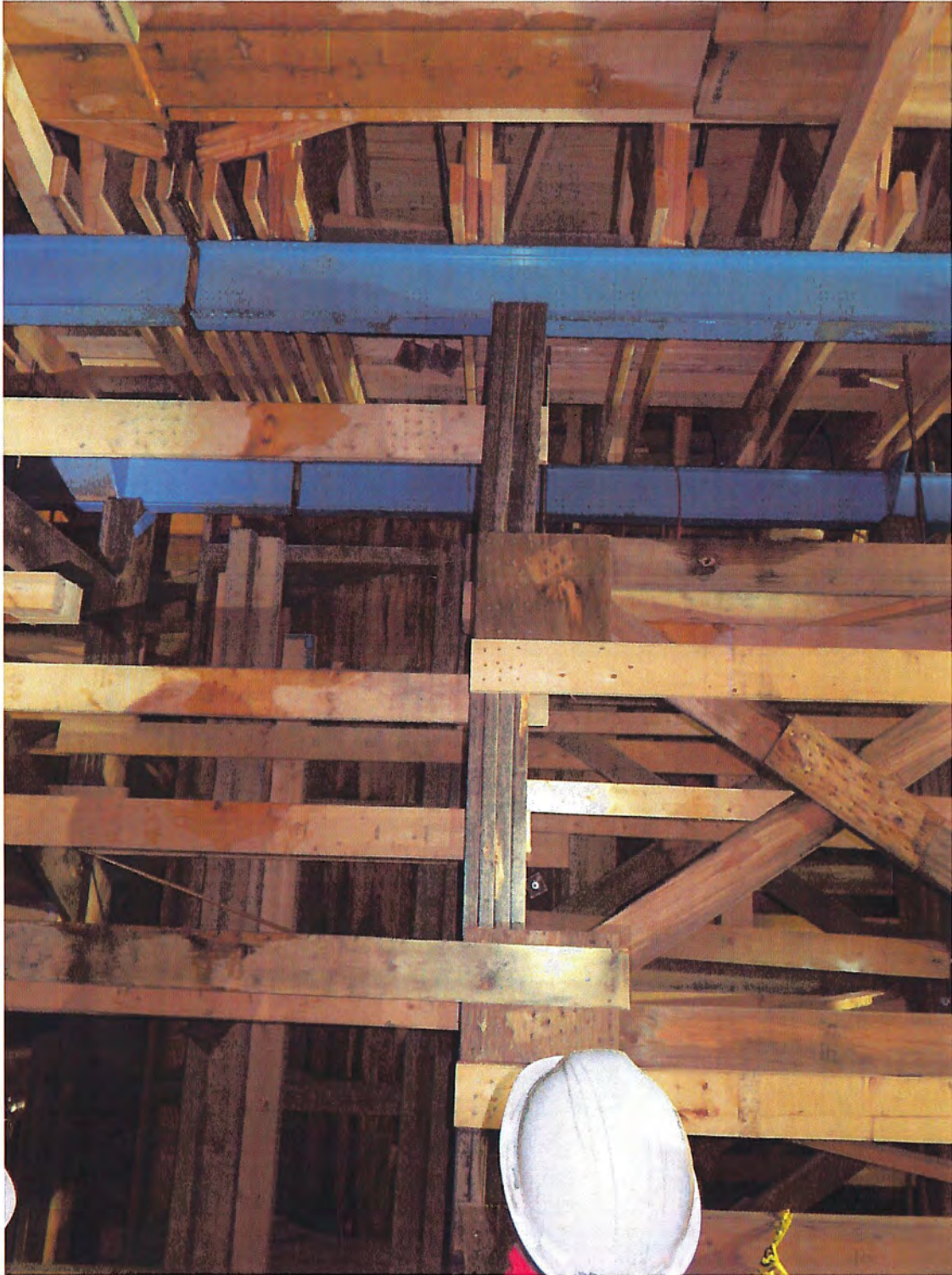
6.3.2 Unit 1 - Tower Buckling



Unit 1 - Tower B2, buckling at top of post

Picture from aDB

Taken June 16, 2016



Unit 1 - Tower B3, buckling at top of post

Picture from aDB

Taken June 16, 2016



Unit 1 - Tower B3, buckling at bottom of post

Picture from aDB

Taken June 16, 2016

6.3.3 Unit 1 - Compression Failure



