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Instability of Solar Power Tower Structures during Construction

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Abstract
It is generally understood that the structure depicted in the final construction drawings represents the completed condition, and that it is the designer’s responsibility to properly check stiffness, strength, and stability in this condition. It is also generally understood that because the contractor chooses means and methods of construction, they must accept responsibility for strength and stability of the structure in all conditions prior to completion. When design firms provide turn-key industrial structures with partial or comprehensive construction directives, these traditional lines of responsibility may not apply. This paper presents such a situation that unfortunately led to catastrophic buckling of a series of industrial steel towers during construction, before the structures were completed and properly braced. Global and local finite element analyses were combined with laboratory testing of the physical evidence to fully explain the nature of the collapse and to investigate possible contributions of reported construction imperfections. The authors hope that lessons from this failure will help to avoid similar disasters in the future.

1. Background

1.1 Site and Structure Description

The solar power plant generates power using long rows of mirrors located on the ground that reflect and focus sunlight onto receiver units suspended overhead from rows of steel towers. The concentrated sunlight heats water in the receiver units to generate steam. Three units, referred to herein as U1 through U3, had been constructed at the site prior to the collapse, at which time a fourth unit, referred to herein as U4, was under construction. Figure 1 shows the status of the solar power plant after completion of U1 through U3 and before the construction of U4. The towers associated with U4 differed in several key aspects from the towers in units U1 through U3, including the use of lighter, more slender leg elements.

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Figure 1: Aerial view of solar plant with U1 - U3 in place

According to design documents for the U4 tower and receiver assemblies, twenty-four receiver units (each approximately fifty-four feet long) are attached end-to-end and are suspended from twenty-five A-frame tower structures. The tower structures are approximately fifty-seven feet tall and consist of hollow structural section (HSS) steel frames with concrete pier foundations. The width (stance) of the towers at ground level is approximately twenty-eight feet and tapered to approximately five feet at the top. A schematic elevation of the towers is shown in Figure 2. The towers are assembled horizontally on the ground and then tilted into place. The receiver units are also assembled near ground level and are raised into position using lifting weldments mounted to the tops of the towers.

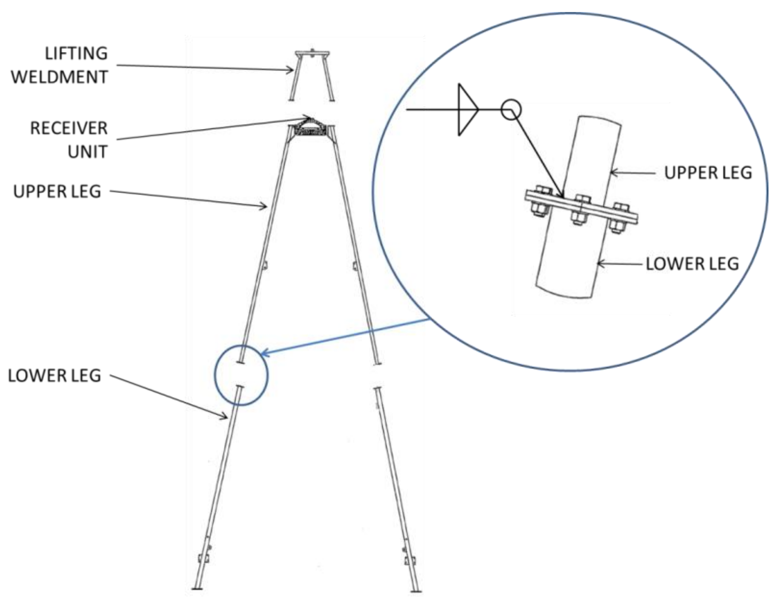


Figure 2: Tower elevation schematic

1.2 The Failure

During installation, the U4 towers failed during assembly of the receiver units. All towers were erected and the receiver was suspended a couple of feet off the ground from the lifting weldment at the top of the tower. As the final section of piping was being loaded into the receiver shell between towers #24 and #25, the towers buckled out-of-plane, collapsing the receiver assembly to the ground. Photographs of the failed towers are shown in Figure 3. Local distortion of the tower legs and failure of the welded connection between the upper and lower portions of the tower legs are apparent at multiple locations.

