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Analytical Investigation of the Stability and Post-Buckling Behavior of Large-Scale Truss Assemblies

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Abstract
The latest generation of standard nuclear power plant designs involves the use of small modular reactors (SMR). The construction of the Nuclear Island (NI) of a SMR project is the critical path activity to construction completion. As such, schedule optimization of the NI is a primary design goal, i.e., in lieu of simply minimizing material quantities and installation job-hours. In order to achieve this, large multi-commodity ground pre-assemblies comprised of stay-in-place formwork, reinforcing steel, penetrations, commodities and commodity supports are created that allow for parallel construction efforts. These large pre-assemblies need to be: rigged, lifted and, when properly positioned through the use of reusable truss-like structures (e.g., three-dimensional long span space frames), able to support the additional wet weight of concrete. This paper will address the unique aspects of these structural designs, including constructability and stability when the range of lifted loads can be up to 1000 tons (including the supporting truss assembly) and the total supported load after being positioned and upon placement of concrete can approach 5000 tons. Parametric buckling analyses for these large truss assemblies now being used for modular construction are carried out for a range of different bracing configurations.

1. Introduction

For standard nuclear power plant projects, select safety-related reinforced concrete slabs may approach 5 feet thick. In select areas, spans may be on the order of 100 feet. Constructing such thick slabs in a conventional manner involves not just installation of reinforcing steel and placement of concrete, but both installation and removal of formwork and shoring. Due to the heavy reinforcement at both the top and bottom of the slab and select shear ties lacing the levels, construction is very labor intensive and time consuming. Alternately, deep steel framing may need to be used in lieu of conventional formwork placement. However, such framing over containments approaching 100 feet in diameter, safety-related equipment, spent fuel pools, water storage tanks, etc. may lead to a significant increase in foundation depth and required excavation/site dewatering.

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To mitigate both costs and the required construction schedule, the design/build methodology involves constructing large-scale, ground assembled super-modules that can be up to 50 - 70 feet wide and over 100 feet long. The super-modules include (Ryan et al, 2012):

- Shop fabricated modular stay-in-place (SIP) formwork panels
- Re-usable formwork support trusses (in lieu of shoring)
- Multiple levels of reinforcing steel, along with shear ties and top steel support frames
- Pipe, electrical and HVAC penetrations
- Embedded plates
- Select saddle supports above the slab
- Under-hung commodities and commodity supports

Fig. 1 illustrates a long-span condition with two primary trusses on the order of 100 ft. long. Perpendicular to the trusses are SIP formwork panels nominally 10 feet wide and 30 to 50 feet long. As clarification, the parallel red beams are situated at the periphery of the SIP formwork panels of the lifted super-module. Reinforcing steel is not shown for clarity. With reinforcing steel, the super-modules may approach 2000 kip (approximately 100 kip/panel), a lift viable only with the latest generation of field assembled ringer cranes.

