Pushover Analysis of the Tower of the New Self-Anchored Suspension Bay Bridge

Abolhassan Astaneh-Asl
The University of California, Berkeley
Astaneh@berkeley.edu

Xin Qian
The University of California, Berkeley
cindyqian@berkeley.edu

Abstract

The new East Span of the San Francisco-Oakland Bay Bridge, which opened to traffic in 2013, is a "Self-Anchored-Suspension" (SAS) bridge with a single tower. The main cables of a SAS bridge are connected to the bridge deck instead of being connected to the anchor blocks as is the case in traditional suspension bridges. The bridge is located in a highly seismic area in northern California and has only one tower with four shafts. Steel shear links connect the four shafts to each other along the height. A cable saddle connects the top of the four tower shafts to each other. High-strength anchor rods connect the base of the tower to the pile cap. This paper presents the results of a series of realistic nonlinear pushover analysis of the single tower of the SAS Bay Bridge.

A detailed nonlinear finite element model of the main tower was constructed using shell elements available in the finite element analysis software ANSYS. The analysis consisted of pushing the top saddle horizontally in five different directions (at 0-, 30-, 45-, 60-, and 90-degree angles with respect to the longitudinal direction of the bridge) until it collapsed. This paper focuses on the behavior in the transverse direction (zero degrees). Gravity loads were included in the pushover analysis. The results showed that local buckling of the tower shaft plates may occur relatively early in the pushover analysis, resulting in a drop of the strength and a reduction in ductility. The original designers of the bridge did not take this behavior into account. Their pushover analysis of the tower used a model with only beam elements instead of shell elements. They concluded that the only nonlinearity in the tower would be yielding of the shear links connecting the tower shafts to each other, while the tower would remain essentially elastic with no local buckling. The realistic modeling and accurate analysis presented in this paper show that this conclusion is incorrect.

Introduction

The new East Span of San Francisco-Oakland Bay Bridge is a "Self-Anchored-Suspension" (SAS) bridge with a single tower. The bridge opened to traffic in 2013 and is located between two active seismic faults: the Hayward and San Andreas faults. Figure 1 shows the probabilities of occurrence of earthquakes of magnitude 6.7 or greater occurring in the greater Bay Area during a 30-year period from 2007 to 2032 [1].