Figure 3: Post-incident photograph of bent towers

1.3 Roles and Responsibilities

Stability for steel structures is governed by Chapter C of the AISC Specification for Structural Steel Buildings (the "Specification") only for the completed structural systems. The philosophy of AISC Chapter C can be summarized as, "Stability shall be provided for the structure as a whole and for each of its elements" (AISC 360 2010, §C1). Coincident with the Specification, the fabrication and erection of steel structures is covered in the AISC Code of Standard Practice for Steel Buildings and Bridges (the "COSP") (AISC 303 2010). Specifically, the COSP states that the designer is responsible for the completed project and the erector is responsible for the "means, methods and safety of erection of the structural steel frame" (AISC 303 2010, §1.8.1). Furthermore, the COSP requires the designer to identify the lateral-load-resisting system and diaphragm elements in the contract documents (AISC 303 2010, §7.10.1).

Interpreting the Specification and COSP together suggests that it is the designer's responsibility to point out the components that comprise the lateral system in the completed structure. The erector is then responsible for ensuring that those components are adequately braced (read: "safety of erection" above) during the construction phase. However, if the designer is aware of a component that is paramount for stability of the system during erection, are they under obligation (contractual or ethical) to inform the erector? A more difficult question may be if the engineer provides explicit information for stability during erection, is the contractor thus relieved from responsibility? These types of questions are addressed in the paper through a close inspection of the contract documents and a thorough analysis of the solar tower system failure.

2. Structural Investigation

The tower structure is analyzed for strength and stiffness to withstand the stresses and strains imparted on it during three distinct phases: 1) tower construction and erection; 2) after erection of the tower, during lifting of the receiver; and 3) after construction completion, where the receiver units are attached to the tower top.

2.1 Modeling Characteristics

All relevant structural members of the tower are modeled, including the lower leg sections, the upper leg sections, and the lifting weldment. Properties of the members, as specified in the contract documents, are presented in Table 1.

Table 1: Member properties used in the analyses

Member	HSS Shape	Structural Section	Specified Material[1]
Lifting Weldment – Beam	Rectangular	HSS 6x4x1/4 (LSH[2])	A500 Gr.C
Lifting Weldment – Column	Square	HSS 4x4x5/16	A500 Gr.C
Upper Leg Weldment	Round	HSS 4 1/2x3/8	A500 Gr.B
Lower Leg Weldment	Square	HSS 6x6x1/4	A500 Gr.B

[1]Denotes ASTM designation for structural steel
[2]LSH ≡ long side horizontal

ASTM A500 grade B is the most common (and preferred) material designation for rectangular, square, and round hollow structural shapes (HSS). In different versions of the drawings, grade C and grade B are both specified for the same components of the lifting weldment. To test sensitivity to the difference between minimum yield strengths for the materials, the lower yield strength associated with A500 grade B is initially utilized in the analysis. If under any loading conditions the stresses reach a point where the material may yield, subsequent analyses are performed with the grade C material and its corresponding yield strength. This consideration is incorporated into the analysis, results, and discussion herein.

The tower is designed and configured such that the base condition is considered fully fixed; i.e., no translation or rotation is allowed at the base of each column tower leg. The receiver is assumed to brace the tower out-of-plane of the frame only after it is lifted and bolted to the top of the tower. In the final configuration, the middle five towers are braced at the top using guy cables. The engineer issued a directive stating that only the guys for the central tower are to be installed prior to lifting the receiver unit. In other words, prior to final attachment of the in-place receiver unit, none of the towers other than middle tower should be braced.

Loading conditions that are considered include: 1) lifting of the tower frame to vertical from its horizontal assembly position during the erection process; 2) lifting of the receiver unit using the lifting weldment on top of the tower, with only the central tower guyed; and 3) service loading of the tower structure with the receiver unit and guy wires secured in-place. Each of the conditions is analyzed including the entire weight of the tower. An average weight for steel of 490 pounds per cubic feet is used; leading to a total tower weight equal to 2.1 kips. A summary of findings with respect to each loading condition is presented in the *Results and Discussion* section.

