



## **Structural stability condition assessment of corroded steel trusses in operating industrial facilities**

Hunter J. Brown<sup>1</sup>, Damon G. Reigles<sup>2</sup>, Perry S. Green<sup>3</sup>

### **Abstract**

Industrial facilities in the United States are typically of conventional steel construction, but often times operate with caustic or high internal humidity environments. As these structures approach and exceed their typical 50 year design life, the long span roof trusses that comprise part of the roofing system are beginning to show severe corrosion. This corrosion causes a loss of cross-sectional area in the truss chord and web members, and affects the integrity of truss internal connections, as well as bracing member connections. Faced with these degraded structural conditions, owners must decide whether to continue operations and face a potential catastrophic failure of one or more roof trusses, or shutdown plant operations to repair or replace any highly corroded trusses. This paper will address the unique structural stability aspects of these corroded trusses, and offer suggestions for reinforcing or adding additional braces to continue operations. Parametric buckling analyses using modern stability methods are used to evaluate a range of different truss conditions.

### **1. Introduction**

Due to economic pressures to avoid additional capital expenditures, many owners of industrial facilities seek to repair existing facilities rather than building new facilities. Many industrial structures are approaching or have exceeded their typical 50 year design life. For industrial facilities that have caustic environments and/or very humid environments the rate of corrosion is often accelerated over the lifetime of the structure, especially if the structure has not been well maintained. Depending on the industrial processes performed, the corrosion may be more pronounced in specific regions of the facility, whereas other regions experience little or no corrosion. One example of this is paper mill facilities, where the pulp processing areas experience very pronounced increases in humidity levels in comparison to the end of the structure where the final paper product is fed onto paper rolls.

---

<sup>1</sup> Project Engineer, Martin/Martin Consulting Engineers, < hunterbrownj@gmail.com >

<sup>2</sup> Principal Engineer, engNoveX, Inc., < dreigles@engnovex.com >

<sup>3</sup> Sr. Civil Engineer and Technical Specialist, Bechtel Nuclear Security & Environmental, < perrysgreen.phd@gmail.com >

Fig. 1 shows some examples of steel roof truss corrosion at two different industrial facilities. The height of the long-span trusses shown in Fig. 1 both have spans of approximately 100 ft. with a truss depth of 10 ft. The corrosion for the particular facilities shown in Fig. 1, which both experienced significant steam in the facility due to industrial processes, tended to experience varying amounts of corrosion over the height of the truss, with the worst corrosion focused on the bottom half of the trusses. Since the bottom chord members are in tension for gravity loading, some small amount of section loss of the bottom chords is not typically a cause for immediate concern. However, over time the corrosion also produces cross section loss of the vertical web members and the integrity of the gusset plates that support the web members.

This paper uses recent stability research (Toğay *et al.* 2015) to evaluate the buckling load of a corroded steel truss taken from an actual facility. Some simplifications are made to limit the number of variables evaluated in this study. The buckling load for the corroded truss structure is compared with the buckling load of the truss in its original constructed state (*i.e.*, no corrosion). The intent of this paper is to outline a practical general approach for evaluating corroded long-span steel truss structures, so that an assessment can be made of whether the existing truss has sufficient capacity to carry the loads for which it was designed.

The existing truss structure is composed of WT chord members of the same size: WT7. These chords frame into wide flange columns with a bolted end plate connection. Web members are double angle members varying in size, increasing from midspan towards the support. Single angle in-plane bottom chord and top chord bracing in the form of angle shapes are used. Additionally, full depth cross frames are used at midspan, and three-quarter span. These cross frames are composed of WT sections at the top and bottom chord elevation, and single angle back-to-back members in an X configuration. The roof framing is composed of W8 purlins spanning 18'-0" between trusses. These purlins connect into the top chord with a bolted connection from the flange of the WT chord member to the bottom flange of the W8. These W8 purlins support 1.5" deep roof deck. The original roof was designed with a 10 PSF superimposed dead load, and 20 PSF superimposed live load.



Figure 1: Examples of Corrosion in Steel Roof Trusses in Two Different Industrial Facilities

## 2. Methodology for Stability of Trusses

### 2.1 Overview

Planar truss systems are inherently complex not only because of the varied geometrical forms that they can have, but also due to their structural behaviors as an overall assembly that is comprised of tension, compression, and flexural members. These individual members can have a variety of

possible local buckling failure modes that may govern the truss design. Alternatively, the overall truss system itself may be controlled by a global buckling mode since its out-of-plane stiffness relative to its in-plane stiffness is very low. The global truss behavior is dependent on the contribution of the top chord, bottom chord, and web members, but also the connectivity between these elements. The web member connections into the top and bottom chords at discrete locations will determine their ability to restrain the out-of-plane movement of the compression chord similar to a beam web restraining the compression flange. Therefore, the out-of-plane stiffness of the web elements will add another degree of uncertainty into the general behavior of the truss system.

When the members of a roof truss, *i.e.*, tension chord, compression chord, tension or compression diagonals and vertical webs, are reasonably proportioned then the overall buckling behavior of the truss system will be dictated by the design of the lateral or transverse bracing (bridging) provided. This is especially true when the chords are essentially parallel. Additionally, secondary members framing into the truss top chord have an impact. These members often act as lateral bracing, torsional bracing or a combination of both. Depending on their strength and stiffness they will affect the buckling behavior of the overall truss. If they are capable of restraining global rotation of the truss, they may prevent global buckling modes. The governing truss failure modes will be global out-of-plane buckling similar to lateral-torsional buckling of a beam or local, in-plane, out-of-plane, or torsional buckling of an individual chord segment between panel points. Additionally, web members may buckle under compression, which would be similar to local flange or web buckling.

