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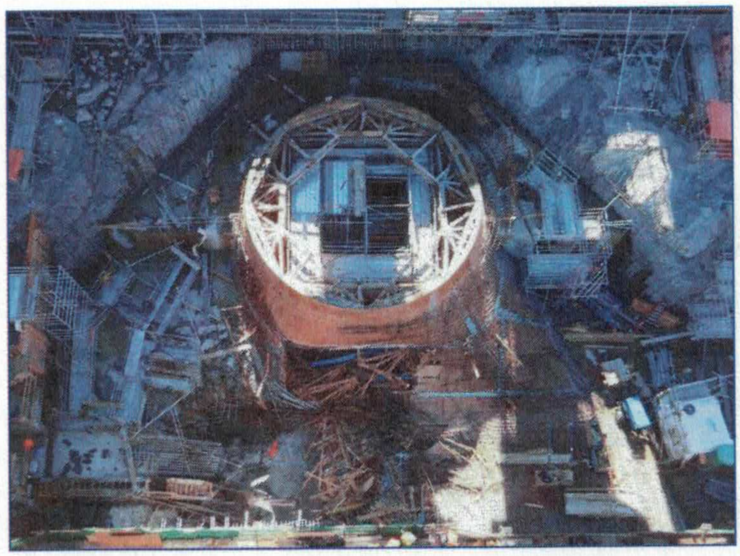

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REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
TIMOTHY C. WELLERT
SIGNATURE
FEB 24, 2017
DATE
NEWFOUNDLAND & LABRADOR

Lower Churchill Project

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

February 24, 2017

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REVISION

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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

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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.

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.

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.

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.

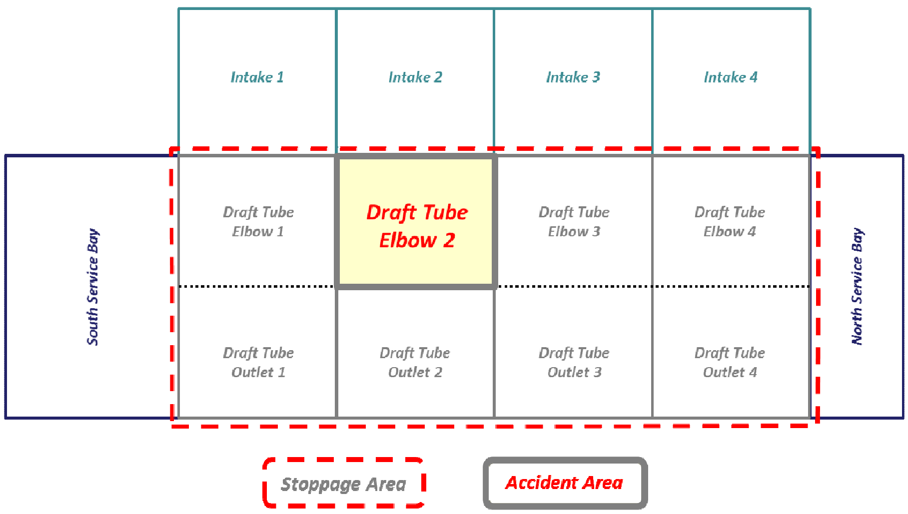


Figure 1: Areas impacted by May 30, 2016 Stop Work Order

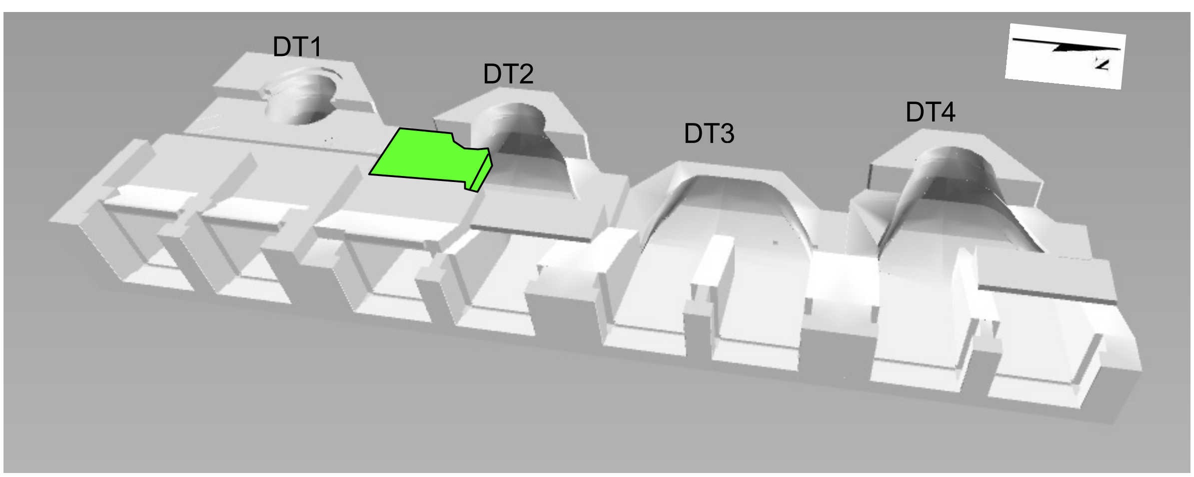


Figure 2: Work progress in the power house June 2, 2016. Failed D2ESB-03 in Green

