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DOCUMENT REVIEW
Comment Sheet

Completed by LCP Representative				Completed by LCPDCC
Document Title:				Record Number:
Final Engineering Assessment Report of Draft Tube #2 Formwork/Falsework Failure				
NE-LCP Document Number:	Revision:	3RD Party Document Number:	Revision:	Transmittal Number:
MFA-AT-SD-331A-EN-A99-0031-01	B1	A-DT000-NA-CV-D31-031-01	A	
LCP Department of Origin:		Purchase Order/Contract Number:		Transmittal Date:
Distribute Comment Sheet to:		Date returned to LCPDCC		

Comments:

LCP Representative:	Lead Reviewer:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
Company Comments				
1		Company is returning Code 3 – Not Accept. Please reference comments provided by Company, Company's 3rd Party Engineer (aDB Structural Engineering Inc.) and Company's Engineer (SLI). Company had no contractual or legal obligation to review the formwork design (as per Articles 3.1(e), 3.12, 3.13 and 11.8). The commentary in this report about Company's Engineer's (SLI) review of the CEI Design Calculation Report is inaccurate and therefore misleading.. Additionally, as per the aDB comments, this report contains errors, and contradictory statements that impact the report's conclusions.		
2		Contractor's 3rd Party Engineer ILF should be aware that, as		

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DOCUMENT REVIEW
Comment Sheet (Cont'd)

Comments:

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		per the Contract, Contractor is solely responsible for the Draft Tube Formwork failure, see Article references above. Company strongly objects to all statements that imply Company reviewed the design or inspected the formwork installation as these statements are false. Company did not review the design calculations, drawings or inspect any portion of the installed work. Company advised Contractor, as per LTR-CH0007001-0115, that it would only be receiving temporary structure drawings "For Information" and that its review would be limited to verification that engineering seals have been applied. Company again advised Contractor, as per LTR-CH0007001-0485 that all formwork drawings will be "For Information". As such, Company had no intent and did not review any portion of the technical contents. As per the Concrete Formwork Specification MFA-SN-CD-3300-CV-TS-0001-05 and Article 3.13 Contractor was required to provide formwork drawings and design calculations authenticated with the signature and seal of a registered NL Professional Engineer (P. Eng). Contractor did comply with this requirement as a P. Eng practicing under Contractor's employment authenticated these documents. As per the Engineers and Geoscience Act 2008 Section 15, and PEG-NL By-law No. 1 Section 9.1, this authentication is assurance that Contractor had thoroughly reviewed and taken professional responsibility for their contents including anything designed under its Subcontractor.		

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		As noted by Company's Engineer (SLI) Ref Comment #6 below the general statement affixed to the "For Information" codification of the design calculations for the draft tube formwork has been misrepresented in this report. The Engineer's review was limited to verifying that the correct design loads had been used. The ILF report inaccurately assumes that the review also included a review of the applicability of the design codes. As per Clause 1.4.1 of the Formwork Specification, Contractor was only permitted to use listed codes unless written approval is obtained from Company to deviate. As Company has no record of such concession, Contractor was required and assumed to prepare Contractor's design using listed codes. As noted by aDB in their comments, there are errors throughout the calculations that ILF uses to support its conclusions in Section 2 Summary related to the design codes. There are inaccurate statements that suggest the allowable capacity of 78kips was "manually input" and therefore the documentation supporting the development of the capacity was not provided in the CEI design calculation report. In addition the report contains contradictory statements, notably section 4.3 that the CSA and NDS codes are comparable within 5%, when compared to the overestimation factors provided in item 2. These issues need to be addressed by the author. Based on aDB's report (specifically section 8.3) the key design omission was the 0.6 factor that was required to be applied		

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		due to the built-up characteristics of the shoring legs. This was a fundamental error, resulting in the shoring being under designed, and it was an error that should have been identified in the contractually required drawing and calculation design review by Contractor's authenticating engineer.		
3		It should be noted that as per the formwork checklist used, Company representative provided a signature at the end of the document. This signature is strictly Company quality assurance verification that Contractor has completed the form and signed for each inspection item indicated. The Company representative was not responsible for or performed any inspections.		
4		Nalcor is not party to the spillway and powerhouse contract CH0007; the contract is between Muskrat Falls Corporation and Astaldi. All references to "Nalcor" should be replaced by "Company" with a note that "Company" is Muskrat Falls Corporation.		
Company's 3rd Party Engineer (aDB Structural Engineering Inc.) Comments				
		Please see attached copy of mark-up comments provided by aDB. In addition a copy of aDB's Final Report is attached for Reference.		
Company's Engineer (SLI) Comments				
2	Page 1/ Section 1.1 Description	**Page 1 / Section 1.1 Description** **Recommend adding the following two paragraphs between the 4th and 5th paragraph of section 1.1 Description**		

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Comment Sheet (Cont'd)

Comments:

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		CEI Calculation package was prepared by an engineer employee of CEI but signed and sealed by an employee of Astaldi, a Professional Engineer registered in Newfoundland and Labrador. It is understood by Company that Astaldi's professional engineer endorsed the design of CEI by: 1) verifying the design criteria, 2) confirming the applicable codes, 3) doing his own check for the formwork/falsework analysis and design in accordance with applicable Canadian design standards listed in the technical specifications, and 4) supervising the preparation of the drawings. The erection and layout drawings were also stamped by Astaldi's professional engineer while the fabrication drawings were not. Both sets of drawings and the calculation package had the stamp of an engineer registered in Kansas and not in Newfoundland and Labrador. Company instructed Astaldi that the formwork shop drawings would not be reviewed by Company. It further clarified that Contractor had full responsibility for ensuring shop drawing quality and conformance with the project requirements. It is understood that "shop drawings", as used herein encompasses also erection drawings and the engineering design supporting the drawings.		
3	Page 1/ Section 1.2- Basis of Review	**Page 2 / Section 1.2 Basis of Review** **Recommend adding the following documents to references in section 1.2**		

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Comments:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
		1. Technical specification : MFA-SN-CD-3300-CV-TS-0001-05 – Concrete Formwork - Section 03 11 00 2. LTR-CH0007001-0485 – Review Philosophy for Powerhouse Shop drawing Sub-Packages - Letter dated on 7 April-2015		
4	Page 3 / 2-Summary First paragraph	**Page 3 / 2 – Summary** First paragraph *"CEI did not perform a sufficiently rigorous analysis of the complex system of draft tube formwork and falsework and their design assumptions contained critical flaws."* **Comment 1: Recommend adding the following clarifications at the end of the first paragraph** All formwork drawings and associated design calculations have been sealed by a qualified Professional Engineer registered in the Province of Newfoundland and Labrador as per contract requirement – Clause 3.1.1 of Technical specification Section 03 11 00 – Concrete Formwork. Note: *The Report should present the role of this engineer in the design and the preparation of the design documents.* **Comment 2: Recommend stating in the report after the end of first paragraph:** Where the failure initiated, how it occurred and why (i.e.		

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Comments:

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		overstress, workmanship). If not possible, the report should state so in an explicit manner and then mention the author's opinion on the failure cause and location.		
5	Page 3/ 2 – Summary Third paragraph	**Page 3/ 2 – Summary** **Third paragraph** ""*.... Nalcor reviewed design criteria within the CEI calculation package and returned the reviewed document identification code 4 "Information only" noting the scope of review was limited to design criteria used for structural calculations."* **Comment 1: The mention that ~~Nalcor~~ Company reviewed the design criteria is wrong, Reference Comment #6. Statement should be factual as noted below:** Company received the document "Draft Tube Formwork Calculation Report" and returned it with the document identification code 4 "information only" with the following general comment: "The review of document was limited to design criteria used for the structural analysis. It does not include the verification of the calculations or the structural models used or any other portion of the document" **Comment 2: Company provides the following comments/Clarifications** 1. The review process of concrete formwork and related design documents is <u>not under</u> Company/SLI scope as per contract requirement and as per letter LTR-CH0007001-0485 sent by Company to Astaldi and		

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Comments:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
		dated on 7-April-2015. 2. As required by the clause 1.4.1 of the Technical Specification Section 03 11 00 - Concrete Formwork *"The Contractor shall comply with the rules and provisions of the listed Codes and Standards. The Contractor shall obtain written approval from the Engineer prior to using other equivalent codes and standards".* No change request was ever submitted for approval to Company in this regard. 3. Approval of the use of NDS (2005) or other equivalent codes and standards, was never given from the Engineer/Company. 4. The code edition for the project is specified in the specification as established upon award of the contract. If a new code edition or other equivalent code and standard is issued during the execution of the contract and it impacts the design, it must go through change management and a concession request or design change request must be raised and shall be submitted by Contractor to Company for approval. 5. CEI Calculation package was signed and sealed by a Professional Engineer registered in Newfoundland and Labrador (from Astaldi). It is understood by Company that Astaldi's Professional Engineer endorsed the design of CEI by doing his own check for the formwork/falsework analysis and design in accordance with applicable Canadian design codes listed in the technical specified.		
6	Page 3/ 2 – Summary Third paragraph	**Page 3 / 2 – Summary** Third paragraph ""*.... Nalcor reviewed design criteria within the*		

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Comments:

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		CEI calculation package" **Comment :** Company's general comment "review of design criteria used for the structural analysis" was intended to refer to design loads only as shown in page 78 within CEI calculations.		
7	Page 3/ 2– Summary Fifth paragraph	**Fifth paragraph** : "1. *Design Criteria: The design codes utilized by CEI were out of date at the time of design (2014) and were not in compliance with the Newfoundland and Labrador legislation requiring local, Canadian codes"* **Comment 1 :** Change "Design criteria" to "Design Codes" **Comment 2: Recommend adding the following sentence (clarification) after Item 1 above** Design codes and edition to be used for the design are specified in Clause 1.4.3 of Technical Specification Section 03 11 00 – Concrete Formwork. Contractor/CEI is to comply with the rules and provisions of the listed Codes and Standards listed in the Technical Specification. Prior to use other equivalent codes and standards, Contractor/CEI shall obtain written approval from the Company. **See also comment 4 under Item No 4**.		

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DOCUMENT REVIEW
Comment Sheet (Cont'd)

Comments:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
8	Page 3/ 2– Summary Fifth paragraph	**Fifth paragraph :** *"2. Faulty Design: CEI omitted critical stress modification factors in falsework tower calculations, resulting in an over estimation of capacity. The overestimation was 1.32 times higher than the allowable capacity determined by ILF using NDS 2015 and 1.86 times higher using CSA O86-9, respectively."* **Comment: Recommend to add the following sentence (clarification) after Item 2 above** Approval of the use of NDS (2005) or other equivalent codes and standards, was never requested by Contractor nor given by the Engineer or Company as required by the clause 1.4.1 of the Technical Specification Section 03 11 00 – Concrete Formwork. As stated above, if a new code edition is issued during the execution of the contract and it impacts the design, a concession request or design change request must be submitted by Contractor to Company for approval. **See also comment 4 under Item 5**		
9	Page 4/ 2– Summary	**Page 4** "5. Erection Deficiencies :" **Comment: Recommend adding the following sentence to Item 5.b above:**		

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DOCUMENT REVIEW
Comment Sheet (Cont'd)

Comments:

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		There is no written evidence that the inspection of falsework/formwork structure, including any reshoring, was carried out by the Astaldi Engineer in accordance with the requirements of the Code CSA/ S269.1 Clause 8.6.1.3 to 8.6.4.		
10	Page 23 and 24/ Section 3.3.3 Quality of Erection of Shoring Towers and Formwork	3.3.3 Quality of Erection of Shoring Towers and Formwork **Recommend adding the following sentence (clarification) after second bullet of section 3.3.3:** There is no written evidence that the inspection falsework/formwork structure, including any reshoring, was carried out by the Astaldi Engineer in accordance with the requirements of the Code CSA/ S269.1 Clause 8.6.1.3 to 8.6.4.		
11	Page 27/ Section 4.1	**Comment:** **Suggest replacing "4.1 Proper Design Criteria" by "4.1 Proper Design codes and regulations"**		
12	Page 27/ Section 4.1	**Comment: Recommend adding the following paragraphs at the end of section 4.1** In addition to above, the Agreement (contract) requires the following: **1.** Design codes and edition to be used for the design of concrete formwork are specified in Clause 1.4.3 of Technical specification Section 03 11 00 - Concrete Formwork. 2. Contractor/CEI is to comply with the rules and provisions of the listed Codes and Standards listed in the technical specification. Prior to use other		

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DOCUMENT REVIEW
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Comments:

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		equivalent codes and standards, Contractor/CEI shall obtain written approval from Company's Engineer, as per Clause 1.4.1 of the technical specification. If a new code edition is issued during the execution of the contract and it impacts the design, any change must go through a change management process, by making a concession request or design change request which must be submitted by Contractor to Company for approval.		
13	Page 27/ Section 4.2	**Comment 1:** Replace "4.2 CEI Design Criteria" by "4.2 CEI Design codes and references" **Comment 2: Recommend adding the following paragraph at end section 4.2** Approval of the use of NDS (2005) or other equivalent codes and standards, was never given by Company's Engineer as required by the clause 1.4.1 of the technical specification Section 03 11 00 - Concrete Formwork.		
14	Page 27/ Section 4.3	**Comment:** Suggest replacing "4.3 ILF Design Criteria" by "4.3 ILF Design Methodology"		
15	Page 27 / Section 4.3 First paragraph	**Section 4.3 First paragraph** *"Both the 2015 NDS design code and CSA-O86-09 Engineering Design of Wood were utilized by ILF to check the existing formwork system. It was determined that the difference between the two codes, when properly utilized within this context, produced results within approximately 5 percent of*		

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Comments:

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		each other." **Comment : Recommend adding this paragraph in Section 2-Summary**		
16	Page 28 / Section 4.4	**Comment 1:** Suggest replacing "4.4 Discussion of Design Criteria" by "4.4 Discussion of applicable design codes and standards " **Comment 2:** Suggest replacing "Codes" by "Standards" in Table 1 and also in the text.		
17	Page 28 / Section 4.4	**Section 4.4 Bottom of page 28** *"Regardless of explicit NDS code requirements for joining built up members, it is expected that sound engineering judgment would have concluded that the nailing of 2x10 plys employed by CEI's fabrication shop was not consistent with assumptions made by CEI in their calculation package"* **Comment : Recommend adding this paragraph in Section 2-Summary at end of sub-section 4. Faulty Fabrication**		
18	Pages 28 and 29 / Section 4.4	**Section 4.4 last paragraph at bottom page 28 and top page 29** *"..... Astaldi then submitted the CEI calculation package and associated drawings to Nalcor who reviewed and returned the documents code "4- Information only" on February 25, 2015 stating within the general comments "This document is being*		

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Comments:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
		returned for information only. The review of the document was limited to the design criteria used for the structural analysis. It does not include the verification of the calculations or the structural models used or any other portion of the document." It appears that both Astaldi's and Nalcor's review were flawed and did not identify the non-conforming design codes employed by CEI" **Comment: Nalcor did not perform any review, it is therefore false to say that Nalcor's review was flawed. Please replace the paragraph above by the following** Company received the document "Draft Tube Formwork Calculation Report" and returned the document identification code 4 "information only" with the following general comment: "The review of document was limited to design criteria used for the structural analysis. It does not include the verification of the calculations or the structural models used or any other portion of the document" **See also comments in Items 2, 3, 4, 5 and 6.**		
19	Page 31/ Section 4.5.1.1	**4.5.1.1 CEI's Formwork Analysis and Design** **last paragraph at bottom page 31** **Comment:** Replace the "engineer of record of the permanent structure		

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DOCUMENT REVIEW
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Comments:

Item No.	Section/Paragraph /Page/Sheet	Comment	Response	Status
		(SNC for Muskrat Falls powerhouse" **By "Designer of the permanent structure (SLI for Muskrat Falls powerhouse)".** **See also comments in Items 2, 3, 4, 5 and 6.**		
20	Page 32/ Section 4.5.1.1 **First paragraph**	**First paragraph on top Page 32** **Comment:** Replace "Nalcor provided direction for reshoring loading..." **By the following** "Company provided suggestion for reshoring loading ... " **The mention that the loading was imposed by Nalcor is false and must be corrected along the following remarks at the end of this paragraph:** The final design load is to be established by ASTALDI/CEI/ILF. To deal with the reshoring, Company answered also the Contractor SQY-CH-0007001-0568 to confirm that the concrete level 3 (*D2ESB-03/ D2ENB-03)* can support its self-weight when the concrete compression strength attains 21 MPa. This site query SQY-CH-0007001-0568 to be included in Appendix H.		
21	Page 39/ **First paragraph**	**4.2.3 Comparison of Falsework Design Load Cases** Comment1: error in section numbering		
22	Page 39/ **First paragraph**	**Top paragraph of Page 39** *"CEI did account for a re-shore load similar to that specified by*		

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Comments:

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		Nalcor after the collapse (Ref. Site Query: AT-SQY-CH0007001-0556 in Appendix H), but supporting documentation was not provided in their calculation package" **Comment:** Replace "Specified by Nalcor" by "suggested by Company" **The mention that the loading was specified by Nalcor is false and must be corrected along the following remarks at the end of this paragraph:** The final design load is to be established by ASTALDI/CEI/ILF. **See same comments in Items 20 above.**		
23	Page 39/ **Second paragraph**	**Page 39 Second paragraph:** *"Using loading requirements imposed by Nalcor after the failure of D2ESB-03, CEI would be required to design for full thickness of lift 3 concrete 10.2 feet (3.1m) and half the 7.2 feet (2.2m) thickness of lift 4 concrete."* **Comment:** Replace "imposed by Nalcor" by "suggested by Company" **The mention that the loading was imposed by Nalcor is false and must be corrected along the following remarks at the end of this paragraph:**		

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DOCUMENT REVIEW
Comment Sheet (Cont'd)

Comments:

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		The final design load is to be established by Astaldi/CEI/ILF. **See same comments in Items 20 and 22 above.**		
24	Page 67/ Section 5.5	Section 5.5 - Astaldi management of change Process for temporary Structures **Comment:** Replace "Engineer of record" by "Designer of temporary Structures (Formwork/Flasework)"		
25	Appendix B / Page 116	Appendix B – Formwork checklist D2ESB-03 **Comment:** SQY-CH0007001-0353 belongs to the intake and is not relevant of DT2, as specified in the site query response.		

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NE-LCP Lead Reviewer:____________________ Date: 1. May. 2017

For Contractor:____________________ Date: ____________________

subcontract, Contractor shall ensure that Subcontractors comply with such requirements. In addition, Contractor shall provide reports in the manner and format described in Exhibit 13 - Provincial Benefits of this Agreement throughout the Term.

ARTICLE 3
CONTRACTOR'S WORK OBLIGATIONS

3.1 Contractor shall carry out all of its obligations under this Agreement and shall perform the Work, including:

(a) all procurement, fabrication, construction, testing, transport, delivery, maintenance, storage, documentation, preservation, installation, testing, commissioning, repair and remediation of the Work;

(b) provision of all supervision, services, labour, trades, drafting, accounting, purchasing, expediting, inspection, testing, Personnel, Contractor's Items, transportation, mobilization and demobilization required for the compliance with and fulfillment of all Contractor's obligations under this Agreement;

(c) provision and installation of all equipment, products and materials required by this Agreement at a Site;

(d) ensuring the Work conforms strictly as to quality and description with the particulars stated in Exhibit 1 – Scope of Work and Company Supplied Data and complies with all Applicable Laws;

(e) any design or engineering which is the responsibility of Contractor under this Agreement;

(f) satisfaction of the performance requirements set out in Exhibit 1 – Scope of Work;

(g) provision of all documents as required under, and in accordance with, the terms of this Agreement;

(h) provision of any work not expressly detailed in this Agreement but which is necessary for the performance of the Work in accordance with this Agreement;

(i) rectification of any and all Defects in the Work as noted by Company, Engineer or any Authority; and

(j) completing the Work, and portions thereof, in accordance with the relevant Milestone Dates.

3.2 Contractor shall review and verify the details contained in Exhibit 1 - Scope of Work and Exhibit 11 - Company Supplied Documents, and represents that it has a full knowledge and understanding of the nature and the scope of the Work, and including weather and all other conditions at Worksites. Contractor shall :

TA JL

3.9 When work is performed by Company's Other Contractors at a Site at which Contractor is performing Work, Contractor shall:

(a) afford Company and Company's Other Contractors reasonable opportunity to introduce and store their products and use their construction machinery and equipment to execute their work;

(b) co-ordinate and schedule the Work with the work of Company's Other Contractors;

(c) participate with Company's Other Contractors and Engineer in reviewing their construction schedules when directed to do so;

(d) where part of the Work is affected by or depends upon for its proper execution the work of Company's Other Contractors, promptly report to Engineer in writing and prior to proceeding with that part of the Work, any apparent deficiencies in such work (failure by Contractor to so report will constitute a waiver of claims against Company by reason of the deficiencies in the work of Company's Other Contractors except for those deficiencies not then reasonably discoverable); and

(e) comply with the requirements of **Article 32**.

provided that if the acts of Company's Other Contractors are impeding the performance of the Work and as a result impacts the Contract Price, a Milestone Date or an Interface Date then Contractor may proceed in accordance with **Articles 14.7** or **14.8**.

3.10 At Company's option, Contractor shall transfer all unused excess materials, if any, to Company at the completion of the Work or sell such excess materials and any amounts realized from such sales shall be credited to Company as a deduction from the Contract Price.

3.11 Contractor shall direct and supervise the Work effectively to ensure conformity with the Agreement. Contractor will have sole responsibility for construction and installation means, methods, techniques, sequences and procedures and for coordinating the various parts of the Work under this Agreement.

3.12 Contractor will have the sole responsibility for the design, erection, operation, maintenance and removal of temporary supports, structures and facilities and the design and execution of construction methods required in their use.

3.13 Contractor will engage and pay for registered professional engineering personnel skilled in the appropriate disciplines to perform those functions referred to in **Articles 3.1(e)** and **3.12** where required by Applicable Laws or by the Agreement and in all cases where such temporary supports, structures and facilities and their method of construction are of such a nature that professional engineering skill is required to produce safe and satisfactory results.

3.14 Contractor Group will confine construction machinery and equipment, storage of products and operations of Contractor Group to limits indicated by Applicable Laws, permits or the Agreement and will not unreasonably encumber the Work with products, materials, or equipment.

TA JK

11.7 Engineer shall notify Contractor when the Site is available for permanent installation of any equipment, materials or products as part of the Work, and Contractor shall not commence any Work at the Site until such notification has been given.

11.8 Where the Agreement calls for the Acceptance by Engineer or Approval by Company with respect to design, manufacture, installation, testing and commissioning of the Work, any such Acceptance or Approval is for general compliance with the Technical Requirements and does not relieve Contractor from satisfying all Technical Requirements. No inspection, review or Acceptance by Engineer or Approval by Company shall release Contractor from compliance with Contractor's obligations under this Agreement or Applicable Law.

ARTICLE 12
COMPENSATION AND TERMS OF PAYMENT

12.1 As full compensation for the performance by Contractor of all its obligations under this Agreement, Company shall pay Contractor the Contract Price in accordance with the terms of this Agreement including **Article 12**, Exhibit 2 – Compensation and Exhibit 3 – Coordination Procedures. Only those rates and prices specifically identified in Exhibit 2 – Compensation shall be paid by Company and any costs not specifically identified in Exhibit 2 - Compensation shall be deemed to be included in such rates and prices. Company shall have no obligation to pay Contractor for the purchase of any goods or performance of any services which have not been Approved by Company prior to delivery of such goods or prior to performance of such services.

12.2 Within thirty (30) days of the Effective Date, Engineer, on behalf of Company, shall provide Contractor with a pro forma invoice that sets out all relevant Company cost codes and required information. Contractor shall utilize said pro forma invoice and cost codes when billing Company.

12.3 Compensation to Contractor shall be paid:

(a) an advance payment of ten percent (10%) of the Contract Price ,

(b) monthly based on progress, and/or

(c) upon achieving a Payment Milestone,

as further specified in Exhibit 2 – Compensation. Contractor shall be paid the portion of the Contract Price applicable to monthly progress or to a Payment Milestone following Approval by Company of a Payment Certificate and in accordance with the provisions of this **Article 12.** Any compensation payable to Contractor pursuant to the Cost Sharing provisions of Section 2.7 of Exhibit 2 – Compensation shall be determined as of Final Completion.

12.4 Contractor shall provide, maintain and issue to Engineer, a detailed listing of the invoiced amounts of the Work and cash flow requirements regarding unbilled portions of the Work in accordance with the requirements set out in Exhibit 3 – Coordination Procedures. Contractor shall develop and present a format for the listing for Company Approval.

TB JL

Muskrat Falls Corporation
Corporate Office
500 Columbus Drive
P. O. Box 15000, Stn. A
St. John's, NL Canada A1B 0M4

Lower Churchill Project Operations Office
350 Torbay Road, Suite 2
St. John's, NL Canada A1A 4E1

7 April-2015

ASTALDI Canada Inc.
358 Hamilton River Road
Happy Valley-Goose Bay, NL
A0P 1C0 Canada

Attention: Giacomo Orsatti, Project Manager

Subject: Agreement No.: CH0007- 001
Title: Construction of Intake and Powerhouse, Spillway and Transition DamsCompany
Re.: Review Philosophy for Powerhouse Shop Drawing Sub-Packages

References:
- LTR-346 Change to Deliverable Review Drawing Codes
- LTR-0332 Further Changes to Deliverable Review Codes
- LTR-0294 Change to Drawing Coding
- LTR-0115 Revision of Document and Drawing Submittals

Dear Sir

We are writing to advise that going forward it is Company intention, in collaboration with Contractor, to adopt a risk based review approach for the powerhouse shop drawings. Employing this approach, Company will avail of the similarity of detail between the four units of the powerhouse and scale back its oversight review to focus on areas of uniqueness only.

In order for this approach to work Contractor and its Subcontractors will need to focus the production of shop drawings such that the sub-packages for one unit are produced, reviewed and accepted by Company prior to starting the production of similar sub-packages of the other units. This is only possible if production of shop drawings can be advanced enough ahead of the work in all units.

With approximately 10,000 shop drawings required to be produced and processed in the next year, this review approach by Company coupled with Contractor' shop drawing sub-package method of production and control there is significant potential to reduce amount of effort and recycle between our engineering teams.

Company review philosophy is summarized as follows. Please note for the purposes of this letter Company assumed that Contractor will focus production of shop drawings on Unit 1. Contractor shall confirm:

- All formwork drawings to be submitted to Company "For Information".
- Company will review 100% the reinforcing and lift drawings for the powerhouse and Intake unit 1, north and south service bay; the stair shaft No. 7 adjacent to intake Unit 4 as well as the fire walls. Unit 2, 3 & 4 drawings to be submitted to Company "For Information" provided similar sub-packages for Unit 1 review is complete.

UH

- Current company review philosophy for miscellaneous steel and powerhouse superstructure steel is summarized in LTR-0449. This may be further refined in subsequent correspondence.

See attached spreadsheet for review criteria for each Contractor shop drawing sub-package. Each sub-package should be subdivided in several workflows to take in account the time frame for the review of the shop drawings by Company.

For Units 2, 3 & 4 Company will review these drawings on an audit basis (approximately 25% of documents). Contractor will be notified when an audit has been completed via an audit form and any drawings which generated commentary will be returned as per normal procedure.
If commentary warrants it, Company will increase its level of review.

This review philosophy is consistent with Company's role in an oversight capacity. Contractor is reminded that notwithstanding the application of the above review process, Contractor is still fully responsible for ensuring shop drawing quality and conformance with the project requirements and specifications. Company has noted with the review completed so far that the quality and conformity of the shop drawings is reasonable. There is however room for improvement and we believe this can be achieved through an enhanced focus on the quality check performed by Contractor of its subcontractors work. Company will pay particular focus to this in the coming weeks through the on-going auditing process.

Company wish to recognize the excellent ongoing collaboration effort between Company/Contractor engineering teams, we believe the approach outlined will further streamline our mutual operations.

Regards,

Muskrat Falls Corporation

Scott O'Brien
Company Representative
Muskrat Falls Generation

Attachment

Muskrat Falls Corporation

350 Torbay Road Suite 2
St. John's, NL Canada A1A 4E1
t. 709.737.1440 or 709.752.3460

LTR-CH0007001-0115

4-April-2014

ASTALDI Canada Inc.
358 Hamilton River Road
Happy Valley-Goose Bay, NL
A0P 1C0
Canada

Attention: Guido Venturini, Project Director
Vittorio Robiatti, Construction Manager
Jack Zhou, Deputy Project Manager

Subject: Agreement No.: CH0007-001
Title: Construction of Intake and Powerhouse, Spillway and Transition Dams
Re: Revision of Document and Drawing Submittals

Dear Sirs,

Company wishes to advise Contractor that with respect to the following clause from Supplier/Contractor Document Requirements LCP-PT-MD-0000-IM-PR-0015-01 rev C1, first paragraph of Section 6.2.1 - Revision Status which states:

"All documents and drawings require a revision status upon submission. The first submission will be submitted as revision status of A1 - Issued for Review, **unless otherwise agreed to by the LCP Responsible Lead/Package Engineer**. All A1 documents are to be resubmitted at revision B or higher and achieve a Review Code 01 to be considered ready for use."

Company advises Contractor, that the agreed process for first revision submission of documents and engineering, fabrication, installation and shop drawings shall be as per the following guidelines:

1) On all document and drawing submittals, Company expects to see proof that Contractor has completed its own internal review process and internally accepted the document prior to 1st submission to Company. Contractor shall use it's own revision classification for their drawings and documents. It is expected that the Contractor will implement the same document control protocols with its Sub-contractor group.

2) For all documents, a Contractor cover sheet should be placed underneath the LCP cover sheet. The Contractor cover sheet should as a minimum contain Contractor revision history and approval sign-offs. First submissions to LCP shall be revision A1 (Issued for Company Review).

Muskrat Falls Corporation

350 Torbay Road Suite 2
St. John's, NL Canada A1A 4E1
t. 709.737.1440 or 709.752.3460

3) Drawings for permanent structure that **do not have a design element** or require a professional engineer to stamp shall be submitted at Rev C1. Contractor is still required to receive drawings back from LCP at Review Code 1 to be considered ready for use.

4) Drawings for permanent structure that **have a design element** or require a professional engineer to stamp shall be submitted at Rev A1 (Issued for Company Review).

5) All drawings for temporary structure **with or without a design element** shall be submitted at Rev C1. Company will be reviewing these drawings for information only and will limit its review to verification that engineering seals have been applied. Contractor is still required to receive these drawings back at either Code 1 or Code 4 prior to starting work.

   Note: Any temporary structures that interface with permanent structure i.e. the powerhouse ICS Structure will be reviewed as per a permanent structure.

Sincerely,
Muskrat Falls Corporation

Desmond Tranquilla
Deputy Company Representative and Site Manager

cc: S. O'Brien, R. Woolgar, T. Vanwyk, B. Knox, P. Oblander, T. Hassanein, N. Ferguson, M. Collins

Document Front Sheet

nalcor energy
LOWER CHURCHILL PROJECT

NE-LCP Contractor/Supplier

Contract or Purchase Number and Description: CH007-001-Construction of Powerhouse, Spillway and Transition Dams	Contractor/Supplier Name: Astaldi Canada Inc.
Document Title: Final Engineering Assessment Report of Draft Tube #2 Formwork/Falsework Failure	Total Number of Pages Incl. Front Sheet 831
Contractor Document Number: A-DT000-NA-CV-D31-031-01	Revision Number: A
Supplier Document Number:	Revision Number:

NE-LCP Document Number: MFA-AT-SD-331A-EN-A99-0031-01	NE-LCP Issue Number: B1

Approver's Signature: Robert Gregoire	Date (dd-mmm-yyyy): 25-Feb-2017	Review Class:
Comments:		Equipment Tag or Model Number:

NE-LCP

REVIEW DOES NOT CONSTITUTE APPROVAL OF DESIGN DETAILS, CALCULATIONS, TEST METHODS OR MATERIAL DEVELOPED AND/OR SELECTED BY THE CONTRACTOR, NOR DOES IT RELIEVE THE CONTRACTOR FROM FULL COMPLIANCE WITH CONTRACTUAL OR OTHER OBLIGATIONS.