If the bracing provided is insufficient in either strength or stiffness then an overall buckling failure mode can occur. Under gravity loading the bracing attached to the top chord needs to be designed to prevent its out-of-plane buckling, whereas when the loading is wind uplift the bracing needs to be attached to the bottom chord to prevent its buckling out-of-plane. When considering a local buckling mode, column buckling provisions are most appropriate. However, overall buckling modes are similar to a flexural member, which would suggest that beam bracing provisions are appropriate. Correctly applying both column and beam bracing provisions to ensure a stable system is non-trivial.

## 2.2 AISC Provisions for Trusses and Beams

Prior to the current bracing provisions being added to the specification, many issues such as secondary stresses in trusses were addressed by Nair (1988). The current commentary to the AISC 2016 Specification, Appendix 6 (AISC 2016) states that parallel chord trusses with both chords subjected only to flexural loading, where both chords are extended to the end of the span and attached to supports, can be treated the same as beams. Cross section distortion effects,  $\beta_{sec}$ , need not be considered when full-depth cross-frames are used for bracing. Therefore, the bracing stiffness requirements reduce to Eq. 1.

$$\beta_T = \frac{1}{\phi} \frac{2.4L}{nEI_{yeff}} \frac{M_r}{C_b}^2 \quad (1)$$

where  $E$  is the modulus of elasticity,  $I_{yeff}$  is the moment of inertia of both chords,  $L$  is the unbraced length,  $C_b$  is the moment gradient factor (equal to 1.14 for uniform loading),  $\phi$  is the strength

reduction factor,  $n$  is the number of braced points, and  $M_r$  is the maximum value of the required flexural strength, equal to the maximum chord compressive force times the depth of the truss.

### 2.3 Wongjeeraphat & Helwig

Extensive work was performed at the University of Texas at Austin in determining stability bracing requirements for truss systems (Wongjeeraphat 2011). The study included both experimental testing as well as parametric finite element analyses (FEA), which looked at the behavior of twin trusses with and without intermediate bracing. The study led to the development of the stiffness requirement for the truss system with a brace at midspan, which is presented below. Eq. 2 is applicable to relatively shallow trusses in the elastic buckling range.

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

where  $B$  is the load height factor (equal to 1.0 for top chord loading),  $I_{chord}$  is the moment of inertia of a single chord, and the other variables are defined in Eq. 1.

The FEA modeling approach is similar to the approach taken in this paper. The truss, which was comprised of wide flange chord and web members oriented such that the flanges aligned, was modeled in ANSYS using 6 degree-of-freedom BEAM44 (ANSYS 2009) elements. Web members were modeled with both fixity and moment end releases at connections to chords. Ultimately, Eq. 2 was derived based on modal buckling analyses using releases at web members, which is consistent with typical design practices.

### 2.4 Effects of Stiffness Reduction at Cross Frame Members

As mentioned earlier, the primary torsional bracing for the roof trusses in this study are full depth cross frames composed of single angle members in a back-to-back orientation. Work at the University of Texas at Austin (Wang 2013) has been performed on accurately modeling the stiffness of cross-frames for bridge girders. These studies have shown that modeling cross-frames with line elements tends to significantly overestimate their stiffness. The analytical solution ignores bending related to eccentric connections of single angle members, reducing the stiffness of the members greatly. While shell element models can accurately capture this effect, this modeling approach adds a large amount of complexity. The results of these studies produced Eq. 3, which provides a stiffness reduction factor that when applied to a line element model more accurately represents the true stiffness of the bracing members.

$$R_{est-SX} = 1.063 - 0.087 \frac{S}{h_b} - 0.159y - 0.403t \quad (3)$$

where  $S$  is the spacing of the two girders,  $h_b$  is the brace height,  $\bar{y}$  is the centroid location of the angle, and  $t$  is the angle leg thickness.

For the cross-frames in the roof trusses evaluated in this study, the stiffness reduction equates to 0.64, which is only slightly greater than the stiffness that would be achieved assuming a tension only configuration. However, the stiffness reduction approach is used here for completeness of the model.

### 2.5 Material Models

The material model used in the ANSYS (ANSYS 2018) model is based on the recent work presented by Green and Varma (2018). The true stress-natural plastic strain curve is shown in Fig. 2. The material used in the original truss construction is unknown; however, for the purposes of this study the behavior of ASTM A36 steel as shown in Fig. 2 is used. If the age of the truss was confirmed, and the material had a yield stress of 33 ksi it would be reasonable to scale the ASTM A36 steel stress-strain curve as shown by the thinner curve in Fig. 2. For a design engineer with a unique structure, additional guidance is provided in AISC Design Guide 15 (AISC 2018).

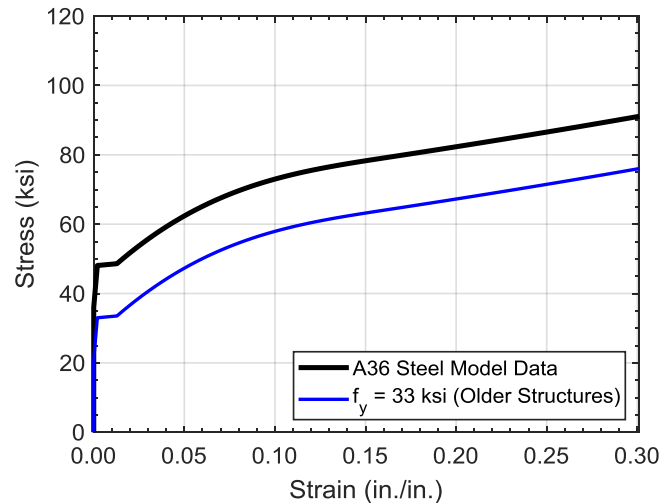


Figure 2: True Stress-Natural Plastic Strain Curve for ASTM A36 Steel Used in ANSYS Finite Element Model -Adapted from Green and Varma (2018)

## 3. Corrosion Effects on Stability

### 3.1 Background

Corrosion is a general term used to describe the degradation of steel due to its interaction with the surrounding environmental conditions, whether due to nature or man-made. Structures or structural members experience accelerated rates of corrosion when in close proximity to salt water in coastal areas as well as when exposed to corrosive chemicals that might be expected in facilities constructed to store road salt or bulk fertilizer.