Unit 1 - Tower B1, compression failure below steel beam

Picture from aDB

Taken June 16, 2016



Unit 1 - Tower B5, compression failure below steel beam

Picture from aDB

Taken June 16, 2016

6.4 UNIT 3

6.4.1 Unit 3 - Lumber Weathering



Unit 3 - Installed shoring tower weathering

Picture from aDB

Taken June 16, 2016



Unit 3 - Installed shoring tower weathering

Picture from aDB

Taken June 16, 2016

6.4.2 Unit 3 - Gaps in Built-up Tower Leg Joints



Unit 3 - Built-up tower with a gap between two planks

Picture from aDB

Taken June 16, 2016

6.4.3 Unit 3 - Fabrication Workmanship



Unit 3 - Saw cut mark in lumber

Picture from aDB

Taken June 16, 2016



Unit 3 - Saw cut mark in lumber

Picture from aDB

Taken June 16, 2016

6.5 UNIT 4

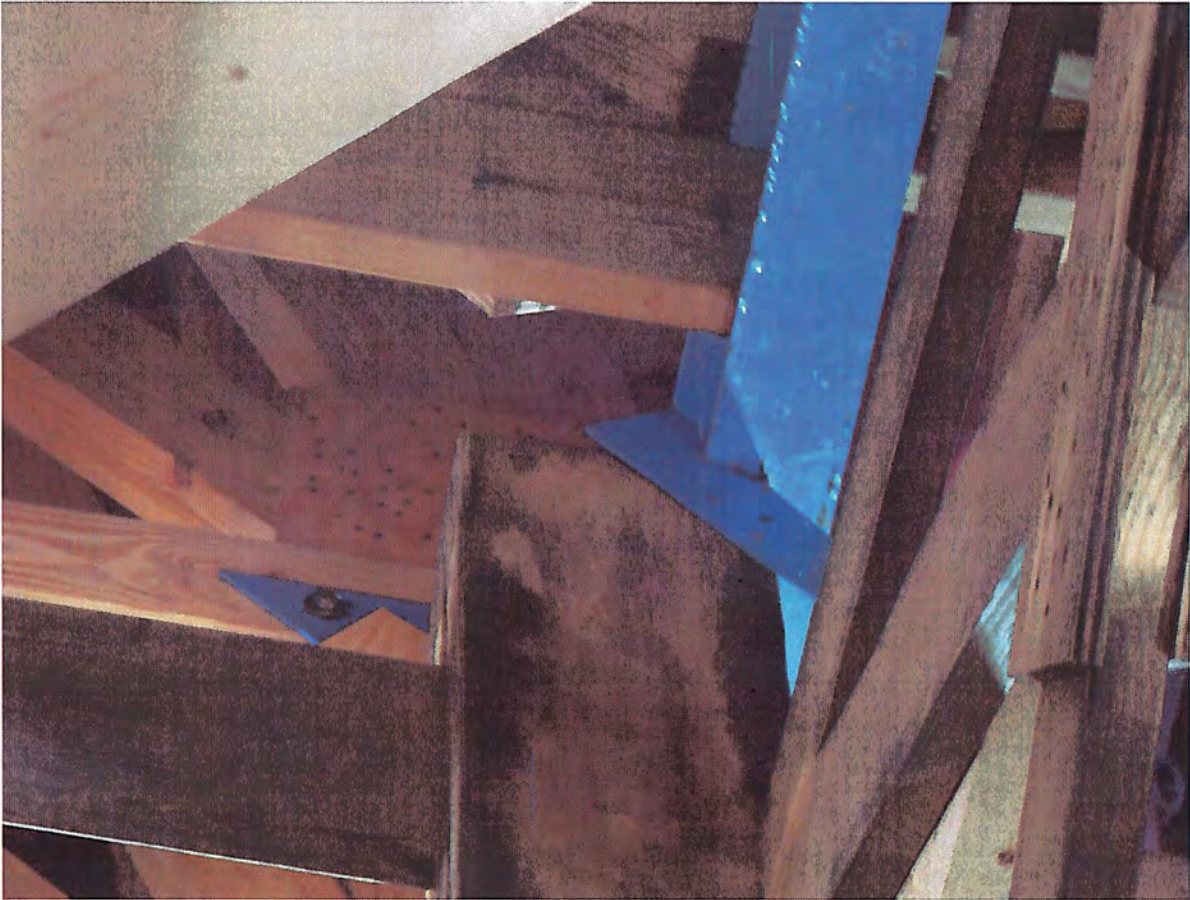
6.5.1 Unit 4 - Installation Workmanship



Unit 4 - Tower misaligned with steel beam

Picture from aDB

Taken June 15, 2016



Unit 4 - Tower misaligned with steel beam

Picture from aDB

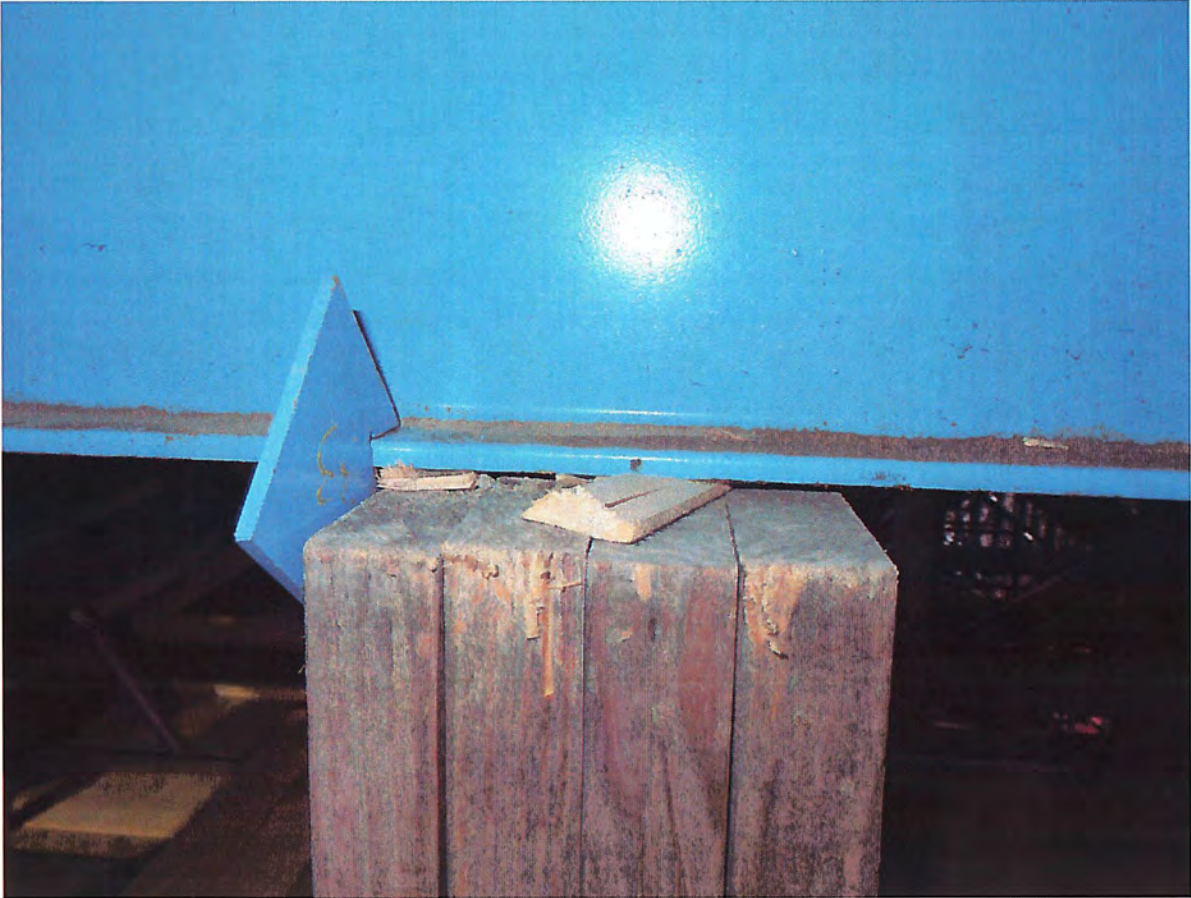
Taken June 15, 2016



Unit 4 - Missing shim between post and steel beam

Picture from aDB