2.2 Analysis Methods

Both eigenvalue buckling and non-linear, load-deflection analyses are used to assess the stability of the solar power tower system. The analyses are performed in the general purpose structural computer program SAP2000 (2012). All elements are modeled as 3D frame objects between pre-defined nodes. SAP then automatically subdivides each frame element into an appropriate number of sub-elements to ensure model accuracy.

A simple means to assess the tower stability is to compare the actual loads to the eigenvalue buckling load. For these analyses, the tower is modeled including self-weight. Then, based on the phase of construction completion selected for analysis, bracing and/or loading is applied at the top of the frame. After the analysis is performed, a load proportionality factor (LPF) is reported. The LPF is the multiple (or fraction) of the self-weight and receiver load (if applied) that can be safely carried by the tower system. If one is interested in determining solely the receiver load allowed by the tower configuration, the applied receiver load must be iterated until the LPF equals one, so as to maintain independence between scaling of the receiver load and the tower self-weight.

While the eigenvalue buckling analysis immediately showed a problem with the tower configuration, the actual buckling mechanism is more complicated. Lateral restraint to finite displacement is provided by the lifting line to the receiver, which is restrained from moving longitudinally by friction on the supporting sawbucks. It is only when sufficient receiver units and tubing are lifted off the sawbucks that the friction resistance becomes too low to resist the lateral component of the P-Delta driving force. In this paper, the friction effects are ignored for the sake of simplicity. However, a more refined analysis is employed in SAP2000 (2012) using a geometrically nonlinear, load-deflection-based model.

SAP2000 (2012) can solve for buckling loads by eigenvalue extraction (discussed above) or by second-order analysis considering geometric and material nonlinearities. Similar to the method used for the eigenvalue buckling solutions, only one line of elements is modeled along the height of the tower. The self-weight of the tower is again distributed along each element. Various imperfection magnitudes are considered for initial out-of-plane displacement between the top and bottom of the tower. The solutions are based on geometric nonlinearity, but material nonlinearity is ignored in these analyses. A two-stage analysis is performed: 1) a second-order analysis subjected to self-weight starting with the initial out-of-plane imperfection, and 2) a second-order analysis with scaling of a point load representing the receiver weight that continues from the final geometry and loading of the previous step. In all analyses, tower top displacement is monitored versus the applied receiver load.

2.3 Results and Discussion

The results and relevant discussion are presented in this section for each of the construction phases introduced above. All the cases are summarized and presented in tabular form to facilitate comparison at the end of this section.

2.3.1 Erection of the Tower Frame

The towers were assembled horizontally and then rotated to vertical using a crane attached to the center of the lifting weldment on top of the tower. At the beginning of the lifting procedure, the frame is in a horizontal position and is supported at the bottom of each leg as well as at the crane attachment point. As the top of the tower is raised, the legs of the tower act as horizontal beams spanning the entire height of the tower and carrying the entire tower self-weight. The deflected shape under the self-weight of the frame is shown in Figure 4. At this phase of construction, the welds joining the upper and lower tower legs are subjected to the highest stresses anticipated for the design life of the tower (indicated by the red circles in Figure 4). In fact, calculated weld stresses during this operation are several times larger than those expected from in-service loads such as wind or earthquake.

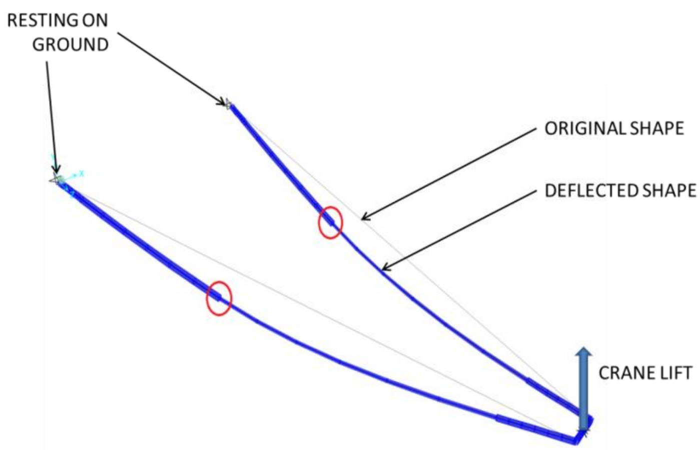


Figure 4: Horizontal tower assembly before crane lift to vertical

During tower lifting, the maximum moment at the joint is determined to be 95 kip-in. There were no reported failures of the welds during this process. The issue of weld capacity versus demand is explored in the following section.