Rusting, for example, is a common type of corrosion that occurs when structural steel reacts with oxygen in the presence of compounds. Unmitigated corrosion of steel (*i.e.*, allowing steel to continue rusting for extended periods of time) leads to reduction in cross section of structural steel members, and can eventually lead to the failure of structural members or assemblies such as trusses.

ISO Standard 9223 (2012) provides a classification system for atmospheric environmental corrosivity that is widely used. This standard breaks down into corrosivity categories from C1 (mild) to C5 (severe) with an additional category, C5M (severe marine) for marine exposures. The expected range of corrosion rate for each classification is shown in Table 1. While this standard is not widely used in the commercial and industrial structural engineering industry in the U.S., it has gained popular use for offshore and utility structures.

Table 1: Carbon Steel Corrosion Rates for Various Environments According to ISO 9223 (2012)

Environment	Carbon Steel Corrosion Rate (mils per year)
C1	0.05
C2	<1
C3	1 to 2
C4	2 to 3
C5	3 to 8
C5M	8 to 28

In the bridge industry, it is commonly observed that the upper surface of I-shaped bottom flanges will corrode before the rest of the I-shape. The lower portions of flanged structures corrode early due to longer durations of wetness for these parts relative to the rest of the structure, since they are horizontal and can hold moisture and debris. The longer duration of wetness is also caused by preferential condensation on the lower portions of a steel element. Similarly, the roof truss examined in this paper experienced most of its corrosion at the bottom chord WT, whereas most the remainder of the truss members experienced only minor, superficial rusting.

Fig. 3 shows the naming convention for the corrosion induced cross section loss noted in Table 2. Note that at the worst-case flange thickness loss, Section A-B, the total loss is approximately 0.4". For an approximately 70 year structural life, that equates to approximately 5.4 mils/year of cross section loss, which is equivalent to a C5 corrosion environment per ISO 9223 (2012). This corrosion environment for building interiors is described as areas with almost permanent condensation and with high pollution.

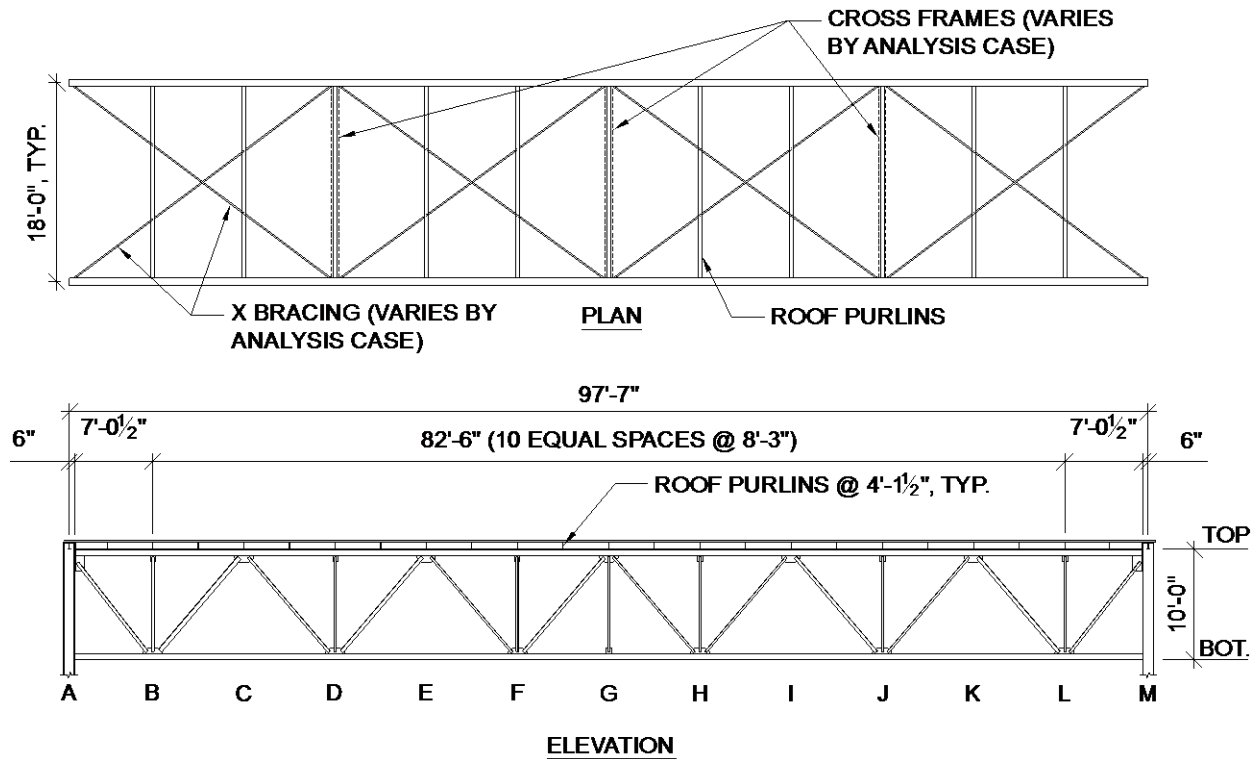


Figure 3: Elevation View Showing Truss Geometry and Naming Convention Key and Plan View Showing Bracing Between Trusses for Various Analysis Cases

Table 2: Bottom Chord Flange Thickness Field Measurements.