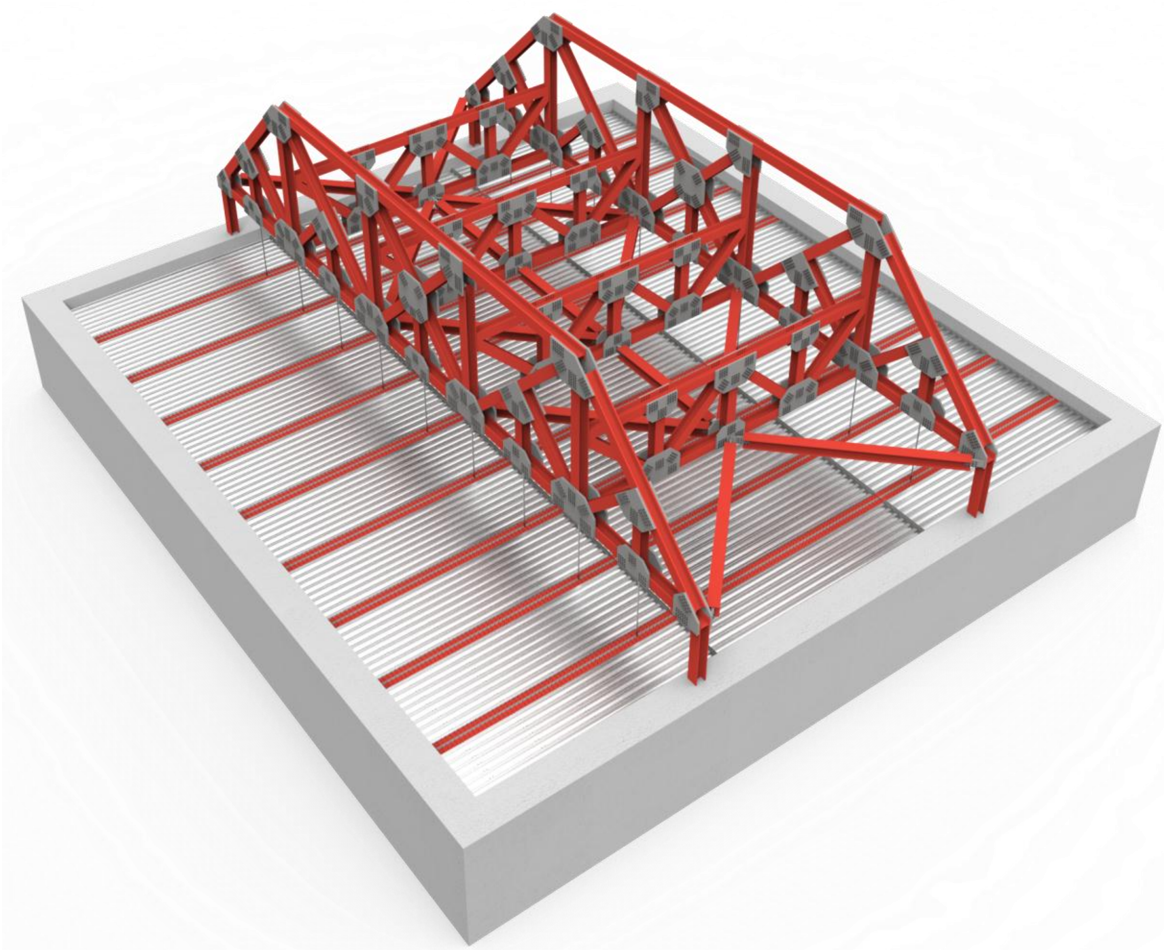


Figure 1: Isometric View of Large Scale Ground Assembled Super-Module

High strength rods transfer the super-module weight and wet weight of concrete to the truss pre-assembly. Couplers are provided at the top of the concrete slab. These couplers allow rods to be removed above the slab, in part for reuse. In addition, the couplers anchor the portions of the rods embedded in the slab and the SIP formwork panels in place, thereby providing a secondary barrier to any concrete spalls from impact-type loading.

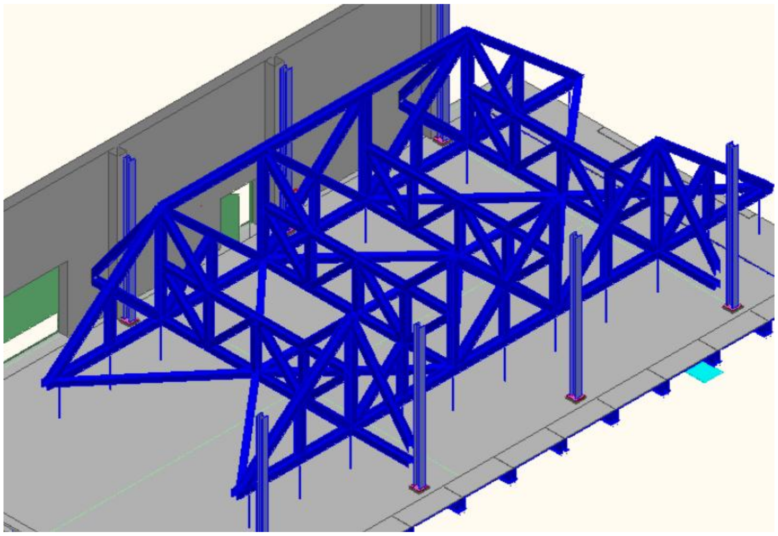


Figure 2: Isometric View of Super-Module with Outrigger Extensions

Significant cost savings may be achieved for standard plant designs, reflecting: the shift of labor from that at height to the ground and/or fabrication shops, schedule compression savings including reductions in indirect costs and the cost of capital and amortization of the re-usable formwork support frames amongst multiple standard nuclear power plant locations.

Fig. 2 illustrates how the truss pre-assembly is modified with four structural extensions (i.e., outriggers) to support a super-module that is an additional 20 feet wide, or 70 feet by over 100 feet long.

1.1. Modular Stay-in-Place Formwork Panels

In the past fifteen years, Bechtel Power Corporation has used modular floor panels to expedite critical path construction. The modular floor panels are shop fabricated, with plan dimensions up to 12 feet wide and 60 feet long. A total of 10,000 panels have been used on projects worldwide, with a maximum of 2000 panels at any single site.

Fig. 3 illustrates the stackable modular floor panels in transit. Shipping premiums are avoided by having weight and volume criteria simultaneously optimized. Whereas steel shipments are typically governed by weight rather than volume, modular steel design includes all labor intensive miscellaneous steel in the floor panels.

The shop fabricated modular floor panels are typically installed at a rate of 10 hours per ton, which is considerably lower than the average rate for conventional “stick-built” structural steel (i.e., 15 hours per ton) and miscellaneous steel (i.e., 50 hours per ton). The improved efficiency can be attributed to two major sources. The first source is the significant reduction in the number of bolts required during installation. The reduction is achieved by specifying shop welds for infill steel beam connections to the primary beams, and by providing seated connections at the ends of the primary beams that require only two bolts per connection, regardless of load. This results in a total of eight bolts per modular floor panel. The second source is the 90 percent reduction in the quantity of structural and miscellaneous steel pieces to be installed at the site. Almost all of the time consuming miscellaneous steel work is performed in a fabrication shop, rather than at elevated heights in the field.

Figure 3: Stackable Modular Floor Panels in Transit and During Installation (Courtesy Bechtel Power Corporation)

For this generation of standard nuclear power plant designs, the modular floor panels are modified to create modular stay-in-place formwork panels. Each modular SIP formwork panel is comprised of two primary parallel beams, infill beams, composite deck (or WT-stiffened high strength plate), closure strips/angles and steel headed stud anchors. For slab thicknesses up to 3 feet thick, composite deck is typically used, while for slab thicknesses of 4 feet or greater, WT-stiffened plate is used. The stiffened plate provides additional diaphragm stiffness and strength for loading during the placement and curing of concrete. Fig. 4 shows a slab from the under-side, illustrating the modular panels, as well as a limited quantity of under-hung commodities and supports (for clarity). When commodities are to be under-hung, the modular panels are supported on a temporary frame with a bottom of steel elevation of between 7 feet and 10 feet above the construction phase required slab on grade.