Taken June 15, 2016



Unit 4 - Insufficient shim between post and steel beam

Picture from aDB

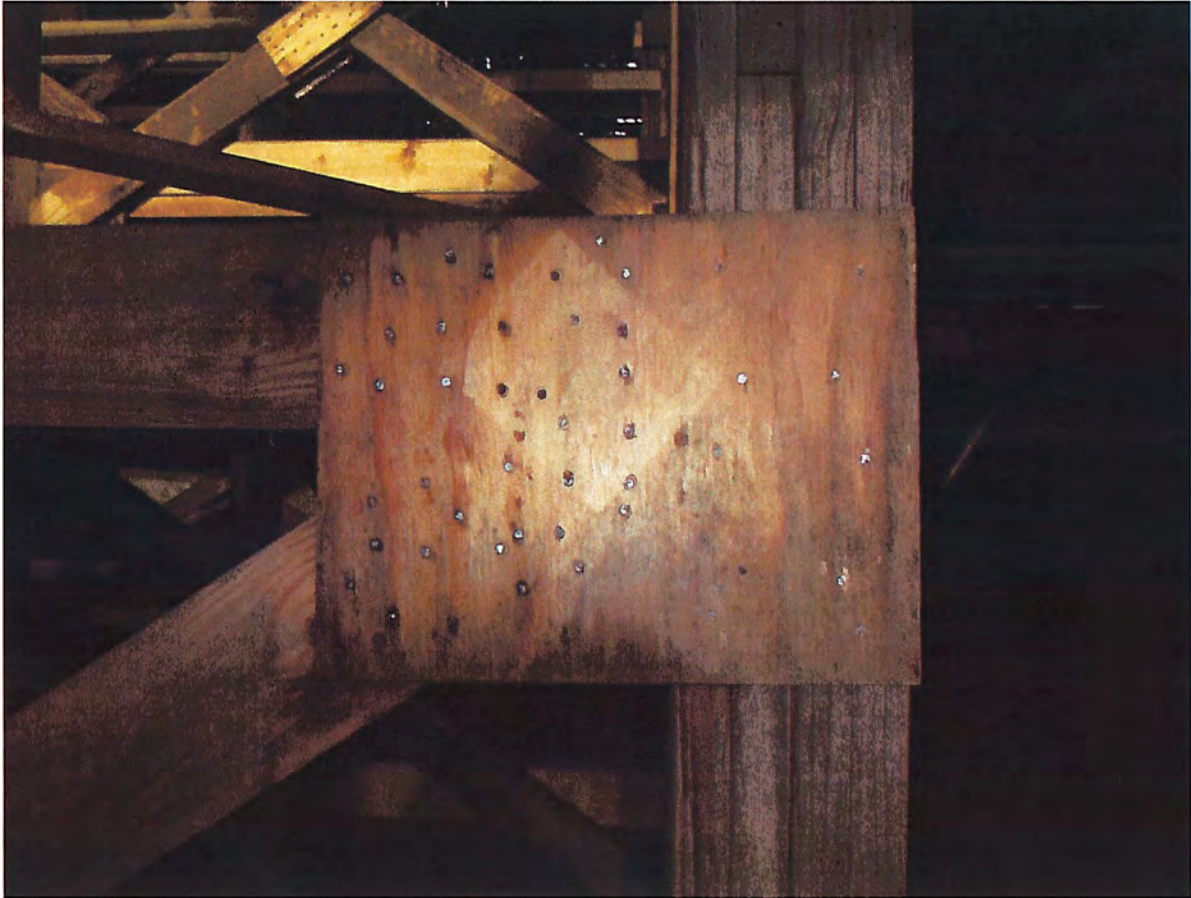
Taken June 16, 2016



Unit 4 - Tower brace installed on site – Insufficiently nailed

Picture from aDB

Taken June 15, 2016



Unit 4 - Tower brace installed on site – Insufficiently nailed

Picture from aDB

Taken June 15, 2016