Section	Flange Thickness (in.)
A-B	0.224
B-D	0.415
D-F	0.505
F-H	0.531
H-J	0.572
J-L	0.622
L-M	0.592

### 3.3 ANSYS Model Description

The FEA model was developed using the commercially available general finite element software ANSYS (2018). The elements used in the models were line elements, including the BEAM188 (ANSYS 2018) element for chord members with moment connections and the LINK180 (ANSYS 2018) element for web members and bracing members. The roof purlins were modeled using BEAM188 elements without moment end releases at truss connections. This approach matches the idealized conditions that are often assumed in design practice for trusses, and is considered a reasonable approach for the truss configuration under study, which involves WT chords with double angle web members. The cross-sectional properties match the existing roof truss structure, except where simplifications were practical. Specifically, the existing truss web members were not symmetric about the centerline of the truss. Instead of matching those member's cross-sectional properties, it was assumed that the truss web members on the south side of the truss were identical to those on the north side. The BEAM188 elements are modeled accounting for actual cross section properties, while the LINK180 elements are only modeled with a cross-sectional area. All BEAM188 elements are modeled using 8 divisions along the segment, and LINK180 elements use only one division. An isometric view of the ANSYS model is shown in Fig. 4.

A major disadvantage of using line elements is the local effects at connections are not captured. Thus, gusset plates and connection details are not accounted for in this model. Also, cross section distortion of individual connection components or framing members is not captured. These modeling choices tend to overestimate the stiffness of the truss, but the LINK180 web elements tend to underestimate the stiffness of the truss.

Boundary conditions are idealized, similar to what is done in typical design practice. The top and bottom chords are provided with out-of-plane and vertical restraints at both ends. Longitudinal restraints are provided at only one end of the truss. The existing truss details do not provide for longitudinal slip in the chord connections to the existing column. However, this was not captured as the approach is atypical, and the flexural stiffness of the column supporting the truss is relatively small compared to the truss.

Two loading conditions are considered: 1) top chord downward (gravity) loading applied to the truss at purlin locations, and 2) top chord upward (wind uplift) loading applied to the truss at purlin locations. Purlins are spaced at approximately 4'-0" on center, whereas the truss panel points are spaced at approximately 8'-0" on center. This means that the top chord experiences both axial compression from the overall behavior, and local bending between panel points.

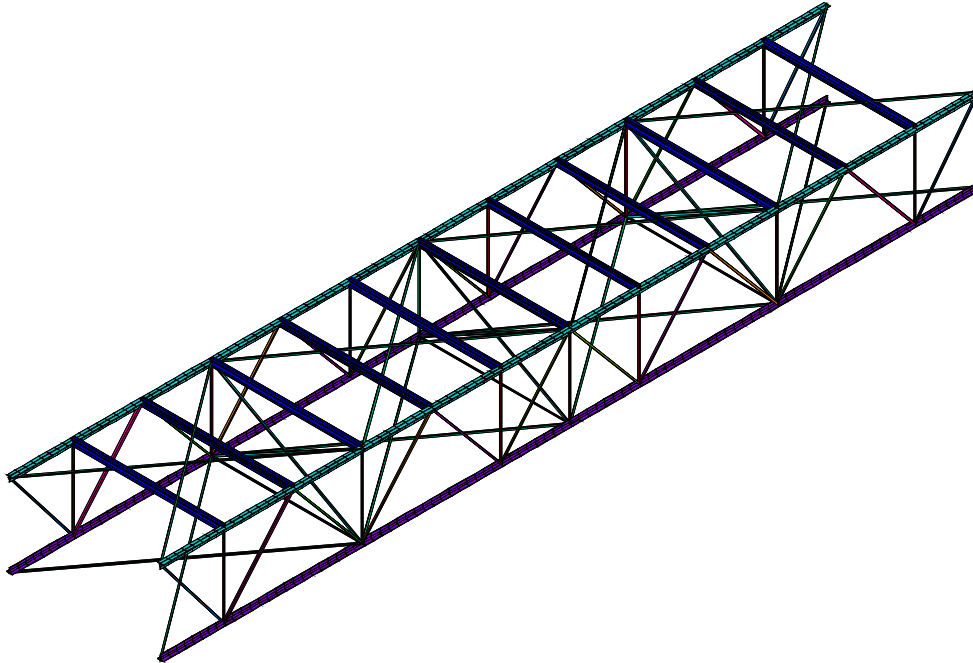


Figure 4: Isometric View of ANSYS Model Elements

Other global effects in the existing structure are not captured in the FEA model. For instance, there are multiple trusses relying on each cross frame – six (6) trusses in total are restrained from rotation from a set of cross frames. In the FEA model, only two (2) trusses are studied. Additionally, because the trusses are built-in to the columns at each end, they will participate in the building lateral system, and this is not captured in the FEA model.