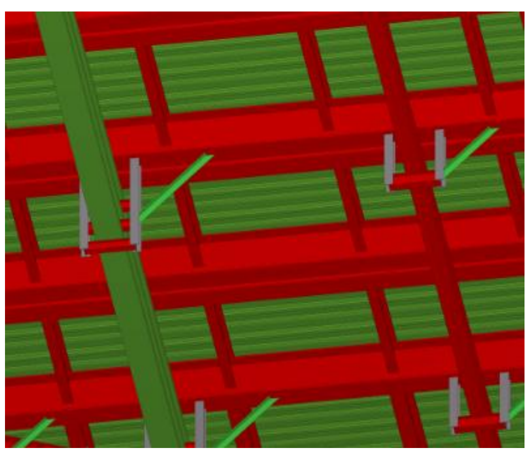


Figure 4: Bottom View of Modular Floor Panels with Under-Hung Commodities/Supports
(Limited Quantity of Commodities Shown for Clarity)

1.2. Truss Assembly Geometry

Each primary truss of the full truss pre-assembly is proposed to have a modified Warren truss geometry (see Fig. 5) using high strength, quenched and self-tempered ASTM A913 (2011), Grade 65 material. The use of A913 steel provides the following benefits:

- Reduced shipping costs of the reusable truss from site to site
- Maximized W-shape lengths for shipment on truck beds
- Minimized truss weight for construction rigging and placement

The rationale for the truss geometry includes several factors. The first reason is that it provides the most direct load path, including at the truss bearing location. With the enormous magnitude of the loading (including the wet weight of concrete), the connections are substantial in any event. The absence of any members above the truss bearing locations mitigates the number of connections. In addition, the ability to transfer the most heavily loaded, outboard diagonal compression load to the truss top chord in bearing connections using a minimal number of bolts greatly reduces shop fabrication costs and field erection job-hours. Finally, a minimalistic load path results in a minimized deflection. With such long spans and the enormous tributary wet weight of concrete, such an attribute is critical.

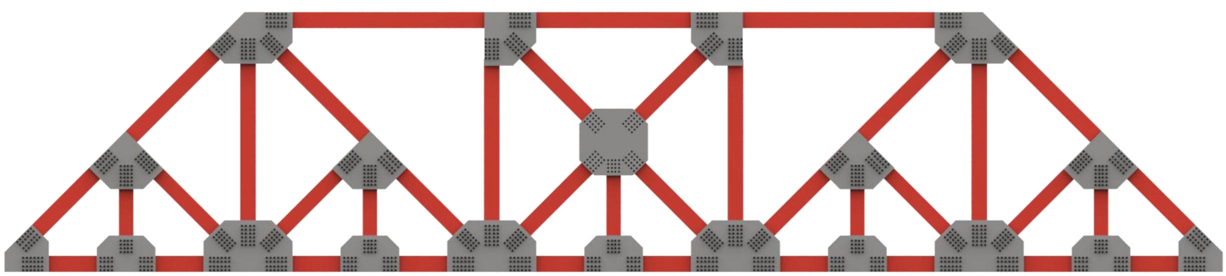


Figure 5: Primary Truss Geometry

The second reason is the successful application of a modified Warren truss in solid fuel plant structural designs for boiler support. As illustrated in Fig. 6, the design replaced conventional plate girders in the support of boilers weighing up to 15,000 tons. The “truss within a truss” modification to the basic Warren truss profile for boiler support side walls is extrapolated for the entire truss length, providing support at 10 feet on center, i.e., the modular SIP formwork panel width.

The cross trusses shown in Figs. 1 and 2 are provided for stability bracing. The two internal cross trusses are envisioned to be half depth in the lower half of the primary trusses, albeit with kickers to the top chord members of the primary trusses. The outboard trusses are envisioned to be full depth, due to their occasional function as transfer trusses, i.e., as shown in Fig. 2. Lateral bracing is provided at the truss bottom chord, in part to mitigate shifting during rigging of the entire super-module.

Figure 6: Modified Warren Truss in Modular Boiler Support Structure (Courtesy Bechtel Power Corporation)

2. Background on Buckling of Trusses and Existing Work

There has been limited research conducted on the lateral stability of trusses, primarily open web steel joists, pony trusses, and top chord stresses. The Steel Joist Institute's Technical Digest No. 2 (Galambos, 1970) documented research that produced reliable analytical methods for computing the critical elastic lateral buckling load of joists braced at equal intervals by lateral braces. Other research directed at determining the buckling load of the compression chord of trusses has been summarized in the Stability Design Criteria for Metal Structures, Chapter 15, Members with Elastic Lateral Restraints (Ziemian, ed., 2010).

2.1 Structural Stability - Trusses

The overall structural system is comprised of a collection of primary trusses with secondary trusses used to provide lateral stability. The factors given in Table 1 will affect the stability of steel trusses when evaluating the physical attributes of the structure and loading. These factors have been adapted from AISC Design Guide 28 (Griffis and White, 2013).