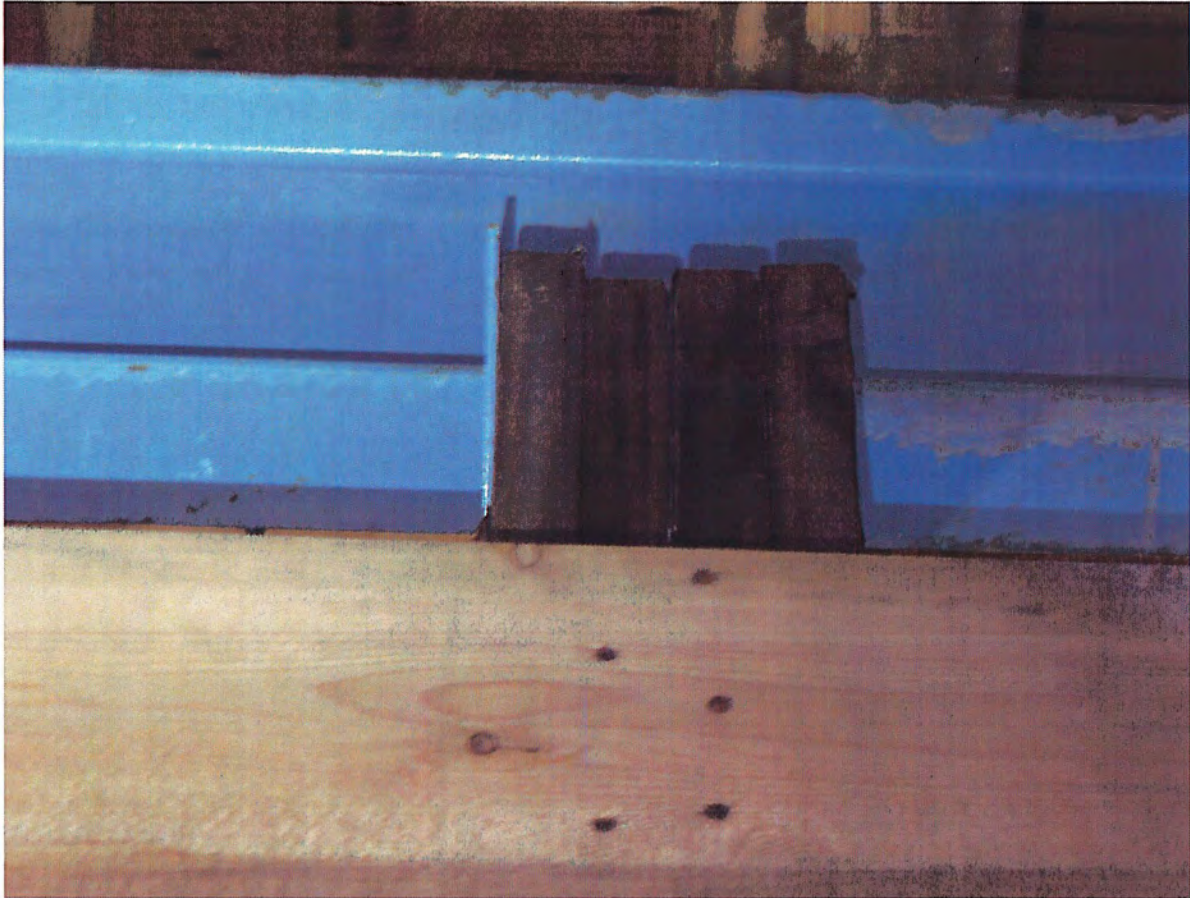


Unit 4 - Damage to tower during formwork or tower alignment

Picture from aDB

Taken June 16, 2016

6.5.2 Unit 4 - Fabrication Workmanship



Unit 4 - Uneven tower top

Picture from aDB

Taken June 16, 2016



Unit 4 - Inappropriate location for built-up post joint

Picture from aDB

Taken June 15, 2016



Unit 4 - Inappropriate location for built-up column joint