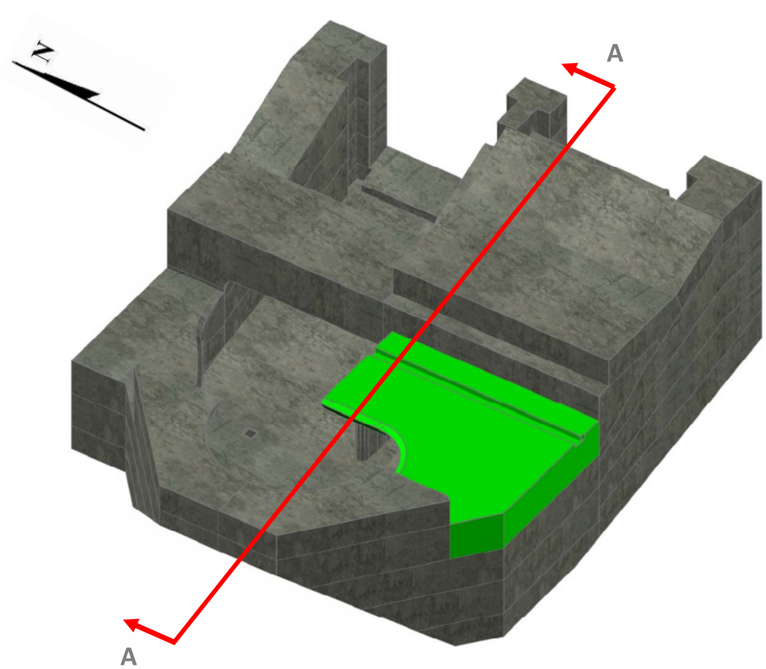


Figure 3: Draft Tube Unit 2 isometric; failed D2USB-03 pour in green

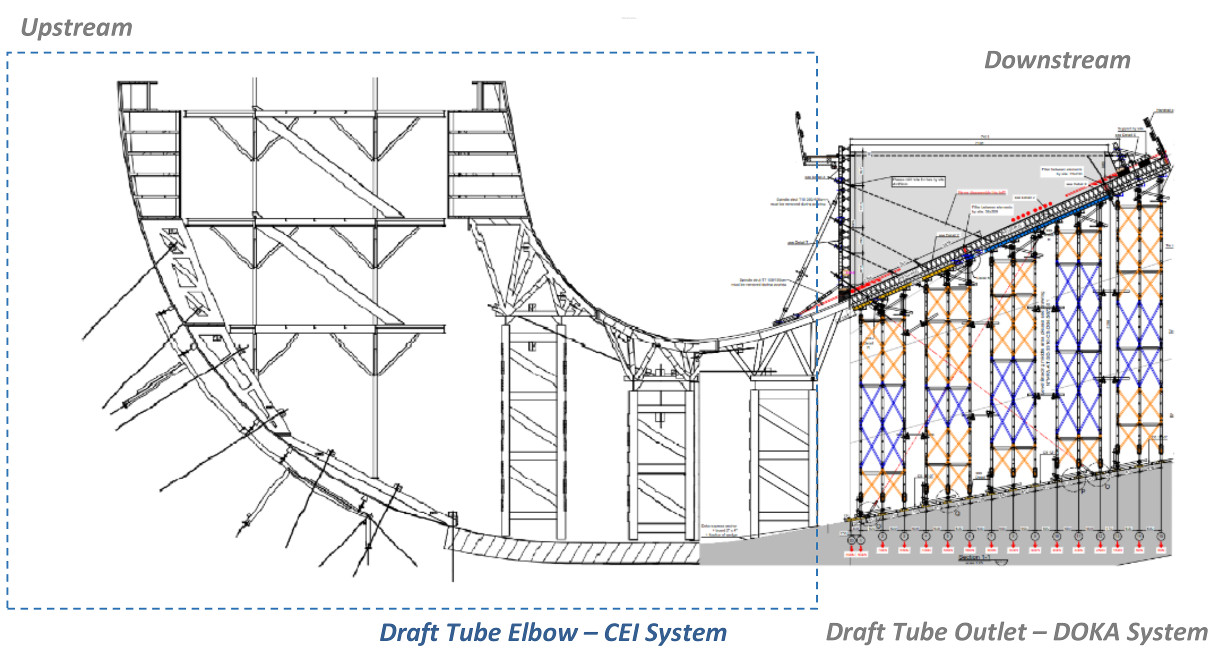


Figure 4: Section A-A from Figure 3: Typical section of draft tube and outlet Formwork Systems

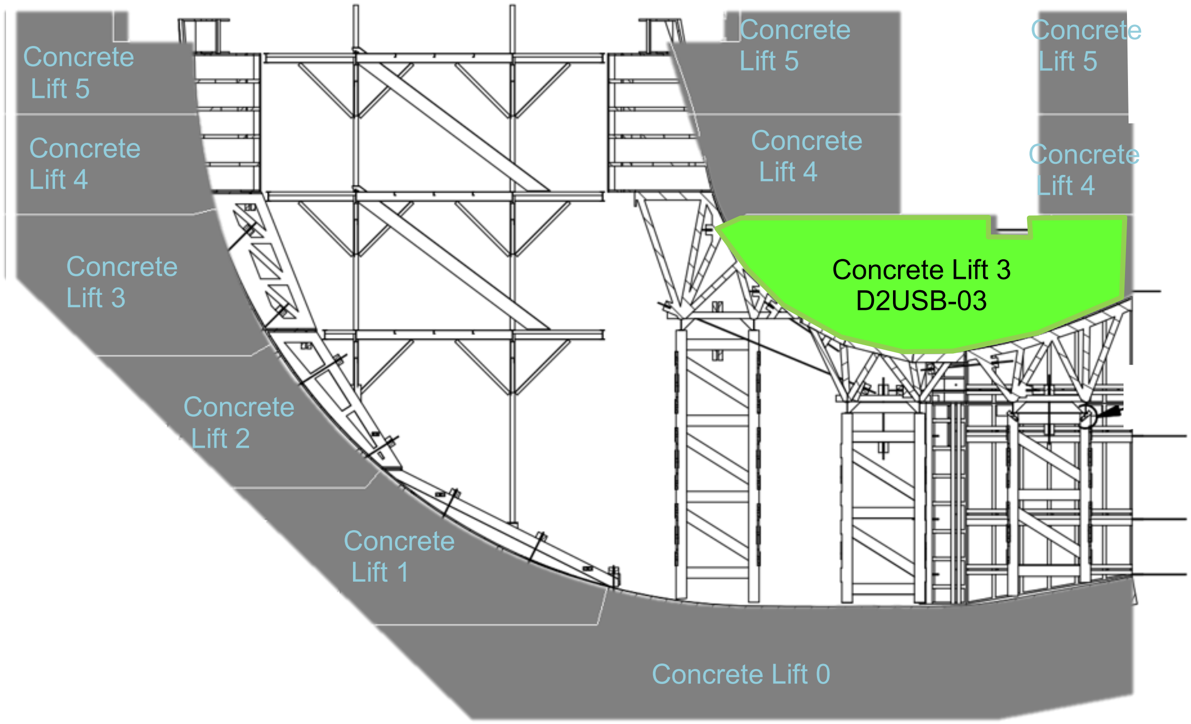


Figure 5: Section A-A from Section 3, Naming convention of concrete lifts in draft tubes.

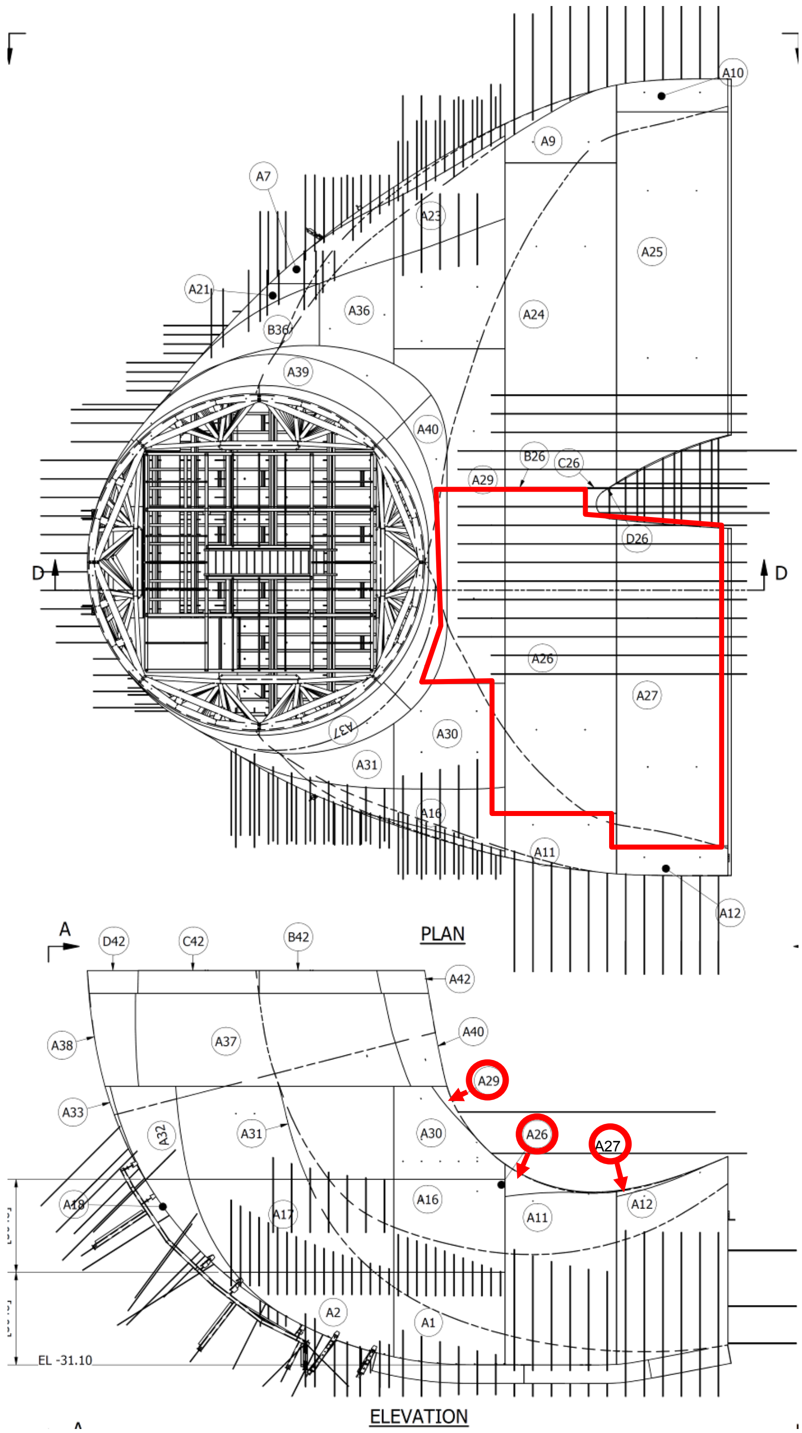


Figure 6: CEI formwork modules involved in collapse

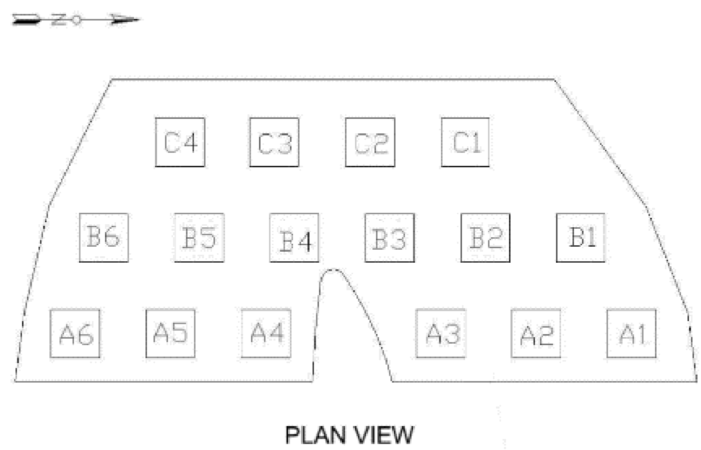


Figure 7: Falsework tower naming convention typical all draft tubes

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.