2.2 Member and Structure Imperfections, Misalignments, and Tolerances

The Code of Standard Practice for Steel Buildings and Bridges, AISC 303 (2010) addresses member and structural tolerances in three sections: Section 5 Materials, Section 6 Shop Fabrication and Delivery, and Section 7 Erection Tolerances. Section 5, by direct reference to ASTM A6 (2013), establishes the wide flange structural shapes tolerances for camber, profile, and sweep. The following tables define the maximum limits permitted without correction:

Table 16 Permitted Variation in Cross Section for W, HP, S, M, C, and MC Shapes

Table 22 Permitted Variations in Length for W and HP Shapes

Table 24 Permitted Variations in Straightness for W and HP Shapes

The Code of Standard Practice (COSP) states "Normal variations in the cross-sectional geometry of standard structural shapes must be recognized by the designer, the fabricator, the steel detailer, and the erector."

Table 1: Factors Affecting Truss Stability

Truss geometry and configuration	Centerline framing dimensions Member geometry Connection details Foundation and support conditions Finite member and joint size effects
Material properties	Elastic moduli Expected vs. nominal strengths Ductility and fracture toughness
Geometric imperfections	Erection out-of-plumbness Member out-of-straightness Incidental connection or loading eccentricities
Internal residual stresses	From fabrication processes From erection fit-up From construction sequencing From incidental thermal loadings or support movement
Loadings	Magnitude and distribution Loading rate and duration

When creating analytical models to represent structures comprised primarily of structural shapes (i.e., beam elements) it is not practical to address all these tolerances other than the member out-of-straightness along the length. The only other tolerance that can be easily introduced into an analytical model is the out-of-plumbness or the allowable erection tolerance. The column plumbness tolerance normal to a building line cannot be greater than 1/500; alignment tolerances for members with field splices not greater than 1/500; and the maximum tolerance between brace points shall not be greater than L/1000.

The COSP makes a Commentary remark directed at trusses where it states, "Trusses fabricated and erected as a unit or as an assembly of truss segments normally have excellent controls on vertical position regardless of fabrication and erection techniques. However, a truss fabricated and erected by assembling individual components in place in the field is potentially more sensitive to deflections of the individual truss components and the partially completed work during erection, particularly the chord members, in such a case, the erection process should follow an erection plan that addresses this issue."

For the top chord, it will be conservative to assume a splice at mid-span. Technically this might not be necessary with a truss span of 100 ft. and a 60 ft. long top chord (the maximum length for a standard truck). However, there are two factors which may drive two sections for the top chord: first, the weight of the truss based on road capacity (20 tons or so); and second, the length of the super-module truss, which may drive the top chord length to greater than 60 ft. long.

3. Analytical Studies

Finite element analysis (FEA) models are created using the commercially available software package ANSYS (2010) to study the buckling behavior of a truss super-module.

3.1 Finite Element Analysis Model Description

3.1.1 General Overview

Several experimental tests have shown the validity of using beam elements in modeling truss systems (Wongjeraphaat, 2011; Eberle et al, 2012). Specifically, these studies were able to show that analytical results from an FEA closely match physical testing when the following analysis requirements are imposed:

- All chord and web members are modeled with beam elements with 6 degrees of freedom at each node
- Each member is composed of several elements to capture $P\delta$ effects
- Initial imperfections are included in the compression chord
- Second-order large deflection analysis is performed

Generally, these requirements are also made in the ANSI/AISC 360-10 (2010) endorsed Direct Analysis Method, along with a reduced stiffness value of 0.8 of the steel elastic modulus to account for residual stresses.

The models reported in this paper satisfy all of these requirements. The ANSYS BEAM 188 element is used, which is capable of 6 degrees of freedom at each node. A cubic displacement function is employed. Each chord, web, or bracing member is composed of 10 BEAM 188 elements, i.e., 10 divisions are used.

Material properties consider 0.8 of steel's elastic modulus (23,200 ksi), with a Poisson's ratio of 0.3. The nominal material yield stress for the main truss members is set to 65 ksi [ASTM A913 (2011) material], while the bracing members have a nominal yield stress of 50 ksi [ASTM A992 (2011) material]. An elastic-plastic material model is considered in the analysis.

To capture this material yielding, an I-section is input directly into ANSYS (2010). This section consists of three plates with their own respective dimensions. This is a simplification to the AISC (2011) manual cross section, which considers a radius at the web-flange junction. Instead, the web depth is taken as the distance between the centerline of the two flanges, and the two flange plates each overlap the web plate by half of their thickness.

Member properties reflect W14x257 top and bottom chord members, W14x176 for larger web members, and W14x90 for smaller web members.

Member out-of-plumbness is considered, as well as member out-of-straightness for the top chord only. The out of plumbness considers a splice point in the top chord at midspan, and an L/500 out-of-plumbness value, where L is 30 ft. An L/1000 member out-of-straightness is also considered for the top chord.

To capture the post-buckling performance of these truss systems, the arc-length method is used, as opposed to the typical Newton-Raphson method. The difference between these two techniques is that the Newton-Raphson method performs linear interpolation between load points, whereas the arc-length method uses an arc to interpolate between load points. Since an arc is being used, this method is able to capture the negative slope on the load-deflection curve of a structure that is losing stiffness (buckling). As a result, the final buckling load need not be known before the analysis is performed. Instead, the initial load step size is chosen, and the analysis is run until a criterion is met (maximum deflection, or maximum number of solution iterations). In these studies, the maximum number of iterations was chosen such that the peak load was achieved, with a declining load-deflection curve when the analysis was terminated.

3.1.2 Modeling of Connections

One of the major difficulties in modeling truss behavior is the connection between truss members. Each web member is connected to either chord, and also to other web members at the working points. Historically, these connections are considered to be pinned connections only capable of transmitting axial forces into the members of the truss. This pinned assumption greatly simplifies the analysis of the truss. However, more recently these connections have been put under great scrutiny.