- ☐ **01 – REVIEWED AND ACCEPTED – NO COMMENTS**
- ☑ **02 – REVIEWED – INCORPORATE COMMENTS, REVISE AND RESUBMIT**
- ☐ **03 – REVIEWED - NOT ACCEPTED**
- ☐ **04 – INFORMATION ONLY**
- ☐ **05 – NOT REVIEWED**

Lead Reviewer:	Date (dd-mmm-yyyy):	Project Manager:	Date (dd-mmm-yyyy):
NE-LCP Management:	Date (dd-mmm-yyyy):		

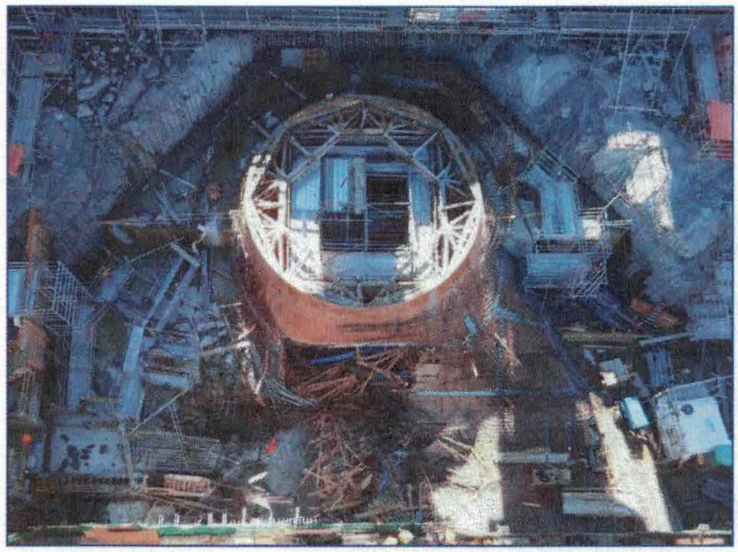
General Comments:

aDB comments are identified in the following pages with blue text in red boxes.
Items identified as unclear should be clarified for better understand the root cause of the draft tube formwork collapse.

ASTALDI CANADA Inc.

Astaldi Canada Inc.

114 Hamilton River Road, HV-GB
NL, Canada A0p 1C0
P.O. Box 177 Station C



PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PERMIT HOLDER
This Permit Allows
ILF CONSULTANTS INC.
To practice Professional Engineering
In Newfoundland and Labrador.
Permit No. as issued by PEGNL 796
which is valid for the year 2017

REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
TIMOTHY C. WELLERT
SIGNATURE
FEB 24, 2017
DATE
NEWFOUNDLAND & LABRADOR

Lower Churchill Project

Final Engineering Assessment Report of Draft Tube 2 Formwork/Falsework Failure

February 24, 2017

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ILF CONSULTING ENGINEERS

REVISION

0	02/24/2017	Initial Submission	TCW TCW	JAM JAM	TCW TCW
Rev.	Date	Issue, Modification	Prepared	Checked	Approved

Contents

REFERENCE DOCUMENTS

AISC, Manual of Steel Construction, Allowable Stress Design – 9th Edition
APA, The Engineered Wood Association, Panel Design Specification – 2004 Edition
ASTM D143-14 Standard Test Methods for Small Clear Specimens of Timber
CSA-O86-09 Engineering Design in Wood
CSA-S16 Design of Steel Structures
CSA-S269.1-1975 Falsework for Construction Purposes
MFA-AT-SD-331A-EN-A99-0001-01 General Report - Engineering & Inspection Plan
MFA-AT-SD-331A-EN-A99-0002-01 Draft Tube Unit 2 Outlet - Civil - General Report - Man Basket Inspection
MFA-AT-SD-331A-EN-A99-0004-01 Draft Tube Unit 1 Elbow - Civil - Phase 2a Report - Level 4 & 5 Formwork Design Check And Inspection
MFA-AT-SD-331A-EN-A99-0009-01 Draft Tube - Unit 3 Outlet - Civil - Phase 1 Report - Inspection Of Draft Tube 3 Outlet Shoring
MFA-AT-SD-331A-EN-A99-0011-01 Draft Tube Unit 3 Elbow - Civil - Site Inspection Of Draft Tube 3 Formwork
MFA-AT-SD-331A-EN-A99-0012-01 DT2 - Civil - Site Inspection Of Draft Tube 2 Replacement Panels A11, A12, A16, A24, A25, A26, A27, A29 and A30
MFA-AT-SD-331A-EN-A99-0015-01 Ground Level Inspection of DT2
MFA-AT-SD-331A–EN-A99-0016-01 Draft Tube 3 – Structural Verification of CEI Formwork – Levels 2 and 3
MFA-AT-SD-331A-EN-A99-0018-01 Draft Tube Unit 4 – Civil Phase 4a – General Report – Site Inspection
MFA-AT-SD-331A–EN-A99-0019-01 Draft Tube 3 Elbow Level 3 – Formwork Panels – A13 to A15 & A24 to A30 – Engineering Review
MFA-AT-SD-331A–EN-A99-0020-01 Draft Tube 2, 3 & 4 Elbow Level 4 and 5 – Formwork Panels – A37 & A40 – Engineering Review
MFA-AT-SD-3310–CS-D04-0001-01 Draft Tube Elbow Wood Formworks – General Notes
MFA-AT-SD-3310–CS-D04-0066-01thru -06 CEI details
M.K. Hurd, Formwork for Concrete - 6th Edition
National Building Code of Canada - 2010 edition
NDS National Design Specification for Wood Construction, Allowable Stress Design – 2005 and 2012 Editions
Newfoundland and Labrador Regulation 45/12
MFA-AT-SD-331A-EN-0016-01 DT3 Elbow- Phase 2B Report- Structural Verification of CEI Formwork –Levels 2 and 3
MFA-AT-SD-331A-EN-A99-0020-01 Draft Tube 2, 3, and 4 Elbow –Level 4 and 5 Formwork Panels A37 to A40 –Engineering Review
ACI 347.2 Guide for Shoring/Reshoring of Concrete Multistory Buildings

FIGURES

TABLES

APPENDICES

1 PURPOSE AND SCOPE OF THE REPORT

This document examines the Draft Tube 2 formwork/falsework failure that occurred May 29, 2016 on the Lower Churchill Project. ILF Consultants Inc. (ILF) has performed field inspections, reviewed design and fabrication documents, care and preservation practices, and erection and inspection documentation in an effort to identify the cause of failure.

1.1 Description

The Lower Churchill Project is an 824 megawatt hydroelectric project consisting 4 turbines in a powerhouse. The project is owned and managed by Nalcor Energy (Nalcor). It is centered about Muskrat Falls on the Lower Churchill River, near Happy Valley – Goose Bay, Newfoundland and Labrador, Canada.

On May 29, 2016, the timber formwork and falsework supporting concrete pour D2ESB-03 in Draft Tube 2 (DT2) failed nearing completion of the 530 cubic meter pour. On May 30, 2016, Newfoundland and Labrador Occupational Health and Safety (OHS) issued Stop Work Order #0671924-01. This effectively stopped all concrete work in the powerhouse until the falsework and formwork systems were evaluated by a Structural Professional Engineer (P.Eng.) licensed to practice in the Province.

Astaldi Canada, Inc. (Astaldi), the prime contractor for power house works, retained the services of ILF on June 1, 2016 to perform structural engineering services including investigation of the failure, analysis of formwork and falsework design, fabrication, and erection, and development of a summary report of contributing factors that lead to the failure of DT2 formwork and falsework.

The formwork and falsework analyzed in this report was designed and fabricated by Contractor's Engineer Inc. (CEI) in Kansas, USA, and was installed at the project site by Astaldi. CEI presents themselves as experts in the industry and have nearly 50 years' experience in design and fabrication of unique formwork systems.

Subsequent to initial engineering analysis and findings, ILF performed additional design work to strengthen and modify formwork and falsework to meet local design codes. ILF also performed pre-pour inspections of the modified structures prior to releasing for concrete placement.

1.2 Basis of Review

ILF's engineering analysis included a review of the following information:

1. CEI Calculation Report dated January 24, 2015. The document presents CEI Design Calculations for formwork and falsework and contains Astaldi (Newfoundland and Labrador) and CEI (Kansas) engineering seals. This document was reviewed by Nalcor dated February 25, 2015. Reference Appendix A.

2. CEI Erection and Layout drawings with title Muskrat Falls Draft Tube Elbow Wood Formworks XXXX, dated October 31, 2014. These documents provide layout and erection drawings for the formwork and falsework designed and fabricated by CEI and contain Astaldi (Newfoundland and Labrador) and CEI (Kansas) engineering seals. Reference documents Appendix J.

3. CEI Shop drawings with title Muskrat Falls Draft Tube Elbow XXXX dated September 12, 2014. These documents provide detailed member sizes for fabrication of formwork and falsework and contain CEI's engineering seal (Kansas). Reference documents Appendix K.

4. Inspections performed by ILF June 3, 2016 through September 16, 2017. Reference documents:

    - MFA-AT-SD-331A-EN-A99-0002-01 Draft Tube Unit 2 Outlet - Civil - General Report - Man Basket Inspection,
    - MFA-AT-SD-331A-EN-A99-0004-01 Draft Tube Unit 1 Elbow - Civil - Phase 2a Report - Level 4 & 5 Formwork Design Check And Inspection,
    - MFA-AT-SD-331A-EN-A99-0009-01 Draft Tube - Unit 3 Outlet - Civil - Phase 1 Report - Inspection Of Draft Tube 3 Outlet Shoring,
    - MFA-AT-SD-331A-EN-A99-0011-01 Draft Tube Unit 3 Elbow - Civil - Site Inspection Of Draft Tube 3 Formwork,
    - MFA-AT-SD-331A-EN-A99-0012-01 DT2 - Civil - Site Inspection Of Draft Tube 2 Replacement Panels A11, A12, A16, A24, A25, A26, A27, A29 and A30,
    - MFA-AT-SD-331A-EN-A99-0015-01 Ground Level Inspection of DT2,
    - MFA-AT-SD-331A–EN-A99-0016-01 Draft Tube 3 – Structural Verification of CEI Formwork – Levels 2 and 3,
    - MFA-AT-SD-331A-EN-A99-0018-01 Draft Tube Unit 4 – Civil Phase 4a – General Report – Site Inspection

5. Shipping, receiving, and care and preservation documents from Astaldi. Reference Appendix D, Appendix E, and Appendix F.

Was the wood of poor quality when the structures were fabricated?

Are you referring the DT1 tower buckling?

2 SUMMARY

CEI did not perform a sufficiently rigorous analysis of the complex system of draft tube formwork and falsework and their design assumptions contained critical flaws. These factors lead to failure. Additionally, CEI did not use design codes recognized by the National Building Code of Canada (NBCC), improper fabrication practices, and poor wood quality negatively impacted the calculated capacity of the falsework towers.

Errors in CEI's calculations and fabrication contributed to failure of the engineered system. For the purposes of this report, failure is defined as excessive deflection or rupture of overstressed members, formwork is a temporary structure supporting a lateral concrete load until the concrete is self-supporting, and falsework is a temporary structure supporting a vertical concrete load until the concrete is self-supporting.

CEI's falsework tower design was flawed. CEI erroneously calculated their falsework towers having an allowable capacity of 78.3 kips (346 kN), which is 1.86 (78.3 ÷ 42 = 1.86) times the value determined by ILF using CSA O86-9 methods. Improper fabrication details exacerbated flaws in design and resulted in further overestimation of tower capacity. CEI sealed the design documents with their Kanas professional engineering stamp. The errors in the calculations and fabrication were passed to Astaldi, who sealed the documents with their Newfoundland and Labrador permit to practice and professional engineer stamp prior to putting the formwork and falsework in service. Nalcor reviewed design criteria within the CEI calculation package and returned the reviewed document identification code 4 "information only" noting the scope of review was limited to design criteria used for structural calculations. Care and preservation practices prescribed by CEI were not adhered to by CEI or Astaldi creating the potential for non-conforming wood strengths.

The following major deficiencies were noted by ILF during review of the failed formwork and falsework in DT2.

1. Design Criteria: The design codes utilized by CEI were out of date at the time of design (2014) and were not in compliance with the Newfoundland and Labrador legislation requiring local, Canadian codes.
2. Faulty Design: CEI omitted critical stress modification factors in falsework tower calculations, resulting in an over estimation of capacity. The overestimation was 1.32 times higher than the allowable capacity determined by ILF using NDS 2015 and 1.86 times higher using CSA O86-9, respectively.
3. Rigor of Design: CEI employed over-simplified design assumptions resulting in numerous overstressed members. CEI utilized incorrect load assumptions that did not account for maximum applied loads.
4. Faulty Fabrication:
    a. CEI did not specify a bearing plate at top of falsework towers. This results in a local overstress of tower leg members that was not accounted for in CEI's analysis.
    b. Fabrication errors resulted in local overstressing of falsework towers.
        i. The built-up falsework tower legs were not joined per NDS or CSA codes and the actual allowable capacity cannot be determined using NDS or CSA guidelines.
        ii. Numerous instances of saw kerfs and gaps in members of falsework tower legs were observed. Both of these defects reduce allowable capacity of structural members.
        iii. Butt joints were identified as being closely spaced within built-up falsework tower legs. The close proximity of butt joints significantly reduces flexural capacity of the tower legs.

The reduction factor forgotten by CEI is Kf = 0.6 for nailed built-up column
Kf=0.6 1 / 0.6 = 1.6
Therefore the overestimation should be 1.6

Is there other factors that were not taken into account by CEI ? Please clarify.

   iv. Uneven bearing surfaces at top of falsework tower legs were identified, resulting in high stresses in taller members due to unequal load sharing.
   v. Missing members were identified in erected falsework modules, increasing unbraced lengths of some members.

5. Erection Deficiencies:
   a. Improper shimming at top of the tower legs was observed. Had a bearing plate been provided, the impact of poor shimming would have been limited to additional deflection only (in the order of mm), as shims are not considered structural members. Poor shimming coupled with lack of bearing plate and uneven bearing surface at top of tower legs, results in uneven distribution of loads into the falsework tower leg members and local overstress in leg members.
   b. Field changes were made to CEI specified column base grout pad and associated anchors, reducing sliding capacity of falsework towers.
6. Care and Preservation:
   a. CEI did not follow their own care and preservation guidelines, resulting in weathering of formwork and falsework members.
   b. Astaldi did not follow CEI's care and preservation guidelines, resulting in weathering of formwork and falsework members.

About beams misalignment above the shoring towers in unit 4?

What is the meaning of "not considered structural member"?

Shims were in some instances inadequate and in other instances missing (the soffit panels were not touching the legs) Please explain how much the soffit panels and the steel beam can be allowed to deflect without affecting the integrity of the structure.

3 FIELD INVESTIGATION OF DRAFT TUBE UNIT 2 FAILURE

Upon arrival on site morning of June 2, 2016, ILF and Astaldi representatives performed limited visual investigation of the power house area, focusing on DT2 and DT1. The following summarizes observed conditions:

3.1 Status of Powerhouse Construction June 2, 2016

At the time of ILF's arrival on site, powerhouse construction had been suspended and the status of respective draft tubes were as depicted in Figure 2 DT 1 was the most advanced of the four draft tubes, with three levels (lifts) of pours in place. DT 2 was the second-most advanced with some outlet roof pours complete. DT 3 was the least advanced with 2 lifts of draft tube concrete complete and outlet roofs unpoured.

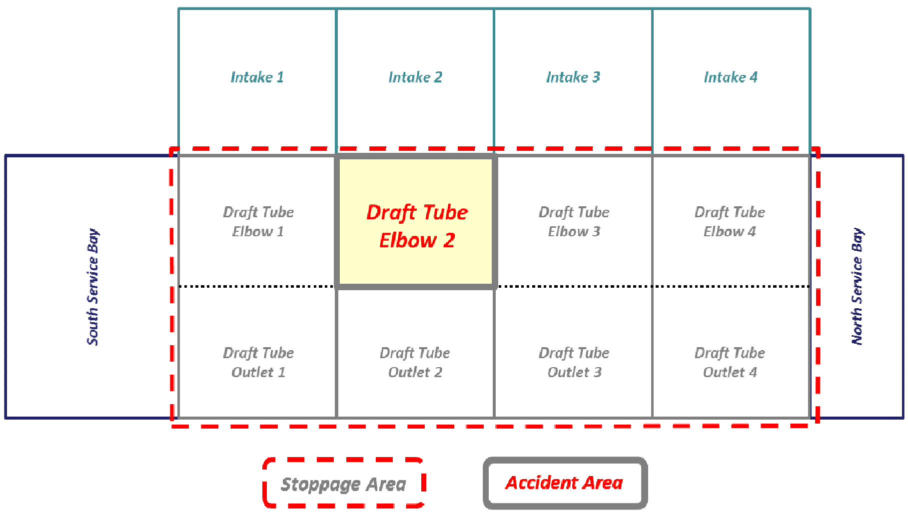


Figure 1: Areas impacted by May 30, 2016 Stop Work Order

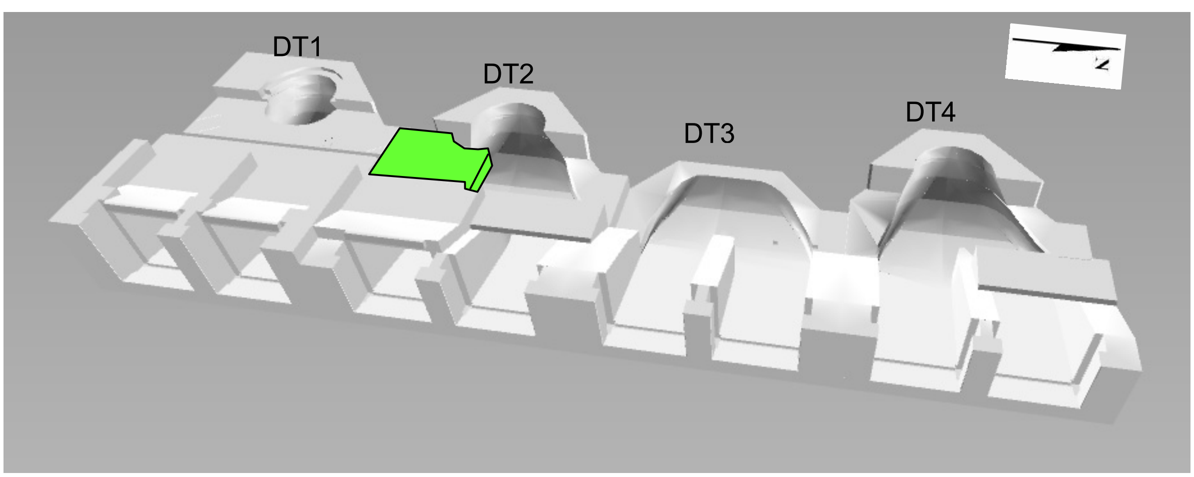


Figure 2: Work progress in the power house June 2, 2016. Failed D2ESB-03 in Green

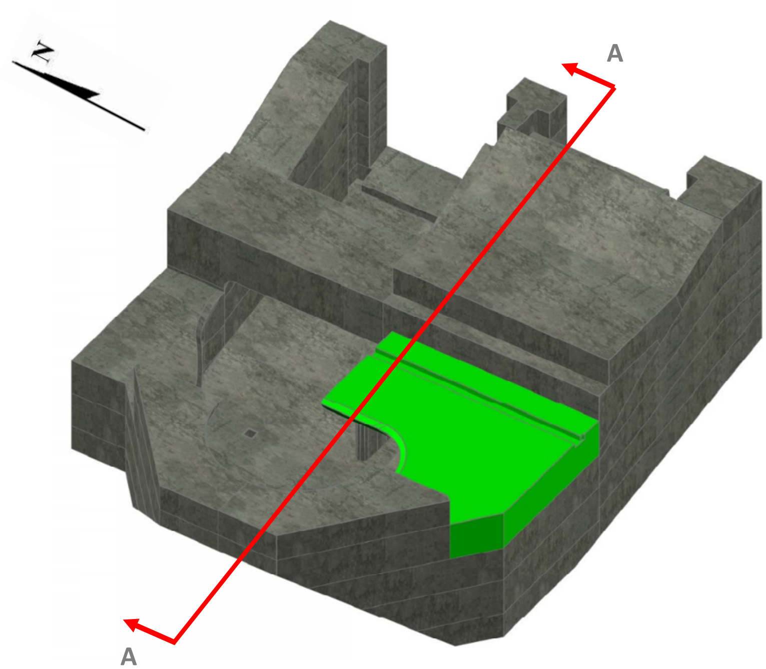


Figure 3: Draft Tube Unit 2 isometric; failed D2USB-03 pour in green

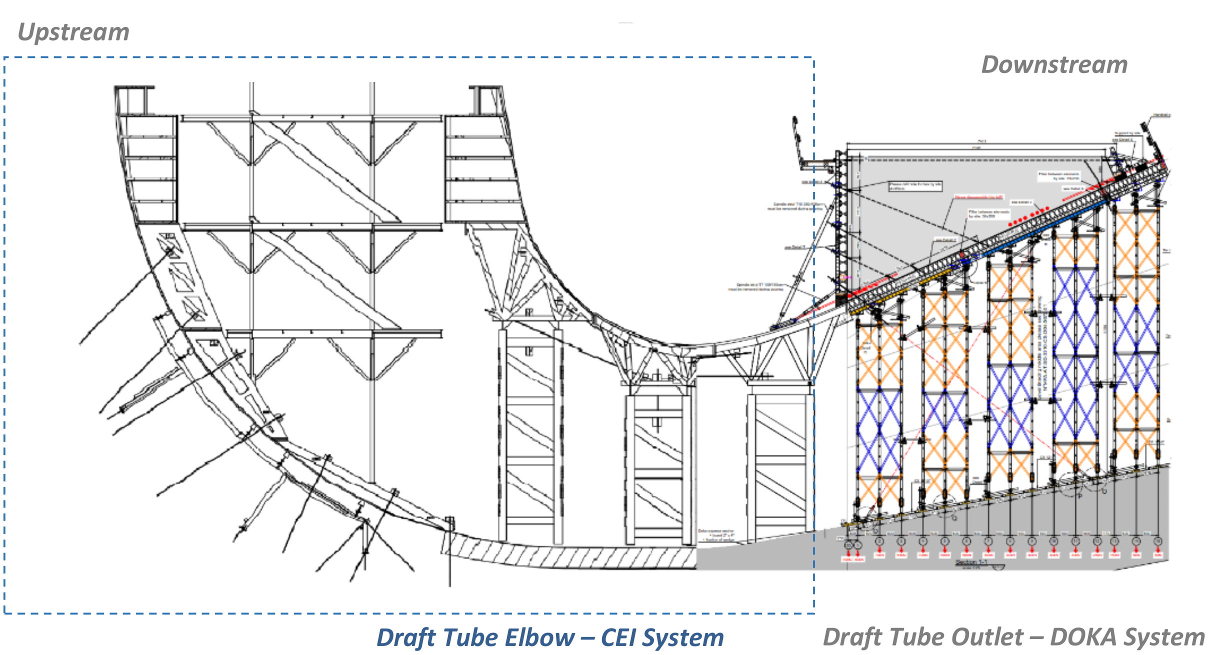


Figure 4: Section A-A from Figure 3: Typical section of draft tube and outlet Formwork Systems

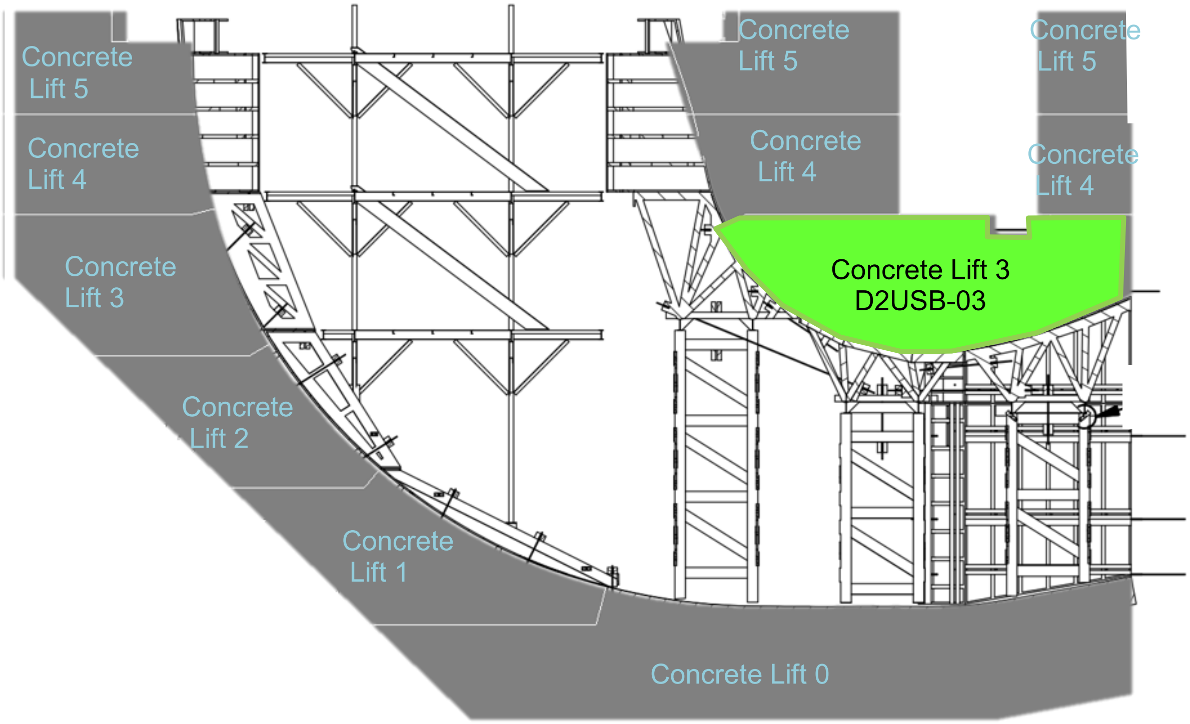


Figure 5: Section A-A from Section 3, Naming convention of concrete lifts in draft tubes.

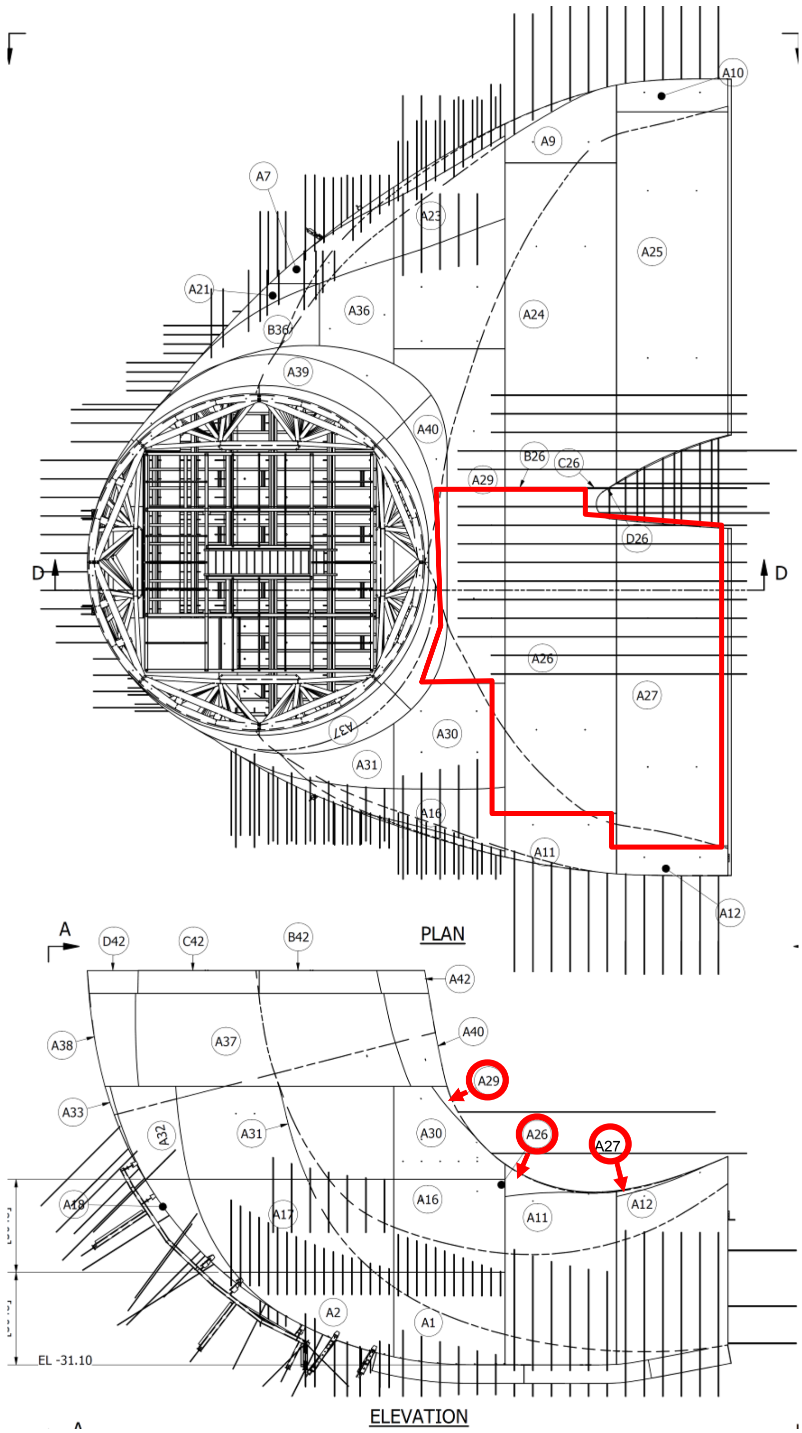


Figure 6: CEI formwork modules involved in collapse

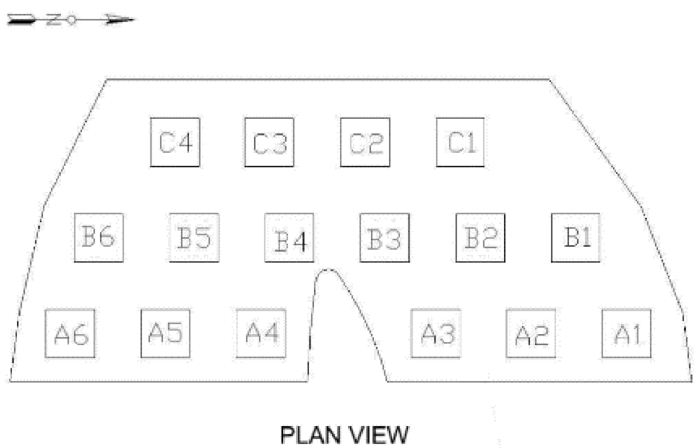


Figure 7: Falsework tower naming convention typical all draft tubes

3.2 Preliminary Field Investigation of Draft Tube Unit 2

Preliminary on-foot inspection from areas surrounding DT2 was performed on the day of ILF arrival on site to identify potential causes of DT2 failure. Access into the failed area of DT2 was limited at that time due to safety concerns by Astaldi safety manager, affording partial views of the collapsed area from surrounding areas above the failure and from the south outlet. Figure 8 through Figure 16 depict conditions on June 2, 2016.