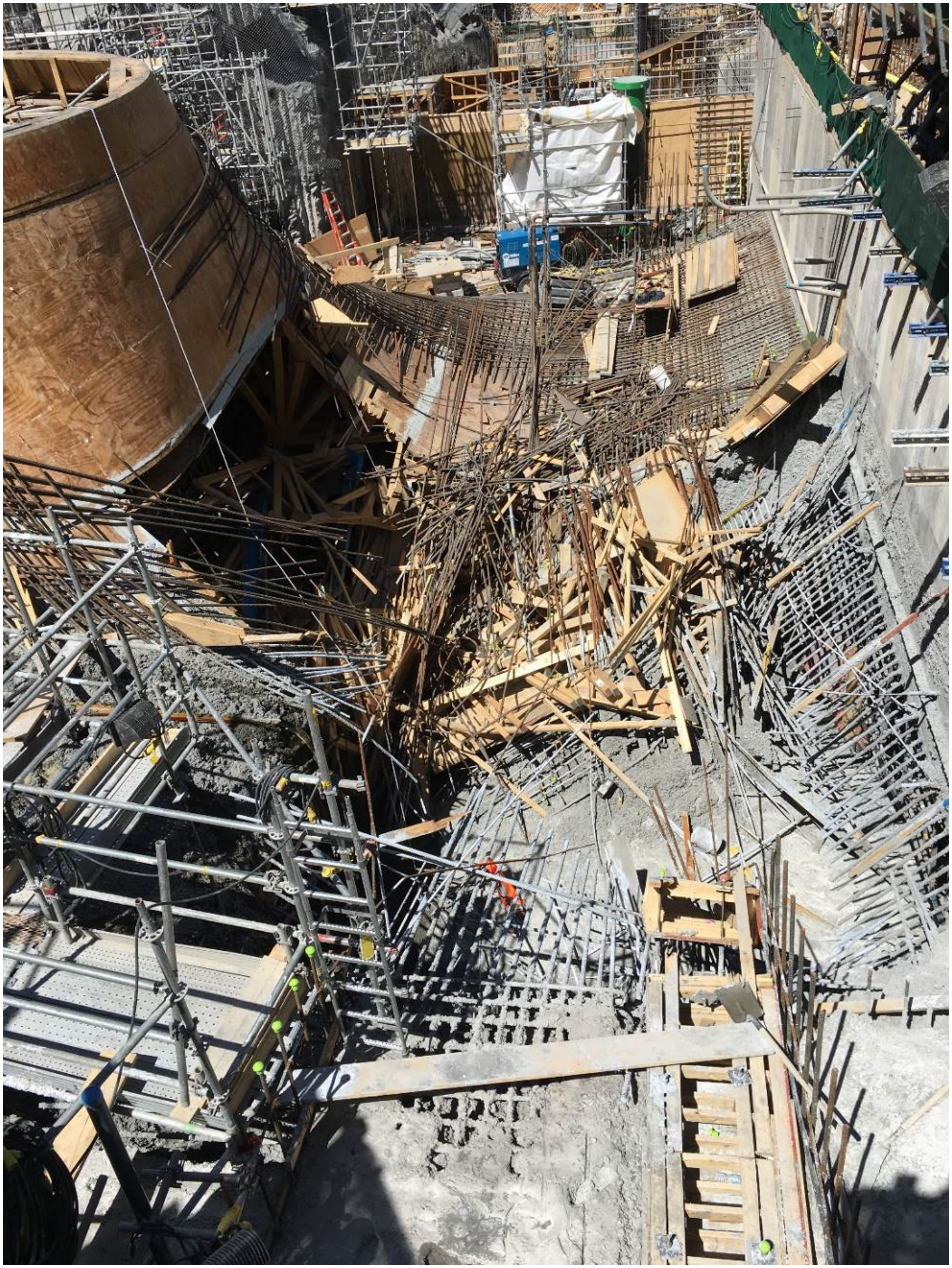


Figure 8: DT2 failure area, looking north

Figure 9: DT2 failure area looking southeast

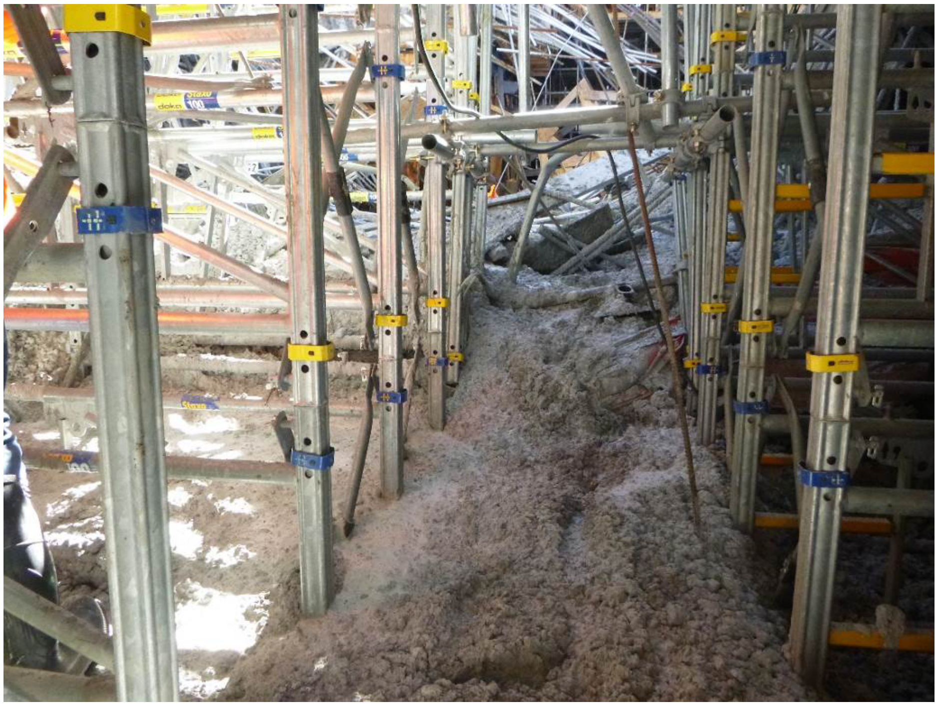


Figure 10: DT 2 south outlet looking upstream (west) into failure area

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.

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

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

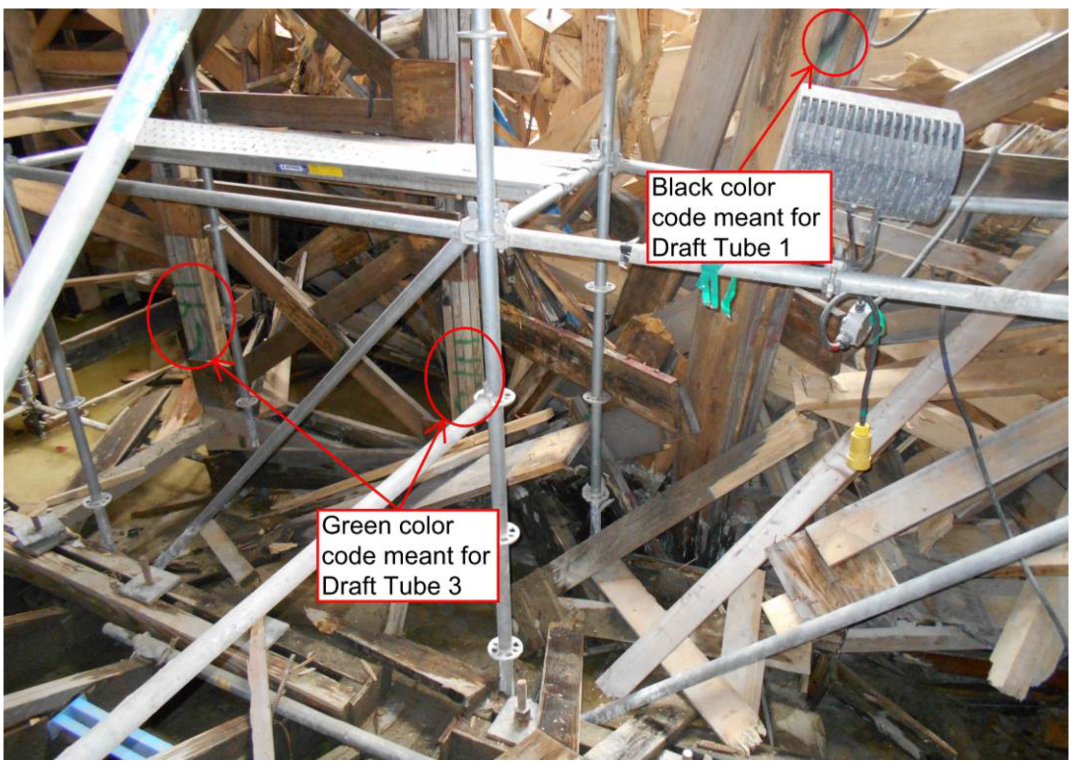


Figure 24**:** Incorrect color coding used in DT 2, multiple colors present

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.