Wongjeeraphaat (2011) and Wongjeeraphaat and Helwig (2011) looked at modeling a truss composed of light W-shaped members with gusset plate connections on both flanges of the cross section. These studies showed that the stiffening effect of the gusset plates must be accounted for when modeling the lateral stiffness of a single truss. In other words, the truss appears stiffer than a frame composed of only prismatic frame members because the cross section at the connection region is built-up. However, when looking at two-truss systems with bracing members between the two trusses it was found that connection flexibility had a major impact on the system stiffness. Wongjeeraphat (2011) showed that when a brace force comes into a two gusset plate connection, the plates locally deform between the gap of the connecting members as illustrated in Fig. 7. This type of local gusset plate deformation can only be modeled with the use of shell-type finite elements.

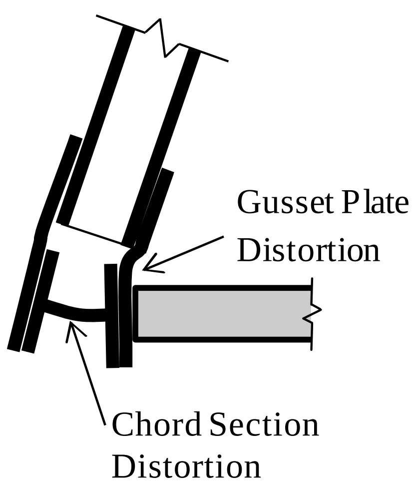


Figure 7: Cross Section Distortion at Brace Connection (Adapted from Wongjeeraphat, 2011)

The analytical models reported in this paper use a rigid connection between the members, with no changes made to account for the stiffening effect of the gusset plates, or their local deformation when bracing is present. This simplification is made to reduce the analysis time, and the time to create the finite element (FE) models.

3.1.3 Brace Configurations

Seven bracing configurations are considered in this study. All configurations are described in the list below:

- Case 1: lateral and diagonal bracing between the bottom chords only
- Case 2: same as Case 1, with the addition of two mid-depth trusses at each vertical of the middle truss panels
- Case 3: same as Case 1, with the addition of two mid-depth trusses at each end-post near end of the truss
- Case 4: same as Case 2, with the addition of kickers extending to the main truss top chord
- Case 5: same as Case 3, with the addition of kickers extending to the main truss top chord
- Case 6: same as Case 1, with the addition of four mid-depth trusses at each panel point
- Case 7: same as Case 3, with the addition of kickers extending to the main truss top chord

Fig. 8 shows a rendering of Case 6 described above while the FE model used to model Case 7 in the analysis is shown in Fig. 9.