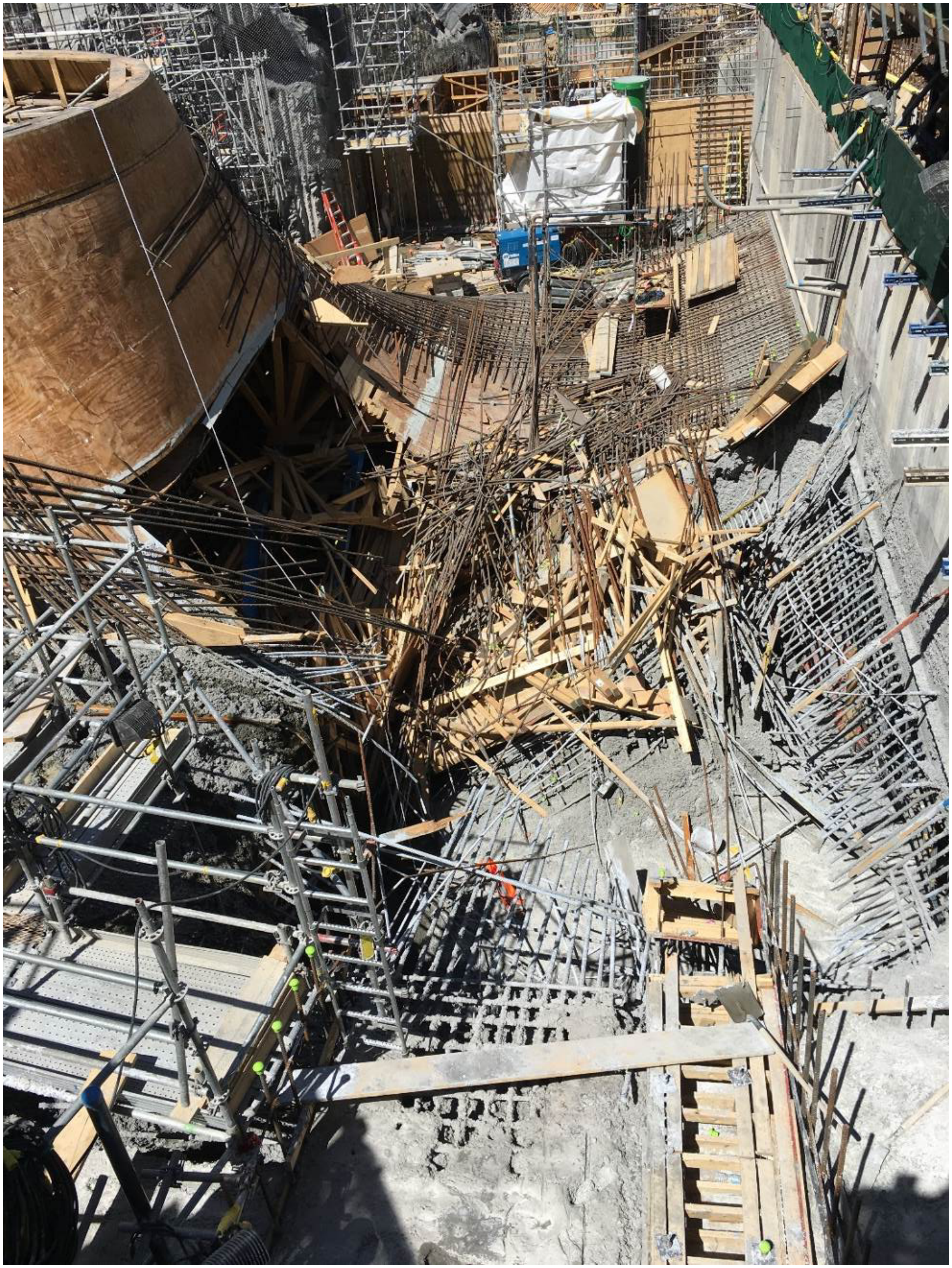


Figure 8: DT2 failure area, looking north

Figure 9: DT2 failure area looking southeast

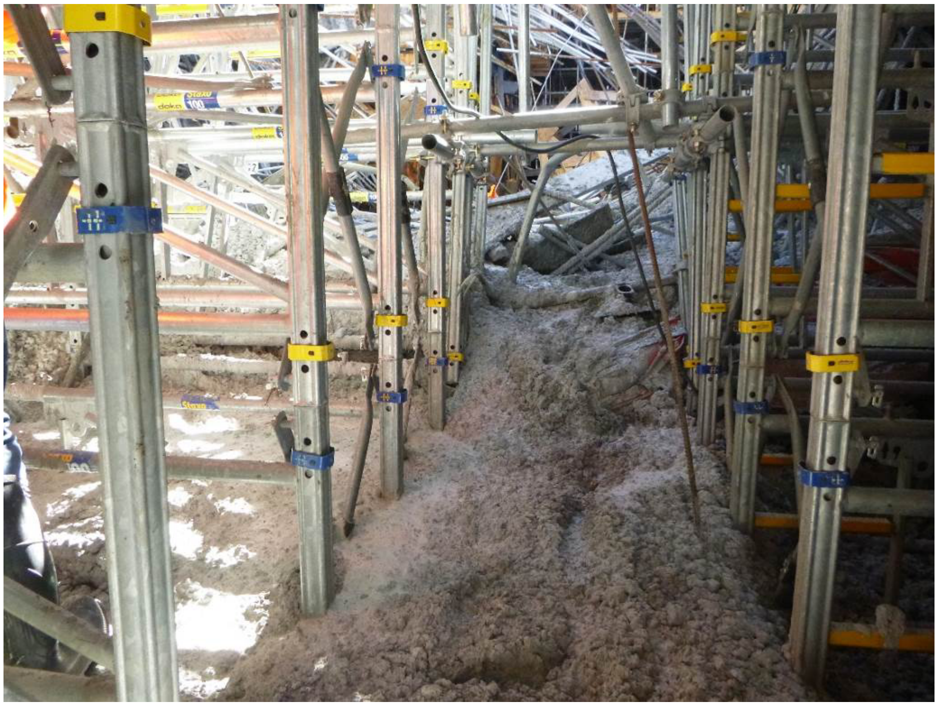


Figure 10: DT 2 south outlet looking upstream (west) into failure area


Figure 11: DT2 south outlet looking west-southwest into failure area

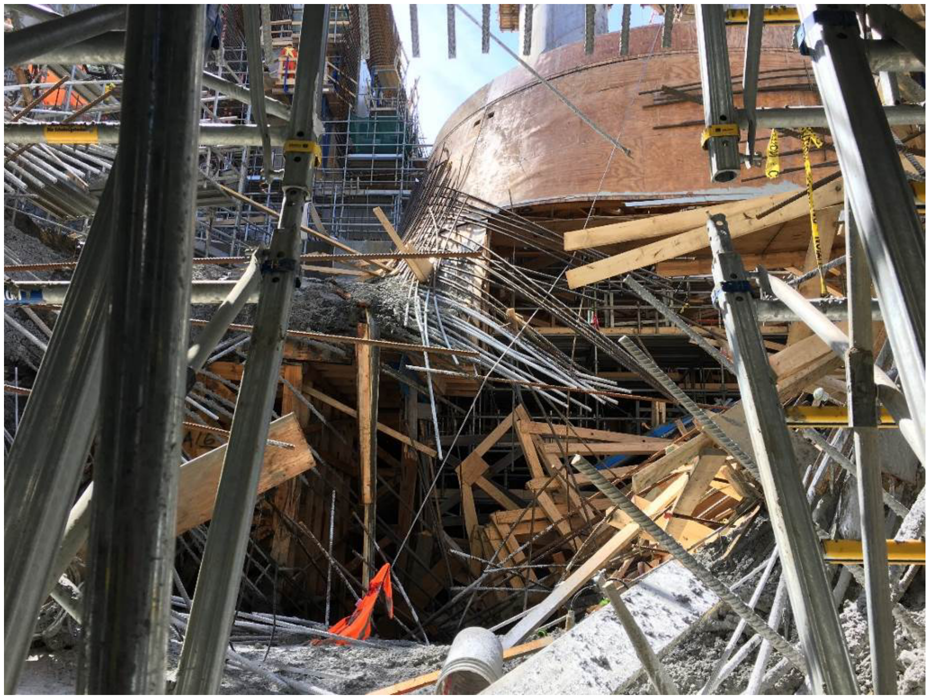


Figure 12: DT2 failure area looking west from northern portion of DT2 south outlet

Figure 13: DT2 failure area looking west from southern portion of DT2 south outlet

General observations from the preliminary field investigation revealed a large mass of concrete approximately 3.5m in thickness in the area underlying the failed pour. The majority of the visible formwork debris was concentrated to the north and upstream (western) edges of the failure area. Formwork and rebar to the north were partially supported by pier nose. Formwork to the west was partially supported by the remaining Row C tower legs, which had shifted to bear against access scaffold behind. The southern and eastern edges of the failure area had a distinct lack of CEI formwork from the failed area visible, suggesting the failure likely propagated from the SE to the NW. The DT2 south outlet had spilled concrete from the failure, which damaged up to 4 rows of Doka Staxo 100 falsework towers downstream from the draft tube.

Additional visual inspections in the DT 2 formwork/false work incident area were conducted by man basket on June 4 and June 11, 2016. Visible tower leg members had signs of weathering and were clearly darkened and grey in color compared to adjacent formwork lumber. It was observed and recorded that several of the upstream legs of towers C3 and C4 had potential indication of inadequate nailing of the 2x10's forming the legs. In some instances, plies were separated. See Figure 7 for Falsework tower naming convention. There were visible indications that the bearing surface on top of the Falsework towers was not uniform, as shown by the column members' end grain being compressed on some 2x10 members but not on others, attributable to the lack of bearing plate and uneven tower leg bearing surface. There was no evidence of a bearing plate being installed at the top of tower legs in the rubble or on remaining erect towers. It was deemed that a ground level inspection would be required to further assess the extent of the deficiencies and damage to the Falsework towers and formwork.

Please clarify.

Figure 14: DT2 photo from man basket

Figure 15: DT2 failure area from man basket

Figure 16: DT2 failure area from man basket looking southeast

See CEI drawing
MFA-AT-SD-3310-CS-D04-0004-03-C1

3.3 **Ground Level Inspection of Draft Tube Unit 2**

A ground level inspection of the DT 2 formwork/falsework incident area was conducted on July 6 and July 7, 2016 to identify damage to the main structural members and note any deficiencies or deviations in the fabrication and construction process and to investigate cause of failure. At the time of this inspection, there was limited access inside DT 2 due to the debris resulting from the failure area. Not all formwork panels were visibly accessible due to the proximity to the failure area and there was limited access to the elevated panels. The majority of falsework associated with D2ESB-03 failure was not visible as it was buried beneath spilled concrete. Portions of C3 and C4 towers remained standing and were visible at time of inspection. During the visual inspections, a number of observations were made with respect to the quality of materials, quality of fabrication, and erection.

3.3.1 Quality of Materials

The visual inspection revealed weathering of falsework tower elements and mold growth on both falsework towers and localized areas on formwork panels. It should be noted that the bottoms of falsework towers were submerged in water at the time of inspection. The overall condition of the wood falsework towers was poor, with evidence of decay and mold growth.

3.3.2 Quality of Fabrication of Shoring Towers and Formwork

- The splice plates used in the fabrication of the formwork ribs overlying the falsework towers were generally too small and did not adhere to the typical splice plate detail shown on CEI fabrication drawing MFA-AT-SD-3310-CS-D04-0001-01_C1. See Figure 17.

- Inadequate nailing of splice plates to falsework tower legs was observed. Nailing patterns were random and did not adhere to the typical splice plate detail shown on drawing CEI fabrication drawing MFA-AT-SD-3310-CS-D04-0001-01_C1. See Figure 25 and Figure 26.

- Column members were inadequately joined (i.e. nails did not have proper penetration through the column laminations). Column members were observed beginning to separate due to inadequate joining. See Figure 18 and Figure 19. CSA O86 and NDS 2012 provide explicit direction for joining of built up members including fastener length, spacing, and edge distance. Requirements for nailing and bolting are provided. ILF was not able to identify any instances of conforming built-up members in visual inspections or in subsequent disassembly of tower legs.

- There were two (2) 2"x10" members missing that were to be located in the web of the two steel beams above the falsework towers B1 and B3. See Figure 20.

- Butt joints in some column members were observed to be in close proximity, in contradiction to sound engineering practice. See Figure 21.

- There was no evidence of bearing plates being installed in any DT2 falsework towers. Despite both CSA and NDS guidelines requiring the use of bearing plates to ensure even distribution of concentrated loads under the W10x22 beams resting on towers, none were specified by CEI.

- CEI labeled falsework tower members and formwork modules with multiple color codes, which are intended to identify the draft tube in which respective members should be installed. Figure 22 and Figure 23 depict falsework and formwork with multiple colors (blue, green, and black) of paint markings as the loads were preparing to ship from CEI's facility.

Falsework towers colored coded green and black (meant for Draft Tubes 3 and 1 respectively) were installed in DT 2. The correct color code should be blue, matching identifying markings for formwork installed in DT2. See Figure 24. The use of multiple color codes poses no practical impact to structural capacity of the formwork and falsework but is poor practice leading to potential for confusion in the field.

Figure 17: Inadequate splice plates should extend to area in red

Figure 18**:** Inadequate nail length (did not penetrate all column laminations)

Figure 19**:** Built-up column members separating due inadequate joining

Figure 20: Missing members between steel beams above Tower B1

Figure 21: Close proximity of butt joints in broken tower leg

Figure 22: Formwork and falsework with multiple color markings "C41" and "A14" at CEI's facility

Figure 23: Falsework with multiple color markings "C41" at CEI's facility

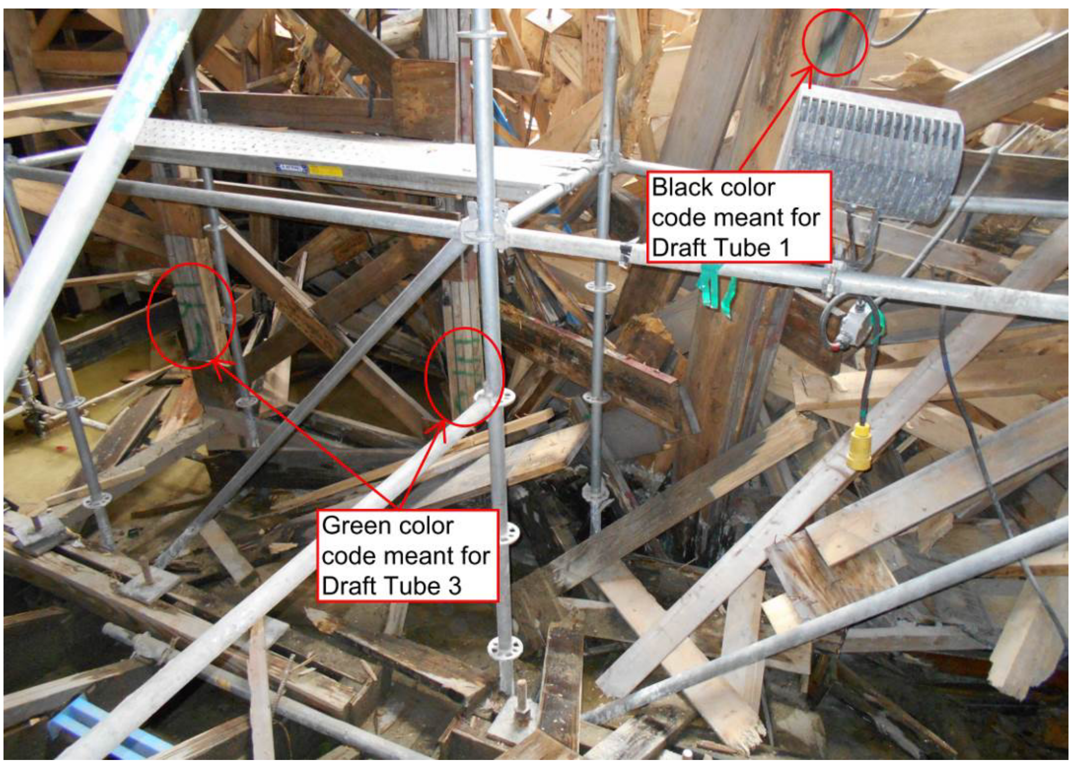


Figure 24: Incorrect color coding used in DT 2, multiple colors present

3.3.3 Quality of Erection of Shoring Towers and Formwork

- Shims between the falsework tower columns and the steel beams were poorly installed. In some cases the shims did not cover the entire bearing surface of the column, potentially overstressing isolated column members. It should be noted that a combination of softwood and steel shims were used. See Figure 27 and Figure 28. Had a bearing plate been specified by CEI, poor shimming would have only resulted in extra deflection in overlying formwork in the order of fractions of an inch (mm). A remaining portion of leg of Tower C3 indicated uneven loading of legs as some plies of tower leg had indication of end grain compression and adjacent plies lacked the same markings. This is indicative of uneven bearing surface at top of falsework tower legs.

- Splice plates used to join falsework towers were inadequately nailed to tower legs. See Figure 25 and Figure 26. Reference MFA-AT-SD-3310-CS-D04-0001-01 within Appendix J for splice plate dimensions and nailing requirements.

See CEI drawing
MFA-AT-SD-3310-CS-D04-0004-03-C1

Figure 25: Splice plates pulling away from column legs

Figure 26: Splice plate pulled away from tower leg

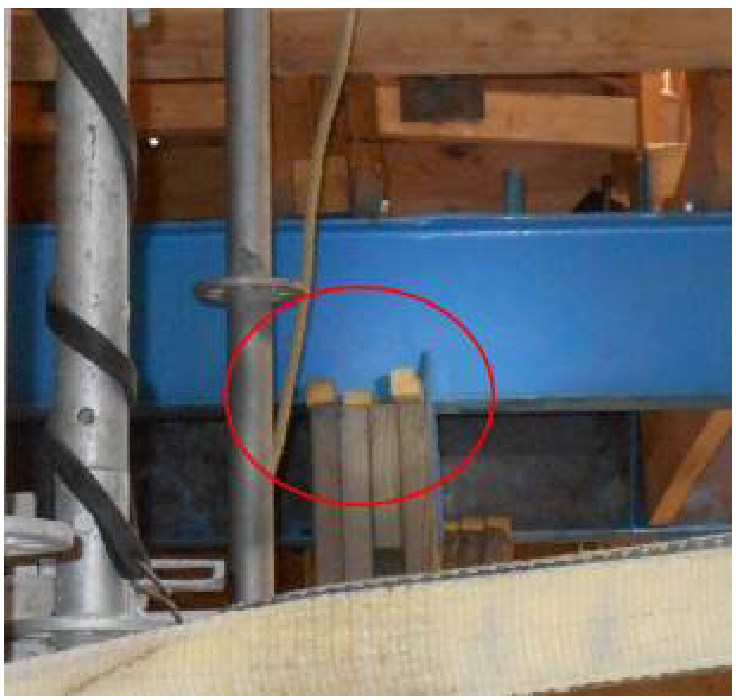


Figure 27: Inadequate shimming on top of falsework towers. No bearing plate installed

Figure 28: Inadequate shimming on top of falsework towers. No bearing plate installed.

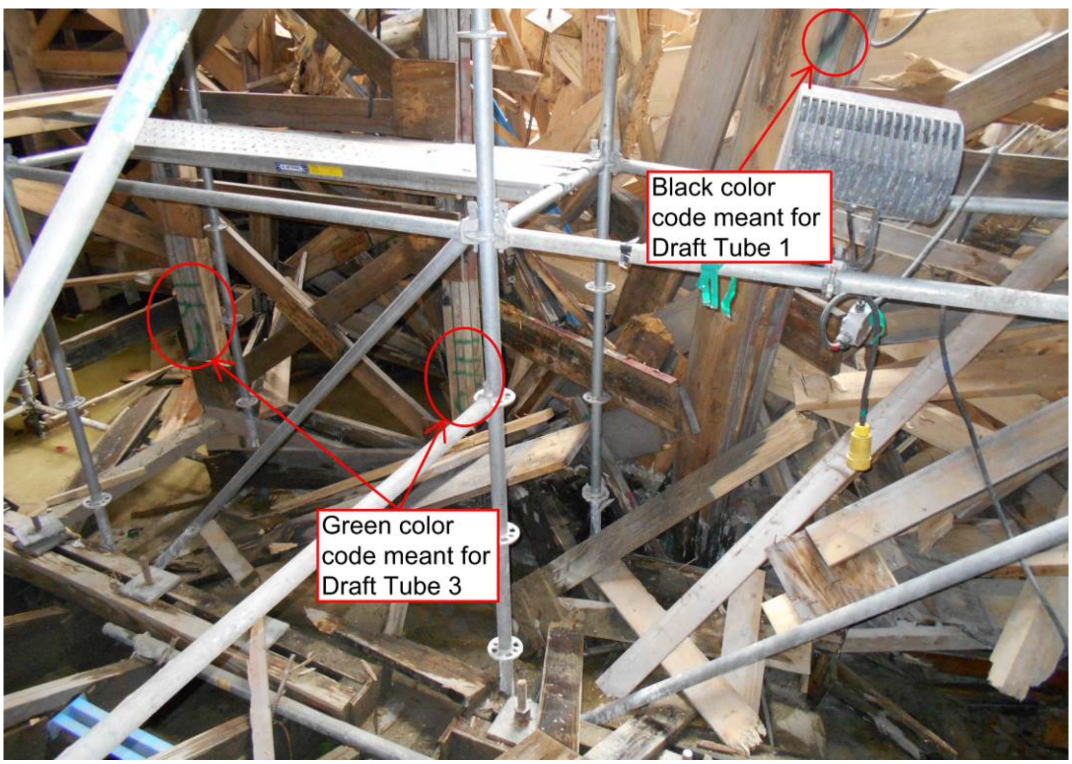


Figure 29: Incorrect color coding used in DT2

4 FORMWORK AND FALSEWORK DESIGN

Can you define 'design criteria'?
Are you making references to design standards, or design loads, or site conditions, or all of the above?

4.1 Proper Design Criteria

A proper design should consider the most recent local design codes and regulations as part of the design criteria.

It is standard practice to require engineered systems manufactured outside Canada and intended for end-use in Canada to comply with local Canadian design codes and standards. Newfoundland and Labrador Regulation 45/12 explicitly states that the National Building Code of Canada (NBCC), 2010 edition should be used for buildings in the province (exclusive of one and two story family dwellings). Section 4.1.1.3 of the NBCC defines "building" as "any structure used or intended for supporting or sheltering any use or occupancy". Item 4 of the same section explicitly requires CSA 269.1 "Falsework for Construction Purposes" and CSA 269.3-M "Concrete Formwork" to be used for design of falsework and formwork. In Section 4.3.1.1 of the NBCC, it states that members made of wood should conform to CSA O86, "Engineering Design in Wood", and in Section 4.3.4.1 it states that members made of structural steel should be in conformance with CSA S16, "Design of Steel Structures". These codes were not employed in CEI's design.

Isn't it 5/12?

Which clause?

4.2 CEI Design Criteria

The design of the draft tube formwork and falsework system was carried out by CEI, stamped on September 14, 2014, and revised on November 30, 2014. The design was based on the following codes and references:

- "NDS National Design Specification for Wood Construction, Allowable Stress Design" – 2005 Edition
- "AISC, Manual of Steel Construction, Allowable Stress Design" – 9th Edition
- "APA, The Engineered Wood Association, Panel Design Specification" – 2004 Edition
- "M.K. Hurd, Formwork for Concrete" 6th Edition

4.3 ILF Design Criteria

Both the 2015 NDS design code and CSA-O86-09 Engineering Design of Wood were utilized by ILF to check the existing formwork system. It was determined that the difference between the two codes, when properly utilized within this context, produced results within approximately 5 percent of each other. As the complex formwork system was designed by CEI using NDS, ILF elected to continue using NDS as to maintain an analysis using similar design methodology (Allowable Stress Design, ASD). ILF continued to perform periodic checks against CSA codes to ensure compliance but did not fully analyze all formwork using CSA as CSA utilizes Load and Resistance Factor Design (LRFD) methodology, which employs load factors. To maintain clarity in design loads and to allow better comparison against CEI's analysis, NDS was primarily relied upon by ILF in formwork analysis.

Both the 2015 NDS and CSA O86-09 and subsequently CSA 269.1-1975 were used for design checks of the falsework towers. For consistency in comparison of design loads, NDS (unfactored loads) was utilized. ILF provides CSA (factored loads) for reference, as ultimately CSA O86 governs in design of falsework towers.

Live load consisting of full 10.2 feet (3.1m) liquid head concrete and a live load of 50 pounds per

Could this number be explained in more details?
Base on Table 5 of this report, the difference between the two standards is about 40%. Base on aDB column capacity calculation, the difference between the two standards is about 20%.

Per CSA O86-09 Table 5.2.1.3 Southern Pine is equivalent to Canadian SPF.
It is not adequate to carry number from AWC to CSA calculation as they do not use the same calculation criteria. It is adequate to compare the end results of both standards.

square foot (2.4 kPa) were assumed for analysis of formwork elements. The analysis assumed a 2 foot (600 mm) tributary area for each rib member per typical rib member spacing, as constructed. Design checks were made for lower formwork modules to ensure reaction loads from overlying pours were adequately supported. This ensured reactions were adequately resisted throughout the entire formwork system as pours progressed sequentially upward. All proposed modifications to formwork systems were designed in accordance with NDS 2015 and checked by ILF against CSA O86-09.

To check CEI original design calculations, ILF used timber code and timber properties based on the US 2005 National Design Specification (NDS). As design values for southern pine are not provided in Canadian codes, all timber was simulated in CSA O86 calculations as Southern Pine No. 1 with 2005 NDS design values for consistency.

4.4 Discussion of Design Criteria

CEI utilized outdated US codes for analysis of formwork and falsework and therefore did not comply with Newfoundland and Labrador regulations. A summary of US codes used in CEI design, US codes available at time of CEI analysis, and appropriate Canadian codes is provided in Table 1.

Table 1: Summary of Design Codes

Codes Used in CEI Analysis (2014)	**US Codes Available at Time of CEI Analysis (2014)**	**Appropriate Canadian Codes at time of CEI analysis (2014)**
NDS 2005	NDS 2012	CSA O86-14 Engineering Design in Wood, CSA 269.3-M92 Concrete Formwork, and CSA 269.1-1975 Falsework for Construction Purposes
APA Panel Design Spec. 2004	APA Panel Design Spec. 2012	CSA O151 Canadian Softwood Plywood
AISC 9th Edition	AISC 14th Edition	CSA S16-14 Design of Steel Structures

As indicated in Table 1, subsequent revisions of NDS, AISC, and APA design guidelines were available at the time of CEI's analysis in 2014. Additionally, Table 1 provides the appropriate Canadian codes that should have been considered by CEI and their design reviewers.

Reviewing US codes available in 2014, ILF identified NDS 2005 does not contain the explicit comments that NDS 2012 and newer publications have regarding nailing and bolting requirements for built-up members. It is likely that the inadequate nailing of built-up tower leg members by CEI would have been addressed had they used 2012 NDS guidelines or newer. Similarly, the appropriate Canadian design codes which ought to have been employed contain explicit requirements for built-up members as described in CSA-269.1. Regardless of explicit NDS code requirements for joining built up members, it is expected that sound engineering judgement would have concluded that the nailing of 2x20 plys employed by CEI's fabrication shop was not consistent with assumptions made by CEI in their calculation package.

2x10

Astaldi reviewed the criteria reported by CEI and affixed their Newfoundland and Labrador permit to practice and professional engineering seal to the CEI calculation package and associated drawings issued for construction. Astaldi then submitted the CEI calculation package and associated

Please refer to NDS 2005 clause 15.3.3.1 where the nailing requirement is clearly explained.

Did Astaldi review only the design criteria?

drawings to Nalcor who reviewed and returned the documents code "4- Information only" on February 25, 2015 stating within the general comments *"This document is being returned for information only. The review of the document was limited to the design criteria used for the structural analysis. It does not include the verification of the calculations or the structural models used or any other portion of the document."* It appears that both Astaldi's and Nalcor's review were flawed and did not identify the non-conforming design codes employed by CEI.

4.5 Formwork and Falsework Engineering Analysis

ILF performed a review of CEI's structural calculations package for the formwork and falsework. In the following sections, the differing assumptions employed by ILF and CEI are explained and compared, and CEI's falsework structural calculations are reviewed.

4.5.1 Formwork Analysis and Design

4.5.1.1 CEI's Formwork Analysis and Design

The CEI designed draft tube formwork system is complex, consisting of 49 individual formwork modules per draft tube and 16 falsework towers, with each module containing multiple trusses or ribs and many having ribs of varying shapes and sizes. CEI presented eight (8) simple 2-dimensional frame models as representative of all formwork rib members within the draft tube system, despite there being nearly 300 unique formwork ribs, each comprised of multiple structural members. Figure 30 is an example of the amount of individual members within one rib of one formwork module. A total of 14 unique members were used to create just Rib C29 alone. Rib C29 is one of 11 unique ribs (Ribs A29 through K29) that are used to create module A29. The total amount of individual and unique members numbers in the thousands for a single draft tube.

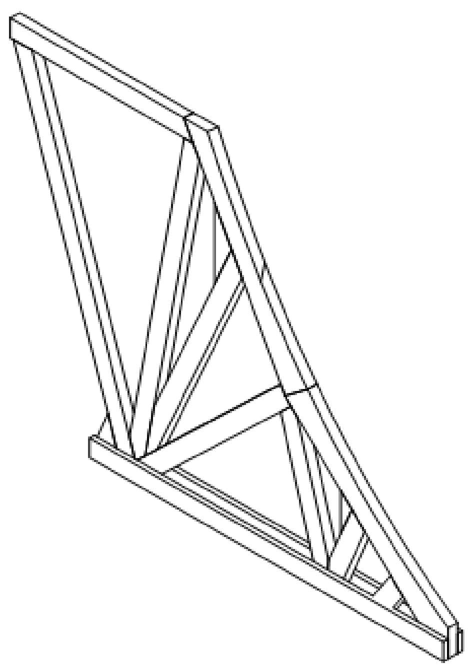


Figure 30: Rib C29 of Panel A29 from CEI shop drawing MFA-AT-SD-3310-CS-D04-0048-01

CEI's use of eight two-dimensional rib models to represent the entirety of the draft tube formwork is not reasonable and does not represent employment of sound engineering judgement. This was especially evident in using the calculations of rib G24 to represent panels A24, A25, A26, A27, and A29. The geometry of these panels varies significantly rib to rib and panel A29 has significantly different geometry and loading when compared to the other panels. Panel A29 has lateral coil rod ties to resist the horizontal loading but that was not represented in the rib G24 analysis. The material impact of this approach is that numerous ribs lacked adequate representation in the design. Based on the complex geometry and variation between adjacent ribs, many structural members were not checked for appropriate loads, load paths, interaction with the formwork skin, and joint connections.

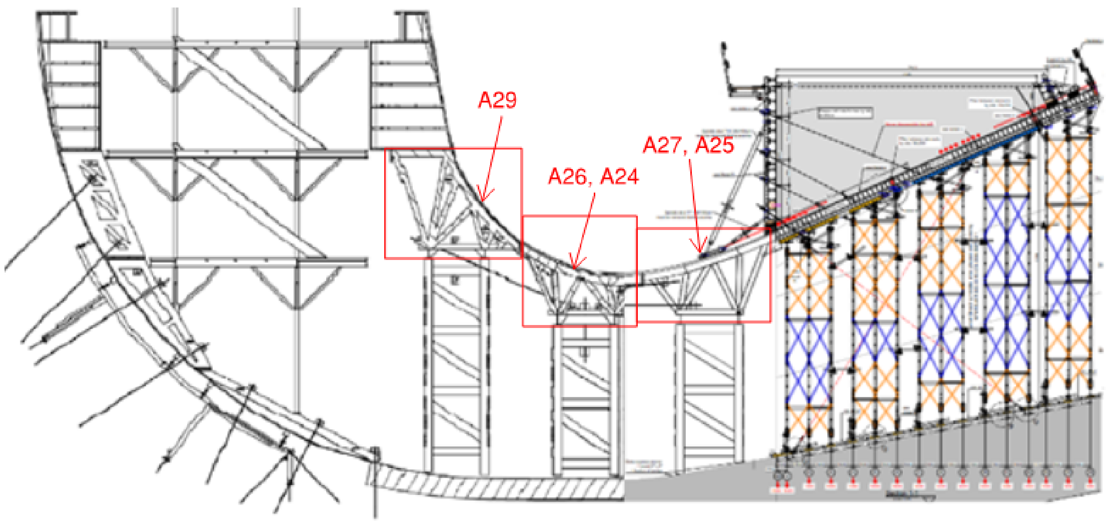


Figure 31: Panel geometry for analysis. Wet ties not shown.

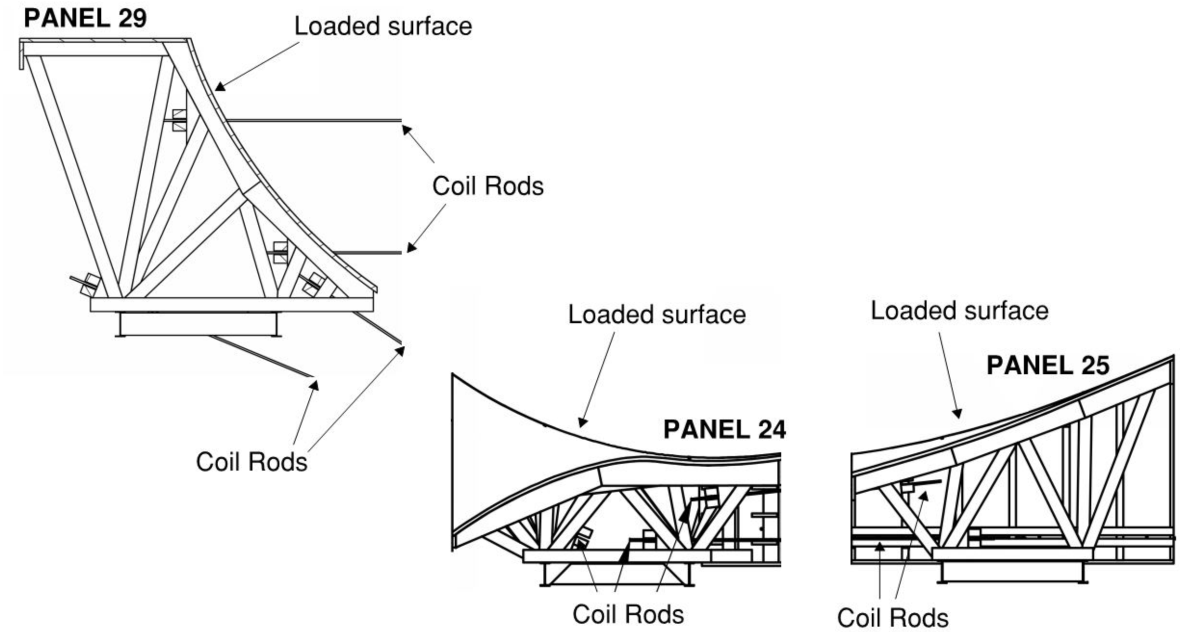


Figure 32: Draft Tube Roof Panel Geometry

Figure 32 provides indication of the differences in the loading and geometry of modules A24, A25 and A29. Modules A26 and A 27 are similar to A24 and A25, respectively. Note the difference in geometry of the module sections and consider CEI modeled all three of these formwork modules using one rib (rib G24) from module A24. CEI's analysis did not consider the difference in applied concrete loading (mostly lateral for A29, mostly vertical for A24) due to formwork curvature nor were connecting forces from coil rods represented in their calculation package. The reactions of the coil rods produce localized stresses and global deflections in the formwork system that result in higher utilization of multiple members.