Figure 25: Splice plates pulling away from column legs

Figure 26: Splice plate pulled away from tower leg

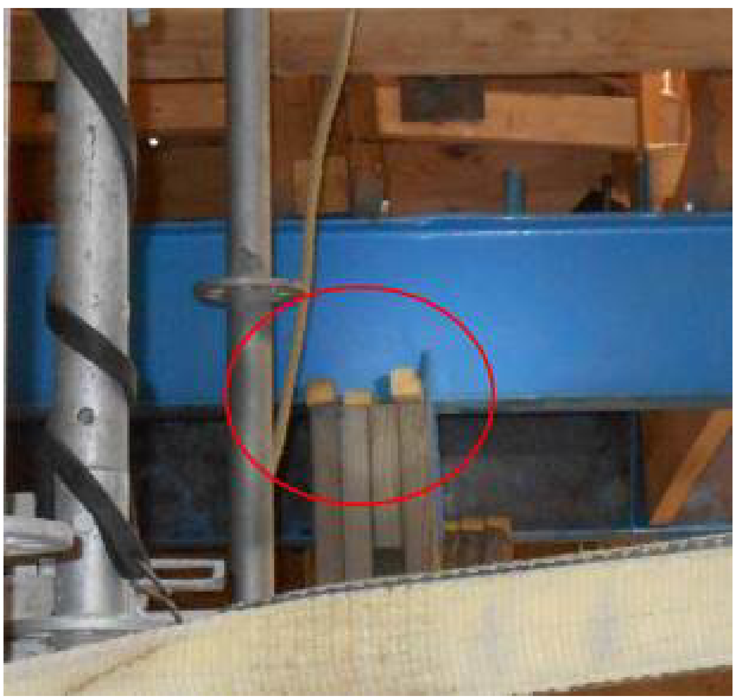


Figure 27: Inadequate shimming on top of falsework towers. No bearing plate installed

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

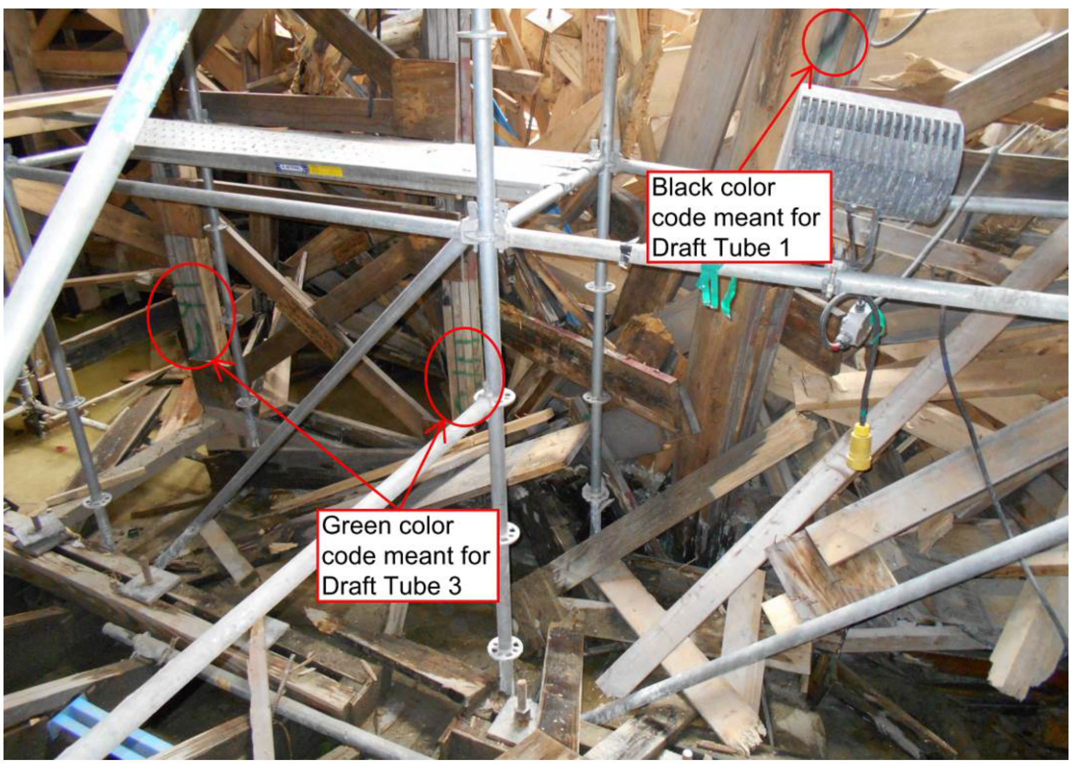


Figure 29: Incorrect color coding used in DT2

4 FORMWORK AND FALSEWORK DESIGN

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.

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 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.

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 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.

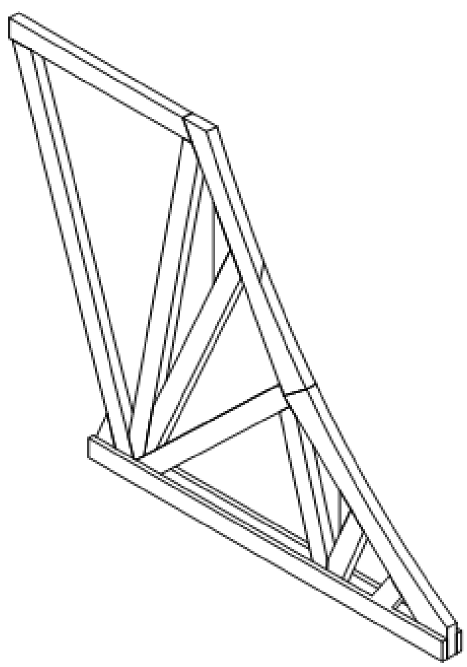


Figure 30: Rib C29 of Panel A29 from CEI shop drawing MFA-AT-SD-3310-CS-D04-0048-01

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.

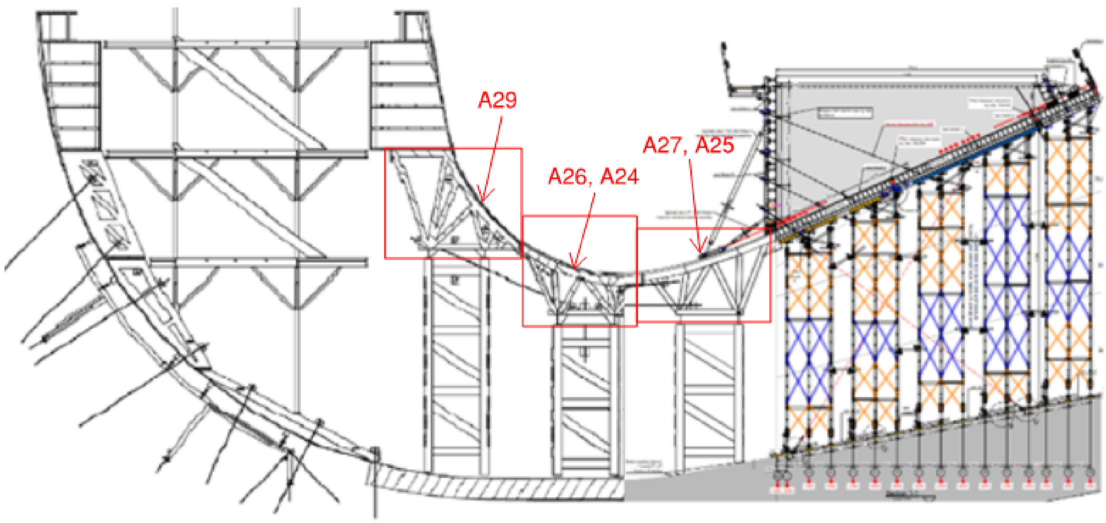


Figure 31: Panel geometry for analysis. Wet ties not shown.