Picture from aDB

Taken June 15, 2016

6.6 GENERAL OBSERVATIONS OF GRAFT TUBE FORMWORK MATERIAL

6.6.1 Identification of Lumber



Wood Identification Stamp

Picture from aDB

Taken June 16, 2016

6.6.2 Grade of Lumber

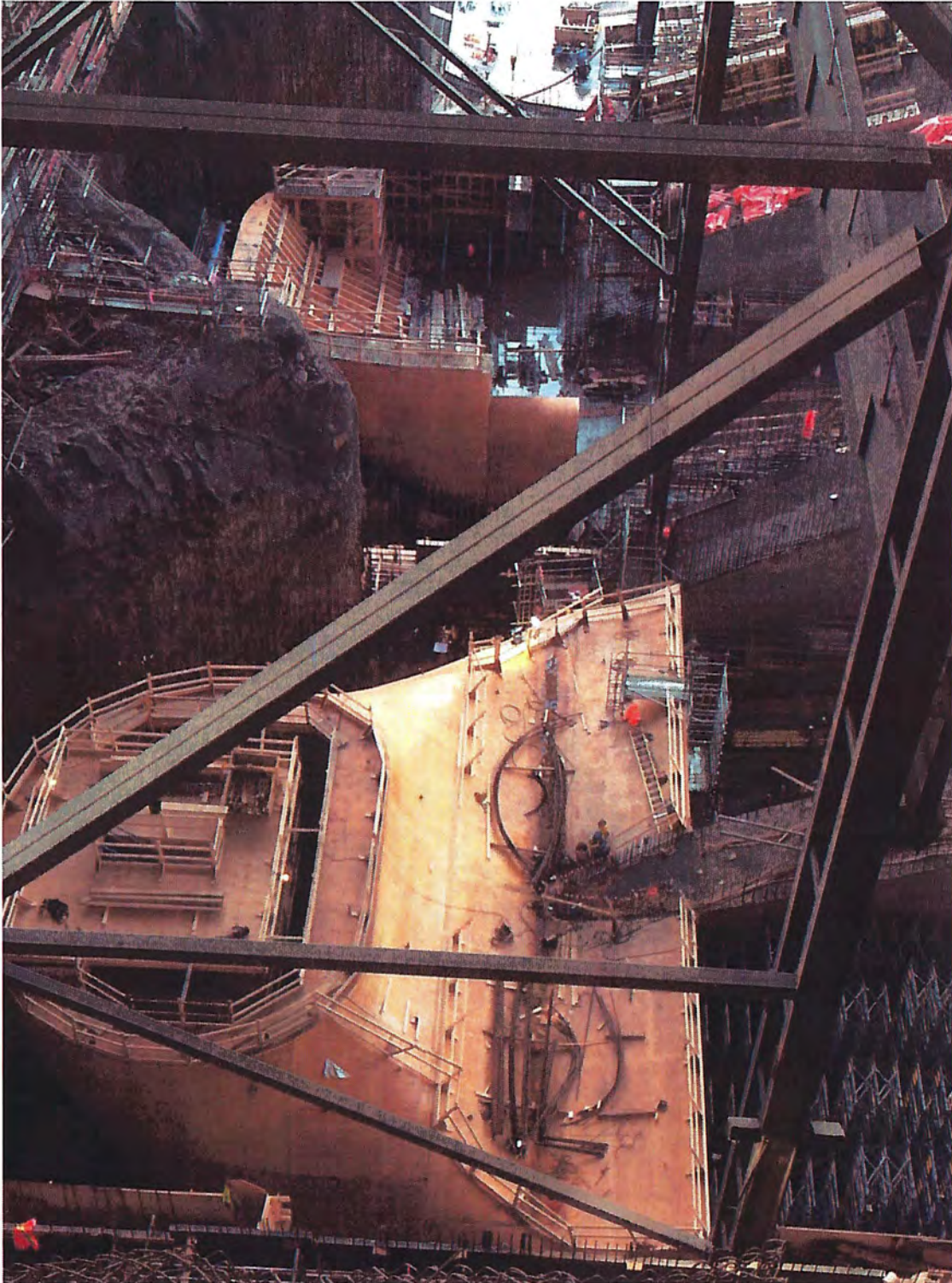


Nuts on Lumber
Picture from aDB
Taken June 15, 2016

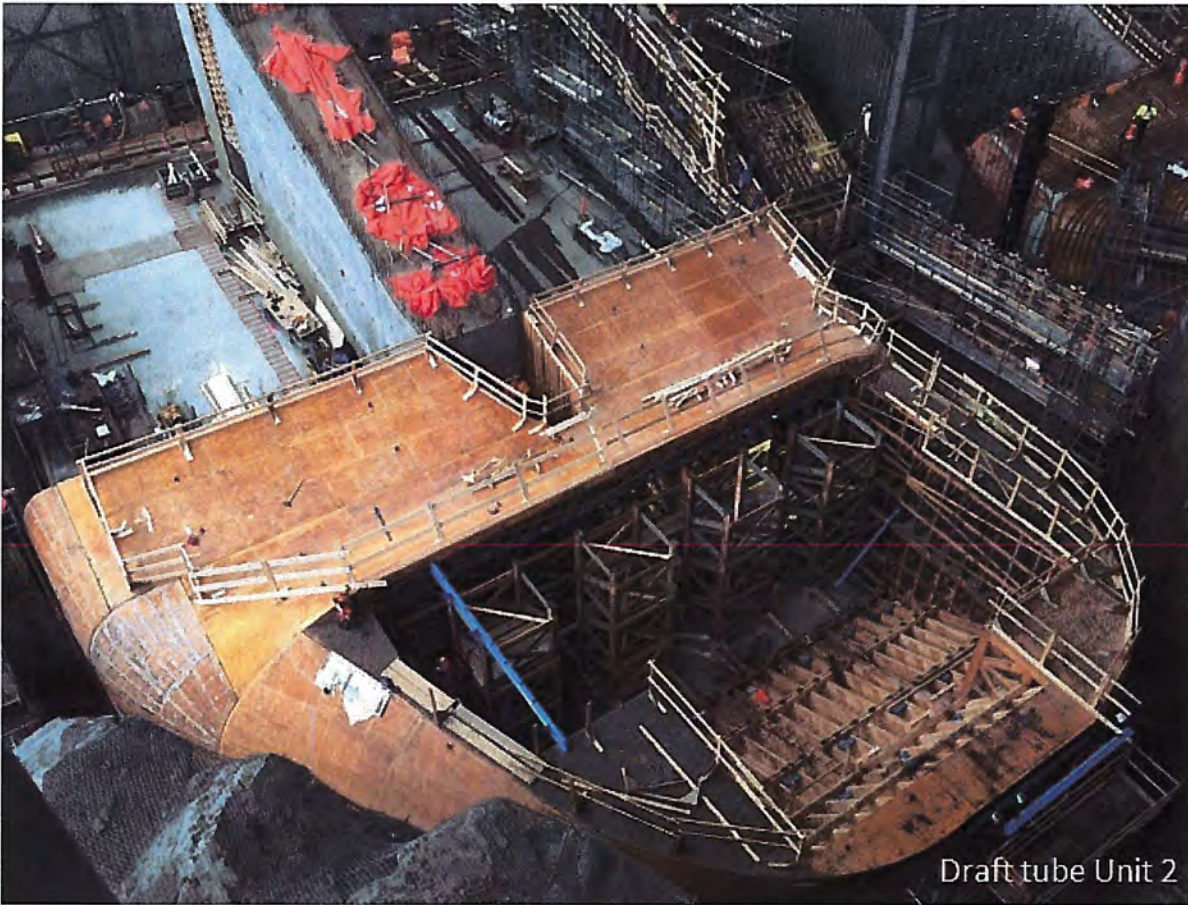