### 3.4 Stiffness Reduction Factor

Multiple approaches exist to evaluate the strength and stability of steel structures. Design for stability requires considering all of the following: member deformations, connection deformations, second order effects, geometric imperfections, inelasticity, partial yielding, and uncertainty in material and cross-sectional properties. The Direct Analysis Method of Design is endorsed by the latest 2016 AISC Specification (AISC 2016), but requires explicit modeling of member imperfections, that is, member out-of-straightness, and member out of plumbness. This approach requires some knowledge with regard to the buckling shape of the structure, which is difficult to predict for a truss system. Recent approaches have provided a comprehensive approach for the design checking of structural steel members and their bracing systems via the use of buckling analysis combined with appropriate column or beam stiffness reduction factors. Details are provided in Toğay *et al.* (2015), and summarized below:

In the 2016 AISC Specification (AISC 2016) column buckling provisions involve two regimes: 1) an elastic buckling range, and 2) an inelastic buckling range, both of which require a  $\phi$  factor equal to 0.90 to capture uncertainty in materials, analysis, and loading. Additionally, elastic buckling is calculated as the Euler buckling load, with a 0.877 factor to account for initial out of straightness of the column. For inelastic buckling additional factors are used to account for initial residual stress effects and the spread of plasticity. These factors are often captured in a stiffness reduction factor,  $\tau$ . The approach presented in Toğay *et al.* (2015) incorporates these stiffness reductions in a direct buckling analysis, such that the structural buckling load from analysis can



directly satisfy the 2016 AISC Specification (AISC 2016) column requirements without further checking.

Furthermore, Toğay *et al.* (2015) suggested a process for solving buckling analysis problems to determine column strengths. First, an overall analysis model is constructed. Second, the desired factored loads are applied to the model. Third, the elastic modulus of the structural members,  $E$ , is reduced by  $(0.9 \times 0.877 \times \tau_a)$ . Lastly, solve for the inelastic buckling load. The applied loads are varied until the stiffness reduction factor converges and the applied load factor,  $\gamma$ , is 1.0.

Note that stiffness reductions in this analysis are not taken for flexural bending because overall bending stresses in the truss are below the threshold for taking additional stiffness reductions. However, the purlins do see a large amount of bending. Also, the torsional restraint provided by the purlins does have a major impact on the primary buckling mode of the truss. The various stiffness reduction factors are shown in Table 3.

Table 3: Stiffness Reduction Factors at Various Members

Member	Reduction
Compression Chord	$0.9 \times 0.877 \times \tau_a$
Tension Chord	0.9
Web	$0.9 \times 0.877$
Purlins	$0.9 \times 0.877$
Bracing Members	$R_{est-SX} \times 2/0.75$

The stiffness reduction factor in the top chord is consistent with the approach presented in Toğay *et al.* (2015). The stiffness reduction for the web elements and purlins does not play a major role in the truss buckling mode, but are reduced for consistency of the approach. The tension chord reduction only accounts for a member  $\phi$  factor. Additional reductions for out-of-straightness are not appropriate since there are no second order effects. Finally, the load magnitudes are low enough that partial yielding in the tension chord does not occur. The bracing member stiffness reduction factors account for second order effects in the angles (Wongjeeraphat 2011), and also consider the required bracing stiffness reduction for column panel bracing compared to the ideal brace stiffness.

## 4. Results

### 4.1. Buckling Behavior of Existing Truss System

Numerous models were created to study the behavior of the existing truss. A summary of results is provided in Table 4. The buckling load listed in kips is the total load, including self-weight, that the truss can support. The buckling load, listed in pounds per square foot (psf), is the total distributed load that the truss can support; this value does not include the self-weight of the truss, which is why case E6 shows a negative value. Isometric views of several of the ANSYS buckled shapes are shown in Fig. 5. The undeformed shapes are also shown as wireframe in Fig. 5 for comparison.

Case E1 represents the capacity of the existing truss without corrosion at any of the members. Although the compression chord stiffness reduction factor is not listed,  $\tau_a$  is equal to 1 for all cases except case E7. Case E2 is similar, but with uplift loading. Note that the buckling capacity is lower because the roof purlins do not provide bracing to the compression chord in this case.

Case E3 to E6 are various bracing configurations. Case E7 considers a lower material yield strength; note that the results are similar to Case E1, as the stiffness reduction factors are only slightly lower (stiffness reduction factor near midspan is equal to 0.766).