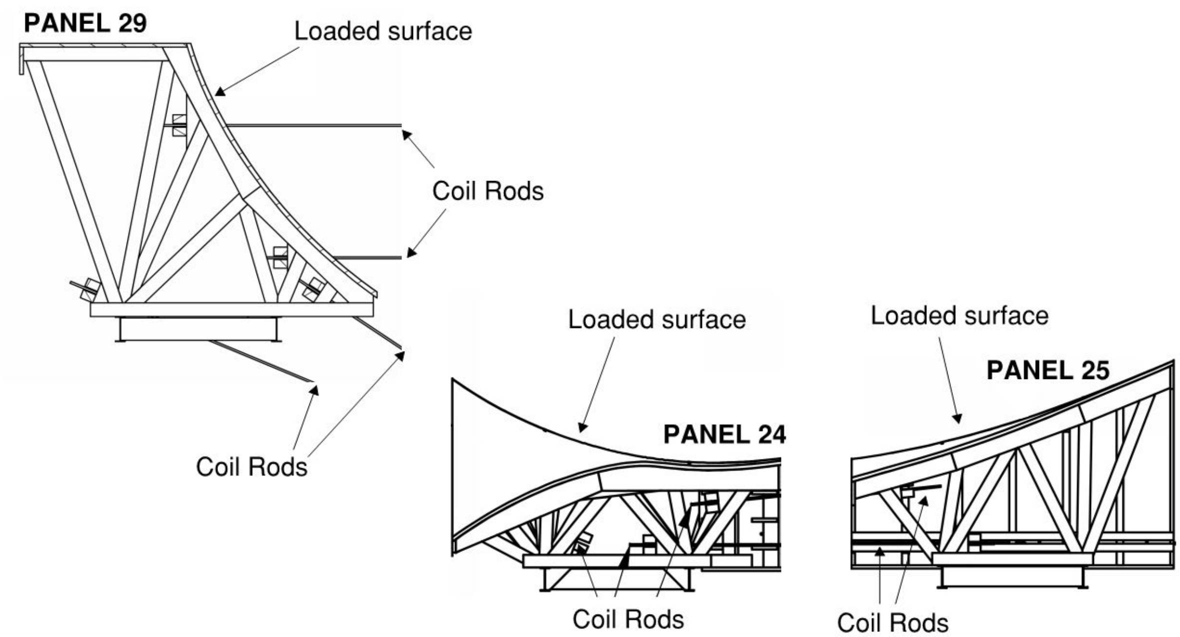


Figure 32: Draft Tube Roof Panel Geometry

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.

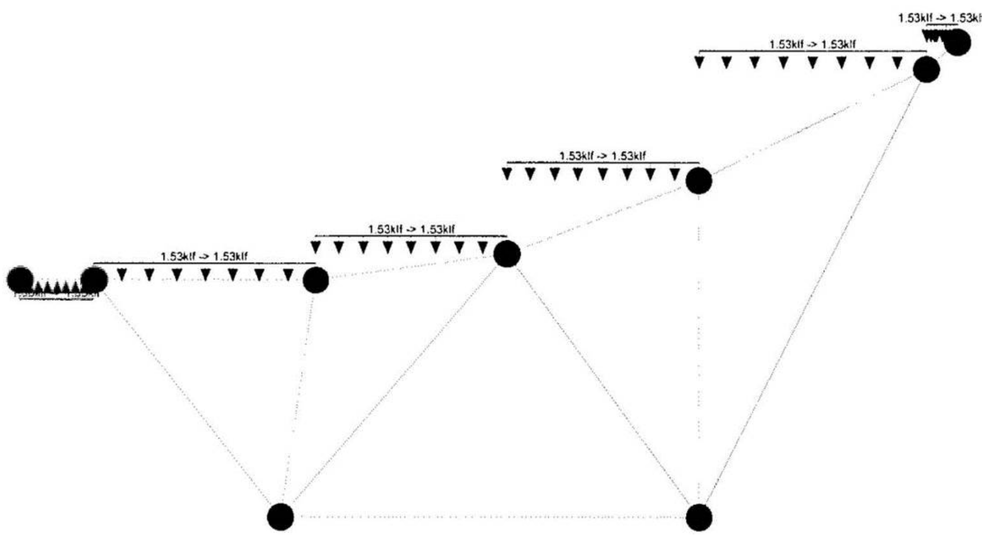


Figure 33: Rib G24 loading in CEI calculations

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.

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.

- 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.

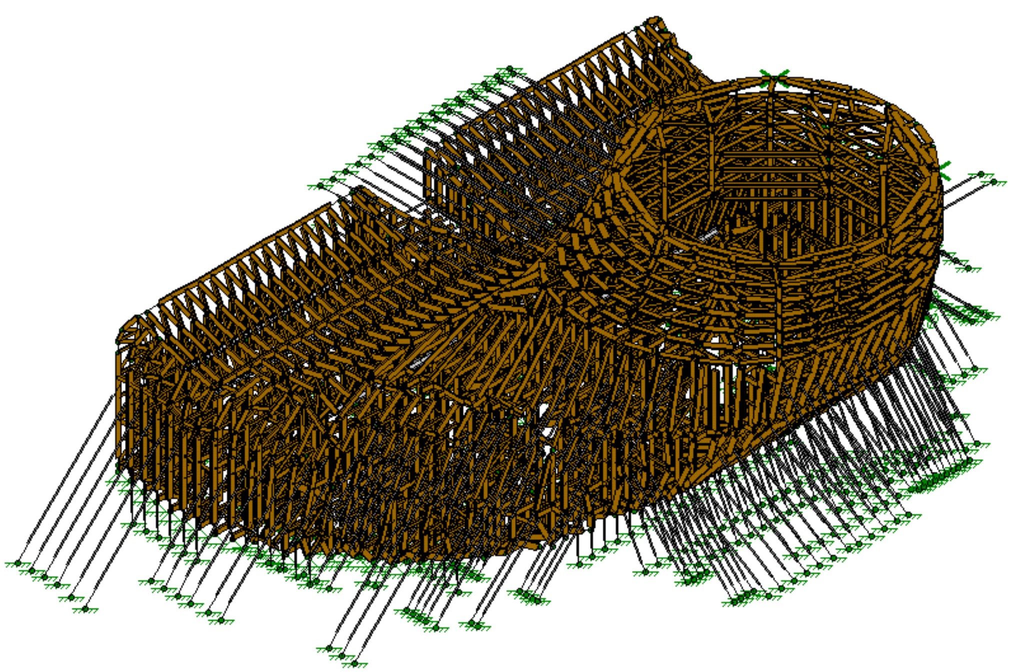


Figure 34: Isometric View of ILF's 3D Structural Model

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.

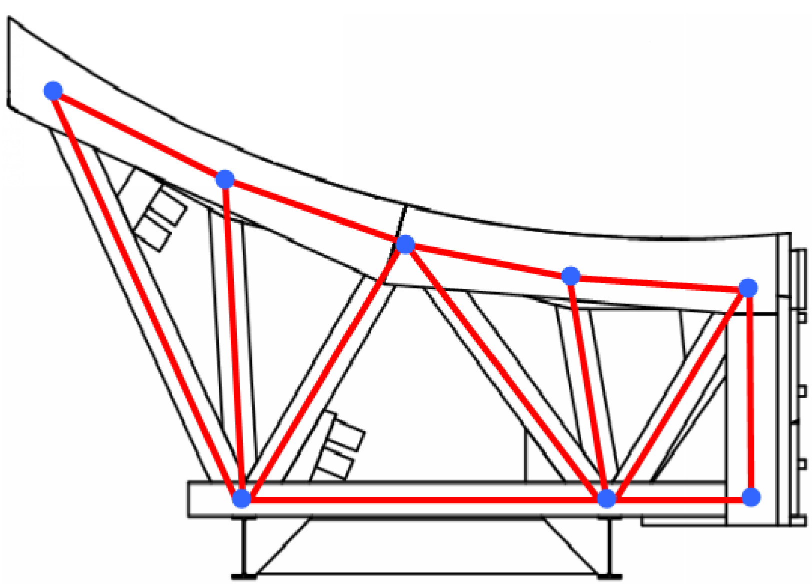


Figure 35: Comparison of stick and node to actual geometry

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.