2.3.2 Stability of the Tower Frame under Self-Weight

In this scenario, the tower frame has been tilted upright and the base has been firmly fixed to the foundation. At this point, neither the receiver nor any guying cables are in place, rendering the tower completely unbraced along its height. Figure 5 shows the primary buckling mode for the tower unsupported laterally at the top and subjected only to self-weight. The original undeflected shape is also shown in the same figure for reference.

Figure 5: Primary buckling mode for the laterally unbraced tower under self-weight only

The eigenvalue buckling analysis reveals that the LPF is 8.5 for the tower subjected only to self-weight. This suggests that, theoretically, the steel could weigh up to 8.5 times its current value and still remain stable. The total load the system could safely support (assuming additional load is distributed in exactly the same proportions as the self-weight) can be calculated by multiplying the LPF by the original self-weight, or roughly 17.8 kips.

2.3.3 Stability of the Tower Frame while Lifting of the Receiver Unit

In this phase, the tower frame is erected vertically and the receiver unit is being lifted into place via a cable from the lifting weldment at the top of the tower. The analysis is simplified by considering only a single tower and its tributary load; it does not consider the geometric stiffening provided by the lift line down to a receiver unit partially restrained by friction. As before, the base of the tower frame is assumed to be fixed. The top of the frame is still considered to be free, since it is not until the receiver is lifted all the way to the top of the tower and bolted into place that the top will be braced against out-of-plane movement. Two different types of analyses are performed for this phase of construction: 1) a point load consisting of the receiver weight is placed at the upper weldment of a "perfect" tower and an eigenvalue buckling analysis is performed, and 2) the same receiver point load is applied to a tower with various initial out-of-plane imperfections and a geometrically nonlinear analysis is performed.

For the eigenvalue buckling case, the weight of the tower is applied and a dummy load of 1 kip is applied at the upper weldment to represent the receiver load. A buckling analysis is performed in SAP2000 and the resulting LPF is 3.0. However, the allowable receiver load is not just 3 times this value since scaling the result by 3.0 would also incorrectly scale the weight of the tower. Therefore, an iterative procedure must be adopted whereby the dummy load of 1 kip from the first analysis is changed and an analysis is performed until a LPF equal to 1.0 is reached. For this case, the LPF is equal to 1.0 when the receiver load is 4 kips. This would indicate a total maximum load on the tower including self-weight of 6.1 kips.

In order to more accurately model the as-built conditions, a suite of initial imperfections were applied to the tower to represent realistic values for out-of-plane construction tolerances. The initial imperfection is applied by canting the tower out-of-plane at the top by the specified dimension. Next, the tower self-weight is applied in a primary step and an analysis is performed. Subsequently, a unit load is applied at the upper weldment corresponding to the location of the receiver lift point. A geometrically nonlinear analysis is now performed and a plot of receiver load versus tower top deflection is recorded. The results of five different load-deflection analyses are shown in Figure 6. The eigenvalue solution is also included in the plot for direct comparison.