7 COMMENTARY ON PROJECT TIMELINE



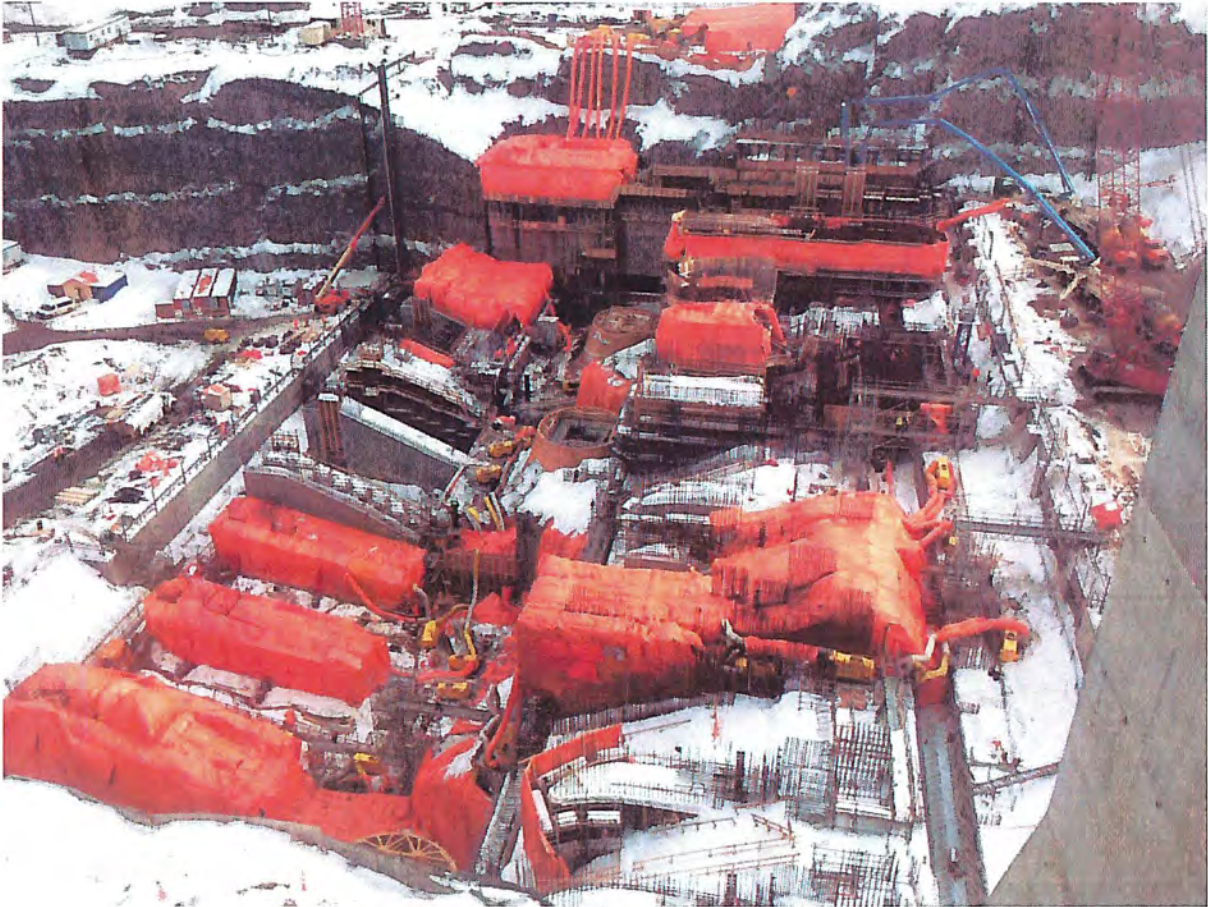
CEI Shop - Nalcor Visit
Picture from Nalcor
Taken November 18, 2014



Draft Tube Unit 1 and 2
Picture from Nalcor
Taken September 19, 2015



Draft Tube Unit 2
Picture from Nalcor
Taken October 10, 2015



Powerhouse View from North Transition Dam

Picture from Nalcor

Taken March 16, 2016