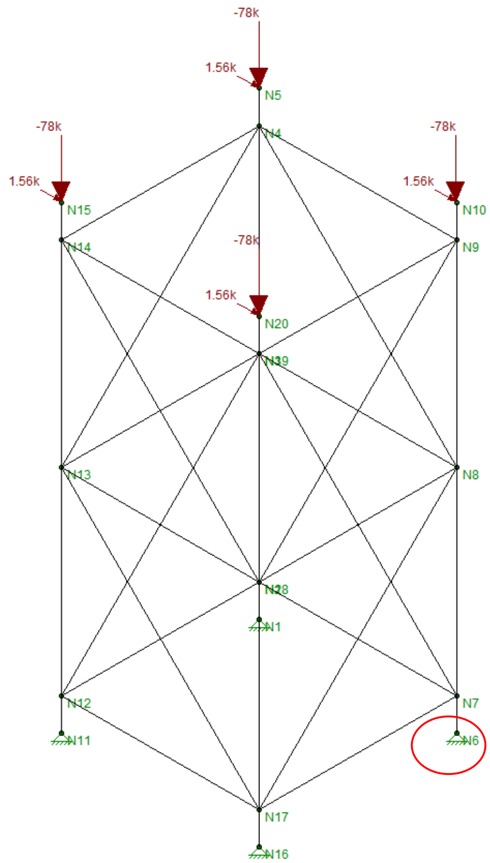
In addition to the overstressed formwork members, ILF identified locations where additional anchor rods or tie-backs were required. These were required due to unbalanced loading during concrete placement. Based on review of CEI's calculation package, it appears they had assumed that the concrete for 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.



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			

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

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 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.

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

	Design Code						
	CSA O86-09		NDS 2005			NDS 2015	
	ILF		CEI	ILF		ILF	
Design Factor	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.

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

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

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.

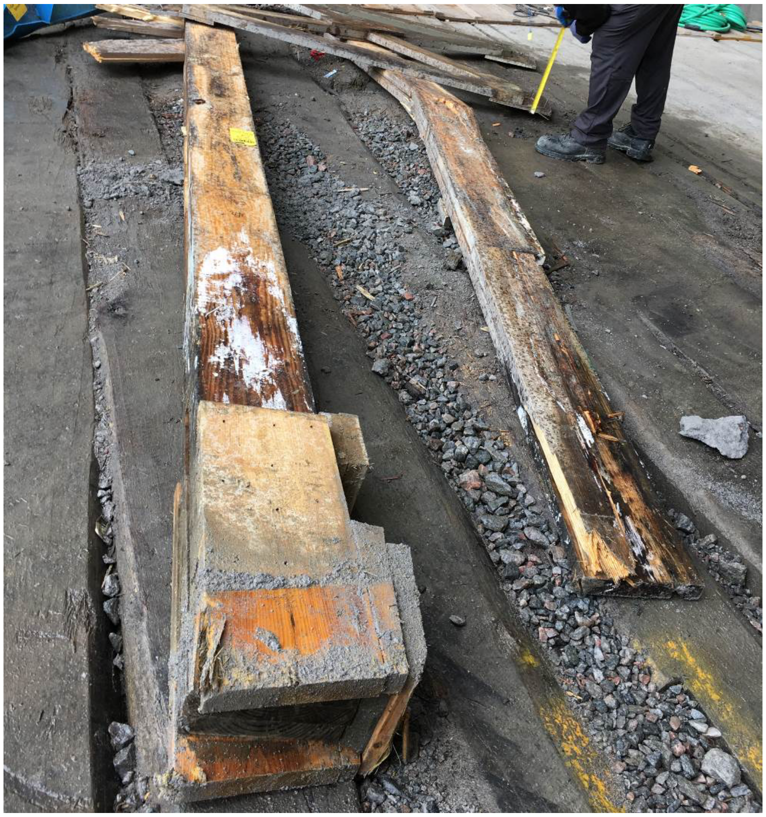


Figure 37: Tower Legs obtained from DT2 Tower C3

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.

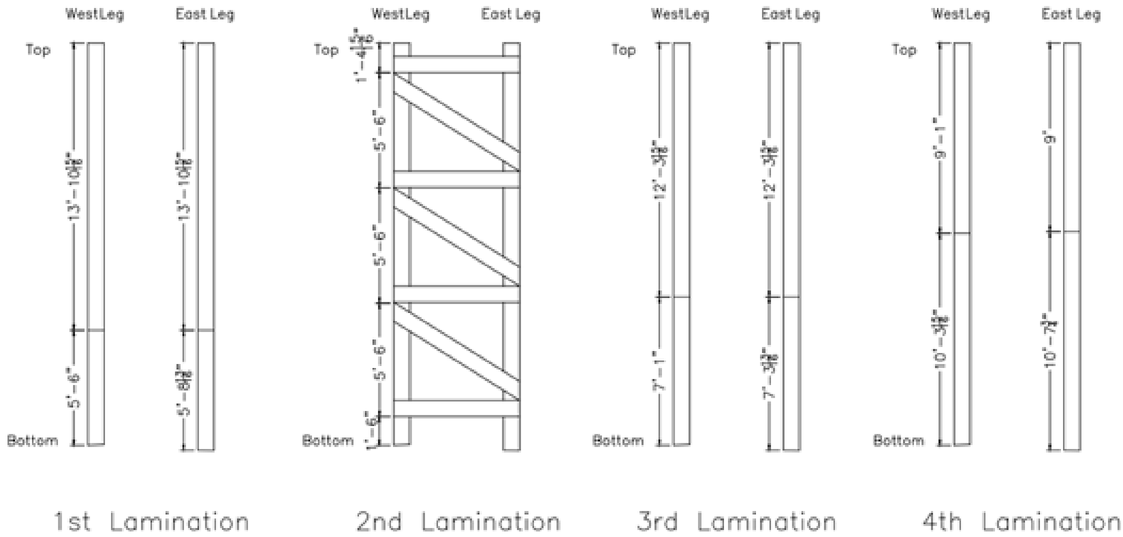


Figure 40: South face of tower C3 lamination components

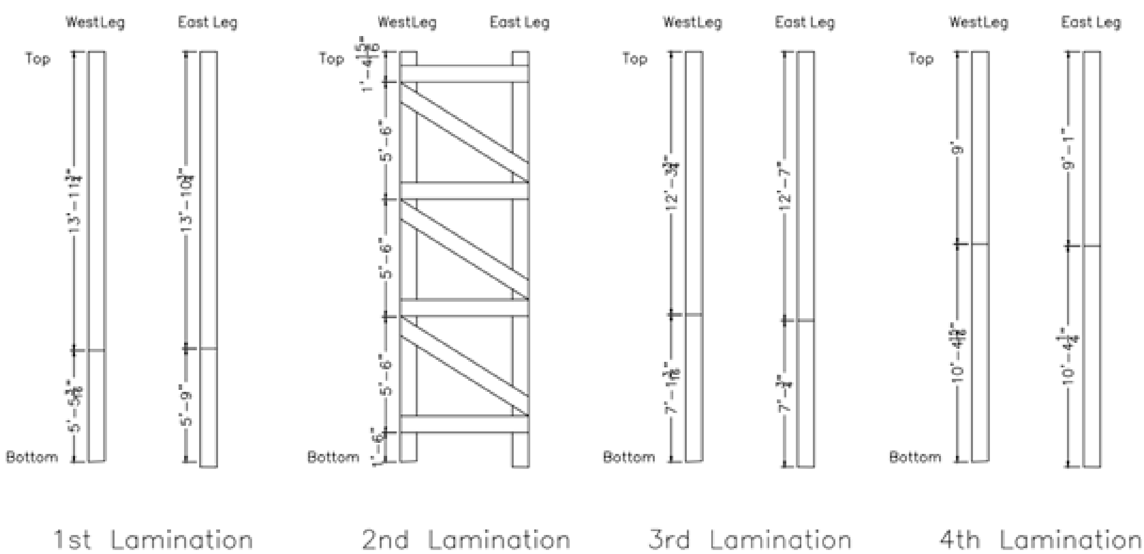


Figure 41: North face of tower C3 lamination components

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.