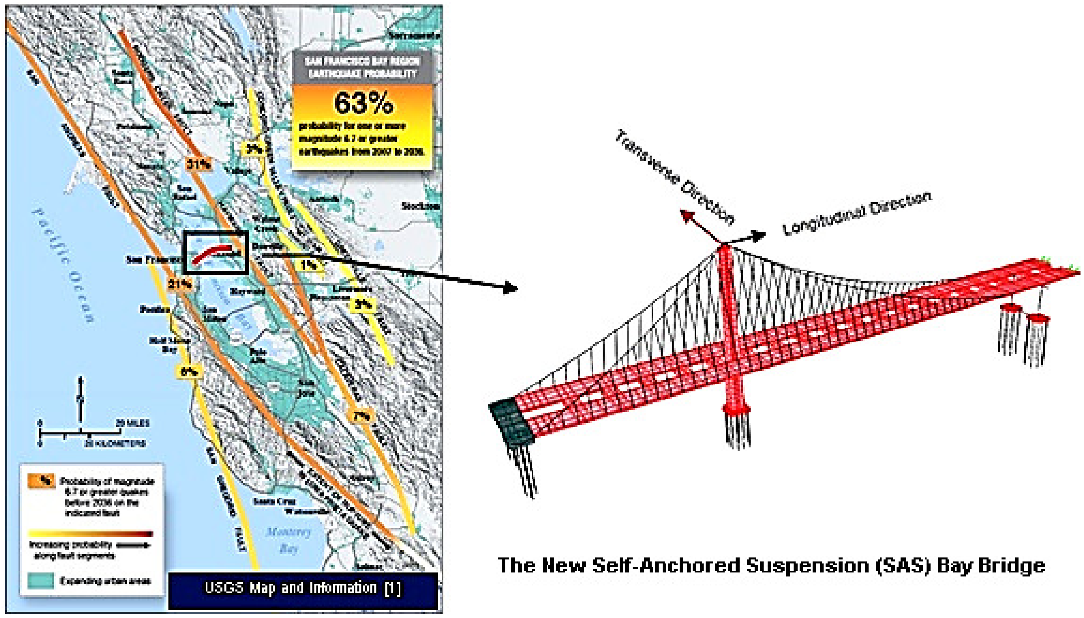


Figure 1: Probability of Occurrence of Magnitude 6.7 or Greater Earthquakes in the Bay Area (left-hand side), and the SAS Bay Bridge Longitudinal and Transverse Directions (right-hand side)

Figure 2 shows the overall structure of the new SAS Bay Bridge, and Figure 3 shows the elevation and plan views of the main tower. Figure 4 shows a typical cross section of the bridge. The 512ft (156m) tower (at the cable intersecting point) consists of four shafts, each shaft being a pentagonal steel hollow box with vertical stiffeners and horizontal diaphragms; see Figure 3.