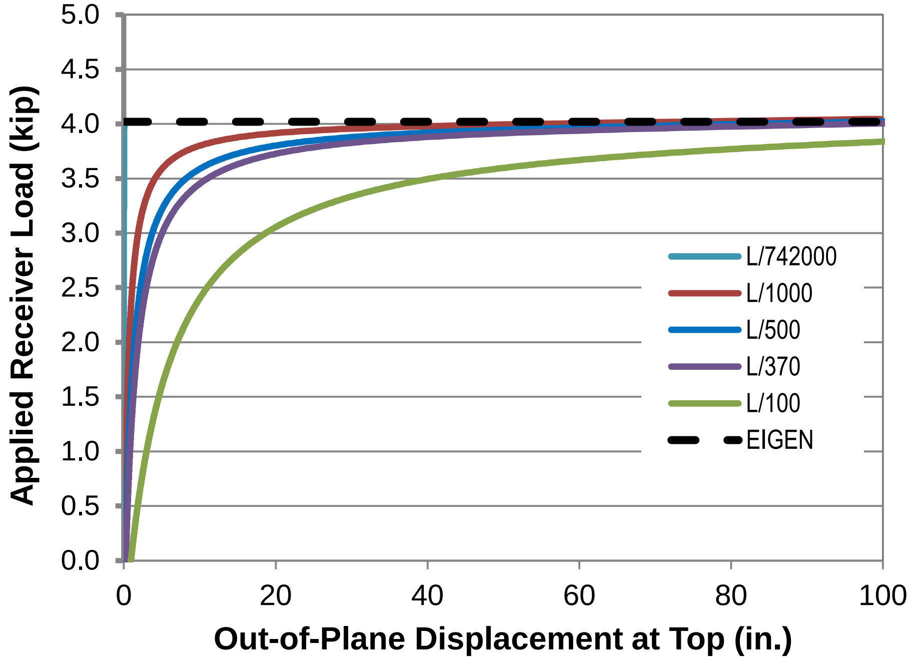


Figure 6: Receiver load versus tower top displacement for various initial imperfections

The results of the analyses are consistent and indicate tower deflections can start to become excessive at an applied vertical load around 3.0 to 4.0 kips (in addition to the tower self-weight), which is well below the specified receiver weight and its piping of approximately 4.8 kips. Also, as expected, as the magnitude of the initial imperfection increases (moving from the upper left of the plot toward the lower right), the displacements increase with less and less applied receiver load.

2.3.4 Stability of the Completed Tower Frame

In this final phase, the tower has been erected and the receiver is bolted to the top of the tower at the lifting weldment. The central five towers in this configuration are guyed out-of-plane and thus, the steel receiver unit acts as a collector and lean-on bracing system. An eigenvalue buckling analysis is performed as discussed above with a dummy load for the weight of the receiver. The first analysis provides a LPF equal to 16.4. Successive iterations lead to a total allowable receiver load of 27.9 kips for a total tower load (including self-weight) of approximately 30 kips.

2.3.5 Results Comparison

The results discussed above are tabulated in Table 2. The table shows the total load on the tower from either the Eigenvalue ("Eigen") or load-deflection ("L-D") for each of the buckling models presented. Since the receiver and its internal piping weighed approximately 4.8 kips and the tower weighed 2.1 kips, any capacity result including the receiver less than 6.9 kips would suggest instability.

Table 2: Analysis results

Model	Top Bracing	Total Load (kips)	
		Eigen	L-D
Self-weight only	None	17.8	-
Receiver + self-weight	None	6.1	5.6
Receiver + self-weight	Fully Braced	30	-

Table 2 is consistent with reports that the collapse initiated after the receiver piping was installed along the suspended receiver; in a condition where the full receiver and piping weight was present without adequate out-of-plane bracing. Furthermore, the shape of the buckling mode is associated with out-of-plane movement along the height of the tower, which is consistent with the tower deflections shown in post-incident photographs (Figure 3).

3. Weld Investigation

According to a post-incident report of the tower failures, at least one welded connection failed at eleven of the twenty-five towers (there were a total of sixteen mid-tower leg weld failures). All but one of the towers with no weld failure exhibited permanent deformation in the form of residual tower lean upon removal of the receiver load (the tower that exhibited no weld failure or residual lean was the only tower guyed out-of-plane at the time of the collapse). As exhibited by Figure 7, inspections of eleven steel plates at the upper portion of the joint between the upper and lower portions of the tower legs revealed permanent deformation of all plates, which is indicative of significant load transfer across the joint prior to the weld failure.