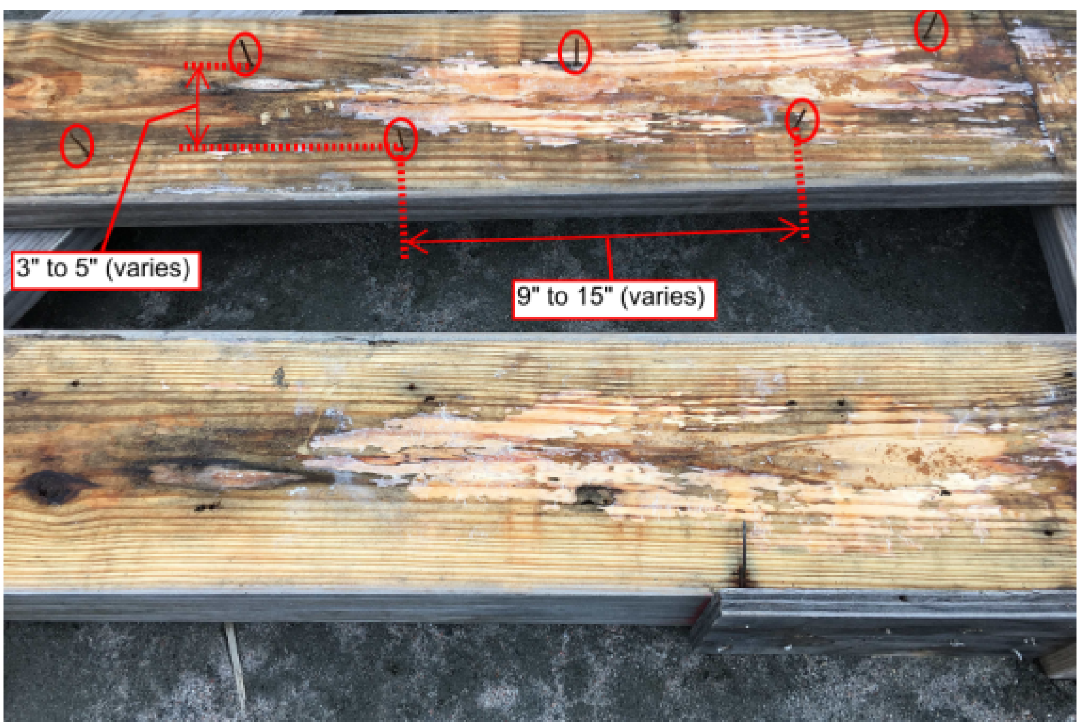
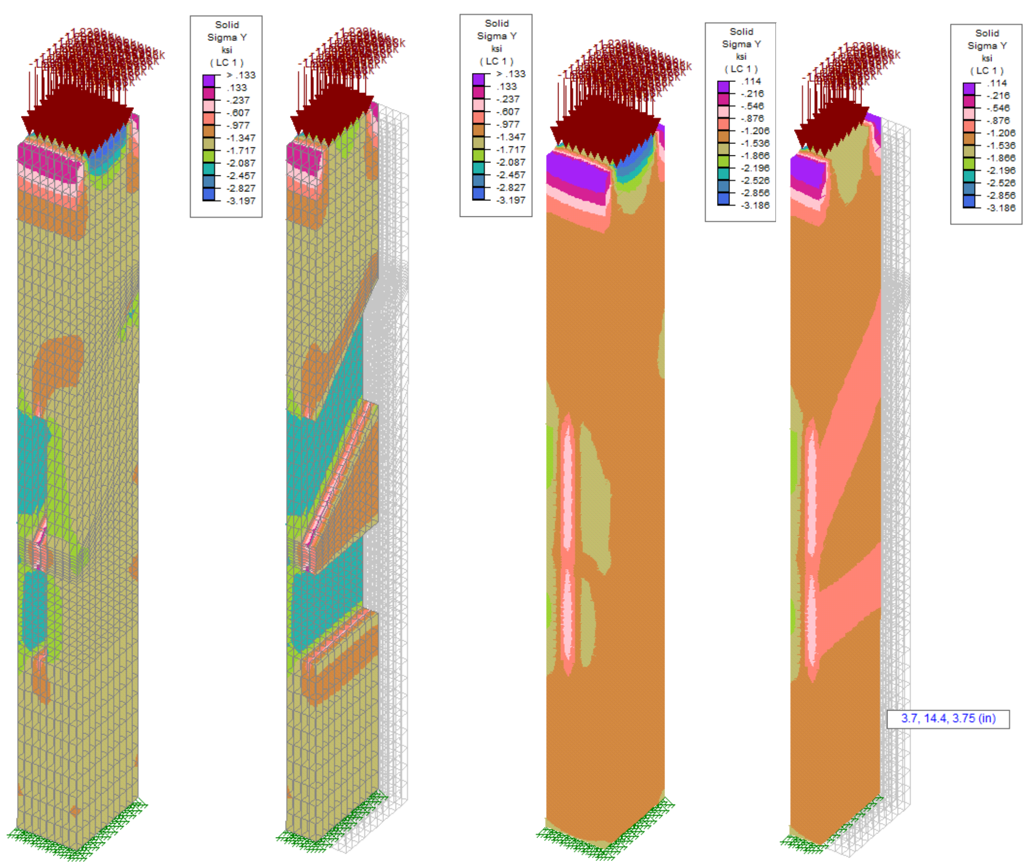


Figure 45: Typical nailing pattern used to build up falsework tower leg member

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.