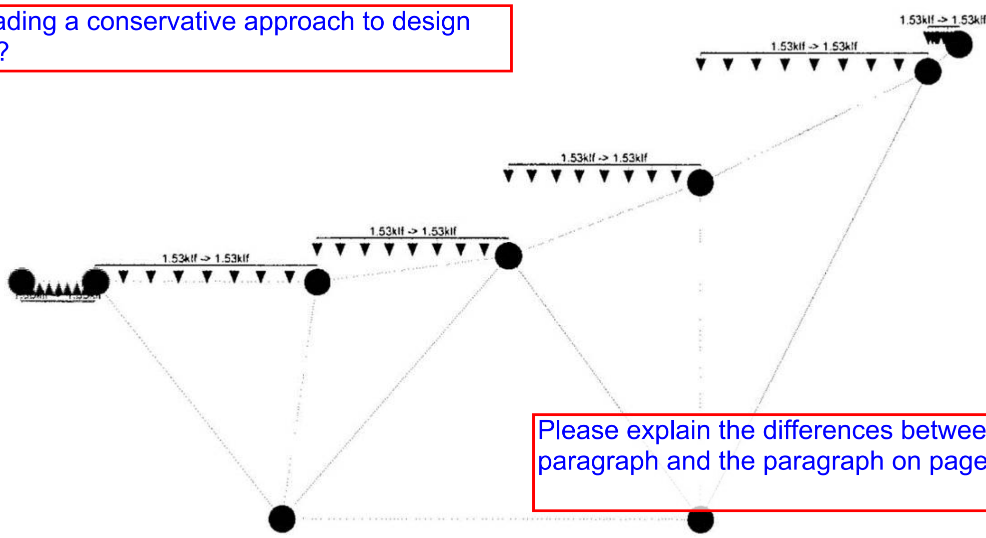
CEI's design did not take into account reshore loading in formwork design for elevated slabs (such as the draft tube roof). When wet concrete is placed to create an elevated slab, a vertical load is locked in place in the formwork and falsework supporting the fresh concrete from below. If construction methodology requires additional concrete pour(s) to be placed over the elevated slab and a mechanism for releasing locked-in loads is not provided (sand jacks or similar), additional loads are imposed through the slab to the underlying supporting structures. Reshore occurs due to the concrete slab being elastic, that is, not infinitely stiff, and deflects when subjected to external loading such as additional pours above. Reshore load required for formwork and falsework design is the portion of additional load felt by structures supporting the first elevated slab when additional concrete is poured. In the case of DT2, this is the portion of lift 4 concrete that must be supported by the formwork and falsework used to support lift 3.

Design codes such as CSA 269.1 and ACI 347.2 provide guidance for reshore and incremental load assumptions in high-rise buildings with consecutive floors. Due to the complexity of the draft tube structure, it is anticipated that the engineer of record for the permanent structure (SNC for Muskrat Falls powerhouse) would provide reshore loads to formwork and falsework designer. If the engineer of record were not able or unwilling to produce this information, a simplified slab analysis could be made to appoximate deflection in cured lift 3 concrete due to lift 4 loads. Note ILF does not have record of CEI requesting or receiving information regarding the anticipated reshore load for their formwork and falsework design.

After the D2ESB-03 failure, Nalcor provided direction for reshore loading in response to a site query requesting reshore values by Astaldi. Ref. Site Query: AT-SQY-CH0007001-0556 in Appendix H. The response indicated formwork and falsework must support full liquid head of concrete for lift 3 and 50 percent of lift 4. This results in a total liquid head of 13.8 feet (4.2m) (10.1‘ + 0.5 x 7.2‘ =13.8‘). CEI utilized 10.2‘ (3.1m) for formwork calculations, as shown Figure 33.

The loading CEI applied to the rib models was inconsistent and incorrect in some of the analysis, specifically for modules A24, A25, A26, A27, and A29. Uniform vertical and horizontal loading was applied where variable loading along members would have been appropriate. The analysis of rib G24 is an example of this. A uniform load of 1.53 kips per linear foot (22.3 kN/m) was applied to the members in the Global Y (down) direction. However lower elevation members experience a higher concrete head load than members at a higher elevation. This method neglects lateral load induced into the rib. As identified in the independent formwork analysis completed by ILF, pressure applied perpendicular to the member creates a bending moment in the vertical elements of the rib truss. Moment coupled with vertical loading overstresses the element to a utilization ratio greater than 1.20. Utilization ratio is defined as the ratio of applied stress divided by the maximum allowable stress, as determined by code requirements.

Is the uniform loading a conservative approach to design the rib members?



Please explain the differences between this paragraph and the paragraph on page 34.

Figure 33: Rib G24 loading in CEI calculations

Module interaction and combined global stability were also not considered by CEI. Modeling each rib individually did not account for the modules interacting as a combined system. For example, Modules A24-27 and A29 are connected through the top and bottom chords of the truss ribs via coil rods. This transfers load between modules, changing the stresses induced in the rib members. In addition, A29 has a more vertical form face, and connecting it to the more horizontal faces of the other panels induces lateral load and moment into the global system. This changes the load path and magnitude to the supporting towers.

Rib model fixity did not accurately represent installed conditions. Tie-backs and push pull bracing were modeled by CEI as pinned supports. Based on the substantial length of some of the tie-backs (29 feet, 8.8m), they would have deflected before providing significant resistance. This assumption would have led to higher than actual reaction loads at tie-back reaction points (whalers) and disregarded the flexibility of the connecting formwork. The flexibility of the system can result in higher stresses among formwork members than CEI anticipated as the tie-back reactions and deflections are distributed throughout adjoining members. The actual impact would have to be determined in a case by case basis.

Please elaborate on the bending capacity reduction when the wood members are assemble together.

To properly account for the deflection due to long tiebacks, CEI should have modeled tie-backs as a spring support to accurately represent interactions within the engineered system. CEI's calculations also assumed a tributary width for loading of one foot but reduced the members to single 2 inch nominal thickness (2x) structural members. In reality, constructed modules had a rib spacing typically of 2 foot (600mm) and consisted of double 2x members. That modeling assumption would produce inaccurate results as laminated members behave differently than single members under load. Based on loading conditions and orientation, built up member capacity should be reduced in bending capacity, or conversely increased for axial capacity.

4.5.2 ILF's Formwork Analysis and Design

ILF's formwork analysis assumes a 2 foot (600 mm) tributary width for most rib members, per typical rib member spacing, as constructed. All proposed formwork modifications have been presented in separate drawing packages detailing the additional member and fastening requirements per the CSA 086 Design Code and NDS.

The NDS separates loading into multiple categories; Live Load, Dead Load, Snow, Seismic, Wind, Temporary Loads, etc. Based on the location, season, structure type, and forming process, ILF used formwork dead load, construction live load (workers, material, etc.), and concrete loads considered as dead load. The formwork should not be exposed to considerable wind or snow loading and seismic is neglected for the temporary aspect of the structure.

The 3 basic load cases for formwork design were defined in the RISA 3D model using the following:

- BLC 1 Formwork SW Load: weight of formwork 70 psf x 2 ft = 0.14 kip/ft (conservative)
- BLC 2 Live load: 50 psf on 2 ft span: 50 psf x 2 ft = 100 lb/ft = 0.1 kip/ft
- BLC 3 Lift 3, 4 or Lift 5 Concrete load:
  Concrete liquid head load: 150 pcf x head ft x tributary width ft (kip/ft varies)

Note that formwork loads vary with depth of concrete.

The load combination utilized in ILF's RISA 3D model is defined as LC 1 = BLC1 + BLC2 + BLC3. These loads are unfactored, per ASD methods. An example calculation for determination of formwork loads for Panel A24 at joint with A25 is as follows:

BLC 1	+	BLC 2	+	BLC 3	=	LC 1
0.14 kip/ft	+	0.1 kip/ft	+	(150 pcf x 11 feet x 2 feet width)	=	3.54 kips/ft

The geometry of the timber formwork model was based upon the three-dimensional AutoCAD file developed by CEI. A simplified stick and node version of the AutoCAD drawing was prepared by ILF to represent the neutral axis of individual members and to ensure proper connectivity between members. Highlights of the structural model include the following:

- Primary and secondary structural members were modeled.
- Typical timber panel members included single and two ply built-up 2"x6" and 2"x10" elements.
- Falsework tower column legs of four ply built-up 2"x10" boards.
- Anchor rods of ¾" diameter were used for all tie backs and coil ties.
- Tie back walers were simulated as unbraced 2 ply built-up 4"x6" members.
- Steel support beams were simulated as W10x17 wide flange beams (grade A992).
- Dead Loads included the self-weight of the primary and secondary members, the formwork skin (1/8" plywood overlying 2"x4" diaphragm), and the individual lift concrete loads.

1/4"

Please clarify

- Concrete loads were applied to perimeter members as an increasing load with depth based on the elevation of the members. Full pour depth gravity loads were applied to members with horizontal elements supporting concrete.
- Vertical loading was considered for the soffit panels on the eastern portion of the pour based on the angle of the formwork. A representative down drag vertical load was also applied to the near vertical walls of the western pours.
- Typical member connections and restraints were modeled as fixed end conditions. Refer to ILF engineering reports MFA-AT-SD-331A-EN-0016-01 and MFA-AT-SD-331A-EN-A99-0020-01 for formwork lifts 2 through 5 for assumptions and analysis regarding fixity.

Two 3D models were developed using finite element software, RISA-3D, to identify the impact of member connections for rib truss members. The first model was created with fully released members (pinned connections that allow for rotation of the connected members) for truss ribs and the second was modelled as non-released members (fixed connections translate rotational forces at the connections). As all truss diagonals were connected to main chord members with gusset plates, in reality these diagonal connections were partially fixed. Under the same load cases and load combinations, the fixed member model is the more conservative approach because it has slightly larger utilization ratios (approximately 5-10% difference). Therefore, the fixed model was used for ILF's analysis.

Formwork truss ribs were assumed to be laterally supported in-plane at anchor rods and push-pull post locations. Anchors and push rods are installed in multiples along whalers that brace the truss ribs. Out of plane lateral supports were also assumed to represent sheathing diaphragms and cross bracing of formwork panels. Anchor rods were modeled based on individual length and applied angle of resistance.

Please explain the differences between this paragraph and the paragraph on page 32.

Are the ties modeled as fixed support?

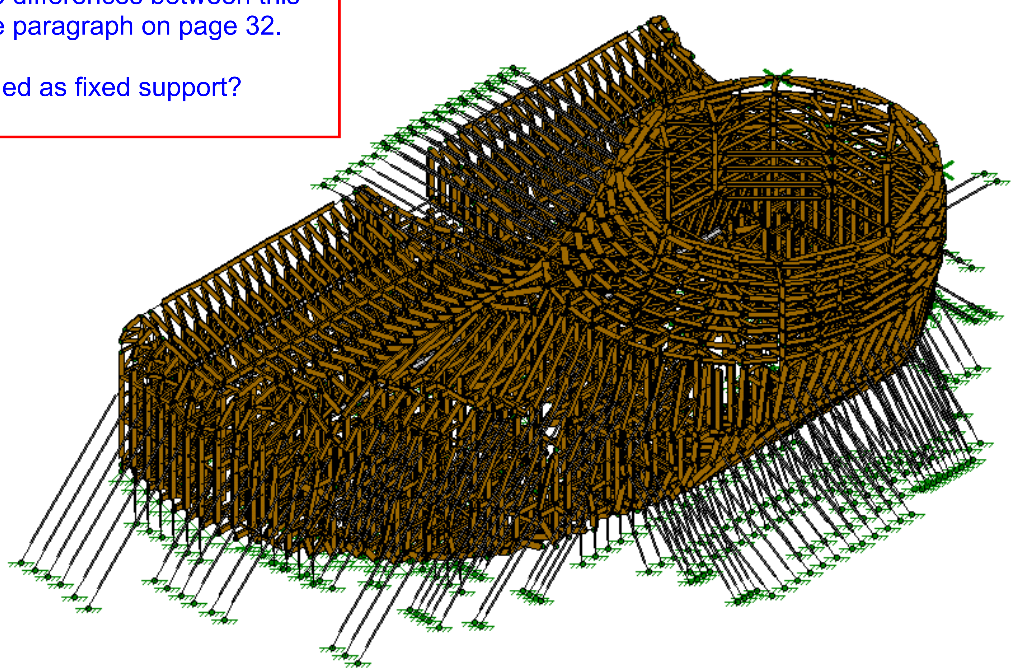


Figure 34: Isometric View of ILF's 3D Structural Model

4.5.3 Comparison of Analysis and Design Assumptions

At the time of ILF analysis, concrete lifts 0 and 1 had been completed in all draft tubes and re-analysis was therefore considered unwarranted for formwork associated with these lifts. ILF performed 20 2-dimensional frame models for formwork associated with concrete lifts 2 through 5. Additionally, ILF performed 3-Dimensional modeling of the entire CEI system to determine the adequacy of the CEI design, and in combination with frame modeling, ILF identified 14 formwork modules having one or more overstressed members that required strengthening. For the purposes of this report, overstressed members are defined as those with utilization ratios over 1.20. Utilization ratios represent the total stress divided by the allowable stress within a member. Reference Table 2 for a summary of formwork modules with at least one overstressed member. ILF also performed a global check of lateral forces on the formwork and found that the CEI design of Panels A25 through A27 had an unresolved net lateral load that required additional ties to prevent the lateral load from transferring to falsework towers.

4.5.4 Summary of overstressed formwork members identified by ILF

Overstressed formwork members required additional review on a case by case basis to determine remedial reinforcing necessary to satisfy NDS 2015 and CSA-O86-09 requirements. Members with a utilization ratio greater than 1.00 were categorized as overstressed. Upon review of the results, it was determined members with a utilization ratio greater than 1.20 required modifications to reduce stress to allowable limits. Members with utilization ratios between 1.00 and 1.20 were deemed acceptable.

Members with utilization ratios of 1.00 to 1.20 were acceptable due to the difference between the center to center span in the RISA model and the as-built clear span of the members. Members in RISA were modelled to centerline of supporting members, versus the actual clear span in the field taking into account support member sizes and plywood gusset plate sizes. Figure 35 indicates how the stick and node models used in RISA overestimate the unbraced lengths, for reasons described above. Stick and node are indicated as red lines and blue dots, respectively.

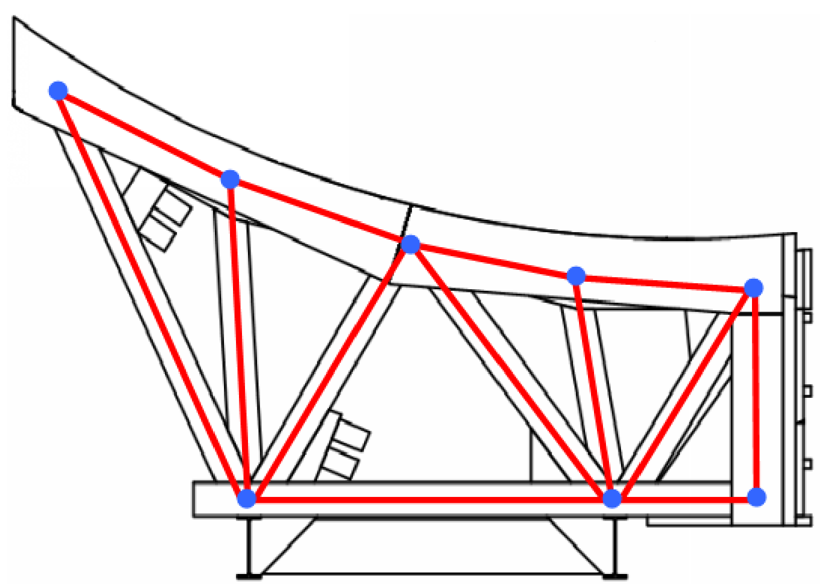


Figure 35: Comparison of stick and node to actual geometry

Reinforcing and bracing with dimensional lumber and plywood were required for members with utilization ratios over 1.20 to achieve acceptable member capacities. Modifications were generally limited to discrete members of formwork module ribs and could be carried out in the field with on-hand materials. Details for the strengthening/bracing of formwork were issued for construction via site instructions and engineered drawings submitted through Astaldi's document control system. See Table 2 below for a summary of modules with overstressed members.

Table 2: Summary of panels with overstressed elements

Concrete Lift	Panel	Shear Stress		Combined Bending/Axial Stress	
		Utilization 1.00≤UR≤1.20	Utilization 1.20<UR	Utilization 1.00≤UR≤1.20	Utilization 1.20<UR
02	A9	X		X	
	A16	X			
	A19	X		X	
	A22	X		X	
	A13		X		
	A14	X			
03	A24		X		X
	A25		X		X
	A26		X		X
	A27		X		X
	A28		X		X
	A29		X		X
	A30		X	X	
	A34	X			
	A35	X			
	B36	X	X	X	
04	A37		X		X
	A38		X		X
	A39		X		X
	A40		X		X
05	A37	X			
	A39	X		X	
	A40		X		X

The panels identified in Table 2 included at least one member that was overstressed. As mentioned earlier, each panel is comprised of multiple ribs, each rib made with several structural members. In some instances, ILF identified formwork modules with multiple ribs having more than one member being overstressed. Refer to MFA-AT-SD-331A-EN-A99-0016-01, MFA-AT-SD-331A-EN-A99-0019-01, and MFA-AT-SD-331A-EN-A99-0020-01 for additional details.

In addition to the overstressed formwork members, ILF identified locations where additional anchor rods or tie-backs were required. These were required due to unbalanced loading during concrete placement. Based on review of CEI's calculation package, it appears they had assumed that the concrete for each lift would be placed in one pour. This assumption was not representative of the multiple pours used during the actual construction of the draft tubes for each lift. This was most obvious within concrete lifts 4 and 5 where the geometry of the draft tube is nearly circular in nature and lateral loads would be approximately balanced if each lift was poured in one pour. When broken into quadrants, unbalanced lateral loads are created, which were not considered by CEI. A revised design was proposed by ILF for construction, including multiple new anchor rods to restrain lateral movement of the now segmented pours within respective concrete lifts 4 and 5.

4.5.5 Examination of Falsework Analysis and Design

4.5.5.1 CEI's Falsework Design Load Case

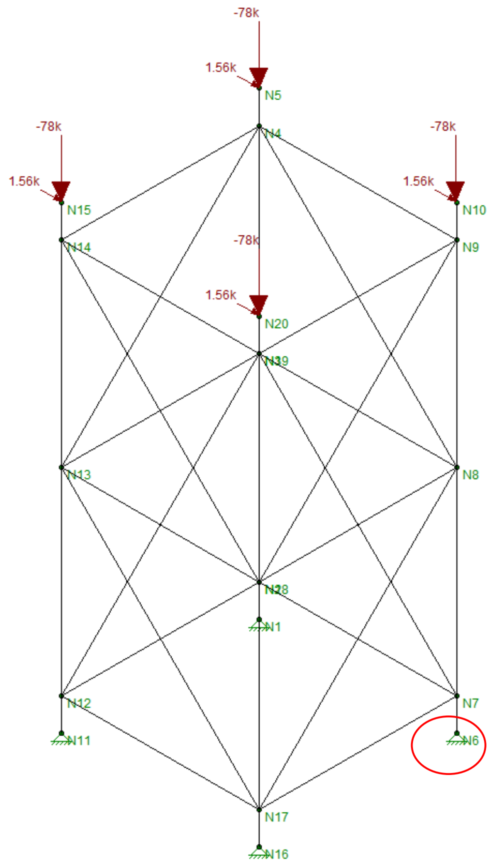
CEI's falsework analysis and design used the load combinations based on allowable stress design, which does not apply load factors to design loads. It appears that live load of concrete was manually inputted in to their calculation sheet as 78 kips per tower leg and a dead load of self-weight of 273 pounds per tower leg was used. Documentation supporting the development of these loads was not provided and the dead load does not accurately represent the self-weight of falsework towers or overlying formwork. Additionally, live loads for concrete placing crews and dead load for overlying formwork were not considered.

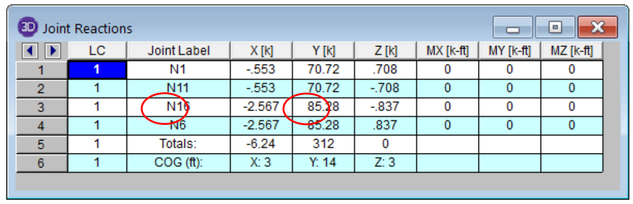
CEI falsework calculations did not account for the required 2 percent lateral load for towers, per CSA 269.1-1975. This requirement is not explicit in NDS codes but sound engineering judgement would expect a portion of vertical load to be accounted for as lateral load and applied at top of the towers. The impact of this error results in approximately 9 percent increase in axial load in the most heavily loaded tower row, Row B. A similar increase would be expected in tower Row A and approximately 13% increase in Row C due to the increased height in those towers. The increase in axial load is due to the moment couple of lateral load and height of tower. Rows A and B have approximately the same heights and Row C is in the order of 6 feet (2m) higher. A lateral load applied at top of tower applies a moment that is resolved through axial loading at the bottom of tower. The increase in axial tower load due to lateral load was not considered by CEI. Reference Figure 36 showing the anticipated axial load increasing from 78 kips to 85 kips (y direction) due to consideration of 2 percent lateral load.

Could you clarify "manually inputted"?

In the CEI calculation document, on PDF page 48 of 79, CEI wrote "(10,139)(6)=60.8k"

aDB's understanding of the CEI note is that the beam line load above the column is 10139 PLF, there are columns every 6ft
10139 PLF * 6ft = 60834 lb
The columns are every 6ft in both directions, 10139 PLF / 6ft = 1689.8 PSF
The 78k value is calculated from the following page (49/79 in CEI calculation document) using the following data
Fc=1 600psi
Emin=620 000psi
all K factor = 1
Section area = 55.5 in^2
Unbraced length = 6ft




Joint Reactions

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	-.553	70.72	.708	0	0	0
2	1	N11	-.553	70.72	-.708	0	0	0
3	1	N16	-2.567	85.28	-.837	0	0	0
4	1	N6	-2.567	85.28	.837	0	0	0
5	1	Totals:	-6.24	312	0			
6	1	COG (ft):	X: 3	Y: 14	Z: 3			

Fc for 2x10
NDS 2005 = 1600 psi
NDS 2012 = 1600 psi
NDS 2015 = 1450 psi

Please validate

Figure 36: ILF model showing impact of neglecting 2% lateral load in Row B towers

4.5.5.1 ILF's Falsework Design Load Case

Full liquid head of concrete and a live load of 50 psf (2.4 kPa) were assumed for analysis of formwork elements due to the slow set time of the concrete mix used. The 2015 NDS (US) design code was utilized to check the structural capacity of the existing formwork system. ILF performed a comparison of NDS 2005 and NDS 2015 and have determined that there are no substantial differences in the methods, formulas, and properties within these documents and performed parallel analysis of formwork producing similar results. Spot checks of ILF's 2015 analysis were performed using the current CSA O86 (Canadian) Engineering Design in Wood to ensure results from NDS were consistent with CSA.

Falsework tower (Row B) loads were developed using the same logic presented in formwork design, except loads were considered over the 6 foot by 6 foot tributary area for one tower leg as follows:

Formwork Load =	63	psf
Placement Live Load =	50	psf
Lift 3 Concrete Load =	1550	psf
Lift 4 Reshore Load =	550	psf
Tributary Area =	36	sf
Column Axial Load =	79,668	lbf
2% Lateral Load Axial =	1,560	lbf
Total MAX Base Reaction =	85,280	lbf

From CSA, 40 PSF live load (workers) and 10 PSF minimum formwork weight for total of 50 psf. Formwork load was included above.

Does the 2% lateral load include the load of lift 4? 2% * 79 668 lbf = 1 593 lbf, please confirm

Which tower height was used to compute the total reaction? Are the tallest towers as loaded as the short ones?

Please explain

4.2.3 Comparison of Falsework Design Load Cases

Both the CEI and ILF design processes used load combinations as directed by ASD guidelines. However, CEI neglected temporary loading conditions due to live loads from workers and installation. CEI did account for a re-shore load similar to that specified by Nalcor after the collapse (Ref. Site Query: AT-SQY-CH0007001-0556 in Appendix H), but supporting documentation was not provided in their calculation package. Neglecting live loads due to workers or temporary conditions during erection is unconservative and increases applied loading by approximately 3 percent. The deviation in reshore load accounted for approximately 7 percent increase from CEI's design load.

CEI calculation package identifies a reshore load for subsequent concrete lifts but a load combination that contained a breakdown of reshore loading was not identified. Using loading requirements imposed by Nalcor after the failure of D2ESB-03, CEI would be required to design for full thickness of lift 3 concrete 10.2 feet (3.1m) and half the 7.2 feet (2.2m) thickness of lift 4 concrete. The overall difference in CEI assumed load case and ILF calculated load case is summarized as follows:

Table 3: Design Load Comparrison

Designer (Code used)	Tower Leg Load
CEI (NDS 2005)	78.3 kips (347 kN)
ILF (NDS 2015)	85.3 kips (387 kN)

Note NDS does not factor design loads. The factor of safety is built into ASD resistance factors and therefore load factors above 1.0 are not required.

Additionally, ILF compared load cases using CSA O86, which employs LRFD method the following loads and factors should have been considered.

Table 4: Proposed Factored Loads using CSA O86 (100% lift 3 and 50% lift 4 per SQ 0556)

Load	Load Value	Load Factor	Factored Load
Formwork Dead load	63 psf (3 kPa)	1.25	78 psf (3.75 kPa)
Placement Live Load	50 psf (2.4 kPa)	1.5	75 psf (3.6 kPa)
Lift 3 Conc.Dead Load	1,550 psf (74.4 kPa)	1.25	1938 psf (92.8 kPa)
Lift 4 Reshore Live Load	550 psf (26.4 kPa)	1.5	825 psf (39.5kPa)
ILF Design Load for 36 sf tributary area (3.35 m2)			105.0 kips (467 kN)

The factored load presented in Table 4 should not be directly compared against the design load in Table 3 due to the difference in design methodologies utilized by ASD and LRFD methods. This is due to LRFD using a load factor reduction in calculations. Reference Appendix M for ILF example calculations using NDS (ASD) and CSA (LRFD) methods.

Had CEI employed a mechanism to relieve loads imposed by lift 3 concrete prior to placing lift 4, such as a sand jack, the reshore load could have been omitted from calculations.

4.5.5.2 CEI's Falsework Design Resistance Factors

Based upon review of CEI formwork and falsework analysis, the design was inadequate for the intended loading of 10.2 feet (3.1m) of concrete in pour D2ESB-03. Numerous deficiencies were identified in the CEI design of the timber falsework towers. Most notably, CEI did not account for the tower legs being constructed as built-up members instead of a solid timber. CEI's calculations assumed the tower legs were a solid timber member with a 9.25in x 6in cross-section (nominal dimensions for a 2x10 are 1.5 inch x 9.25 inch), whereas CEI fabricated the respective legs by making a built-up member with 4 plies of 2“x10“ lumber. NDS and CSA codes require a 0.60 stress modification factor to be applied to the compressive resistance of the gross cross section of a built-up member joined with nails. In addition, wet service conditions were not considered in calculations but would have been appropriate for draft tube construction. NDS applies a 0.8 wet service factor when wood moisture content exceeds 19 percent whereas CSA provides two values for wet service, pending dimension of the lumber. A wet service factor of 0.69 is to be applied for members 89 mm and less, whereas 0.91 may be used for members greater than 89 mm in least dimension. As falsework towers were fabricated using multiple members thinner than 89 mm, the 0.69 factor is utilized for ILF’s calculations. For this reason, CSA O86-9 governs when comparing NDS and CSA design methodology for falsework towers.

4.5.5.3 Resistance Factors Discussion

Through applying the omitted stress modification factors and re-analyzing CEI’s constructed falsework tower capacity in conformance with NDS 2015, ILF has determined the allowable load for one CEI tower leg to be 42 kips (185 kN) assuming bracing, fabrication details, and wood quality all met code criteria (which ILF’s investigation identified that, in many respects, they did not). The allowable load under CSA O96-9 falsework tower leg is 42 kips (185 kN). CEI erroneously calculated their falsework towers having an allowable capacity of 78.3 kips (346 kN), which is 1.86 (78.3 ÷ 42 = 1.86) times the value determined by ILF using CSA methods.

A summary of allowable tower capacities is provided in Table 5, providing CEI’s assumed tower capacity, and properly calculated capacity for same configuration using NDS and CSA codes. Additionally, capacities for the 7 ply and 9 ply configurations required to adequately support dead loads for lift 3, and for lift 3 and 50 percent of lift 4, respectively are provided. Both nailed and bolted allowable tower leg capacities are provided for respective built member configurations, for reference. As noted in preceding sections, CEI did not provide a mechanism for releasing locked in falsework loads from lift 3 prior to pouring lift 4 and therefore reshore loads must be considered.

Table 5: Comparrison of allowable tower leg capacities

Tower Configuration	Design Code						
	CSA O86-09		NDS 2005			NDS 2015	
	ILF		CEI	ILF		ILF	
	Nailed	Bolted	Nailed	Nailed	Bolted	Nailed	Bolted
4 Ply 2x10	42 kip (185 kN)	52 kip (231 kN)	78 kip * (347 kN)	59 kip (262 kN)	98 kip (436 kN)	59 kip (262 kN)	98 kip (436 kN)
7 Ply 2x10	78 kip (346 kN)	97 kip (432 kN)	-	106 kip (472 kN)	176 kip (783 kN)	106 kip (472 kN)	176 kip (783 kN)
9 Ply 2x10	100 kip (445 kN)	125 kip (556 kN)	-	136 kip (605 kN)	225 kip (1001 kN)	136 kip (605 kN)	225 kip (1001 kN)

*CEI calculated 78 kip allowable capacity in error.

Calculations performed to generate Table 5 utilized SP No 1 wood properties, per NDS 2015, as this was the material used by CEI for fabrication of the falsework tower legs. CSA O86 does not provide wood design values for SP No 1 so NDS 2015 values for SP No 1 were applied for

Published data for southern pine in NDS 2005 and NDS 2012 are the same, but different compared to NDS 2015. How can the column capacities on this table be the same between the NDS 2005 and NDS 2015 calculation?

consistency. The largest difference between NDS and CSA calculated capacities reported in Table 5 lies within the built up member factor, wet service factor, and load duration factors prescribed by the respective codes. Table 6 provides a comparison of values used by ILF and CEI for design of falsework towers. Reference Appendix M for sample ILF falsework tower leg capacity calculations and Appendix A for CEI calculations.

Can you elaborate on load duration? CSA S269.1 for column Kd = 1

Table 6: Comparison of design factors using CSA O86 vs NDS

Design Factor	Design Code						
	CSA O86-09		NDS 2005			NDS 2015	
	ILF		CEI	ILF		ILF	
	Nailed	Bolted	Nailed	Nailed	Bolted	Nailed	Bolted
Wet Service Factor	0.69	0.69	1.00	0.80	0.80	0.80	0.80
Load Duration Factor	1.15	1.15	Not Used =1.00	1.25	1.25	1.25	1.25
Built Up Member Factor	0.60	0.75	1.00	0.60	1.00	0.60	1.00

Per comments on fabrication details and material quality within this report, both CSA and NDS criteria were not met in CEI design and therefore the actual allowable capacity of the towers was less than properly calculated 42 kips (185 kN), as determined using CSA O86.

Please clarify which clause has been used.