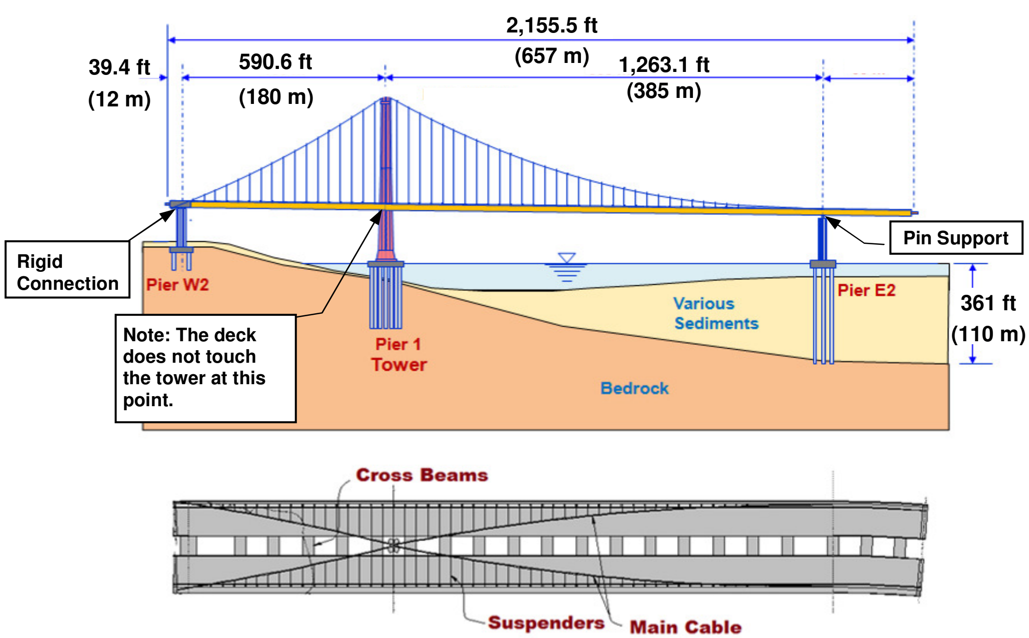


Figure 2: Elevation and Plan Views of the SAS Bay Bridge

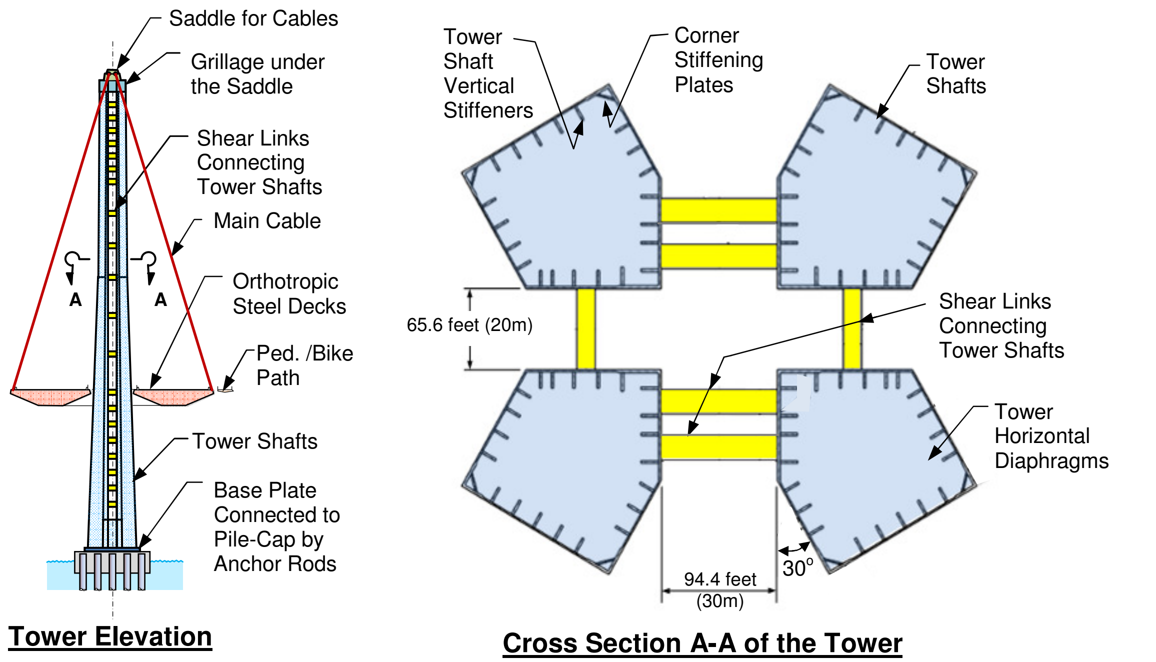


Figure 3: Elevation and Cross Section of the Tower of the SAS Bay Bridge

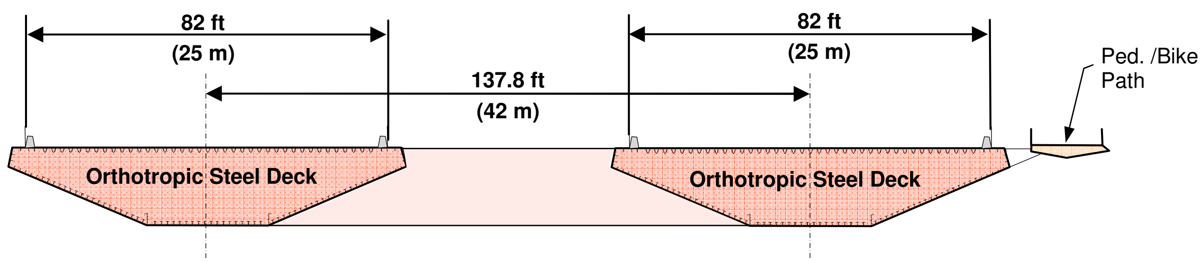


Figure 4: Cross Section of the SAS Bay Bridge Showing Two Separate Orthotropic Decks Connected to Each Other by Transverse Steel Box Girders

The steel used in the tower shaft is ASTM A709 Gr. 50 steel with a yield stress of 50 ksi (345 MPa) and an ultimate strength of 65 ksi (448 MPa). As is standard, the shafts have horizontal stiffeners every 9.9ft (3m). I-shaped shear links connect the shafts to each other along the height of the tower and by a saddle at the top. The main cable is a single cable connecting the top of the tower to the orthotropic steel deck and passes through the saddle.

High-strength A354 BD zinc coated (hot-dip) galvanized anchor rods with a yield stress of 115~130 ksi (793~896 MPa), and an ultimate strength of 140~150 ksi (965~1034 MPa) connect the base of the tower to the pile cap; see Figure 3. The use of the A354 BD zinc coated hot-dip galvanized anchor rods in this key traffic corridor exhibited undesirable behavior when a few months before the opening of the bridge, 32 of the anchor rods connecting seismic shear keys to the top of the Pier E2 on the east end of the SAS Bay Bridge fractured when tightened.