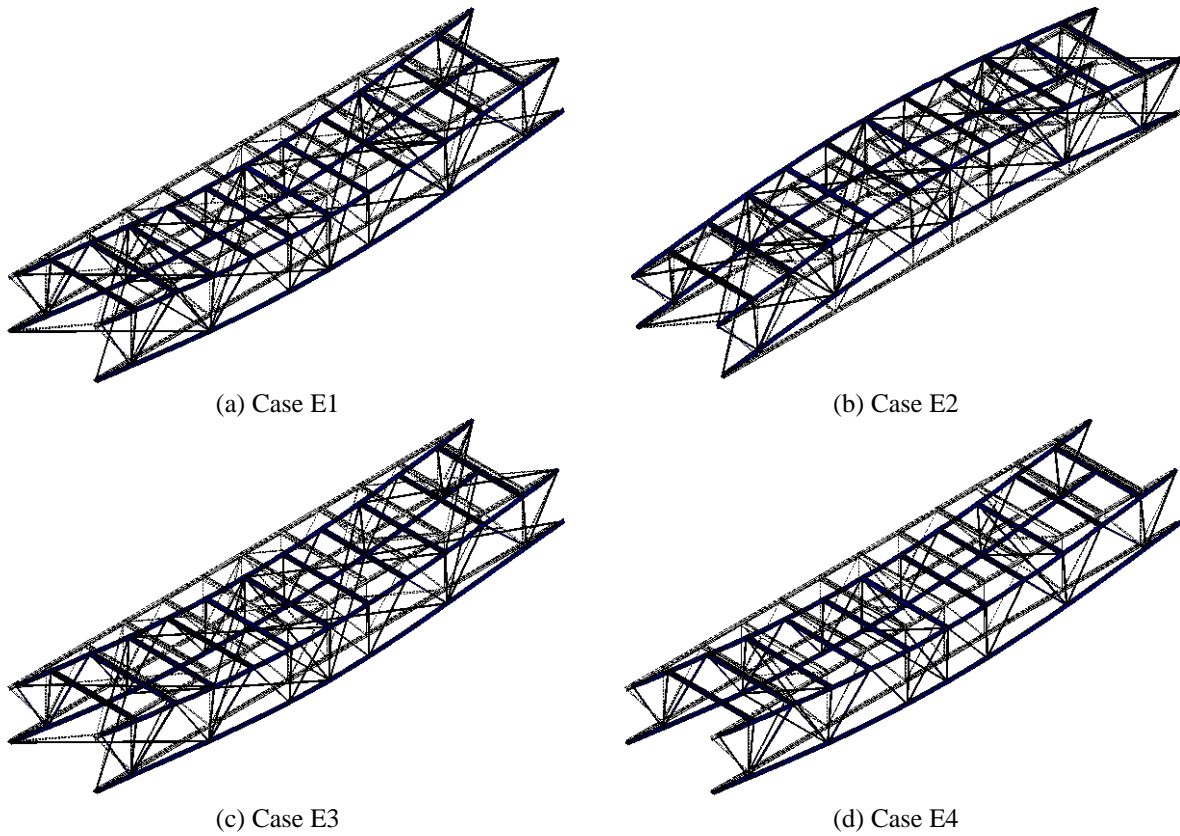


Figure 5: Isometric View of ANSYS Model Showing Buckled Shapes for Various Cases

Table 4: Buckling Load Results for Various Cases

Case No.	Load Factor	Comment	Buckling load (kip)	Buckling load (psf)	Bracing SRF	Ten. Chord SRF	Web SRF <sup>1</sup>	Comp. Chord SRF <sup>1</sup>	Purlin SRF <sup>1</sup>
E1	3.56	Actual structure $F_y = 48$ ksi, no corrosion	81.88	39.8	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E2	3.13	Uplift loading	71.99	41.0	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E3	3.47	Single cross frame	79.81	45.4	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E4	3.27	No in-plane chord bracing	75.21	36.0	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E5	1.02	Same as #4, single cross frame	23.46	6.5	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E6	0.33	Same as #4, no cross frames	7.59	-2.5	0.24	0.9	0.789	$0.789 \tau_a$	0.789
E7	3.56	$F_y = 33$ ksi	81.88	39.8	0.24	0.9	0.789	$0.789 \tau_a$	0.789

1.  $0.9 \times 0.877 = 0.789$

#### 4.2 Effects of Stiffness in Purlin Connection at Top Chord

There are numerous complexities at the purlin to top chord connection. The bottom flanges of the purlins are connected to the top flange of the truss top chord, with a bolted connection between

the two flanges. Since the existing drawings for this detail were not available, it was difficult to quantify the stiffness of this connection. As a result, a sensitivity study was performed to understand how the purlin stiffness affected the buckling capacity of the system. Additionally, the analyses performed do not account for inelasticity due to out-of-plane bending of the purlins. The results in Table 5 show that the overall buckling capacity is fairly insensitive to the purlin stiffness. As a result, the stiffness reduction factor used at the purlins was taken at 0.789 for all other cases.

Table 5: Buckling Load Results for Purlin Sensitivity Study

Case No.	Load Factor	Comment	Buckling load (kip)	Buckling load (psf)	Bracing SRF	Ten. Chord SRF	Web SRF <sup>1</sup>	Comp. Chord SRF <sup>1</sup>	Purlin SRF
P1	3.56	Purlin stiffness sensitivity	81.88	39.8	0.24	0.9	0.789	0.789 $\tau_a$	1
P2	3.55	Purlin stiffness sensitivity	81.65	39.7	0.24	0.9	0.789	0.789 $\tau_a$	0.3
P3	3.52	Purlin stiffness sensitivity	80.96	39.3	0.24	0.9	0.789	0.789 $\tau_a$	0.2
P4	3.25	Purlin stiffness sensitivity	74.75	35.7	0.24	0.9	0.789	0.789 $\tau_a$	0.1
P5	3.09	Purlin stiffness sensitivity	71.07	33.6	0.24	0.9	0.789	0.789 $\tau_a$	0.05

1.  $0.9 \times 0.877 = 0.789$

#### 4.3 Truss Behavior Without Purlin Framing

To better understand the behavior of the truss system without the purlin roof framing, several analyses were performed. The results shown in Table 6 show the buckling load capacity of the roof truss system in various configurations excluding the purlins. The results show the impacts of the number of cross-frames, and the in-plane bracing at the top and bottom truss chords. These results show that both cross-frames, when provided in an adequate number, and in-plane bracing provided at both the top and bottom truss chords are able to adequately brace the truss system. Essentially, if the bracing system is capable of providing torsional stiffness to the trusses, then it is capable of adequately bracing the system. Isometric views of several of the ANSYS buckled shapes are shown in Fig. 6. The undeformed shapes are also shown as wireframe in Fig. 6 for comparison.