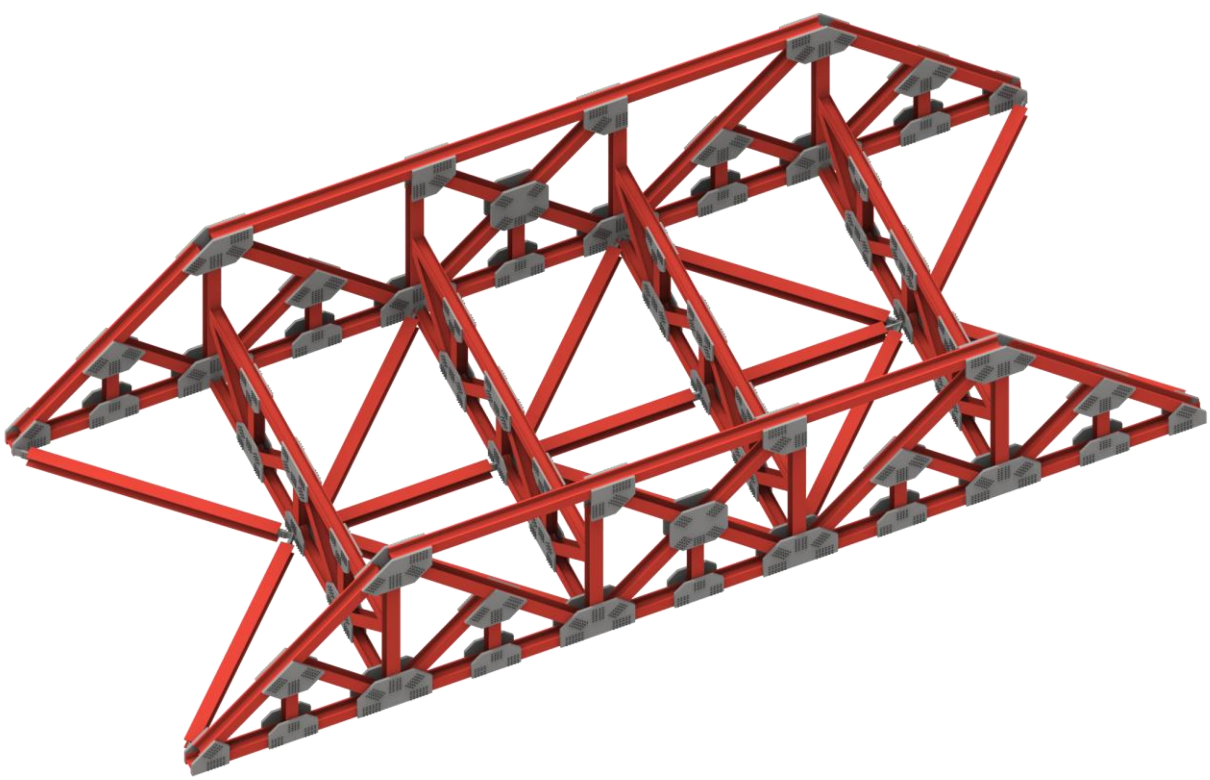


Figure 8: Rendering of Super-Module Truss Assembly

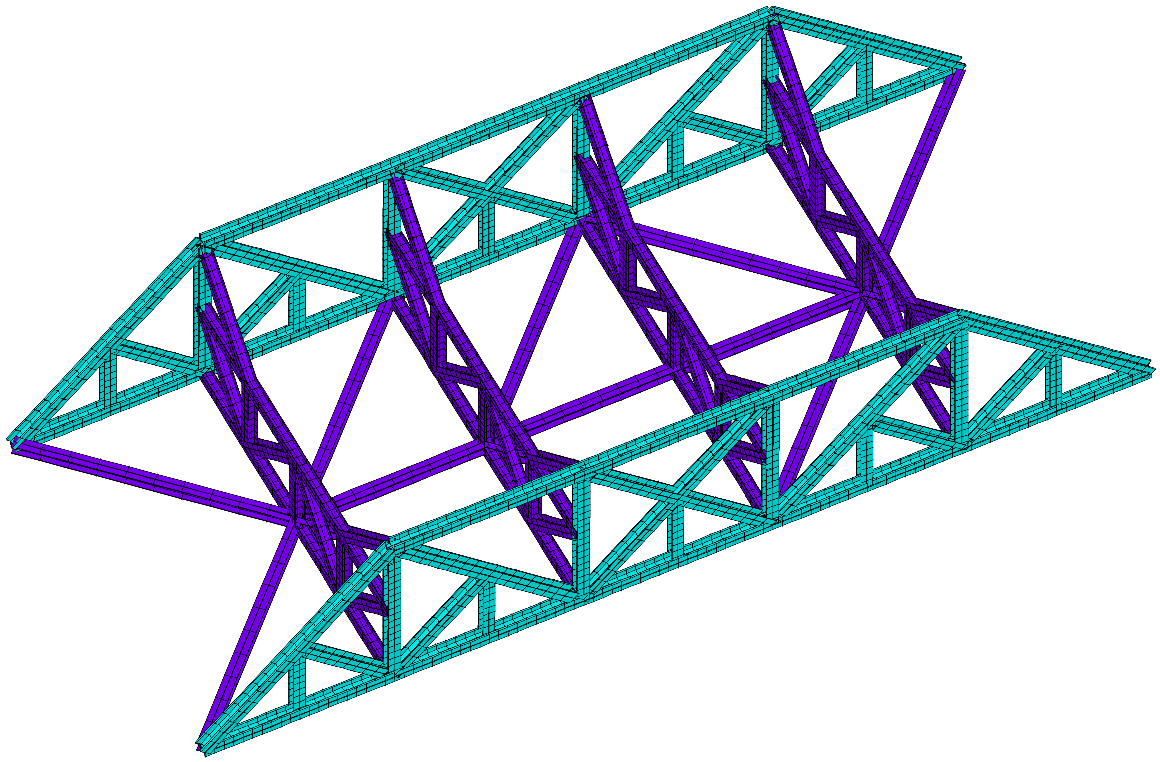


Figure 9: Isometric View of ANSYS Model

3.2 AISC Member Design Equations and Proposed Equations from Research

The ANSI/AISC 360-10 (2010) specification provides specific design equations for the buckling of columns and beams; however, no design equations are provided for truss systems. In lieu of global buckling equations, it is common for member design equations to be used, with brace points being considered as supports. The braces used within these systems are typically designed with the 2% rule: that the brace must be capable of taking 2% of the axial compression of the member being braced. Often the stiffness requirements of these braces are ignored.

The Direct Analysis Method has been endorsed by the AISC specification since 2005. While this approach does address effects such as member out-of-plumbness, out-of-straightness, and residual stresses, it still presents some difficulties in practice. The Direct Analysis Method recognizes that analysis and design are tied together – that the analysis (member forces) drives the design and the design (stiffness) drives the analysis. As a result, the Direct Analysis Method pushes the design to the analyst by requiring additional structural modeling requirements. For instance, the Direct Analysis Method requires the analyst to apply notional loads at the level where the loadings are being applied. However, for a truss with bottom chord loading and bottom chord only bracing, there is no clear requirement to account for the out-of-plumbness or out-of-straightness of the top chord. If these geometric imperfections are included in the model, then the analyst must determine the worst-case imperfections.

Wongjeeraphat (2011) offered design equations for the required brace stiffness (Eq. 1), and the critical buckling load (Eq. 2) of a truss system with a torsional brace at midspan:

$$\beta_{tot} = \frac{M_{cr}^2 L}{2EI_{chord} C_b B} \quad (1)$$

$$P_{cr} = P_o + \left[\frac{B^2 EI_{chord}}{2L^3}\right]\left(\sqrt{(F_o h)^2 + (\frac{2E}{B^4 I_{chord}} + 1000)\beta_T} - f_o h\right) \quad (2)$$

Where P_{cr} is the buckling capacity of the braced truss per joint, P_o is the buckling capacity of the unbraced truss, B is the load height factor, E is the elastic modulus, I_{chord} is the moment of inertia of the chord, L is the span length, F_o is the maximum unbraced chord axial force, h is the truss depth, β_T is the total brace stiffness, β_{tot} is the total required brace stiffness, M_{cr} is the critical moment for a uniform moment, and C_b is the moment gradient factor.