Should be 0.75

4.6 Design Material Properties

4.6.1 CEI Material Properties

CEI calculations used Southern Pine No. 1 with the following design values:

Bending F_b=1.8 ksi

Tension parallel to grain F_t=1.1 ksi

Shear parallel to grain F_v=0.2 ksi

Compression perpendicular to the grain $F_{c\perp}$=0.6 ksi

Compression parallel to the grain F_c=1.9 ksi

Modulus of elasticity E=1,700 ksi

They are not the values used by CEI for the column calculation on PDF page 49 of 79

Additional material specifications include:

WOOD MATERAILS	
Sawn Lumber	S4S
Species	Southern Pine
Commercial Grade	No. 1
Application	Exterior
Plywood Grade	APA A-C , CLASS 1
Thickness	1/4"
Grade Stress Level	S-3
Species Group	1
Application	Exterior
Plywood Grade	APA C-D, EXP 1
Thickness	23/32"
Grade Stress Level	S-2
Species Group	1
STEEL MATERIALS	
W-Shapes	ASTM A 992
Channels, Angles, M and S-Shapes	ASTM A 36
Plate and Bar	ASTM A 36
Hollow Structural Sections, Round	ASTM A 500, Grade B, Fy = 42ksi
Hollow Structural Sections, Rectangular	ASTM A 500, Grade B, Fy = 46ksi
Steel Pipe	ASTM A 53, Grade B, Fy = 42ksi, Type E

WELDED CONNECTIONS

Welding shall comply with AWS D1.1 requirements

1/4"

They are not the proper values for 2x10 southern pine No1

4.6.2 ILF Design Material Properties

Material properties for wood members used in the analysis were the same as those reported in 2015 NDS material property tables, which reasonably match CEI values.

Bending F_b=1.850 ksi

Tension parallel to grain F_t=1.050 ksi

Shear parallel to grain F_v=0.175 ksi

Compression perpendicular to the grain $F_{c\perp}$=0.565 ksi

Compression parallel to the grain F_c=1.850 ksi

Modulus of elasticity E=1,700 ksi

4.6.3 ILF check of formwork skin and subskin members

ILF performed a check of formwork skin and subskin members. The formwork skin consisted of 1/8" thick plywood over 2"x4" dimensional lumber subskin. The skin and subskin was modeled as a 2D plate element in RISA with a thickness of 1.5 inches. The maximum stress was calculated as 413 psi (2.85 MPa) which was less than Southern Pine allowable stress of 711 psi (4.90 MPa). Therefore, the skin was deemed adequate for the design loads.

4.7 Testing of Falsework Tower Material Properties

During visual inspection in the field, ILF identified that the wood used to fabricate the towers had undergone significant weathering and had reason to believe the degree of weathering affected the towers' strength. Some portions of tower legs, especially the lower 4 feet (1.2m) had undergone decay, changing appearance to a dark color and weakening such that portions of tower members could be easily penetrated with a probe, e.g. a mechanical pencil, including samples from towers C3 and C4 removed from the rubble in DT2 on August 25, 2016. Astaldi submitted samples of wood from towers A1, B1, and C2 to the University of Toronto to determine the material properties for back-analysis. Testing was to report NDS 2005 base design values for Southern Pine, including modulus of rupture, modulus of elasticity, and compressive strength parallel to the grain.

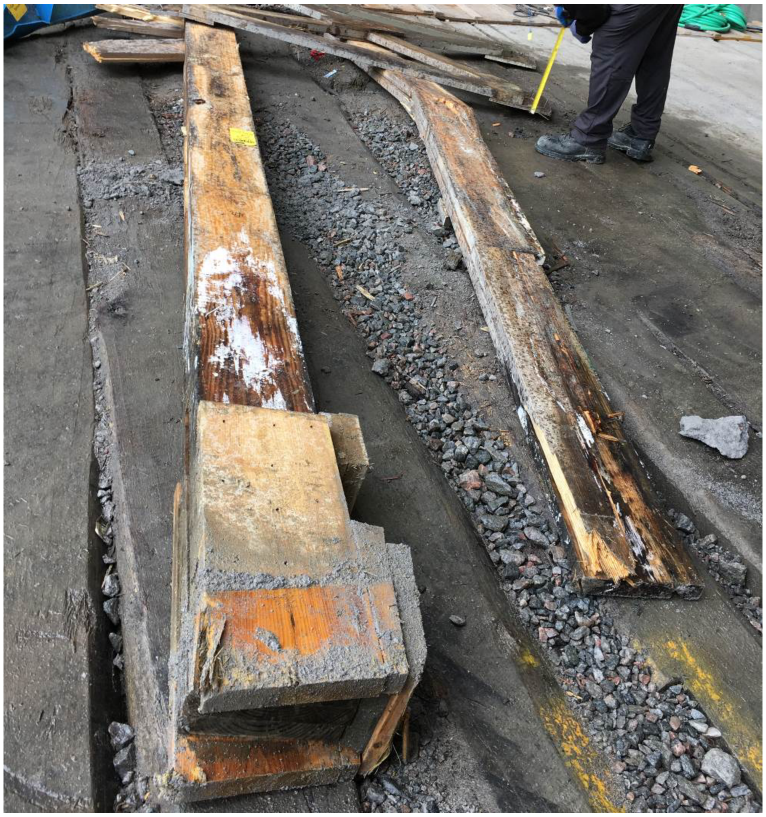


Figure 37: Tower Legs obtained from DT2 Tower C3

Figure 38: Bottom of Tower Leg obtained from DT2, Tower C3

4.7.1 Wood Testing Method

Samples of DT 2 tower legs and bracing were recovered from the north end of DT2, outside the failed area and shipped in protective packaging to University of Toronto. ILF witnessed the collection of the samples and labeled each ply of every leg member to identify the origin of each member submitted for testing.

Figure 39: SW Tower Leg of B2 recovered from DT2, submitted for testing

Two leg samples and two brace samples approximately 5 feet (1.5 meters) in length, respectively were collected from each of DT 2 towers A1, B1, and C2. Note that the use of samples from Tower C2 was a change from the plan proposed in MFA-AT-SD-331A-CV-K99-0005-01 as tower C1 was required to remain in place to support panel A28, which remained intact (a new tower was constructed around Tower C2 to support concrete loads prior to subsequent pours). ILF has no reason to believe that the material properties in Tower C1 varied from those observed in tower C2.

4.7.2 Wood Test Method

ASTM D143-14 “Standard Test Methods for Small Clear Specimens of Timber” was utilized to determine the intact wood properties of tower legs and associated bracing. The method of testing required 1 inch wide by 1 inch thick members, clear of defects for axial compression and flexural testing. Specimens of 4 inches length are required for axial compression testing and 16 inches for flexural testing, respectively.

The tower samples were delivered to the University of Toronto (U of T) on September 19, 2016 and air-dried at ambient room temperature before being processed and dimensioned into specimens for testing.

No material testing of the other draft tubes was planned to be performed at the time of this report; visual inspection indicated that the samples from DT 2 were representative of the wood in Draft Tubes 1 and 2. Draft Tubes 3 and 4 were observed to have less weathering and decay but the difference was not quantified through testing.

4.7.3 Testing Results

Results provided by the U of T testing facility indicated the absence of waterproofing material in the leg samples. This corresponds to the significant wood staining and both incipient and advanced wood decay that had been evident from the initial visual inspection. Initial moisture content testing of tower brace samples, as received by the lab indicated wood moisture content ranged from 9% to 14%. Initial moisture content testing of tower leg samples indicated the wood was above the saturation point of 25%. Note design procedures required by NDS 2005 and CSA 086 reduce the calculated structural capacity of lumber in compression when moisture content exceeds 19%.

Flexural testing of leg samples produced significant variability in strength values. The average modulus of rupture ranged from 48.1 to 60.6 MPa, and the modulus of elasticity ranged from 5.1 to 7.3 GPa. Brace specimens produced more consistent results. The average modulus of rupture for braces ranged from 63.2 to 73.7 MPa, and the modulus of elasticity ranged from 6.3 to 8.0 GPa. Compressive testing parallel to the grain for leg samples ranged from 20.9 to 31.0 MPa. Compressive strength for brace samples were within a range of 33.3 to 39.1 MPa.

Table 7: Range of test values for leg and brace samples

	NDS 2015 Base Design Values	Leg Samples	Brace Samples
Modulus of Rupture (MPa)	10.3	48.1-60.6 (mean 53.6)	63.2-73.7 (mean 67.0)
Modulus of Elasticity (GPa)	11.0	5.1-7.3 (mean 6.2)	6.3-8.0 (mean 7.0)
Compressive Testing Parallel to Grain (MPa)	11.4	20.9-31.0 (mean 27.7)	33.3-39.1 (mean 37.0)

The strength values determined from testing exceeded the typical design values provided in NDS 2005. Modulus of elasticity and moisture content of the legs did not conform to the standard design values.

U of T used ASTM D143-14 as basis of testing, which provides direction that samples with defect such as knots or irregular growth to be discarded. It is inferred that defects such as decay are to be discarded as well, and this is the practice adopted by the U of T lab. U of T observed excessive decay in many of the tower leg samples, rendering evaluation impossible for these portions of the tower legs. U of T reported that tower bracing did not have significant decay.

ILF questioned U of T as to the proportion of material that was not testable due to decay and they advised that based on recollection (no inventory was taken during sample preparation), untestable material due to decay ranged between 20 and 50 percent of respective tower leg samples. Some of the untestable material was in such an advanced state of decay that 1 inch by 1 inch samples could not be cut without the material crumbling during preparation. Therefore ILF believes that the testing performed over estimates the tower leg strength and does not provide lower bound strength values. The U of T did not quantify the degree of decay that may have occurred between time of collapse in May, 2016 and time of testing in October 2016, nor was the impact of field conditions quantified prior to shipping samples off of the Muskrat Falls site for testing.

As some portions of the wood was decayed to the point it was not testable, it can be inferred that decay in tower leg members would most likely have resulted in localized strengths below NDS values. However, the lower-bound strengths of overall tower members and timing of the decay cannot be determined at the time of this report.

5 FABRICATION AND INSTALLATION OF FORMWORK AND FALSEWORK

5.1 Fabrication of Formwork and Falsework

5.1.1 CEI Specified Fabrication

Fabrication details were specified by CEI in drawing MFA-AT-SD-3310-CS-D04-0001-01_C1. ILF observed nonconformities in the fabrication of the formwork and falsework, and those deficiencies are described and shown in the following section. In summary, ILF identified the following errors in review of fabrication drawings:

1. CEI's shop drawings indicate falsework bracing penetrating vertical members of the falsework tower legs. This detail results in considerable overstress in members adjacent to these penetrations.
2. Bearing plates were not specified at top of falsework tower legs to ensure even load transfer from formwork above. The lack of bearing plate results in overstress at top of falsework tower leg.
3. Proper specification for nailing or bolting built up members is not provided in CEI erection or fabrication drawings. ILF was not able to identify any nailing or bolting requirements in the documents provided. Nailing requirements are provided for joining cross bracing splice plate to falsework legs (by contractor) using 10 -8d nails.

5.1.2 Observed Fabrication

Despite fabrication details being outlined in the CEI design drawings and construction drawings, there were numerous fabrication deficiencies noted by ILF in the formwork and falsework. ILF has identified fabrication deficiencies that decreased the load bearing capacity of the formwork and falsework, but without full scale testing the combined impact of these deficiencies is unquantifiable. The practicality of full scale testing is questioned due to the amount of variables that could be considered and may be moot given the nature and extent of design errors identified.

5.1.2.1 Documenting tower leg fabrication

In order to determine typical as-built towers, as fabricated by CEI, Draft Tube 3 falsework tower C3 was disassembled, exposing the penetrations in the 2nd layer of the tower legs, as shown in Figure 42 and Figure 43. This layer contains the internal bracing that does not conform to CSA and NDS guidelines. ILF also observed saw kerfs and gaps between tower leg members in many tower sections, which reduces effective tower section areas.

Leg layers 1, 3, and 4 were constructed using multiple pieces of 2x10, whereas shop drawing #W-41b specified one continuous member. This resulted in multiple butt joints within a given tower leg. If butt joints were adequately spaced, the impact on structural capacity is negligible. However, as, shown in Figure 40 and Figure 41, butt joints were close spaced among layers 1, 2, and 3. Members in the second lamination closely matched the dimensions specified on CEI shop drawing # W-41b. Reference MFA-AT-SD-331A-EN-A99-0010-01 "Muskrat Falls Draft Tube Quality Report, Manufacturing quality –shoring Tower Disassembly" report.

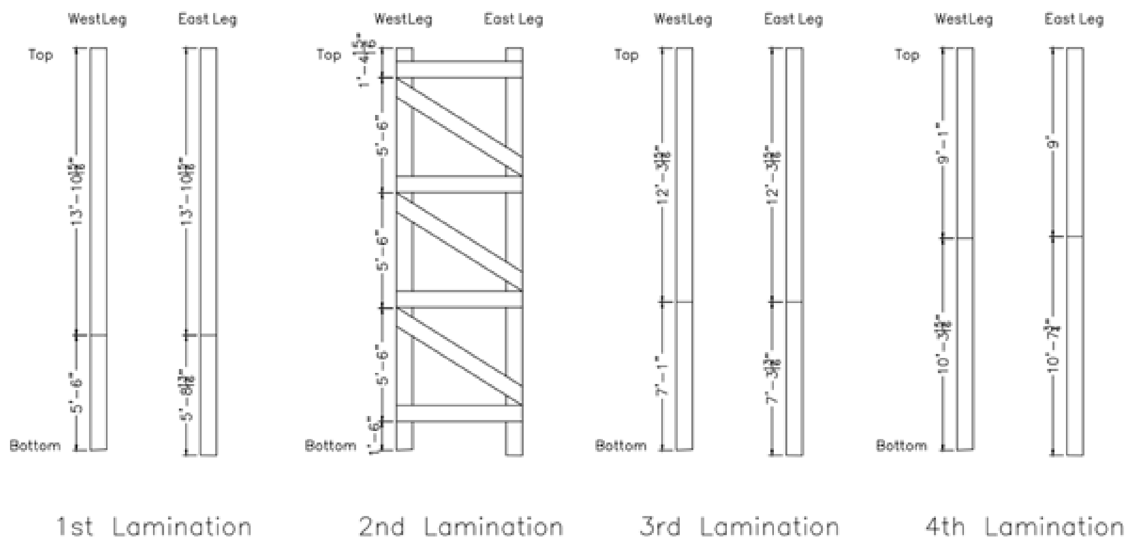


Figure 40: South face of tower C3 lamination components

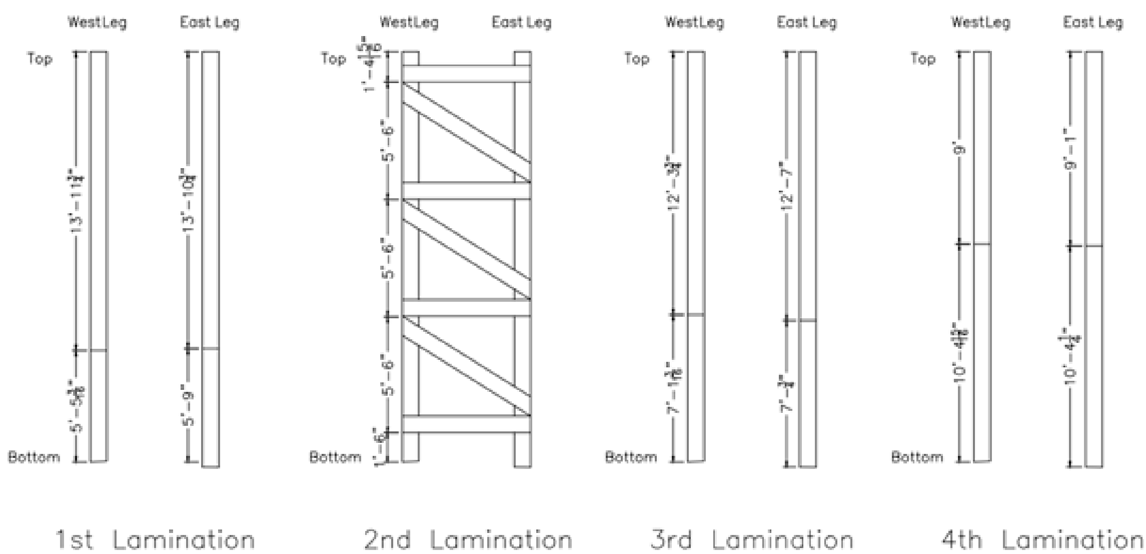


Figure 41: North face of tower C3 lamination components

Figure 42: Disassembled Leg at Bracing from DT3

Figure 43: Gaps in Tower Leg Butt Joints (left) and saw kerfs (right) from DT3

Figure 44: Gap in Tower Leg Butt Joint in DT3

5.1.3 Inadequate nailing

During disassembly of tower legs, ILF documented the size of nails and approximate spacing. It was observed that two rows of 2 ¾ inch ring shank nails were used, with nail spacing varying between 9 and 15 inches (230 and 380 mm). This does not meet NDS or CSA requirements for joining a built up member as nails must be in two rows at 9 inch (230mm) centers (max) with nails penetrating all members, at least ¾ through the farther outer lamination. A nail with length over 5.5 inches (140mm) would be needed to meet this requirement.

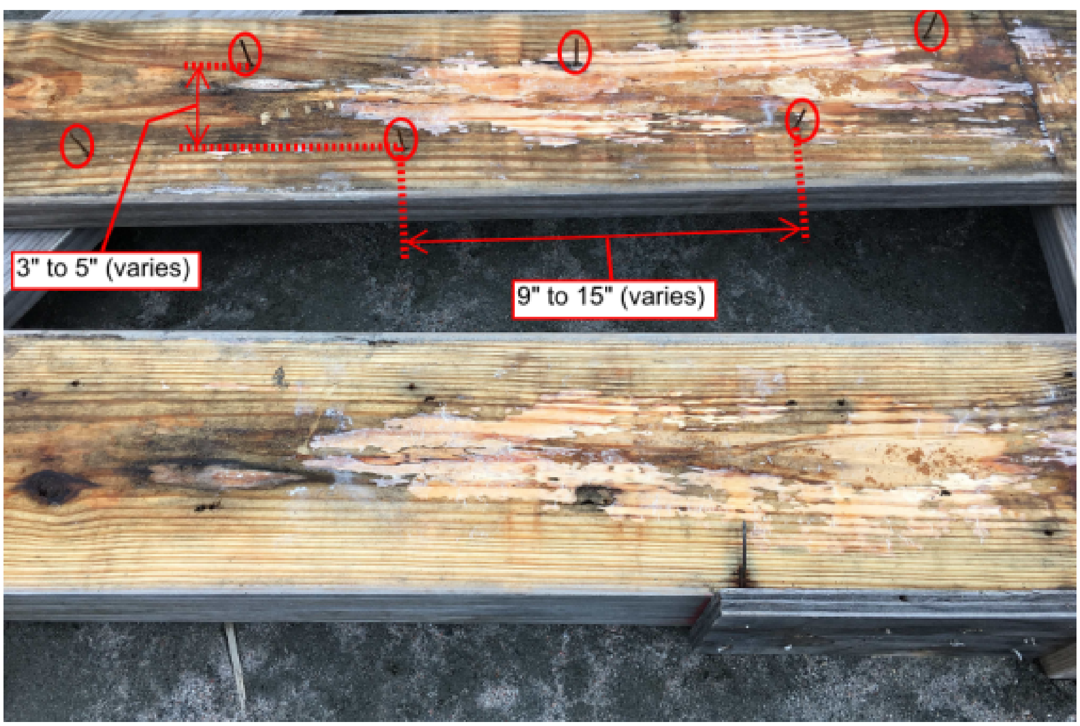
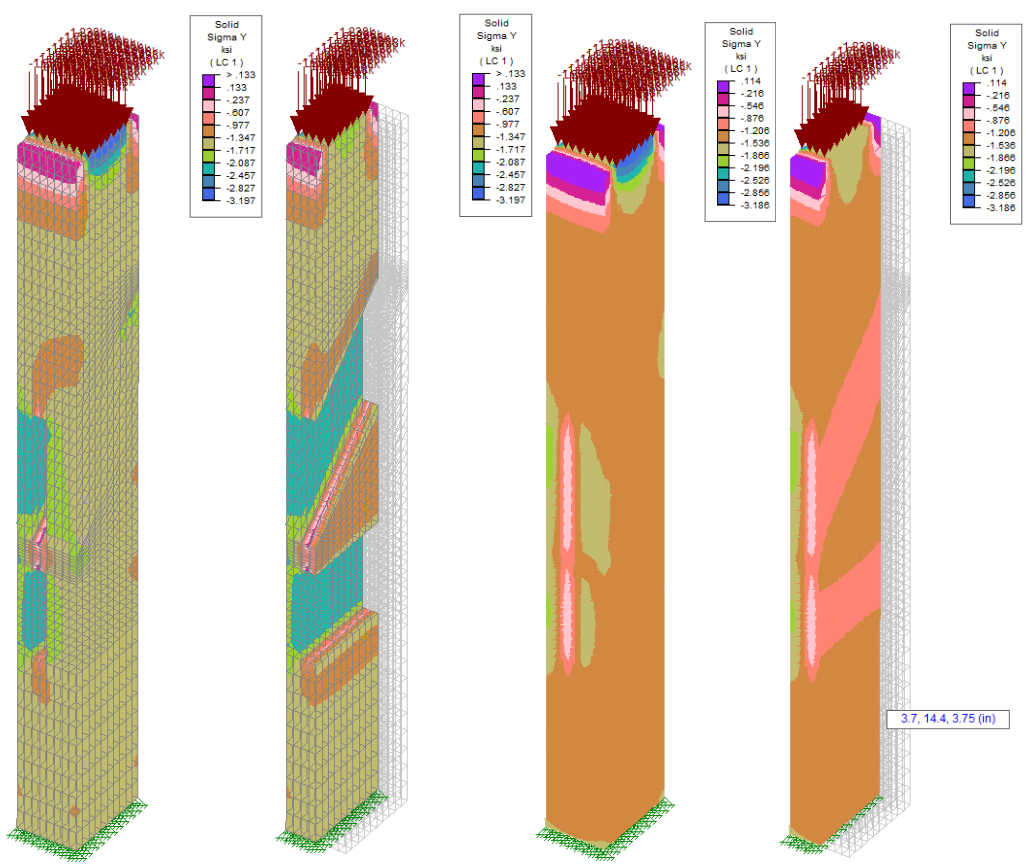


Figure 45: Typical nailing pattern used to build up falsework tower leg member

5.1.4 Improper Bracing

When CEI's Tower C3 from Draft Tube 3 was disassembled, ILF found that the horizontal and diagonal bracing members of the towers penetrated vertical members of the tower legs. This compromised the structural capacity of the composite tower leg section because the compressive strength of wood perpendicular to the wood grain is approximately 30% of compressive strength parallel to the grain. The result of a significantly less-stiff portion of the composite section being sandwiched between stiffer members results in the majority of load transferring to stiffer members. The load sharing by stiffer members to account for a less stiff adjacent members results in potential overstressing of stiffer members. Additionally, in Draft Tubes 1, 3, and 4, ILF observed gaps between falsework leg members that would result in overstressing of adjacent plies of wood. These observations were not considered in CEI's calculation package.

ILF performed 3D structural modeling of the tower legs at brace locations to examine the effects of discontinuities in plys of the built up members. The model consists of solid elements representative of the 4-ply 2x10 leg with a 78 kip (Ref. CEI calculations) axial load applied. The results presented in Figure 46 represent two conditions. Figure 46A and Figure 46B represent a void where the braces penetrate the leg. This assumption comes from the observation of gaps in butt joints (see photographs below) that may not adequately transfer vertical loading. Figure 46C and Figure 46D represent the braces modeled as a weaker material, assuming material properties for wood compressed perpendicular to the grain.



A B C D

Figure 46: 3D model of Tower leg at bracing interface

Both modeling assumptions show significant stress increases at the bracing interface. A vertical stress of 1.4 ksi is developed in the continuous portion of the leg model as would be expected from a simple force per area calculation. At the discontinuities vertical stress increases to 2.5 ksi for the void model and 2.2 ksi for the weaker material model. This is a stress increase of 78% that was not considered in CEI's calculation package. The model is intended to be representative of conditions that should have been explored by CEI during original design, given fabrication methods later used by CEI.

5.1.5 Lack of Bearing Plate at Top of Falsework Towers

Additional 3D models were created by ILF to examine the effects of uneven loading at top of falsework legs. The purpose of this is to examine the effects of tower legs where the plys are not flush at the top and lacked a bearing plate. Figure 48 illustrates a loading condition where two adjacent plys are taller than the others. Figure 49 depicts alternating taller plys. The results of both simulations indicate significant overstressing of portions of the tower leg. Stress concentrations develop at the loading areas and voids caused by the penetrating bracing members. These stress concentrations greatly exceed the nominal (unadjusted design value) Compression Parallel to Grain Design value of 1.85 ksi.

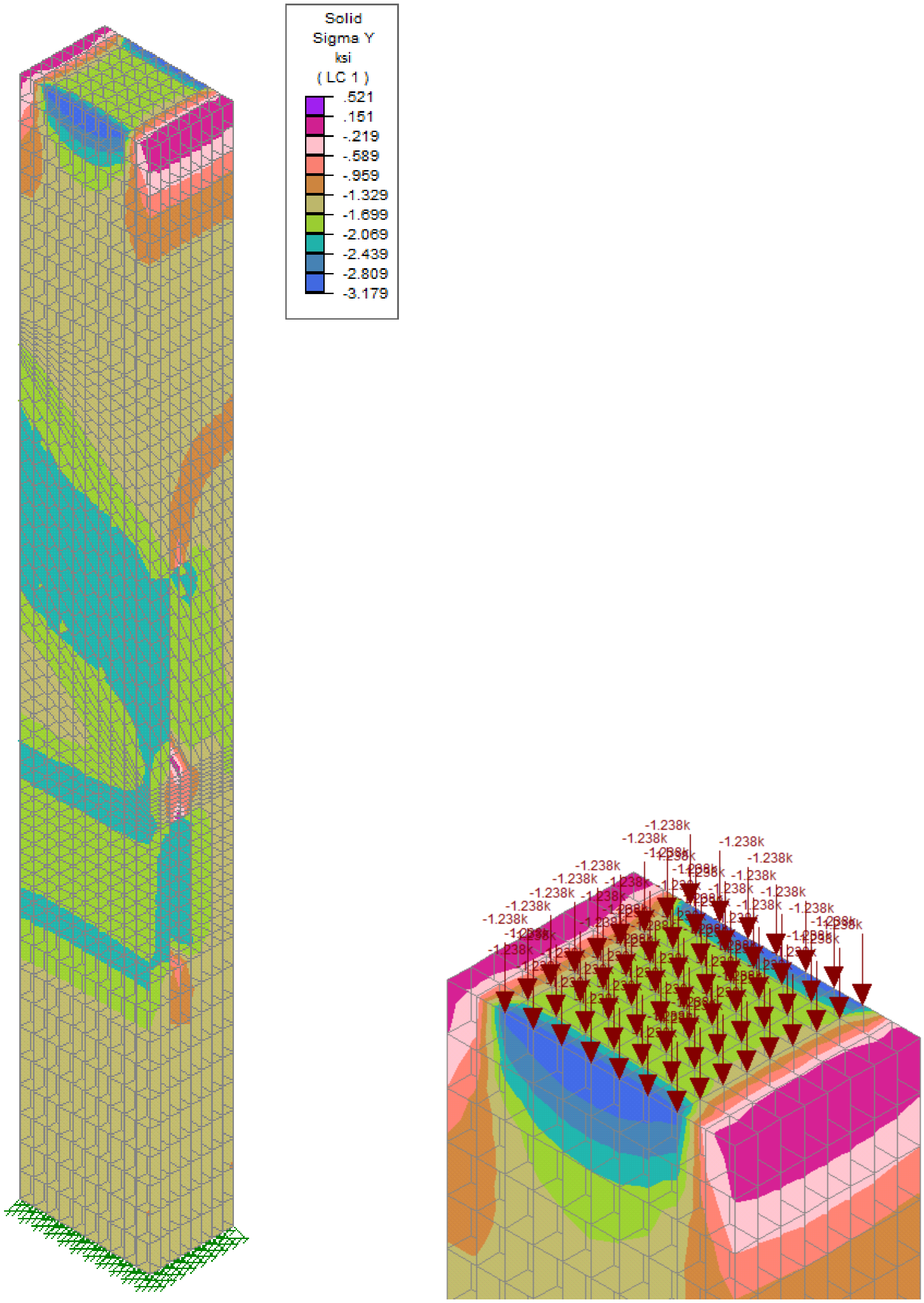
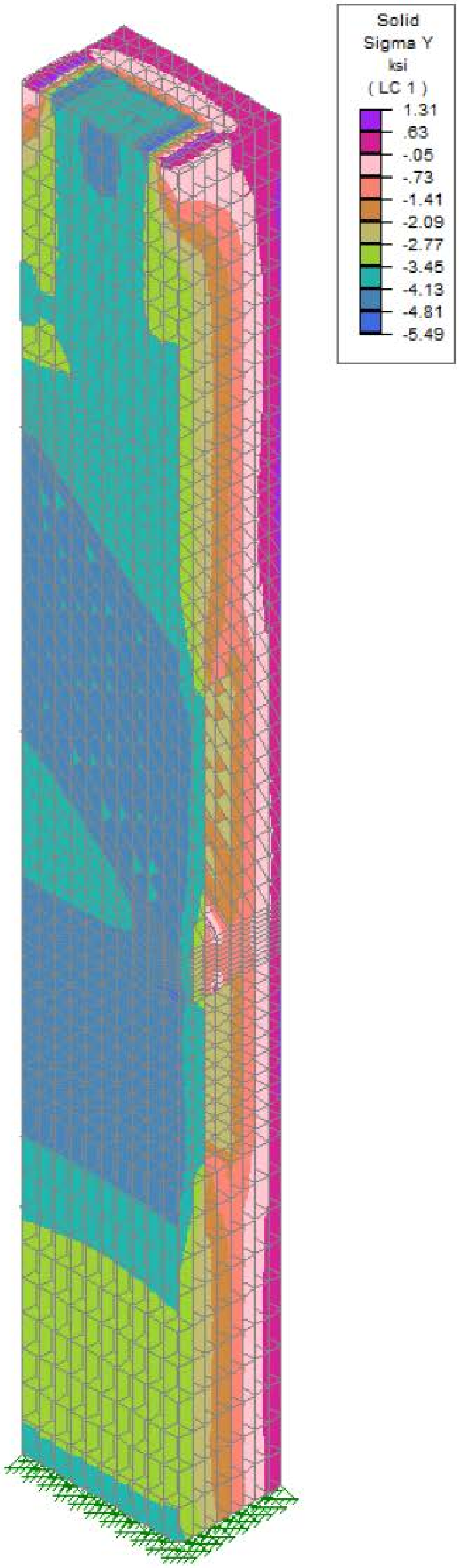


Figure 47: Vertical Stress Analysis - Load applied uniformly over all plies


Solid
Sigma Y
ksi
(LC 1)
1.31
.63
-.05
-.73
-1.41
-2.09
-2.77
-3.45
-4.13
-4.81
-5.49

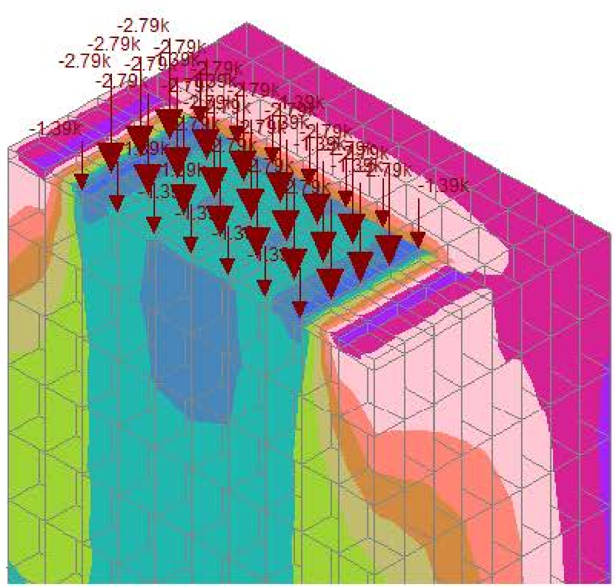

-2.79k
-2.79k
-2.79k
-2.79k
-1.39k
-1.39k
-1.39k

Figure 48: Vertical Stress Analysis – Load applied to two adjacent plys

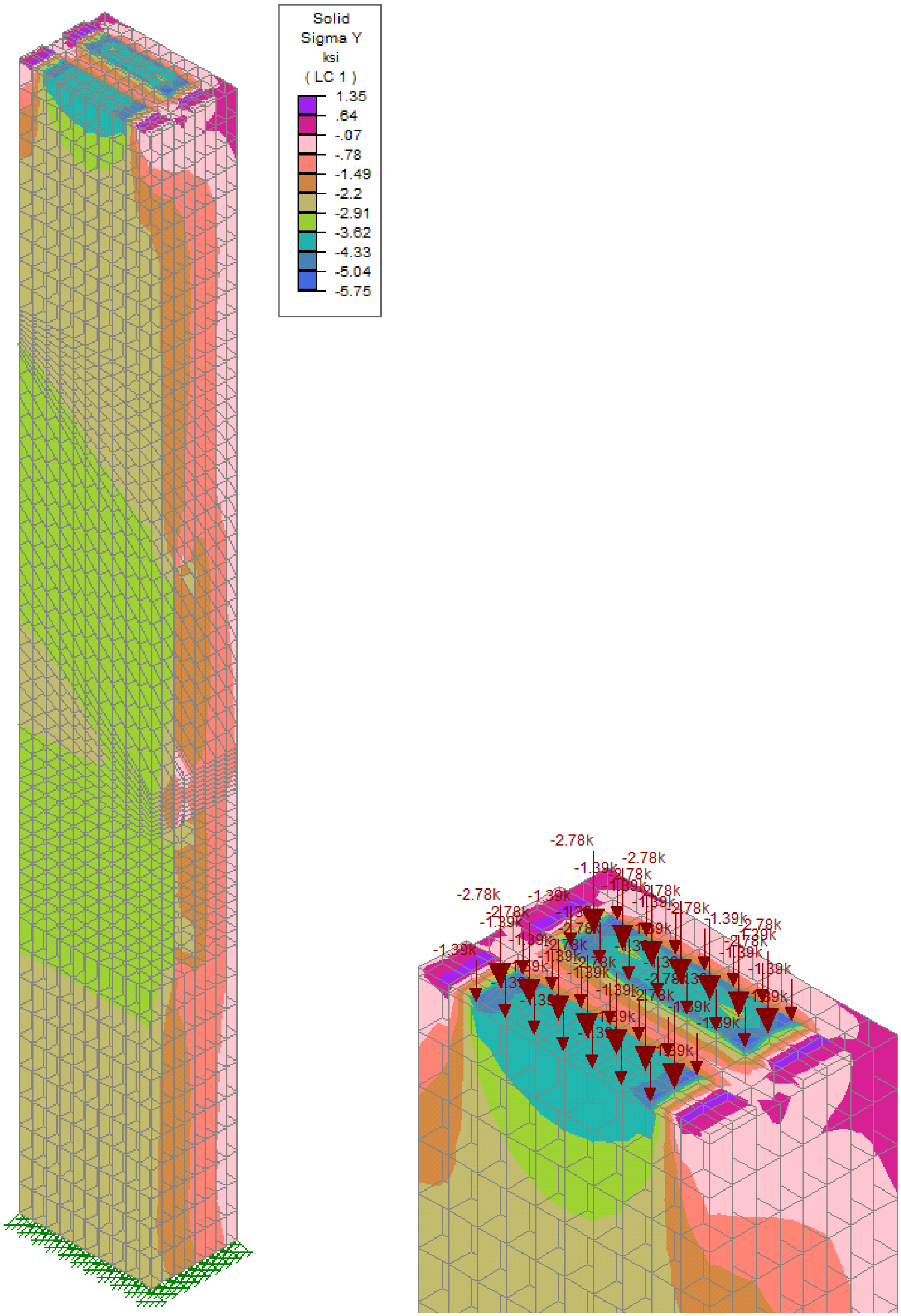


Figure 49: Vertical Stress Analysis – Load applied to two alternate plys

5.1.5.1 Examples of Observed Overstress and Failure in DT1

Figure 50 through Figure 52 indicate the overstress due to lack of bearing plate and penetrating bracing members results in failure of tower legs. The performance of falsework in DT1 is indicative of the types of failure anticipated to have occurred in DT2 prior to collapse. These photographs were taken by ILF in Draft Tube 1 on June 10, 2016.