Table 6: Buckling Load Results for Cases Without Purlin Framing

Case No.	Load Factor	Comment	Buckling load (kip)	Buckling load (psf)	Bracing SRF	Ten. Chord SRF	Web SRF <sup>1</sup>	Comp. Chord SRF <sup>1</sup>	Purlin SRF
B1	0.10	No bracing	2.30	-5.5	0.24	0.9	0.789	0.789 $\tau_a$	N/A
B2	0.76	Single cross-frame	17.46	3.1	0.24	0.9	0.789	0.789 $\tau_a$	N/A
B3	1.19	Two cross-frames	27.32	8.7	0.24	0.9	0.789	0.789 $\tau_a$	N/A
B4	3.19	Three cross-frames	73.35	34.9	0.24	0.9	0.789	0.789 $\tau_a$	N/A
B5	2.78	In-plane bracing both chords	64.03	29.6					
B6	0.35	In-plane bracing top chord only	8.00	-2.3	0.24	0.9	0.789	0.789 $\tau_a$	N/A

1.  $0.9 \times 0.877 = 0.789$

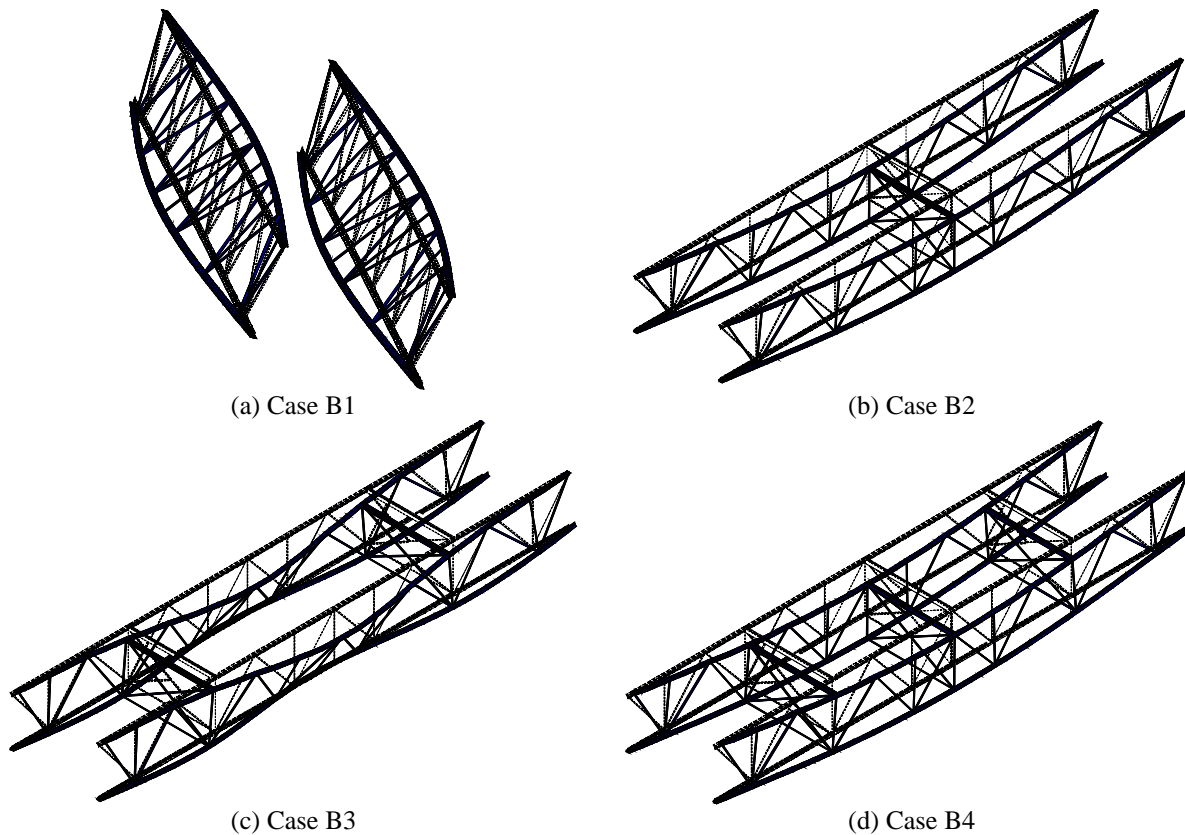


Figure 6: Isometric View of ANSYS Model Showing Buckled Shapes for Cases with No Purlins

The load factor factors noted in case B4 (3.19) and case B5 (2.78) are very similar, suggesting that both bracing configurations provided similar effectiveness. However, both values are lower than case E1 (3.56). This is because case E1 includes the lateral and rotational stiffness of the purlin framing. These analyses can be compared to a more traditional analysis: if a designer were to assume the top chord was adequately braced between cross frames, and it buckled as an assumed half-sine shape ( $KL = L$ ), then a load factor of 2.24 can be calculated. This value is less than the results shown in Table 6 since continuity effects are accounted for in these analyses, while a more traditional approach does not capture continuity.

#### 4.4 Effects of Bottom Chord Corrosion

The results in Table 7 provide structural capacities for the truss, including the effect of bottom chord corrosion. Cases C1 and C2 represent the actual cross section loss in the bottom chord as listed in Table 2. Cases C3 to C11 represent an assumed cross section loss along the entire length of the bottom chord. For instance, Case C3 accounts for a 10% reduction in the bottom chord WT flange thickness.

Interestingly, the truss bottom chord yield strength never controls over the buckling load listed below. This is somewhat counter-intuitive as one might think that the bottom chord stiffness does not contribute to the overall buckling capacity of the truss. However, the results presented below are evidence against that idea.