These equations are useful when considering a truss composed of W-shapes with a single brace and a span-to-depth ratio of approximately 20. Unfortunately, the large industrial trusses considered in this paper do not fit those criteria.

3.3 Buckling of Super Module Truss as Function of Brace Configuration

With the geometry of the main truss set, an investigation was made into the buckling capacity of a super-module truss with various brace configurations, which were described in Section 3.1.3. The normalized load, defined as load per panel point/maximum load versus lateral deflection is shown for each case in Fig. 10. Note that the starting displacement represents the lateral deflection under gravity loading and the maximum load used for normalization is 680 kips.

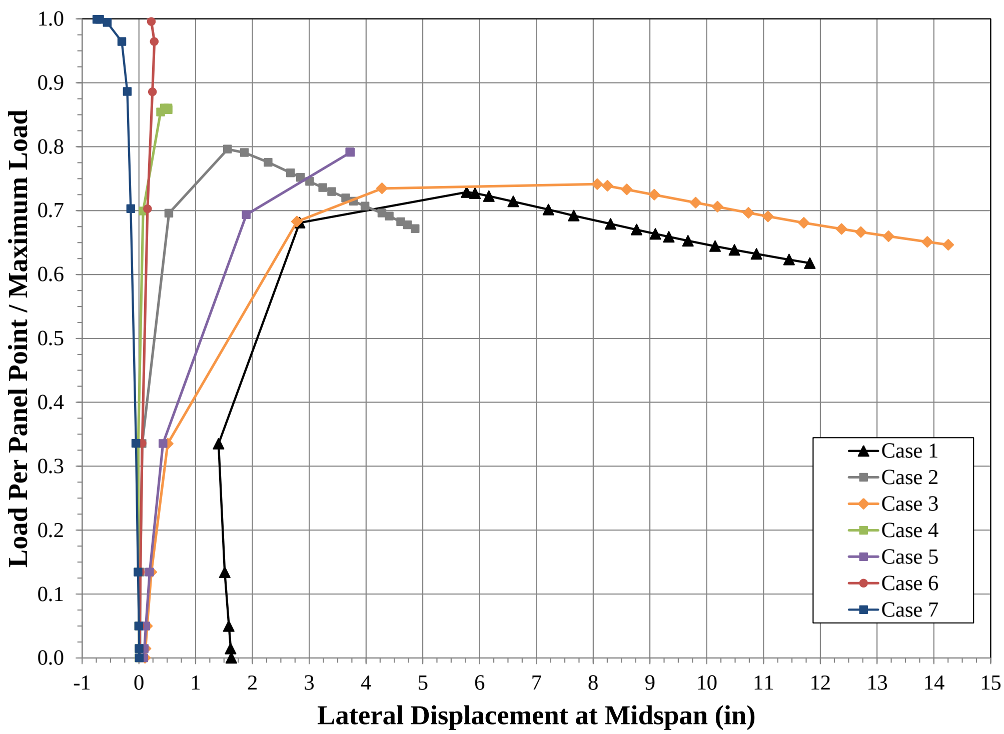


Figure 10: Normalized Load vs. Lateral Deflection Comparison of Bracing Configurations
All Bracing Members are W14x90

Table 2 shows the deflected shapes (a scale factor greater than unity is used for clarity) for Case 1 and Case 6 along with the out-of-plane nodal displacement contours for various levels of normalized buckling load (load per panel point, P_{pp}/maximum load, P_{max}). It should also be noted that since each exaggerated deflected shape has a variable and unique multiplier, the relative deflected shape should only be compared within a figure and not between separate plots.

The results for Case 1 show buckling of the top chord as observed from the large out-of-plane displacements. As the normalized buckling load is increased, the lateral out-of-plane deflections are increased until the top chord achieves a full half-sine buckled shape greater than the length of the member. For Case 1, it is observed that the web members are ineffective at preventing buckling of the truss top chord. Conversely, Case 6 shows very small out-of-plane deflections in the top chord, even as the maximum load for the truss is approached in the analysis. Thus, the half-depth cross trusses at each panel point sufficiently brace the primary trusses.

Finally, a few qualitative observations can be made:

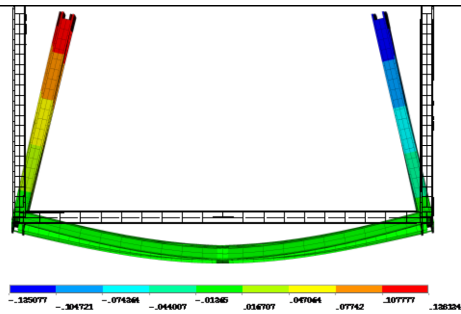
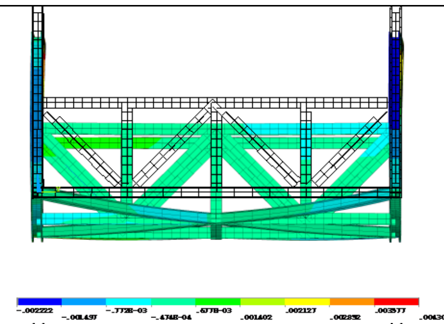
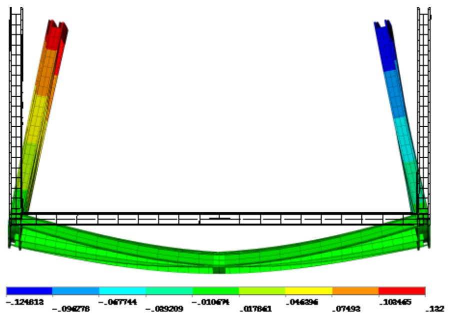
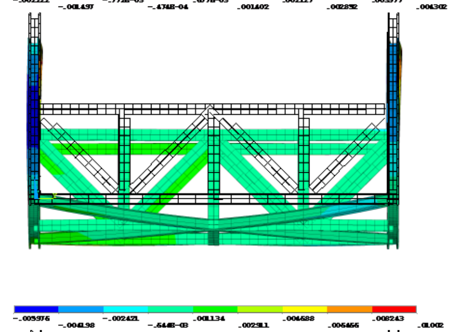
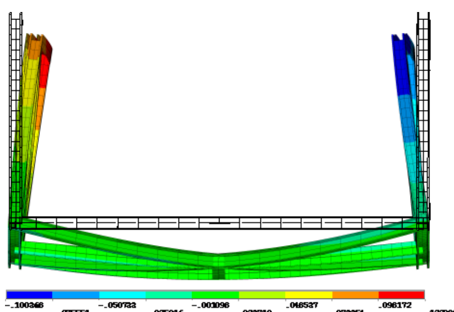
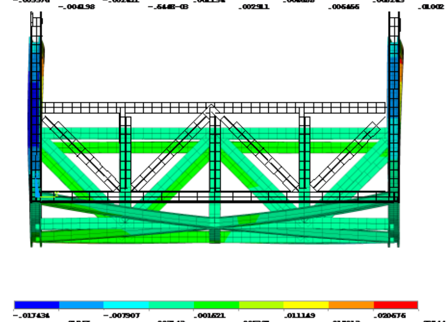
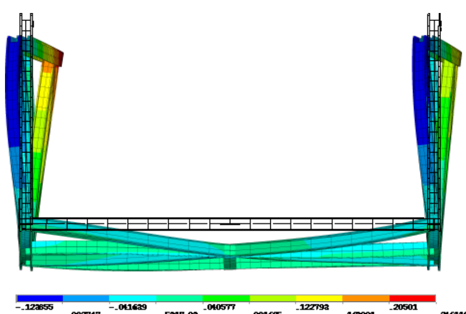
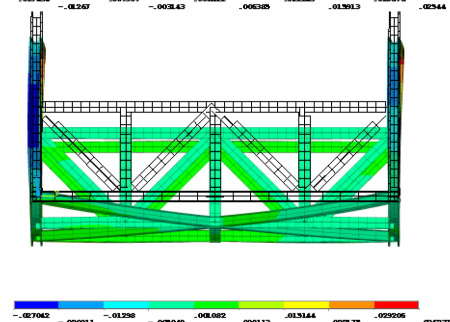
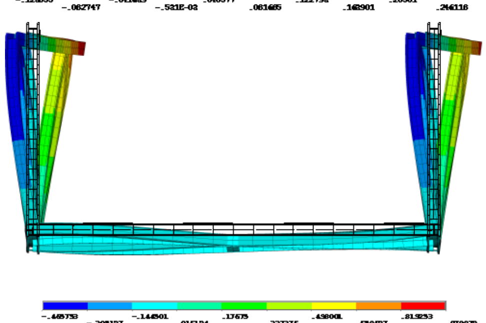
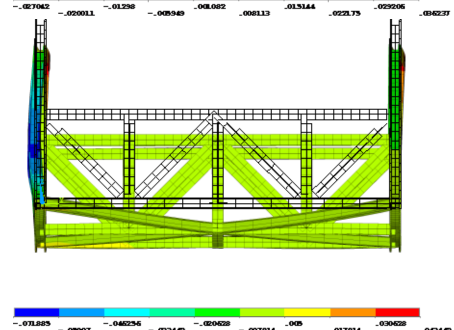
- Case 1 (bottom chord only) shows by far the largest lateral displacement under gravity load, but is still able to achieve over 70% of the maximum load
- Case 1 has large gravity lateral displacement because of the rotation introduced by the bracing member, and the configuration's low out-of-plane stiffness
- Case 2 (two cross trusses near midspan) has a higher buckling capacity than Case 3 (two cross trusses near the supports), as shown in Fig. 10
- Case 6 (four cross trusses) and Case 7 (four crosses trusses with kickers) achieved the maximum load, and show very small lateral deflections at midspan

4. Conclusions

This paper presented a reusable, super-module truss structure for use in the construction of the latest generation of standard nuclear power plant designs. These trusses will be used to support large multi-commodity ground pre-assemblies, then rigged, lifted, and used as support during concrete placement. Details about the design approach of these trusses have been provided, including material selection, and the approach used for bracing.

A parametric study has been performed to evaluate the maximum load-carrying (buckling) capacity of the primary trusses of the super-module when the stiffness and locations of the bracing members or cross trusses (i.e., frames) is varied. It was found that mid-depth cross trusses are effective at bracing the top chords of the trusses. Regarding the cross trusses, this paper does not address the expansion with outriggers, but given the work previously completed by others, working through the specifics of an actual application might have significant value. Thus, such consideration will be the subject of a later study. The results of this study demonstrate that the outer cross trusses function as transfer trusses as well as stability trusses, and the inner cross trusses function as stability bracing.

Table 2: Deflected Shapes of Super-Module with Various Brace Stiffness

P_{pp}/P_{max}	Case 1	P_{pp}/P_{max}	Case 6
0.07		0.13	
0.18		0.33	
0.45		0.71	
0.95		0.88	
1.00		1.00	

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