Figure 50: Local failure of outer ply of tower leg in DT1

Figure 51: Local failure of tower leg at bracing penetration in DT1

Figure 52: Local failure of tower leg at beam interface and bracing penetration in DT1

5.1.5.2 Observed Nailing Deficiencies

Inadequate nailing of splice plates to falsework tower legs was observed. Nailing patterns were random and did not adhere to the typical splice plate detail shown on drawing #MFA-AT-SD-3310-CS-D04-0001-01_C1. See figures below for examples observed in DT2.

Figure 53: Splice plates pulling away from column legs in DT2

Figure 54: Inadequate nailing of splice plate in DT2

It was also observed in a global nature that the nails used to join column laminations into a built-up member did not adequately penetrate the four members that formed a tower leg. CSA O86 and 2012 NDS both identify the requirement for through penetrating nails or bolts, exacerbating the impact of the built up member capacity that CEI did not address in their calculation package. Column members were observed beginning to separate due to inadequate nailing. ILF was not able to identify any instances where built-up falsework tower legs met code requirements for nailing. See Figure 55 and Figure 56 below for examples observed in DT2.

Figure 55: Inadequate joining of built-up member

Figure 56: Inadequate joining of built-up member, members separating

5.1.5.3 Missing Members

There were two (2) 2"x10" lateral supporting members missing that were to be installed in the web of the two steel beams above the falsework towers B1 and B3. The members reduced the unbraced length of the steel beams and provided lateral stability. Their omission reduced the capacity of the steel beams. See Figure 57.

Figure 57: Missing members between beams. DT3 tower B1

The soffit panels in unit 3 were never installed on these shoring towers

5.1.5.4 Inadequate Splice Plate Size

The splice plates used in the ribs overlying the falsework towers were generally too small and did not adhere to the typical splice plate detail shown on drawing MFA-AT-SD-3310-CS-D04-0001-01_C1.

Figure 58: Inadequate splice plate size DT2

5.1.5.5 Improper Staggering of Butt Joints

Butt joints in some column members were not appropriately staggered and were observed to be too close together. See figure below.

Figure 59: Close proximity of butt joints in broken tower leg

5.2 Observed Installation of Formwork and Falsework

5.2.1 Tower Anchors

In Draft Tube 1, 14 of the 16 falsework towers did not use the anchorage detail indicated on drawings MFA-AT-SD-3310-CS-D04-0066-01 through MFA-AT-SD-3310-CS-D04-0066-06. An alternative detail consisting of a polyethylene sheet under a grout pad without anchorage to the concrete invert was utilized. The figure below shows Tower A6 in DT1 with the alternate tower grout pad and CEI designed column anchors uninstalled in the background. The alternate detail reduced sliding resistance of the columns, but was compensated for by additional bracing at the bottom of tower legs. Lateral movement at tower base was not observed in DT1. Based on observations in field, it is anticipated that the alternate anchorage was used in some of DT1 and all of DT's 2, 3, and 4. ILF was not able to identify use of the CEI designed "shoe" in the DT2 rubble and found examples of the alternate on all observed towers. Similarly, the alternate was exclusively observed in DT3 and DT4.

Figure 60: DT1: Uninstalled tower base anchors (blue steel clips) in foreground and piled in background. Alternate grout pad with form installed in foreground left and right.

At the time of the collapse, in unit 3, the tower was only standing on the draft tube invert without anchors

Which of the CEI drawings specified shims?

Figure 61: Alternate grout form at base of falsework tower leg. Recovered from Row C towers DT2

5.2.2 Shims

Shims between the falsework tower columns and the steel beams were poorly installed and a combination of softwood and steel shims was used. CEI specified shims to be installed by Astaldi but did not provide specification for shim material or tolerances. Sound judgement would expect hardwood or steel shims to be used and installed to ensure full bearing under the beam. Poor shimming practices allow for compression of shims which result in settlement of overlying formwork.

5.3 Discussion on Observed deficiencies

In summary the material condition deficiencies and fabrication/construction non-conformities observed in DT2 were as follows:

Material

- varying degree of weathering on shoring towers (minor to major)
- mold growth
- wood decay
- saturated wood

Fabrication

- inadequate nailing of splice plates and column members
- improper sizing of splice plates
- missing bearing plates
- uninstalled/missing members
- localization of butt joints within the falsework tower legs
- Gaps between elements within falsework tower legs
- Saw kerf on structural members
- Uneven tower leg bearing surface

- Multiple color code marking on towers

Erection

- improper use, size, type, and placement of shims
- inadequate nailing of falsework tower splice plates
- alternate grout pad design for falsework towers

Fabrication deficiencies were widespread throughout the falsework and formwork. These included improper bracing of towers (horizontal and diagonal braces interrupt tower leg plys), incorrect staggering of butt joints within the tower legs, insufficient joining of built-up members used in the towers, inadequate splice plate size and/or location in formwork, nonconforming materials, inadequate nailing, multiple color code markings on formwork and falsework, and lack of bearing plate at top of falsework towers.

Erection deficiencies were limited to inadequate nailing of splice plates to falsework tower legs, inadequate shimming, and use of alternate grout pad design for falsework towers.

5.4 Inspection of High Risk Temporary Structures

A pre-pour inspection of the formwork and falsework for pour D2ESB-03 was conducted and approved by an Astaldi Foreman, Field Engineer, QC Representative, and Nalcor representative prior to pouring concrete. The inspection included checking of lumber grade and quality as well as ensuring formwork/falsework conformity to approved shop drawings. No deficiencies were noted on the pre-pour inspection sheet for the failed DT2 pour. Reference Appendix B and Appendix C.

As outlined in Section 5 Fabrication and Installation of Formwork and Falsework, ILF has identified numerous deficiencies/nonconformities that should have been noted in a pre-pour inspection and either corrected or accepted in writing by the engineer of record prior to pouring concrete.

5.5 Astaldi Management of Change Process for Temporary Structures

Per documents reviewed/requested by ILF after the DT2 failure and through conversation with Astaldi engineering staff, it is evident a formal management of change process was not utilized for temporary structures. Site instructions, site queries and and/or requests for information were only used for permanent work in practice and changes to the design of temporary structures were tracked through as-built drawings or revisions to drawings, albeit imperfectly. An instance of the CEI systems being modified without documentation of engineer of record approval was the omission of the steel “shoe” and anchor bolts at tower base, adding the alternate polyethylene sheet bond breaker under tower leg grout pads. It is standard practice to obtain written permission from the engineer of record prior to modifying an engineered system when the modification results in a change to the safe working load of a structure.

6 CARE AND PRESERVATION OF MATERIALS

6.1.1 Specifications for Care and Preservation of Formwork and Falsework

CEI’s specifications for the care and preservation of formwork and falsework are outlined in MFA-AT-SD-3300-CV-A11-0001-01 and MFA-AT-SD-0000-QC-Q03-0014-01_B, CEI Formwork Preservation and CEI Quality Plan respectively. CEI’s formwork preservation document specifies how the panels are to be supported when they are shipped, and it adds that “shielding the panels from direct sunlight, rain, and multiple cycles of high/low temperatures and humidity will help maintain the quality of the panel". CEI's Quality Control Plan also outlines requirements for handling, storage, and preservation of materials. It states that products are to be handled, stored, and preserved in clean, protected environments where periodic inspections are made to verify the integrity of products in storage. Reference Appendix E for CEI's specified care and preservation requirements. Based on Astaldi's audit of CEI quality control processes on December 18, 2014, information documenting periodic inspections was not available. Reference Appendix L for Astaldi's findings.

6.1.2 Observed Storage Conditions Under CEI

During two separate site visits to the CEI facility, Astaldi observed the manufacturing progress of formwork. Photos taken from the visits revealed that some of the formwork and falsework was stored outdoors unprotected. The figures below show progressive weathering of some of the formwork taking place over the course of two months. The storage conditions and length of time in those conditions cannot be verified for all formwork members due to inadequate documentation from CEI. ILF provides Astaldi's warehouse logs for receipt of shipments in Appendix F. These logs indicate when the CEI materials were received by Astaldi.

Standard practice of placing lath between layers of face-to-face lumber was not practiced by CEI, as shown in Figure 62 through Figure 66. The lumber marked "C41" are tower legs stored face-to-face, which does not promote air circulation between layers of tower legs. This practice can encourage damp conditions that are conducive to biological attack.

Figure 62: May 15, 2014. Formwork and falsework at CEI facility. Note fresh, unweathered wood.

Figure 63: July 23, 2014. Formwork and falsework at CEI facility. Note weathering of wood compared to same material in Figure 62.

To facilitate the loading of panels onto trucks for shipment, Astaldi had an expeditor present at the CEI facility. Photos of the panels were taken by the expeditor, but formal inspections of formwork and falsework quality were not performed prior to shipment. Photos at time of shipment indicate weathering of formwork had occurred due to CEI's lack of adherence to their own care and preservation guidelines. The weathering is observed as darkened and grey wood surfaces.

When viewing Figure 64 and Figure 65, reference Figure 62 and Figure 63, noting the progression of weathering over a 15 month period. Also reference Figure 29 in Section 3.3.3, which depicts the same black and green paint markings "C41". ILF believes the towers depicted in Figure 62 through Figure 66 were installed in DT2 at time of the collapse.

Figure 64: June 16, 2015 at CEI yard. Right hand view of falsework towers believed to have been installed in DT2.

Figure 65: June 16, 2015 at CEI yard. Left hand view of falsework towers believed to be installed in DT2

Figure 66: June 16, 2015 DT1 formwork and falsework at CEI yard in Kansas. Note weathering of tower legs relative to formwork module A26.

6.1.3 Observed Condition upon Receipt of Shipments

DT 2 formwork panels were shipped directly to Muskrat Falls between August 28, 2015 and September 24, 2015 as reported in Astaldi's receiving inspection reports, delivery slips, and warehouse logs. Reference Appendix F for this information. ILF has reviewed shipping and receiving documents and have the following observations:

- At the time of this report, receiving inspection reports for formwork panels A27 and C41 were not made available to ILF to review.
- Receiving inspection reports indicate damage to two truckloads (including panels A16, A17, A18, A25, and D41).
- At the time of this report, NCR's for the damaged formwork panels were not made available to ILF for review.
- Formwork was stored on site at C1 Laydown according to warehouse logs provided.

Photos of the shipments were taken once the formwork arrived and was offloaded on site. Reference Figure 67 taken at C1 laydown. Although the photos reviewed by ILF show the formwork stored without protection from the weather (i.e. not tarped), it cannot be verified whether the formwork remained stored in these conditions as records of periodic inspections were not provided to ILF for review. Figure 67 depicts the state of weathering of the falsework tower legs when received at site.

Figure 67: September 14, 2015. DT2 Falsework offloaded on site and stored at C1 laydown

Astaldi did not follow their NCR process for correcting damages to formwork noted upon receipt or on site. NCR's were not opened to track the repair of observed damage. Additionally, ILF cannot verify if observed damages were repaired prior to putting the damaged modules in service as we did not receive inspection reports stating such.

Astaldi also did not practice the care and handling of the material recommendations provided by CEI. This was evident after installation when falsework towers were submerged in standing water in the draft tubes, as observed by ILF in June.

Astaldi and Nalcor inspectors did not identify weathered and decayed wood in their pre-pour inspections in Draft Tube Units 1 and 2. ILF would expect weathering and decay of the extents observed in June 2, 2016 to have been documented by the inspectors and either corrected or accepted in writing by the engineer of record prior to pouring concrete.

6.2 Care and Preservation of Materials Summary

Materials used for the construction of the temporary structures were not cared for in conformance to CEI specifications. CEI's quality control process was not followed at their own fabrication facility as discovered by the Astaldi audit carried out in December 2014. CEI's own care and handling of materials recommendations were not followed by CEI. Formwork and falsework was exposed to the elements over multiple months, resulting in weathering of structural members. No inspections by CEI were documented while the formwork and falsework was in their custody. Astaldi audit of CEI's quality control processes during fabrication found CEI was not able to produce documentation on quality control and periodic inspections of the formwork.

During shipment to the project site, some formwork was damaged and records of NCR's being generated or completed by Astaldi were not provided to ILF documenting repair to the damaged members. Once formwork and falsework was received by Astaldi, the temporary structures were not stored according to CEI requirements.

CONSULTING ENGINEERS

ILF CONSULTANTS, INC.
Telephone: (425)209-4989

Project:	N289 Muskrat Falls	Designed By:	GCE	Date:	2/22/2017
Subject:	Falsework Tower Calculation for Full Lift 3 and 50% Lift 4 Concrete Loads	Checked By:	TCW	Sheet	of

Compression Resistance of Built-up Column *NDS 15.3*

Nailed Built-up ==> 4 plies 2x10 SP No 1 Lumber

Nominal Design Values Table 4A NDS Supplement

These are Southern Pine AWS-NDS 2005 data for 2x4 lumber

F_c = 1850 psi - Compression perpendicular to the grain

E= 1,700,000 psi – Modulus of Elasticity

For 2x10
From AWS-NDS 2015
Fc= 1 450 psi
E= 1 600 000 psi
Emin= 580 000 psi
From AWS-NDS 2012
Fc= 1 600 psi
E= 1 700 000 psi
Emin= 620 000 psi
From AWS-NDS 2005
Fc= 1 600 psi
E= 1 700 000 psi
Emin= 620 000 psi

Strength Adjustment Factors

$F_c' = F_c \times C_D \times C_M \times C_t \times C_F \times C_i \times C_p$ .1 NDS

Load Duration Factor

C_D = 1.25 Seven day construction load .2 NDS

CSA specifies normal duration for wooden post Kd=1
From AWS-NDS 2005
Seven Day Loads Cd=1.25
Two Month Loads Cd=1.15
Normal Loading Cd=1
Please explain your factored selection?

Wet Service Factor

C_M = 0.8 Wet service conditions

Temperature Factor

C_t = 1.0 Wet with T≤100º Table 2.3.3 NDS

Per CSA code CEI could have designed for dry service conditions. Cm=1
Is this based on actual site conditions?

Size Factor

C_F = 1.0 Eqn. 4.3-1 NDS

Incising Factor

C_i = 1.0 No incising Table 4.3.8 NDS

Column Stability Factor

$$C_p = K_f \left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \right]$$

Eqn 15.3-1

K_e = 1.0 Table G1 NDS

CONSULTING ENGINEERS

ILF CONSULTANTS, INC.
Telephone: (425)209-4989

Project:	N289 Muskrat Falls	Designed By:	GCE	Date:	2/22/2017
Subject:	Falsework Tower Calculation for Full Lift 3 and 50% Lift 4 Concrete Loads	Checked By:	TCW	Sheet	of

$l_e = K_e$ l=1.0 x 72 in = 72 in

F_c*=1850 psi x 1.25 x 0.8 = 1850 psi

Emin should be used

$F_{cE} = \frac{0.822 E_{min}'}{(\ell_e/d)^2}$ =(0.822 x 1700000 psi)/(72 in /6 in)^2=9704

K_f = 0.6

c = 0.8 for sawn lumber

Emin should be 620 000 psi
See comment above

C_p = .574 Column Stability Factor

Adjusted Strength

1850psi * 1.25 * 0.8 * 0.574 = 1061.9 psi
please confirm

F_c' = 1850 psi x 1.25 x 0.8 x 1.0 x 1.0 x 1.0 x 0.574 = 948 psi

Tower Capacity = 58935 lbf

What is the surface area utilized in this calculation?
(4) 2x10 ply = 4 * 1.5in * 9.25in = 55.5 in^2

58 935 lbf / 948 psi = 62.2 in^2

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

Date: April 20, 2017

Prepared by:
Mathieu Légaré, P.Eng
Construction Engineer, aDB Engineering

Sean Dingley, P.Eng
Principal, aDB Engineering

LICENSED PROFESSIONAL ENGINEER
08578
MATHIEU LÉGARÉ
2017-04-20
SIGNATURE
DATE
NEWFOUNDLAND & LABRADOR

LICENSED PROFESSIONAL ENGINEER
08577
SEAN DINGLEY
SIGNATURE
Apr. 20, 2017
DATE
NEWFOUNDLAND & LABRADOR

Sean Dingley Digitally signed by Sean Dingley Date: 2017.04.20 13:31:17-07'00'

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador PROFESSIONAL ENGINEERS AND GEOSCIENTISTS
PERMIT HOLDER
This Permit Allows
ADB STRUCTURAL ENGINEERING INC.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL 0454 which is vaild for the year 2017

aDB ENGINEERING
A DINGLEY BOETTCHER COMPANY

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.

aDB ENGINEERING
A DINGLEY BOETTCHER COMPANY

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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.

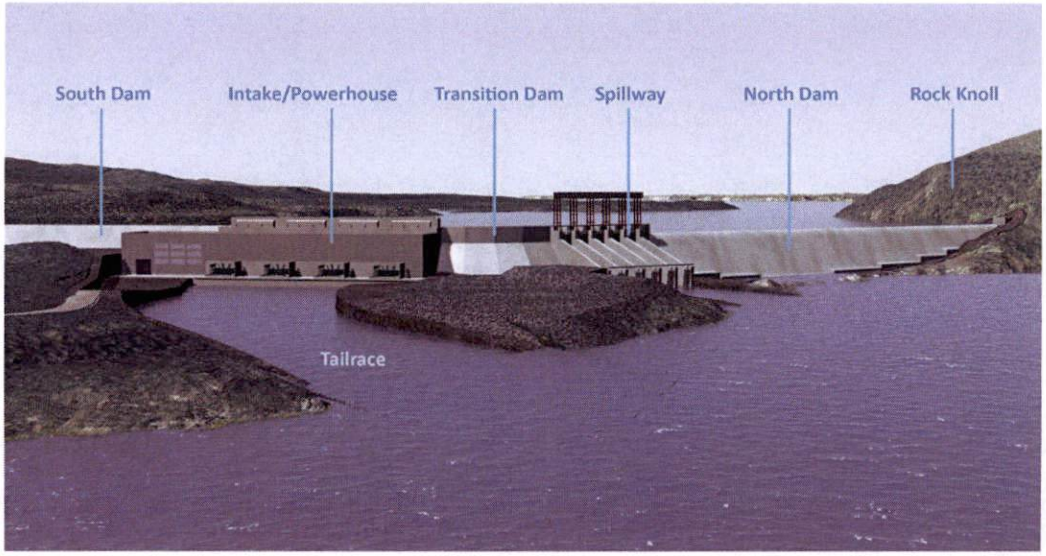


Figure 1: Muskrat Falls Generating Project

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 $530m^3$. The collapse happened near the end of the pour, with only $4m^3$ 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.

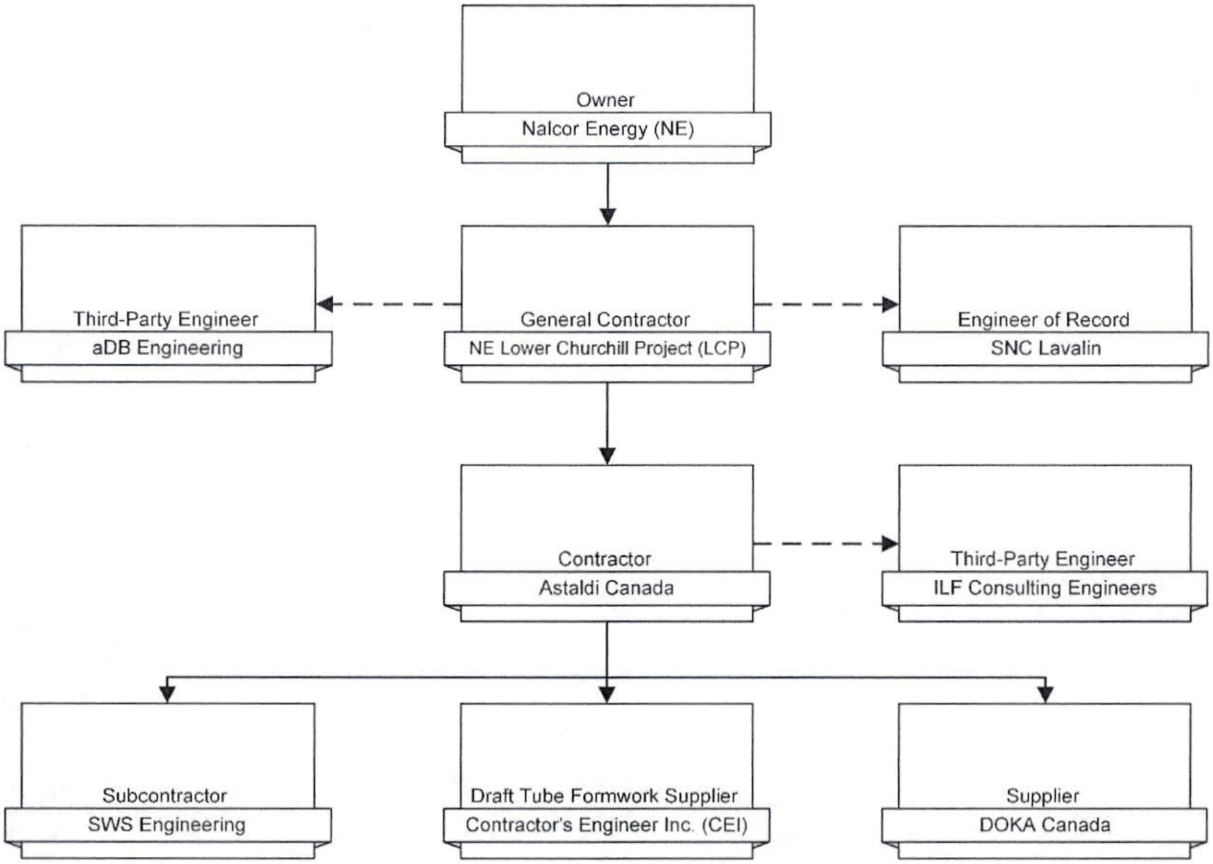


Figure 3: Project organizational chart

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

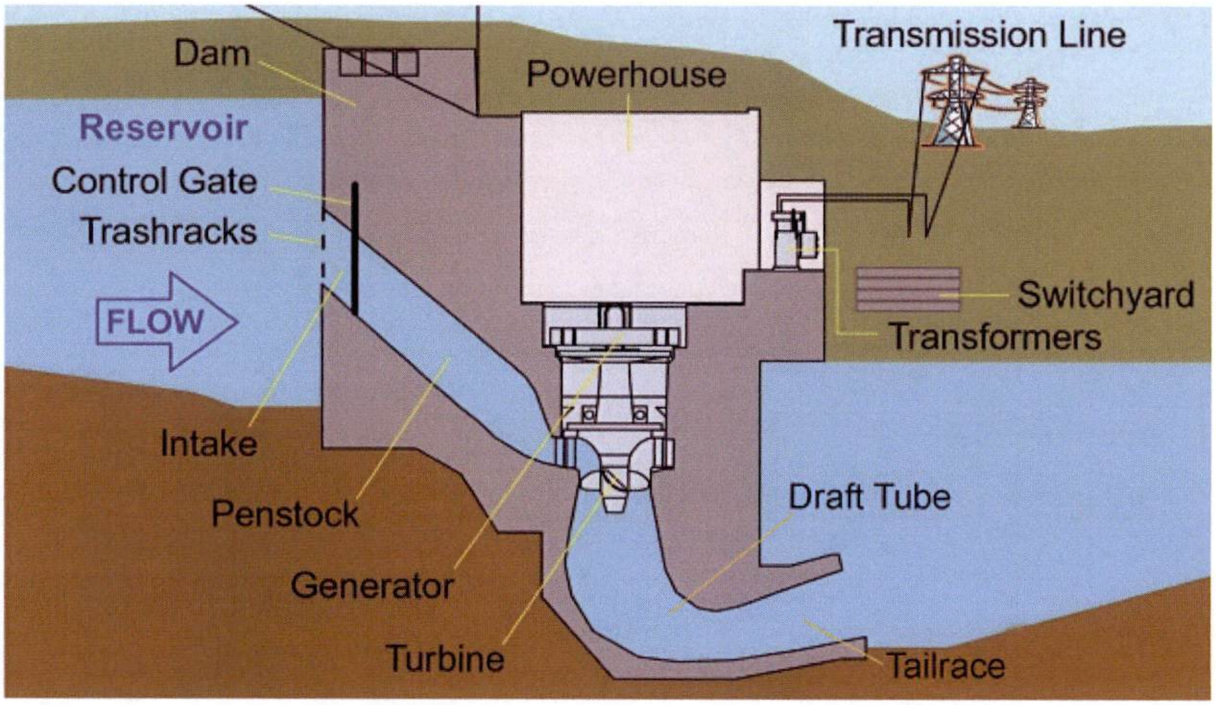


Figure 4:Cross-section of the powerhouse with draft tube below the turbine

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 m^3. The pour started at 10:00 AM, and the collapse occurred at approximately 11:55 PM, with 4 m^3 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.

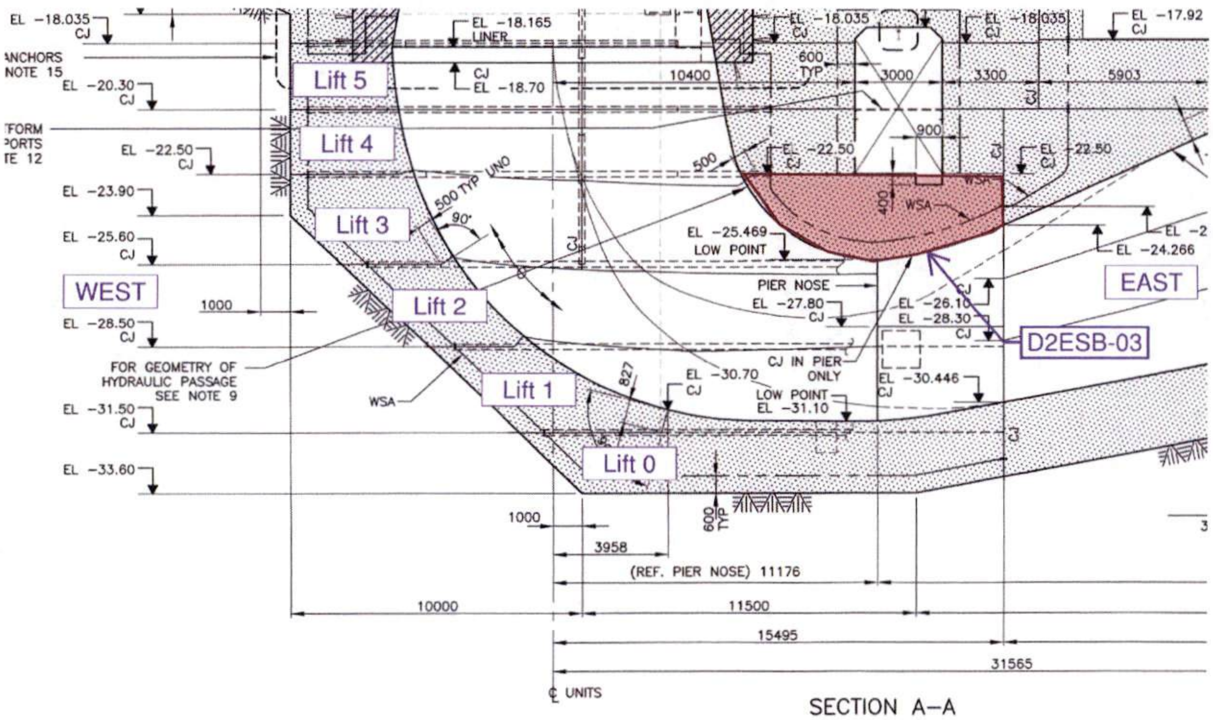


Figure 5: Typical cross-section of the draft tube

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 m^3 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.