Figure 7: Permanently deformed plate between weldments

It was suggested by a party to the case that a premature weld failure induced the collapse of the solar power tower system. This hypothesis may come as a surprise after considering the above buckling results indicating that the tower system was ill-proportioned to sustain the receiver and piping weight without bracing at the top. Despite the obvious chain of events that suggest the towers buckled which caused several welds to fail (not vice versa), subsequent analyses were performed to gain an understanding of the connection capacity between the upper and lower tower legs. Specifically in this paper, only the deformation characteristics of the connection plates are presented; i.e., a metallurgical analysis of the weld is not considered herein.

As discussed in the previous sections, after assembly of the tower on the ground, the tower is lifted vertically into position using a crane. It is at this time that self-weight leads to the largest internal force on the connection; around 95 kip-in. Abaqus (Simulia 2013) is employed to perform the connection plate analyses in order to determine the expected capacity of the plates assuming a competent weld. Also, it is important to ascertain what level of forces on the plate would lead to the permanent deformations observed in the plate shown in Figure 7.

The lower leg, upper leg, and ½" thick ASTM A572 Grade 50 carbon steel connection plates are modeled in Abaqus (Simulia 2013). In the actual configuration, a connection plate is welded to each the upper leg and the lower leg. The two connection plates are then bolted together. It is assumed that separation and slip between the plates is sufficiently prevented at the area covered by the washer due to bolt pretensioning. Figure 8 shows (from top to bottom) the HSS4.5x4.375 upper leg, the top connection plate in red, the bottom connection plate in blue, and the HSS6x6x1/4 lower leg.

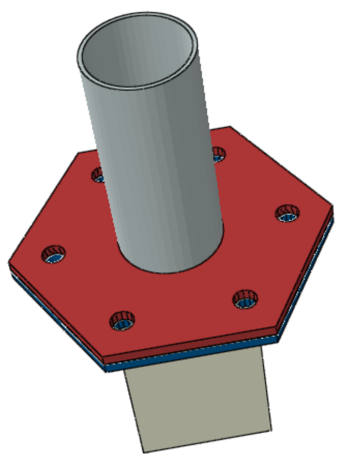
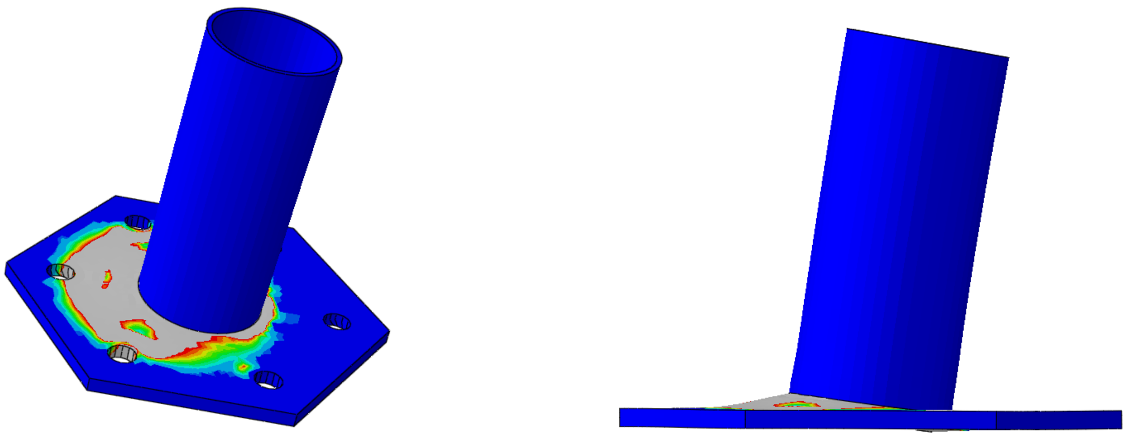


Figure 8: Abaqus connection model

The bottom of the 10 inch segment of the lower leg is considered fixed. An axial load equal to 3.14 kips is applied at the center of the upper leg representing the tower self-weight plus a receiver load of 5 kips. A rotation between 0 to 0.06 radians is applied at the top of the upper leg in order to induce bending in the connection plates. The equivalent plastic strain is monitored versus rotation in order to determine at which point the onset of yielding occurs in the connection plates.