This paper focuses on the pushover behavior of tower itself and assumes that the base plate is rigidly connected to the pile cap/pile substructure. A second paper by Astaneh-Asl, Tabbakhha, and Qian [2], presented at the conference and included in these proceedings, focuses on the pushover behavior of the tower in a scenario where anchor rods have fractured and can no longer resist the tension created by the bending moment at the base of the tower.

Objective

The main objective of the research summarized in this paper was to establish the stiffness, strength, buckling behavior, and ductility of the as-built main tower of the SAS Bay Bridge when pushed by a horizontal force at the top.

Background

Four papers [3, 4, 5, and 6], co-authored by M. Nader, B. Maroney, R. Manzanarez, J. López-Jara, and C. Mibelli, who are the chief designers of the SAS Bay Bridge, provide information on the analysis and design aspects of the bridge. This paper will cite excerpts from these papers on the *Performance Criteria and the Expected Behavior during Seismic Events* and then discuss validity of the assumptions made and how accurate the results based on those assumption are.

Dr. Marwan Nader, the co-author of these papers, was Chief Engineer of Record for the SAS Bay Bridge designed by T.Y. Lin International of San Francisco. Dr. Brian Maroney was the Chief Engineer for the SAS Bay Bridge for the California Department of Transportation (Caltrans), the state agency that owns the SAS Bay Bridge. The other co-authors were the engineers from T.Y. Lin International involved in the analysis and design of the SAS Bay Bridge. Heretofore, they will be referred to as the Bridge Design Team. The co-authors of these publications played a critical role in the analysis, design, construction, and inspection of the bridge, and are directly responsible for its design. The information contained in these publications represents the official record of the bridge design. The research presented herein focuses on the results presented in these publications, discusses the methodology performed, and the assumptions made in the analysis and design, with particular emphasis on the pushover analysis of the tower.

***Performance Criteria presented by the Bridge Design Team*:**

Following are excerpts from Ref. [3] authored by the designers of the SAS Bay Bridge:

> "*SEISMIC PERFORMANCE CRITERIA – The Bridge is designed to provide a high level of seismic performance. It is designed to resist two levels of earthquake, a functional evaluation earthquake (FEE) and a safety evaluation earthquake (SEE). After a functional evaluation earthquake, the bridge will provide full service almost immediately and there will be minimal damage to the structure. Minimal damage implies essentially elastic performance and is characterized by minor inelastic response, narrow cracking in concrete, no apparent permanent deformations, and damage to expansion joints. After a safety evaluation earthquake, the bridge will provide full service almost immediately and will sustain repairable damage to the structure. Repairable damage is damage that can be repaired with minimum risk of losing functionality; it is characterized by yielding of reinforcement, spalling of concrete cover and limited yielding of structural steel.*[3]

Expected Behavior during Seismic Events

According to the bridge design team, the design criteria for the SAS Bay Bridge required that the bridge must be operational almost immediately after a major earthquake. According to the bridge design team, "*Seismic analysis was performed using the ADINA general-purpose finite element program. Three forms of analysis were employed: time history analysis (global model), push-over analysis and local detailed analysis.*[4]*"*

Bridge Model and Pushover Analysis used by the Bridge Design Team

This paper focusses on the "pushover" analysis used by the bridge design team; they stated: "*Push-over analysis was primarily used to evaluate ductility of critical elements and to establish failure mode sequence*" [3].

Figure 5 shows the ADINA global analysis model used by the bridge design team in their pushover analysis. The model consisted of only linear and some selected nonlinear "truss" and "beam" elements, and did not have any shell elements [3]. In Reference [3] they state: "*The shear links between the shafts were also modeled with inelastic moment-curvature beam elements.*"

The base of the tower where the four shafts are connected to each other by steel plates to form a single multi-cell tower was modeled as a single shaft, with elastic beam–column "stick" elements [4]. "*The shear links, connecting the tower shafts to each other, …, were modeled with inelastic "moment-curvature" beam elements, calibrated using the shear displacement relationship from a detailed local model*" [3].