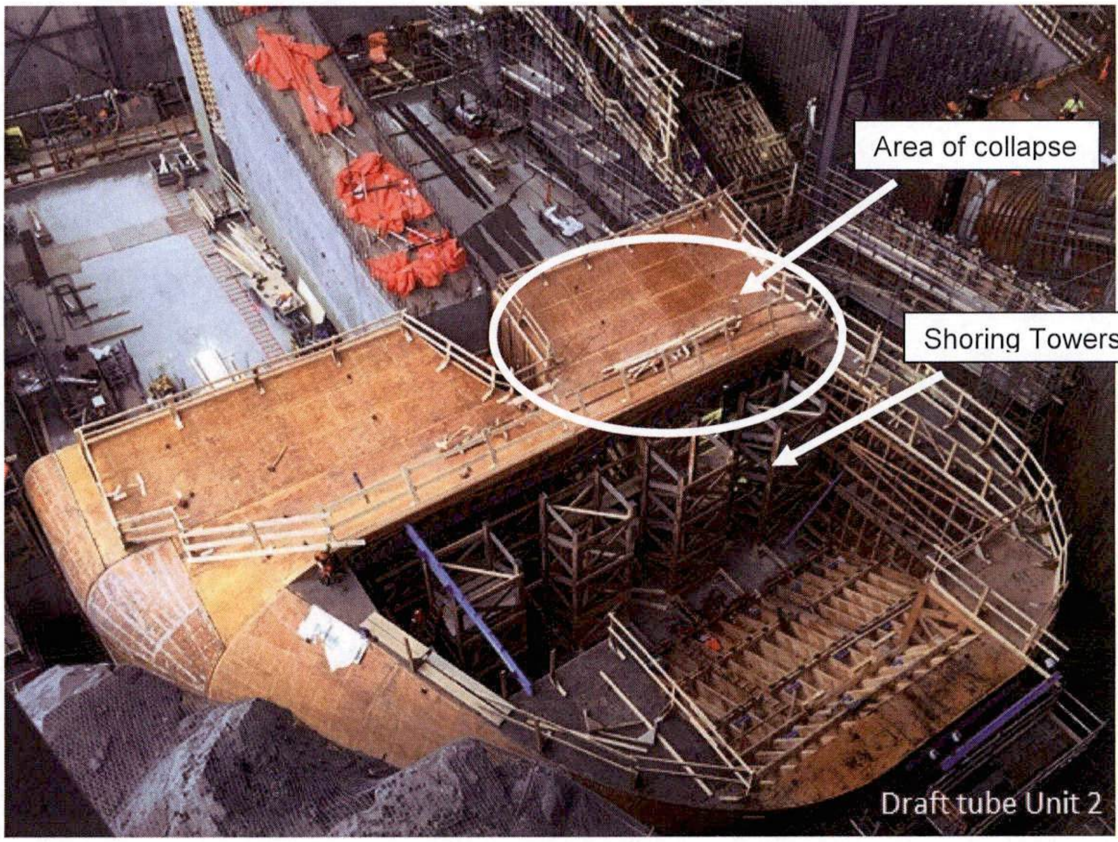


Figure 6: Unit 2 draft tube on October 10, 2015

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.

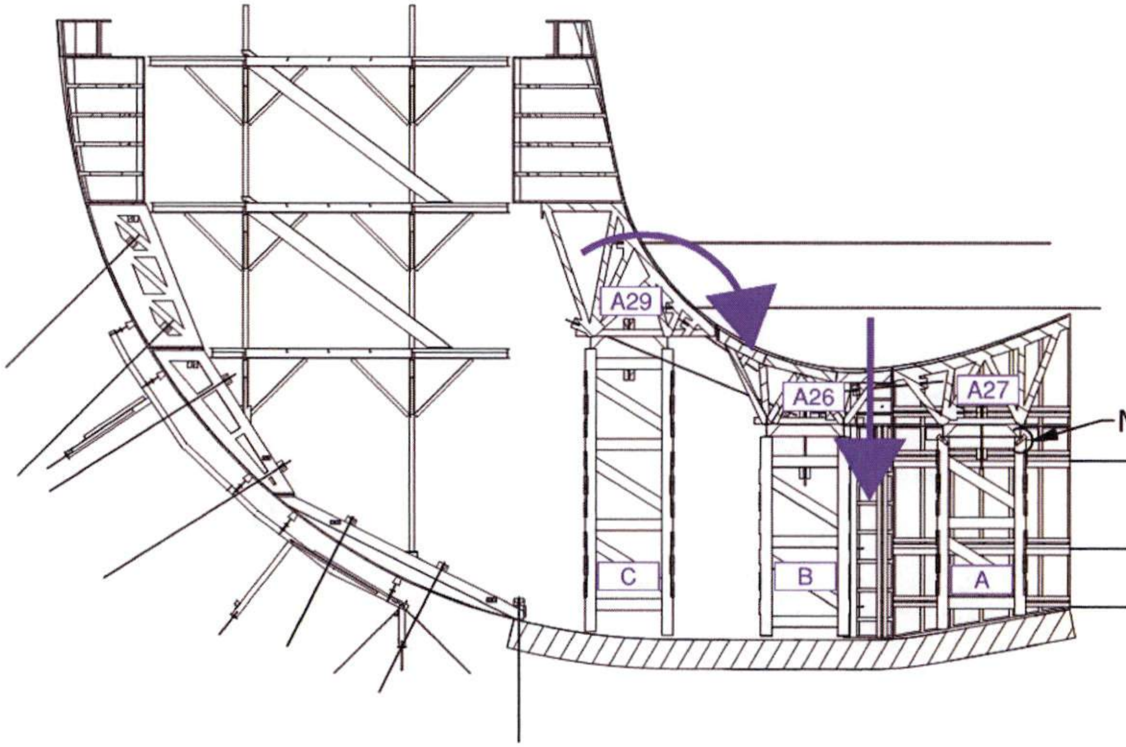


Figure 7: Section view of the draft tube formwork with assumed collapse sequence

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.

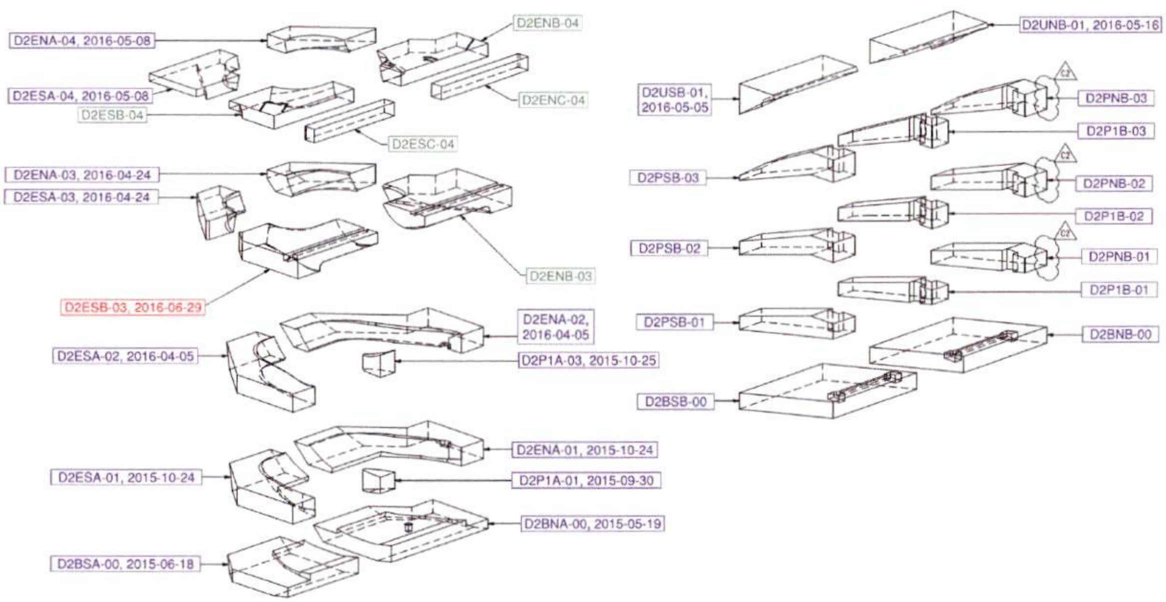


Figure 8: Exploded view of Unit 2 with scheduled pour dates

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 6th Edition
APA	The Engineered Wood Association
AISC	Steel Construction Manual 9th 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/ft^3.

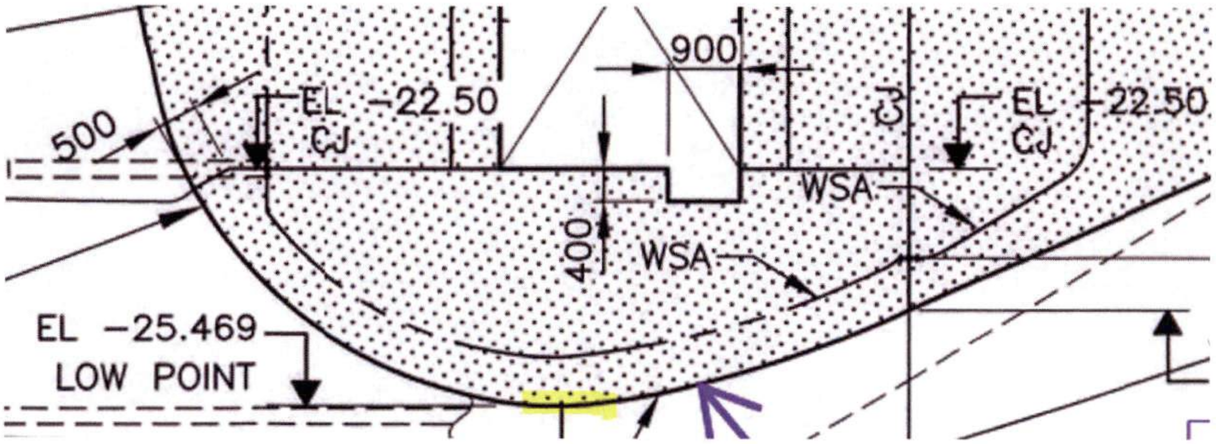


Figure 9: Design concrete pressure based on lift 3 thickness

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.

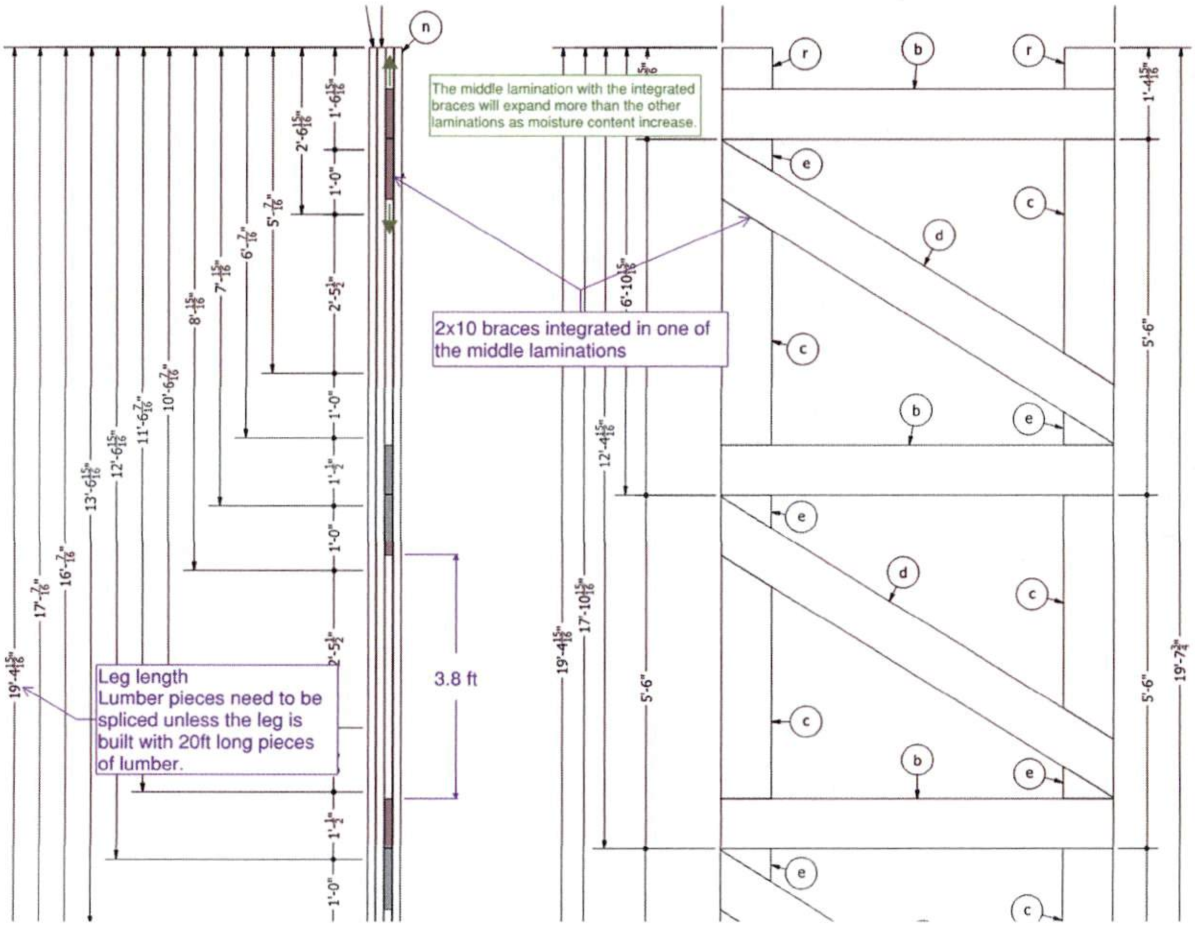


Figure 10: Shoring tower braces assembly detail

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.

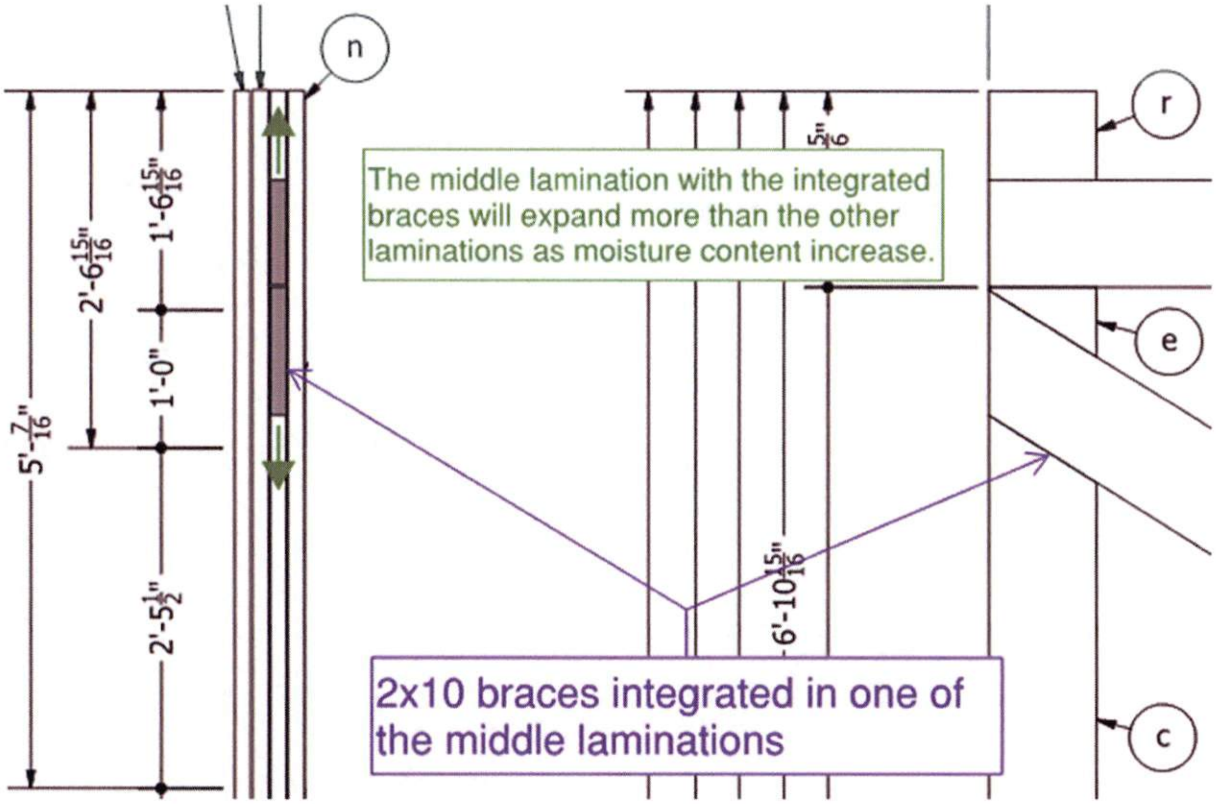


Figure 11: Shoring tower braces assembly detail, drawing "W-41b, Rev A – Shoring Tower C41"

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.

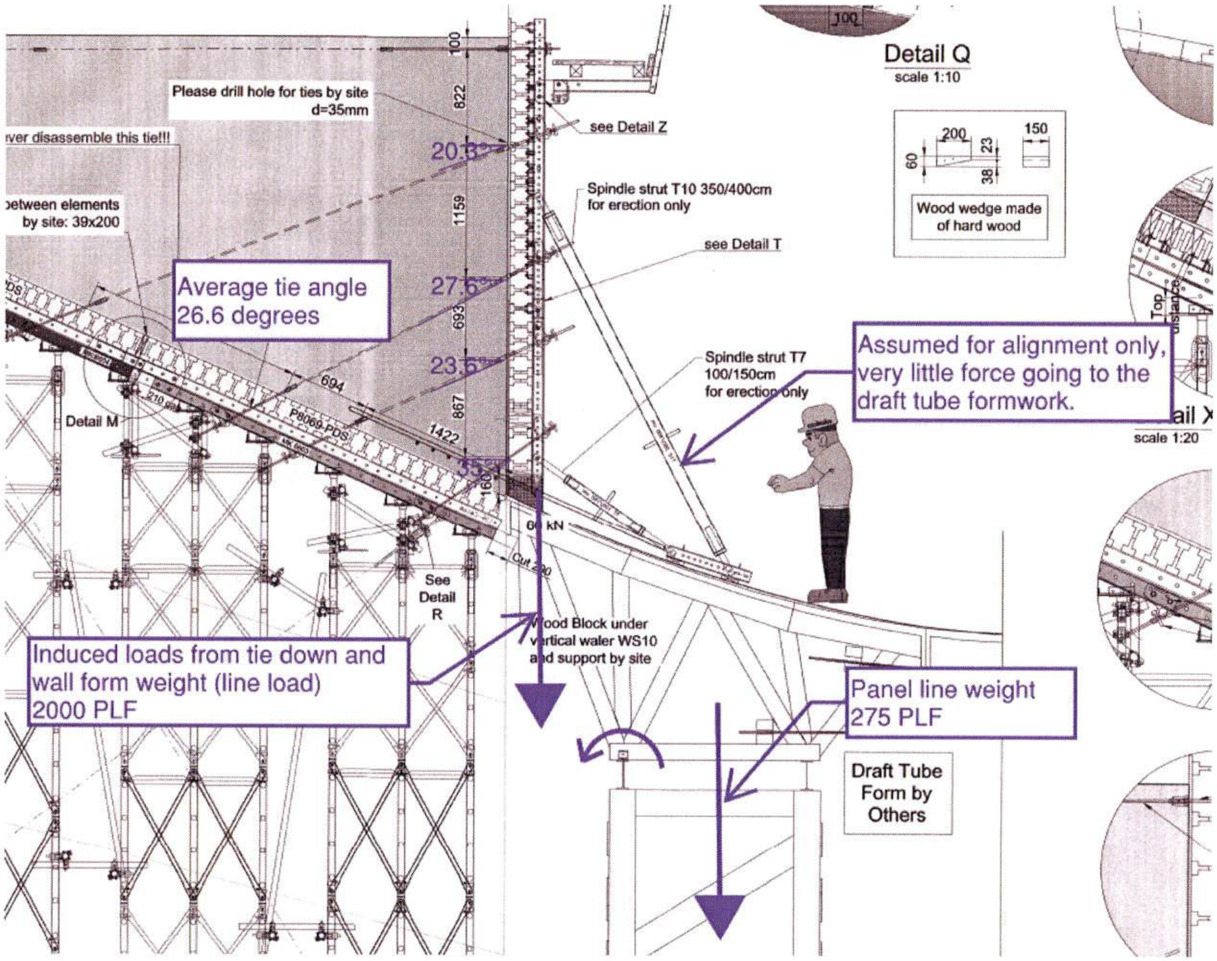


Figure 12: Tailrace wall-form sitting on the draft tube formwork

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 repellant 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 on 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.

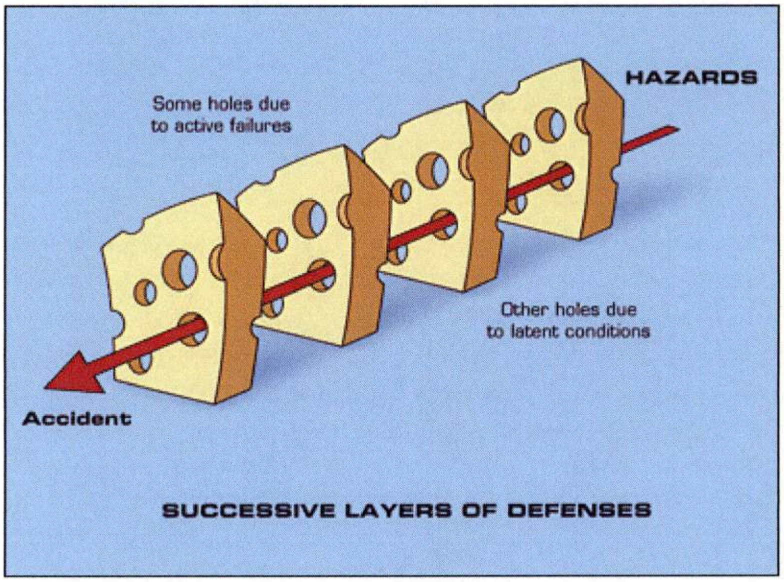


Figure 13: James Reason's "Swiss Cheese Model" per *Human Error* (1990)

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.

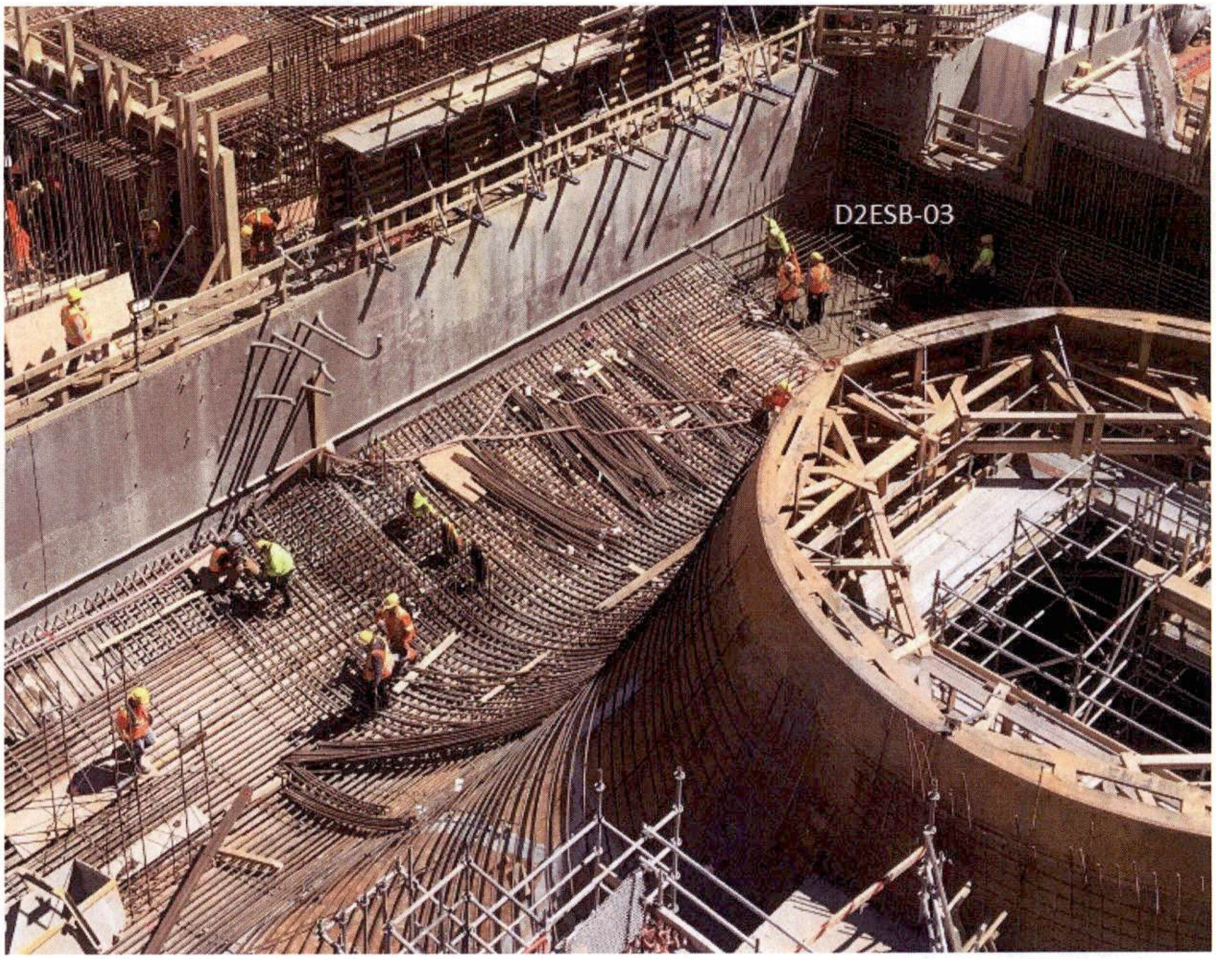
Mathieu Légaré, P.Eng
Construction Engineer, aDB Engineering

Sean Dingley, P.Eng
Principal, aDB Engineering

Appendix

TABLE OF CONTENTS

5 INCIDENT DESCRIPTION

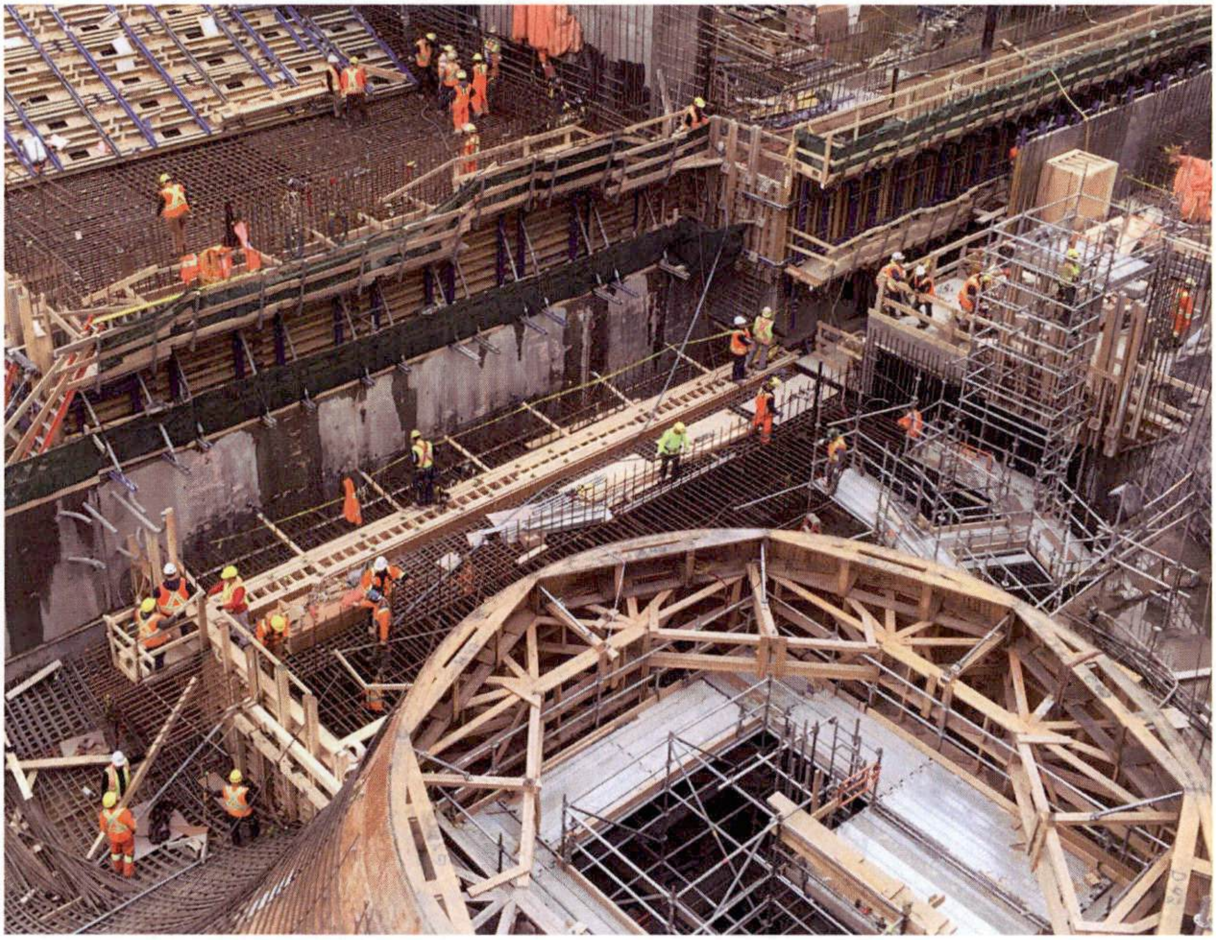


Unit 2 draft tube - View from the North West corner of unit, looking below

Picture from Nalcor

Taken May 22, 2016

aDB ENGINEERING
A DINGLEY BOETTCHER COMPANY



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

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Unit 1 - Decayed wood on tower B3

Picture from aDB

Taken June 16, 2016

aDB ENGINEERING
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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

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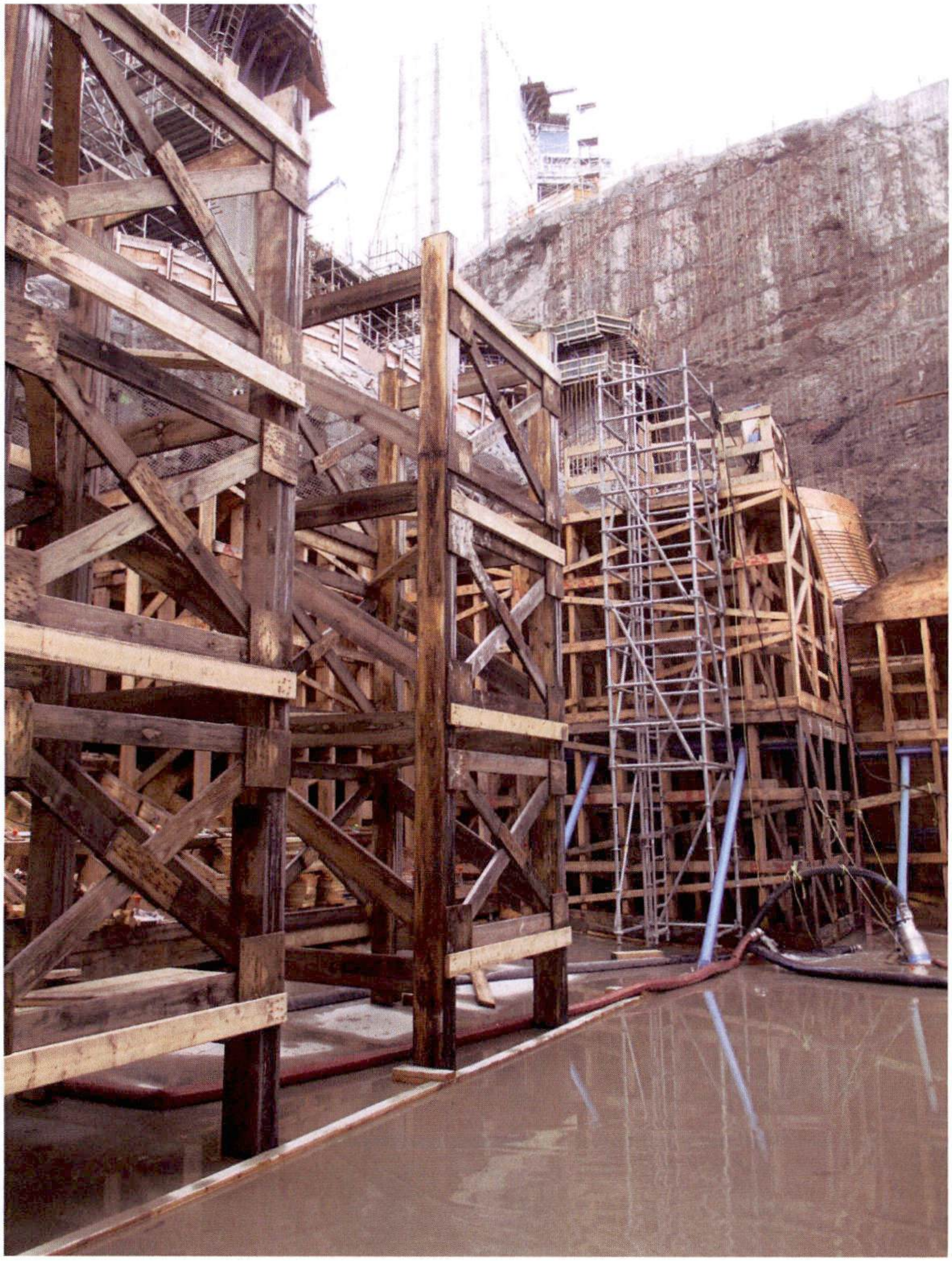
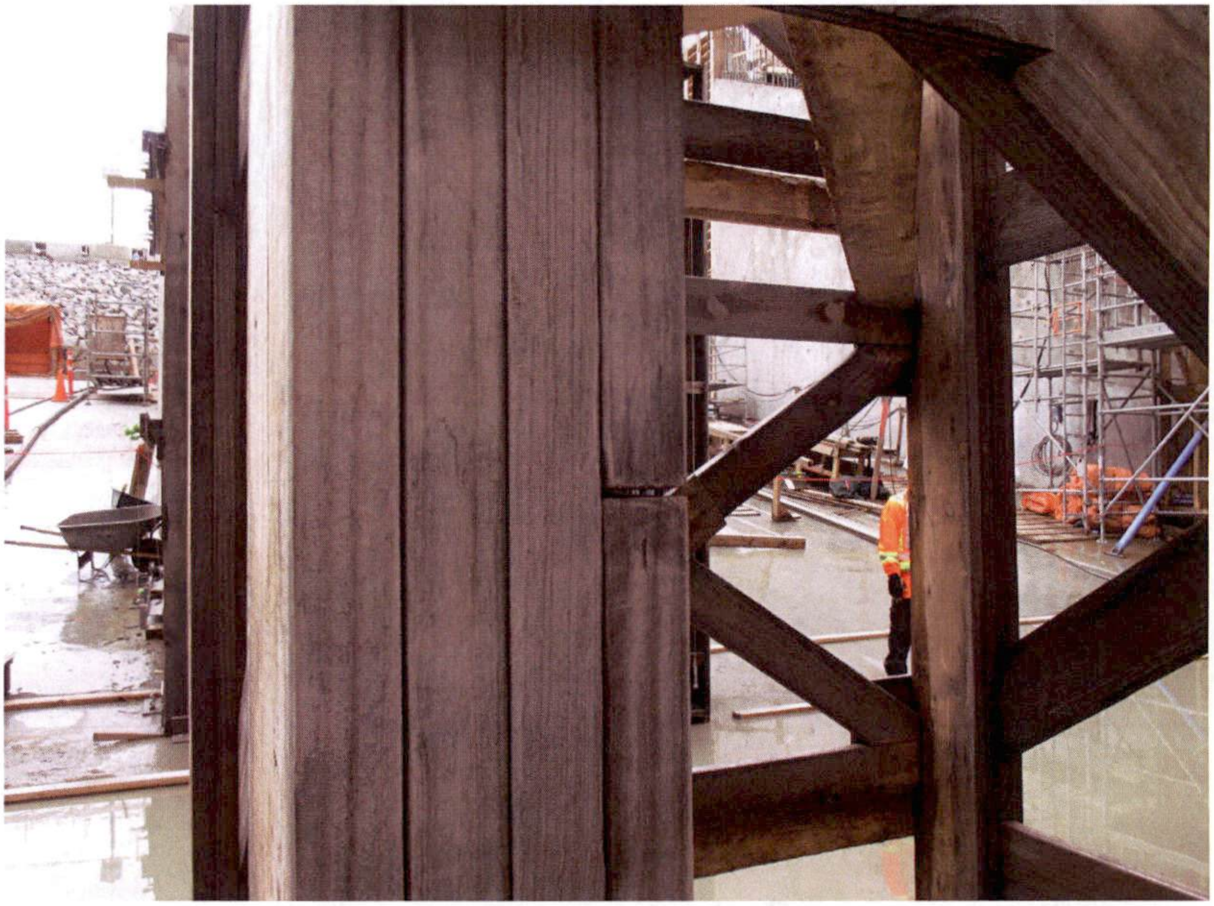