The analysis results indicate that the connection plate begins to yield around 98 kip-in corresponding to a rotation of 0.014 radians. The analysis continues until 0.06 radians of rotation are applied at which point the maximum moment induced in the plate is 249 kip-in. The equivalent plastic strain (PEEQ) for an isometric and side view is shown in Figure 9. In this figure, plastic flow is indicated by the gray areas in the model. Conversely, dark blue represents areas that remain elastic. The figure indicates that significant plastic deformation is occurring at 0.06 radians and that the deformed shape predicted in Figure 9(b) closely matches that which was observed on the actual structure (Figure 7). Also, the analysis shows that the internal forces at this connection at the time of failure are essentially pure compression, indicating that this plastic deformation of the connection plate is a secondary damage that occurred due to the tower buckling.



(a) isometric view (b) side view

Figure 9: Equivalent plastic strain at M = 249 kip-in.

It is important to note the differences between the design, capacity, and actual demand on the connection plate welds. Figure 10 compares the original design moment check by the engineer, the nominal weld moment capacity determined by Abaqus (Simulia 2013), the amount of moment on the weld during the tower lifting procedure, and the amount of moment on the weld at insipient buckling. The results are normalized to the moment checked by the engineer. The plot indicates that the bending moment induced by lifting the tower is on the order of five to six times those contemplated by the engineer. Furthermore, the moments based on the weld design or tower lifting both exceed by a large margin the moments induced in the tower legs by the receiver.

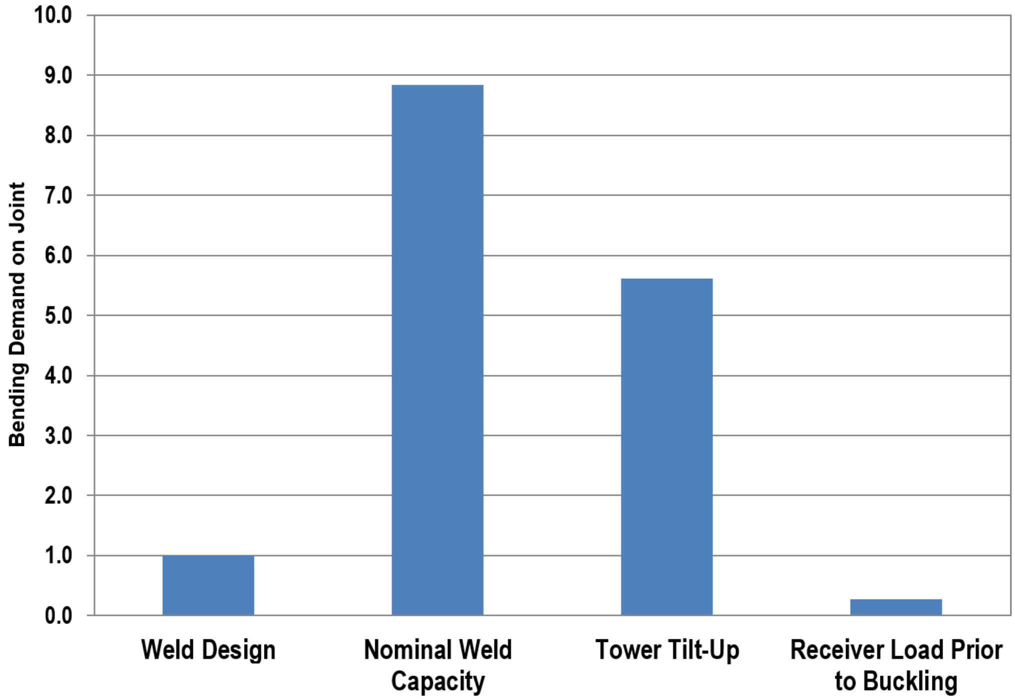


Figure 10: Connection plate weld design, capacity, and demand

4. Contract Documents

The tower design calculations included in the contract documents from the designer detail multiple loading conditions. A buckling analysis is performed for the tower self-weight and the receiver weight. However, as depicted in Figure 11, the tower is modeled to be restrained against longitudinal movement at its top. While this boundary condition may be appropriate to assess the capacity of the frame in its final configuration after the receiver unit is in place and fully attached or the capacity of a tower that has been guyed prior to supporting the receiver unit, it is not appropriate for the erection of U4, where only the center tower was guyed and no lean-on bracing was installed.