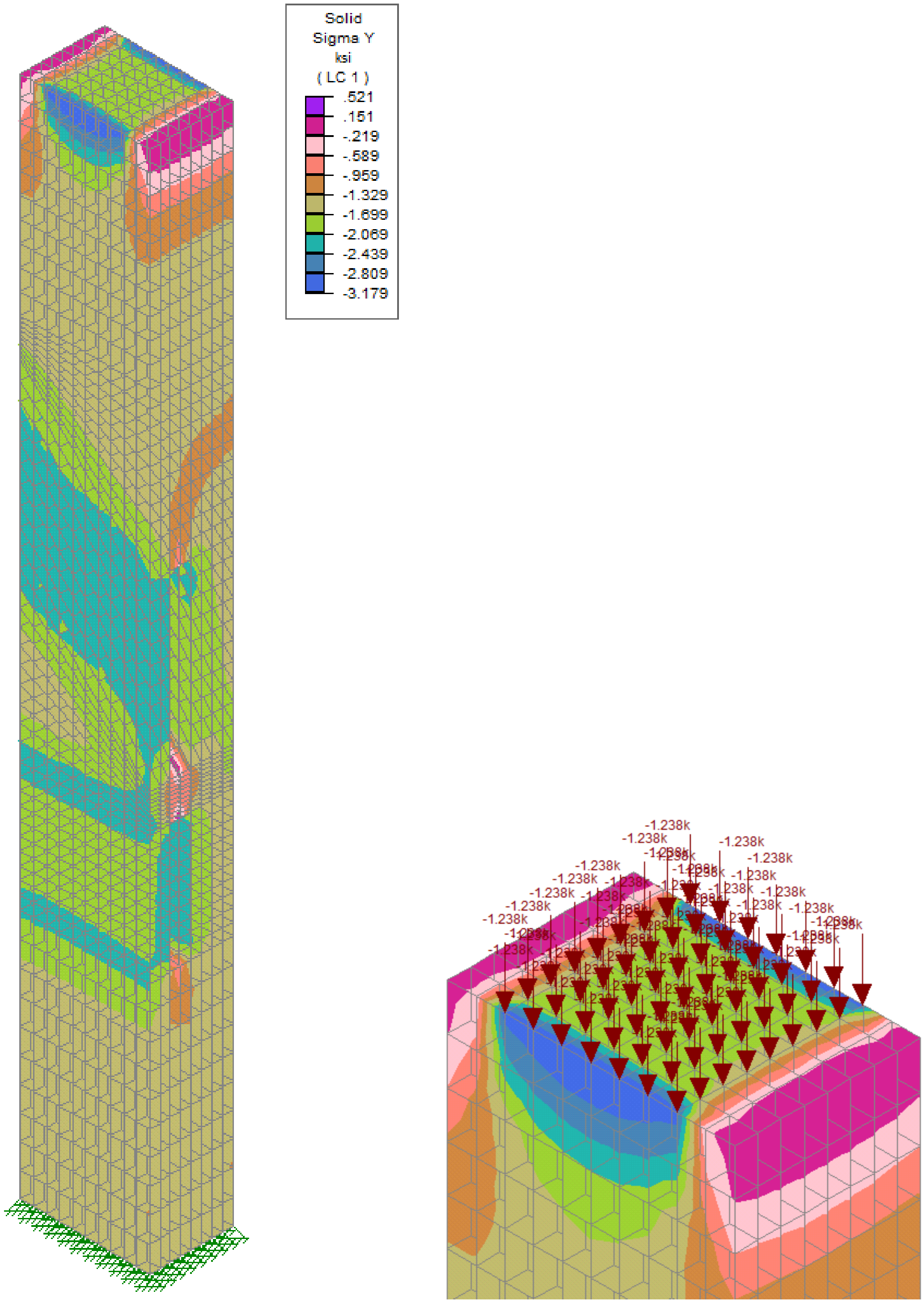
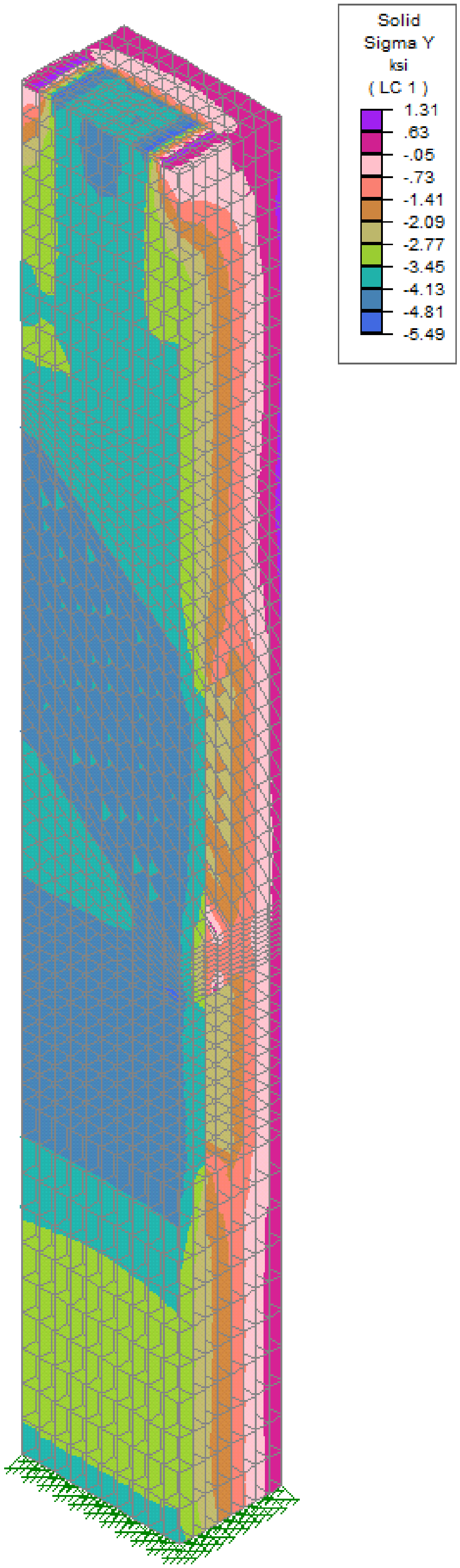


Figure 47: Vertical Stress Analysis - Load applied uniformly over all plies


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

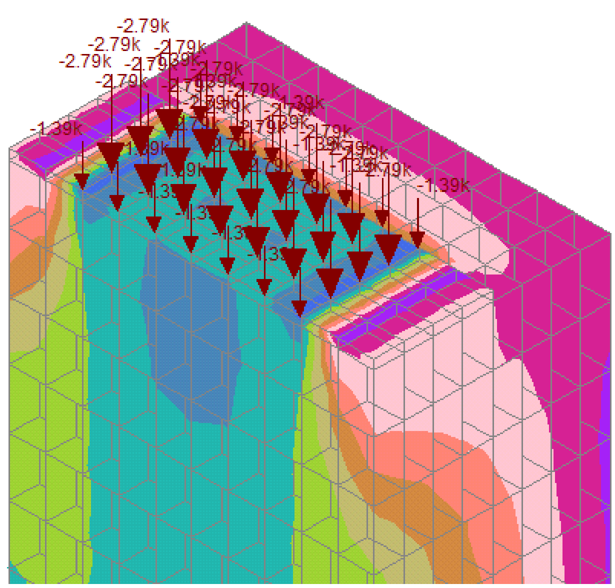

-2.79k
-1.39k

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

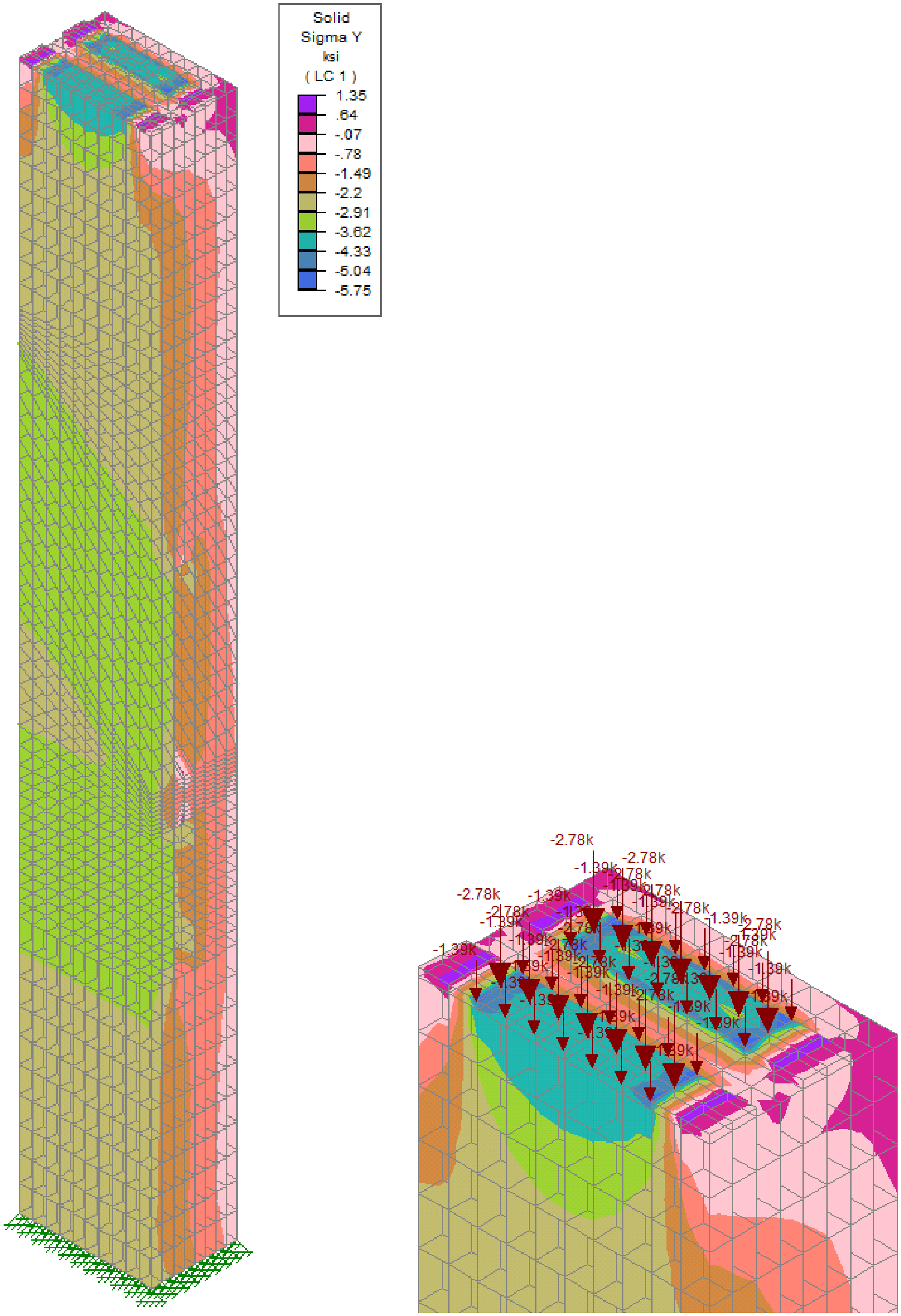


Figure 49: Vertical Stress Analysis – Load applied to two alternate plys

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

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.

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.