Unit 3 - Installed shoring tower weathering

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

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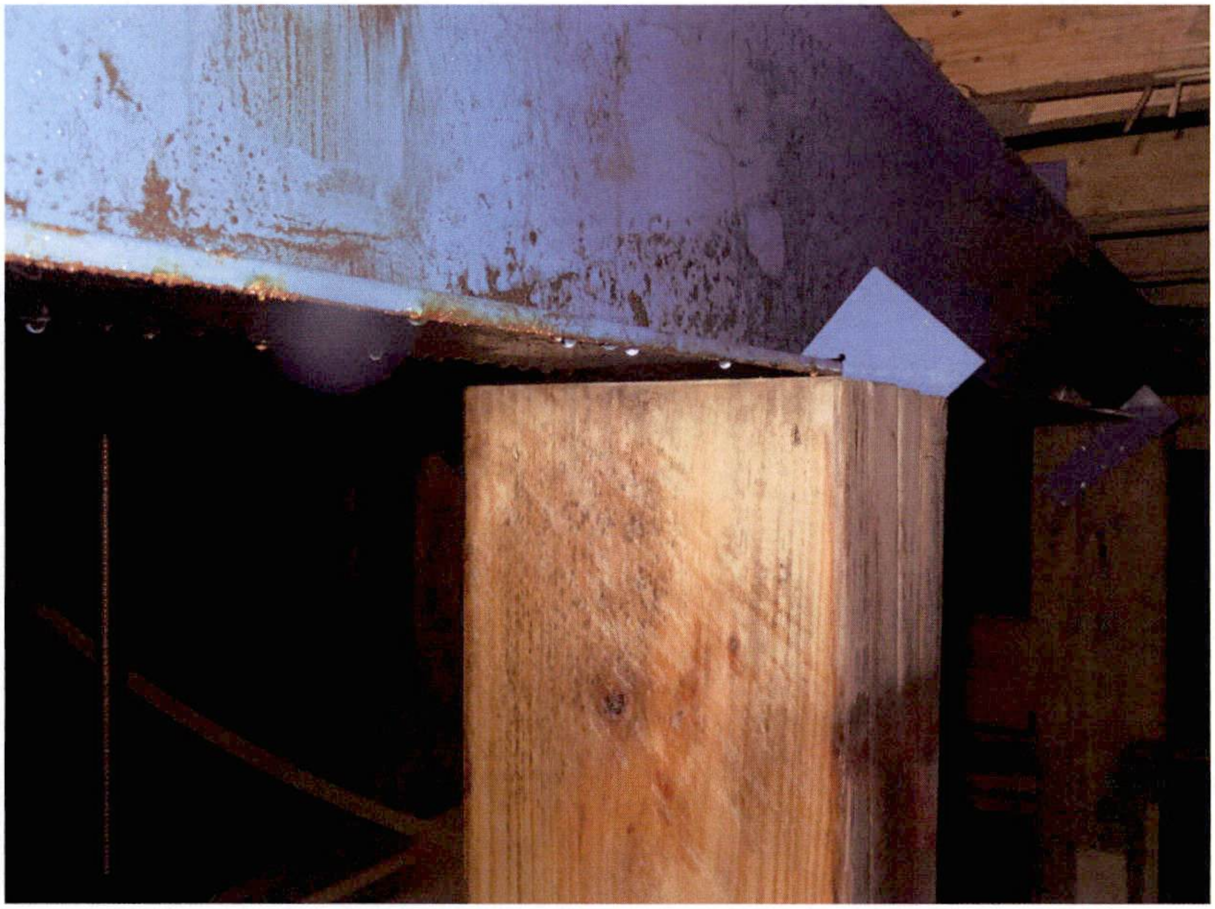
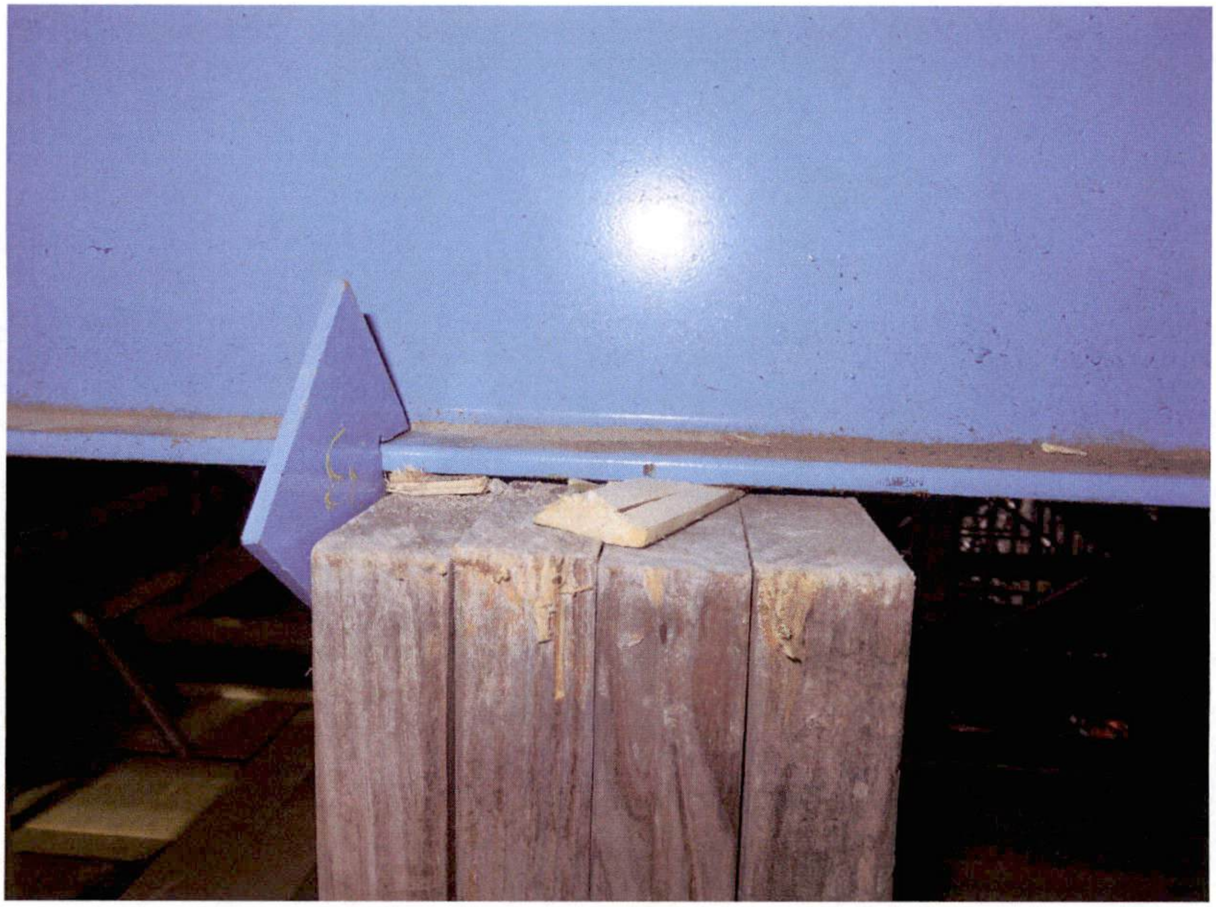


Unit 4 - Missing shim between post and steel beam

Picture from aDB

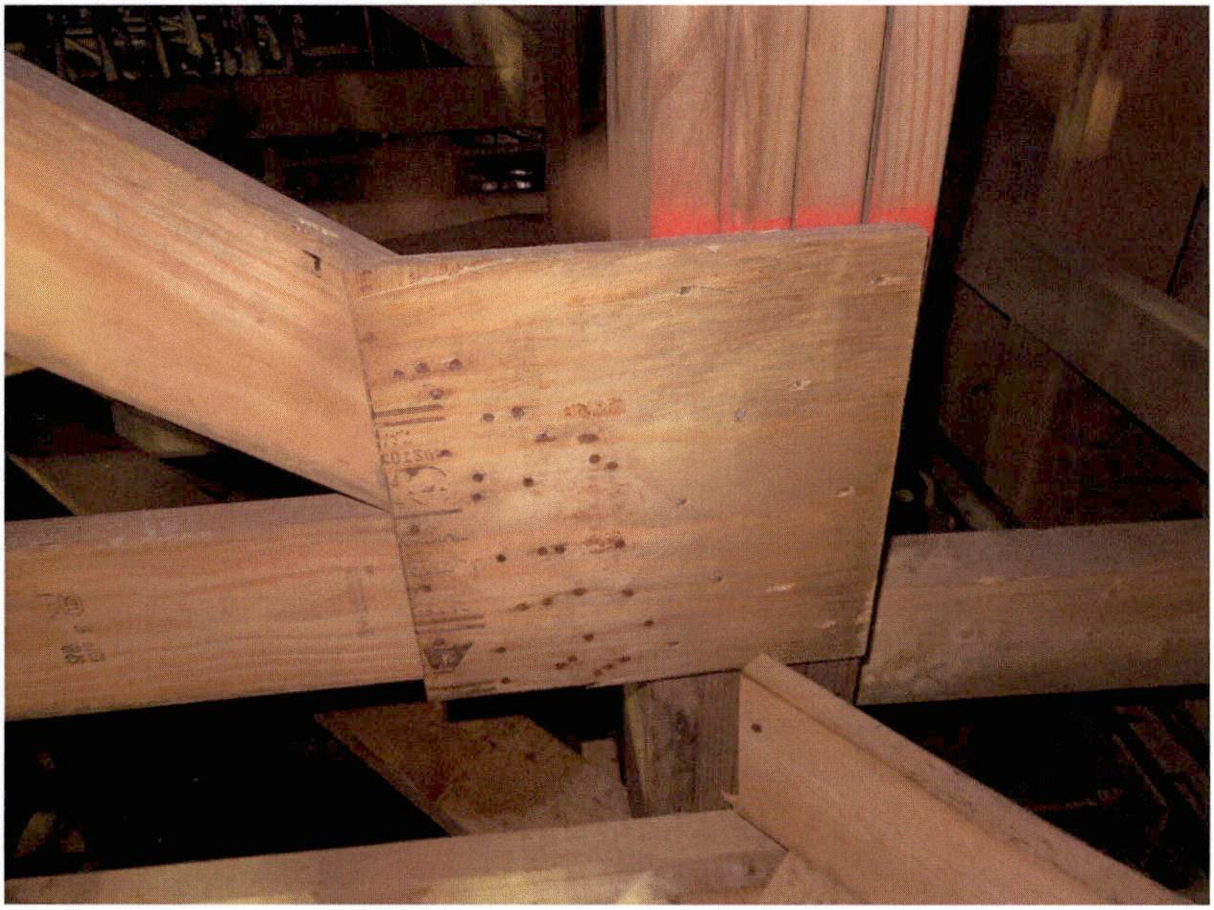
Taken June 15, 2016



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

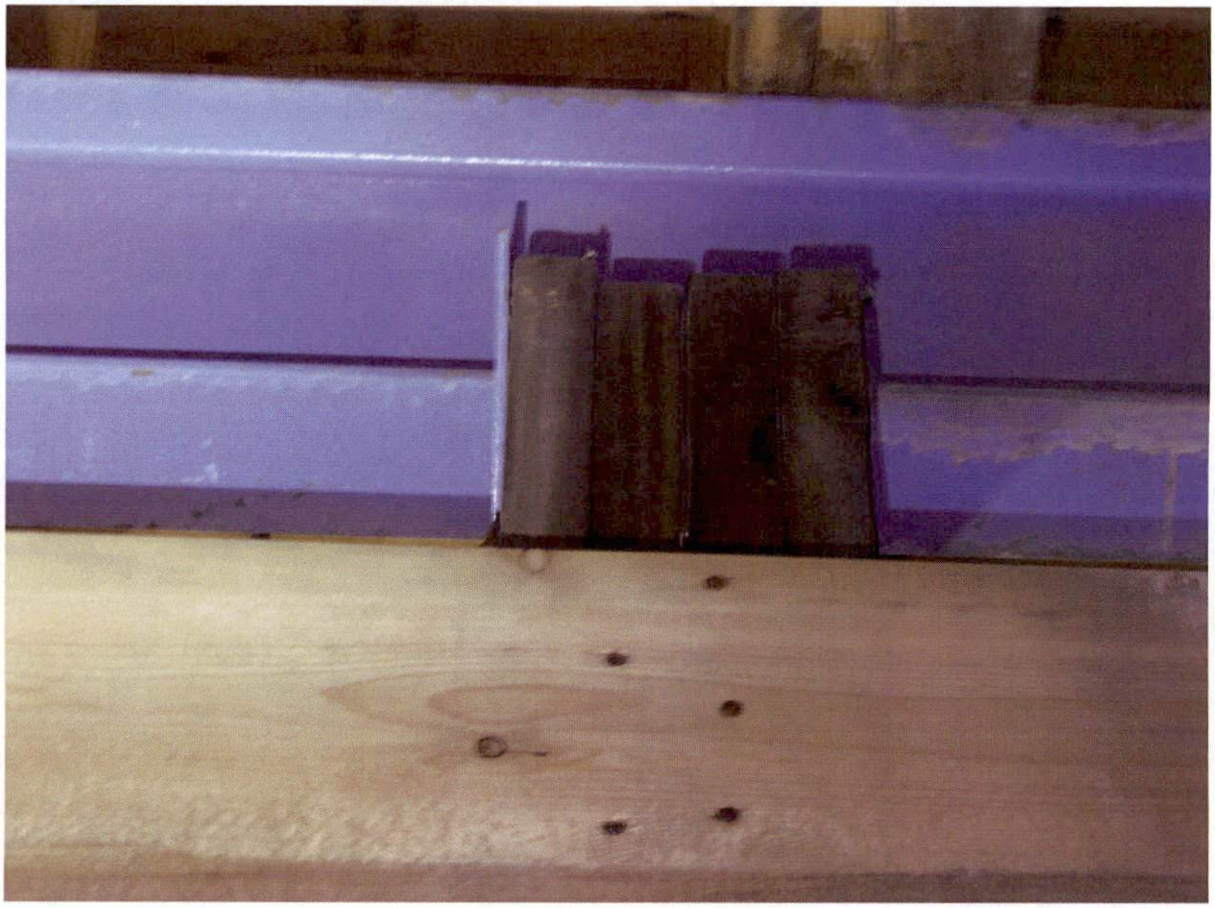
Taken June 15, 2016

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

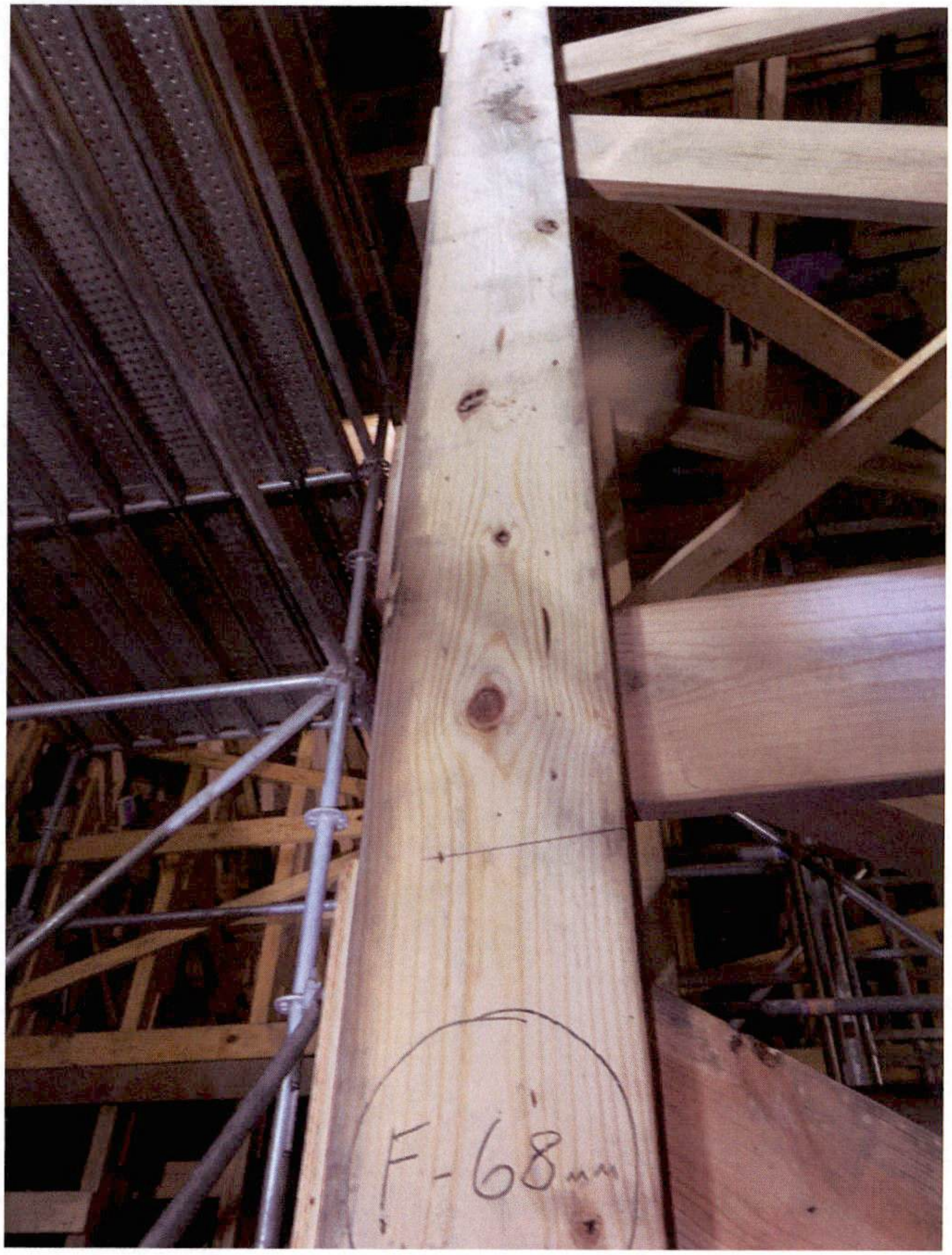
6.6.1 Identification of Lumber

Wood Identification Stamp

Picture from aDB

Taken June 16, 2016

6.6.2 Grade of Lumber



Nuts on Lumber

Picture from aDB

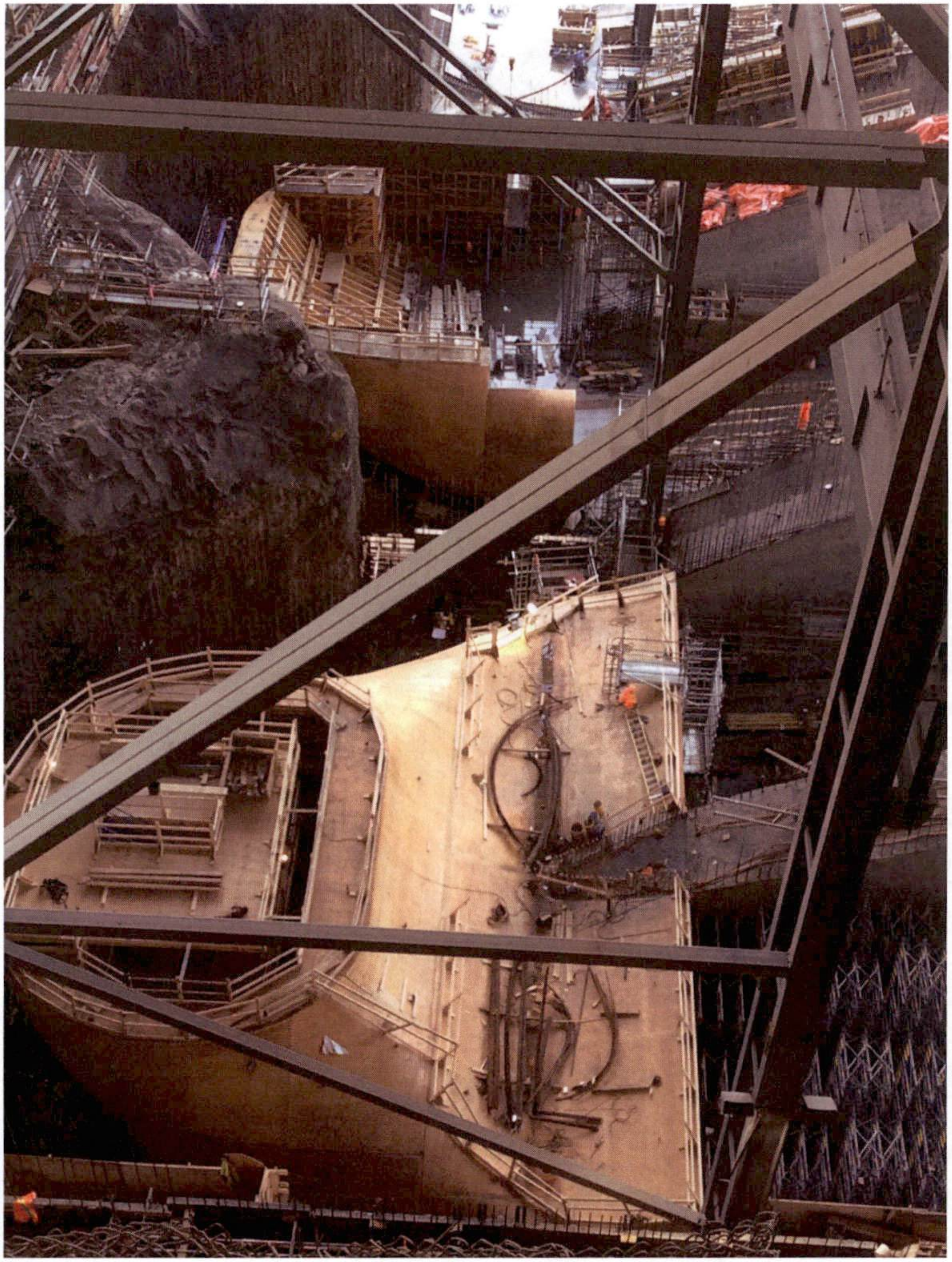
Taken June 15, 2016

7 Commentary on Project Timeline

CEI Shop - Nalcor Visit

Picture from Nalcor

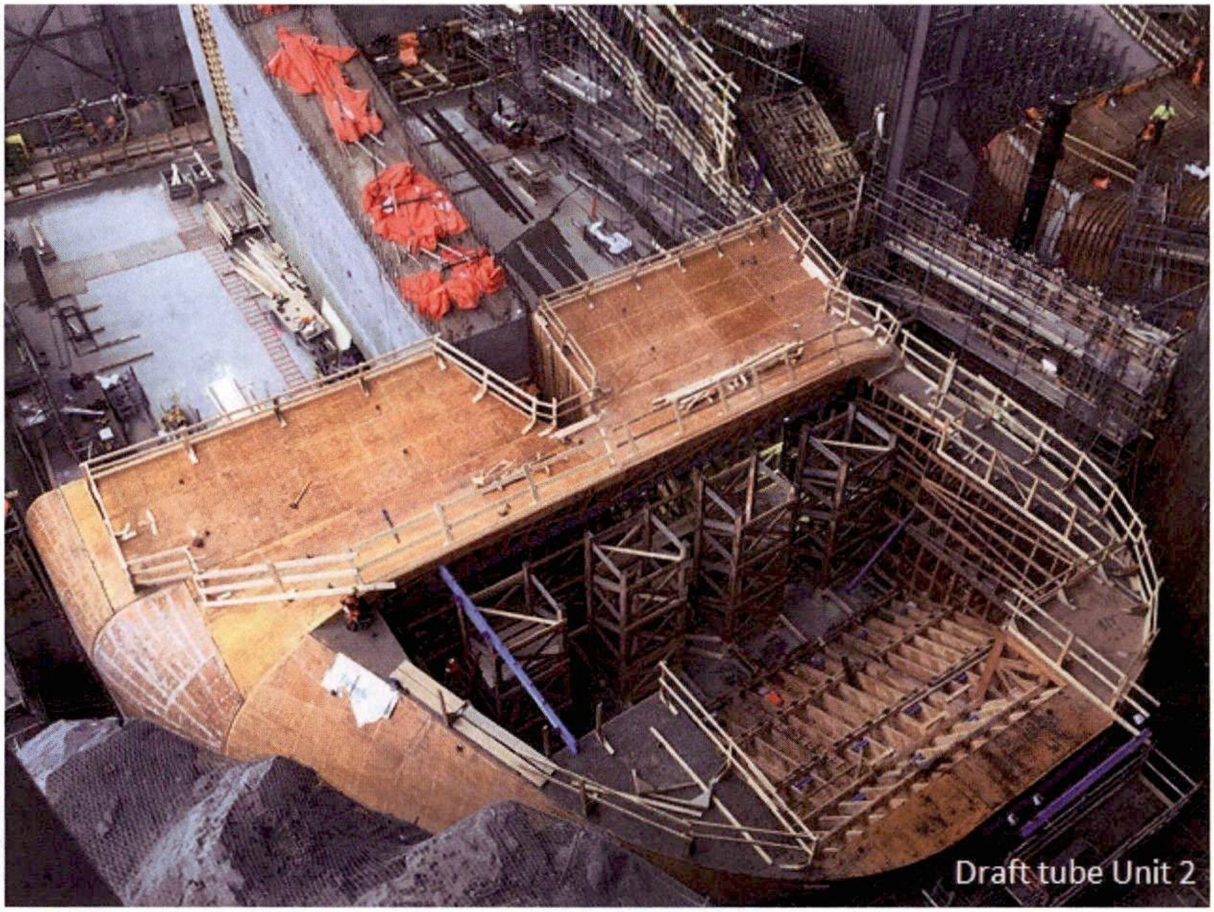
Taken November 18, 2014



Draft Tube Unit 1 and 2

Picture from Nalcor

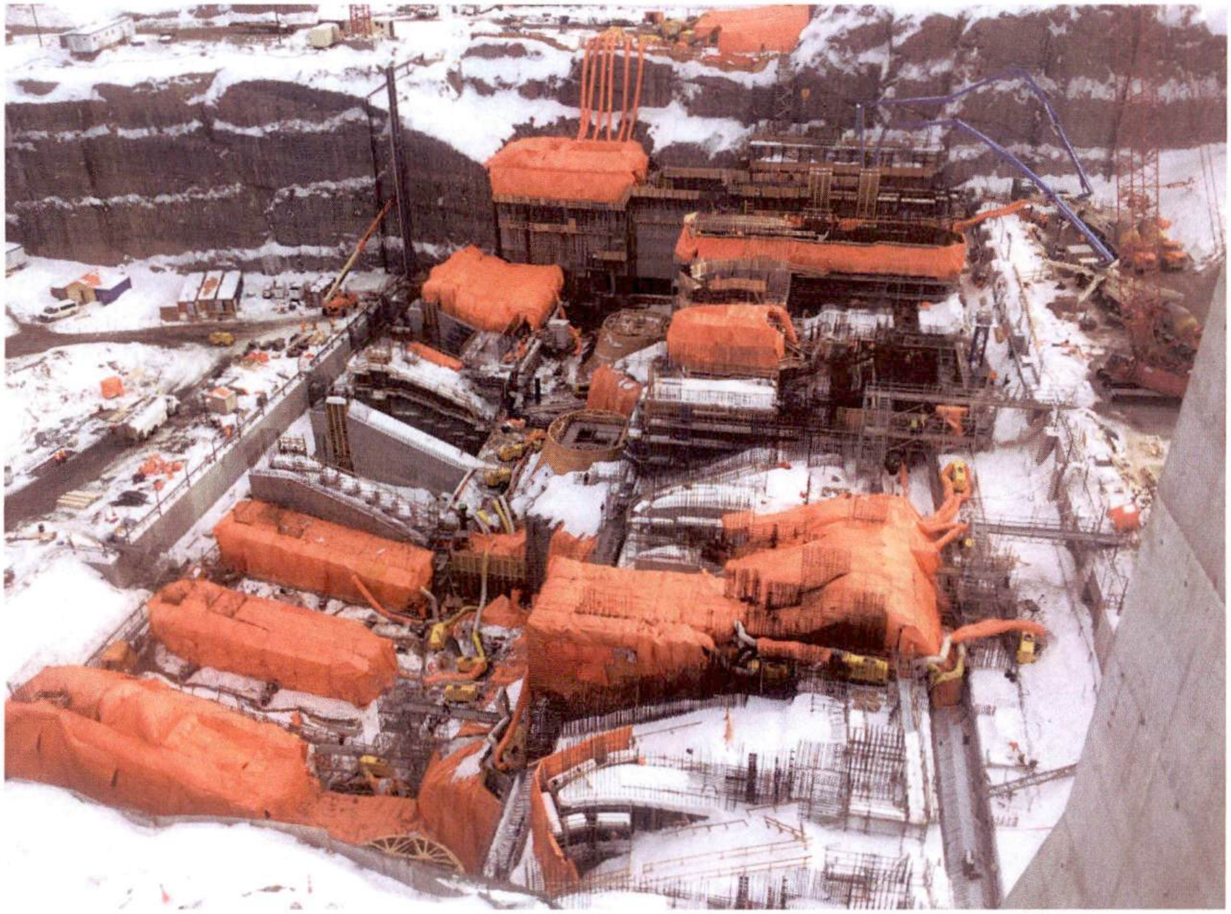
Taken September 19, 2015



Draft Tube Unit 2

Picture from Nalcor

Taken October 10, 2015



Powerhouse View from North Transition Dam

Picture from Nalcor

Taken March 16, 2016