Table 7: Buckling Load Results for Various Cases

Case No.	Load Factor	Comment	Buckling load (kip)	Buckling load (psf)	Bracing SRF	Ten. Chord SRF	Web SRF <sup>1</sup>	Comp. Chord SRF <sup>1</sup>	Purlin SRF <sup>1</sup>
C1	3.11	Measured corrosion in bottom chord	71.53	33.9	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C2	2.74	Same as #11, uplift loading	63.02	35.9	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C3	3.19	Bottom flange thickness x 0.9	73.37	34.9	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C4	2.82	x0.8	64.86	30.1	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C5	2.44	x0.7	56.12	25.1	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C6	2.06	x0.6	47.38	20.1	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C7	1.69	x0.5	38.87	15.3	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C8	1.32	x0.4	30.36	10.5	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C9	0.987	x0.3	22.70	6.1	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C10	0.671	x0.2	15.43	2.0	0.24	0.9	0.789	0.789 $\tau_a$	0.789
C11	0.355	x0.1	8.17	-2.2	0.24	0.9	0.789	0.789 $\tau_a$	0.789

1.  $0.9 \times 0.877 = 0.789$

## 5. Discussion

Aging industrial facilities present challenges to both owners of these facilities, as well as structural engineers working on them. Owners, faced with rising maintenance costs and increasing corrosion issues, want to know how long they can operate a facility before a structural issue will become critical. Maintenance is often deferred as a cost savings measure. This can lead to circumstances where structural corrosion of the primary building structure can pose a large financial risk, and a risk to life safety.

The truss studied in this paper is one such instance. The existing literature for this condition – heavily corroded structures – is very limited. Solutions for this problem are numerous and vary depending on a case by case basis, but include replacing the entire structure, or a portion, shoring the structure, providing strategic reinforcement, installing a corrosion protection system, or continuing operation despite the corrosion. Structural engineers must know how to assess the structural capacity of these heavily corroded structures.

Often times structural engineers act within a team to solve these problems: the owner, a general contractor, and a corrosion specialist are also involved. There are several limitations to the study presented here. While the global buckling capacity of the truss was analyzed, the study did not analyze local buckling capacity of web members or gusset plates that may have reduced capacity due to corrosion. Additionally, a uniform cross section loss was assumed across portions of the span; however, it's likely that localized areas of the existing structure were more heavily corroded. It is possible that entire portions of the cross section were lost due to corrosion. Other unique conditions can exist at connections such as packing of rust causing movement of connecting plates, and localized corrosion around bolted connections.

The results presented in this paper show that for this truss configuration bottom chord corrosion has the effect of reducing truss buckling capacity upon any loss of cross section. Given the design of the truss studied, which included a consistent chord size and corrosion induced cross section

loss in the tension chord, the top chord and overall buckling behavior of the truss still controlled the design of the truss system.

Future research is needed in both the impacts of corrosion on structural stability, as well as global truss buckling behavior. The local effects associated with corrosion at connection elements or web members also needs further study. Localized corrosion of web elements likely impacts the stability of those members, and as a result the stability of the truss as a whole. While this paper builds on the work by Wongjeeraphat (2011), there is a lack of research on the numerous configurations that steel trusses may take. The effects of purlin framing, various bracing configurations including cross-frames, and end supports still need further study. However, the approach presented by Toğay *et al.* (2015) is considered to be a good approach for unique conditions where structural engineers are looking for more insight than provided by the current AISC Specification (AISC 2016).

## 6. Conclusions

Aging industrial structures present a unique stability problem both in their structural system, and extensive corrosion that may exist. Long span roof trusses are complicated because they can have many geometrical forms, and also because they exhibit both local buckling behavior, and global buckling behavior, with interaction between the two. This analysis shows that the stiffness reduction methodology is well suited for this type of problem. Conservative approximations can be made with regards to web member fixity (pinned), and purlin connections where appropriate. Additionally, research exists for more accurately modeling of the stiffness of full depth cross frames that can account for the un-conservatism in using line elements. This paper showed that for the truss system being studied that corrosion impacts the buckling capacity of the truss system, and as a result the overall load carrying capacity of the truss.

## References

- AISC (2016). "ANSI/AISC 360-16 Specification for Structural Steel Buildings." American Institute of Steel Construction, Chicago, IL.
- AISC (2018). "AISC Design Guide 15: AISC Rehabilitation and Retrofit Guide – A Reference for Historic Shapes and Specifications." Second Edition, American Institute of Steel Construction, Chicago, IL.
- ANSYS (2018). "Element Reference." Release 19.2, ANSYS, Inc., Canonsburg, PA, August.
- ANSYS (2009). "Element Reference." Release 12.1, ANSYS, Inc., Canonsburg, PA, November.
- FHWA Publication No. FHWA-HIF-16-002. (2015). "Steel Bridge Design Handbook: Corrosion Protection of Steel Bridges." U.S. Department of Transportation, Federal Highway Administration.
- Green, P.S., and Varma, A.H. (2018). "A New Stress-Strain Material Model for Ductile Steels in Numerical Analysis – A Rational Approach." *Eighth International Conference on Thin-Walled Structures – ICTWS 2018*, Lisbon, Portugal, July 24-27.
- ISO 9223. (2012). "Standard 9223 – Corrosion of Metals and Alloys – Corrosivity of Atmospheres – Classification, Determination, and Estimation." *International Standards Organization (ISO)*. Geneva, Switzerland.
- Nair, S. (1988). "Secondary Stresses in Trusses" *Engineering Journal*. American Institute of Steel Construction, Chicago, IL.
- Toğay, O., Jeong, W.Y., Bishop, C.D., and White, D.W. (2015). "Toward A More Comprehensive Approach for Design Using Buckling Analysis." *Proc. of the Annual Stability Conference Structural Stability Research Council*, Nashville, TN. March 24-27.
- Wang, W. (2013). "A Study of Stiffness of Steel Bridge Cross Frames." Doctoral Dissertation, The University of Texas at Austin, Austin, TX.
- Wongjeeraphat, R. (2011). "Stability Bracing Behavior for Truss Systems." Doctoral Dissertation, The University of Texas at Austin, Austin, TX.