Also included in the calculations is a load combination titled, “Tower Evaluation During Erection.” The load combination includes the receiver load applied at the lifting weldment, as well as a lateral wind load. The analysis is described as a “stress analysis” and does not appear to contemplate tower buckling. Further, the analysis does not appear to account for stresses associated with lifting of the tower frame from its horizontal to its vertical position.

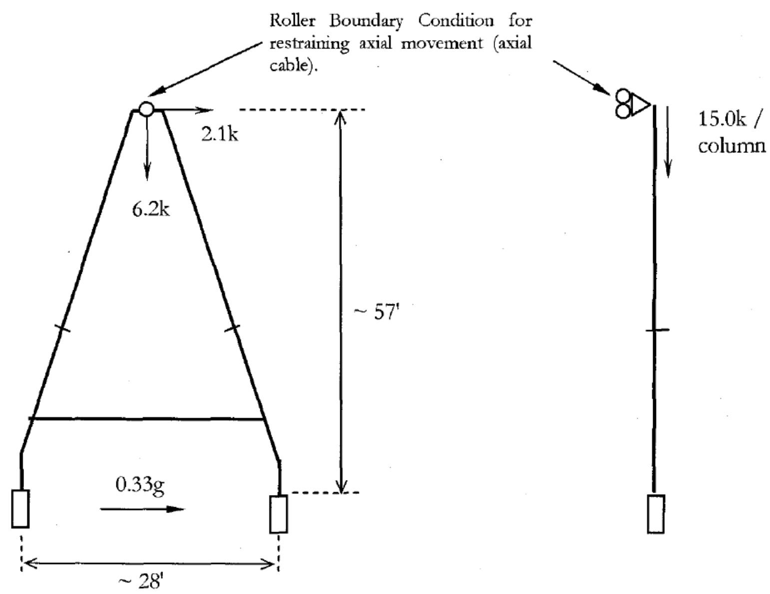


Figure 6.1 – Tower Buckling (G + Seismic X) *Figure 6.2 – Tower Buckling (G + Seismic Y)*

Figure 11: Engineer's tower model for buckling analysis (note the top restraint indicated)

The contract documents also included explicit engineering instructions for tower erection. Those instructions dictated that only the center tower should be guyed during lifting of the receiver units, in order to allow some flexibility for fit-up of the receiver. The instructions for installation were explicit and did not include any provisions for stabilizing the towers. Testimony by the erector during the ensuing litigation indicate that, had it not been for the explicit instructions from the engineer, and given the slenderness of the tower legs, that they would have guyed each tower during construction. It should be noted that tower units U1 through U3 were all erected without guying; however, the tower legs for those units are less slender. While the factors of safety are relatively low, the towers did not buckle under the self-weight and receiver load.

In summary, the design calculations do not predict the mechanism that led to the tower collapse; i.e., buckling of the towers during construction before the tops of the towers are braced by the receiver unit. In addition, the design does not address lifting the assembled towers to their vertical position, which induces the highest weld stresses during the service life of the tower.

5. Conclusion

Based on the AISC Specification (2010) and the COSP (2010), the engineer is typically responsible for stability of the finished structure, and contractor is responsible for stability during erection. However, some contracts, especially for industrial structures, can blur this distinction. This paper presents a case study in which the engineer’s explicit erection instructions apparently relieved the contractor from responsibility to provide construction bracing. Had an experienced structural engineer recognized that the construction method precluded lean-on bracing from the guyed tower, the unstable condition could have been easily identified and avoided. Unfortunately in this case, the stability during tower erection was never evaluated, resulting in collapse of twenty-five of twenty-five towers.

Disclaimer

The views expressed herein are solely those of the authors and do not necessarily represent the position(s) of Exponent or any other individuals therein.

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