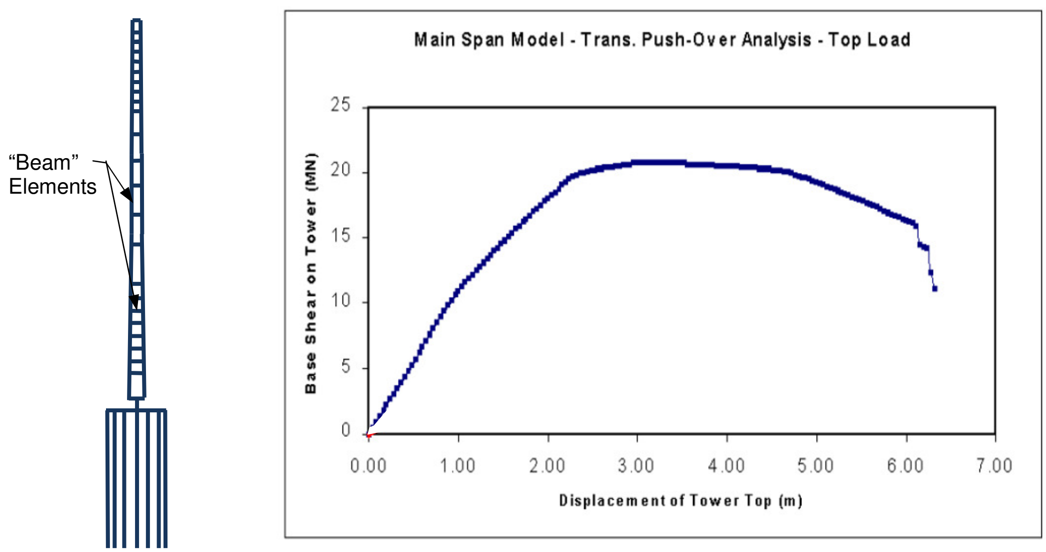


Figure 5: The simplified ADINA "Beam" Model of the Tower (on the left) used by the Bridge Design Team in Their Analysis and Results of Pushover Analysis Done by the Bridge Design Team [3]

Basis of Design: According to the bridge design team [3]:

> “ *..the bridge is designed based on a limited ductility design in which plastic deformations are clearly defined and predetermined. … the bridge is designed to remain largely elastic with the exception of the east and west piers which are designed to form plastic hinges. … The shear links between the tower shafts are also designed to yield in shear during the SEE earthquake.*”

Seismic Response of the Bridge: Reference [3] states that during the Seismic Safety Evaluation Earthquake, which was used as the design earthquake, the top of the tower will move maximum horizontally 1.3 m and 1.0 m in the transverse and longitudinal directions, respectively. The bridge design team stated that the only inelastic areas will include: plastic hinge formation at the top and bottom of R/C Pier W2, plastic hinge formation at the bottom of R/C Pier E2, and yielding of shear links connecting four shafts of the main tower to each other.

Pushover Analysis Conducted by the Bridge Design Team: According to Reference [3], pushover analysis of the main tower was performed:

> “*… to evaluate the base shear versus top of tower displacement relationship, to optimize the design of the tower shear link and shaft, to evaluate the lateral ductility of the tower before collapse and to evaluate the ductility demands on the shear links and tower shafts at various levels of displacement demand during an earthquake.”*

Figure 5 also shows a pushover curve performed by bridge design team [3], resulting from pushing the tower at the top in the transverse direction.

The studies summarized in the remainder of this paper done by the authors, Astaneh-Asl and Qian, demonstrate that the pushover performance of the main tower of the SAS Bay Bridge does not represent the actual behavior of the tower when subject to ground shaking. The main inaccuracy is that the analysis model of the tower, shown in Figure 5, uses only “beam” elements to represent the actual steel plate members, which are “shell” elements. These “beam” elements are unable to predict local buckling phenomenon, which is the main cause of instability in structures designed utilizing steel plates, such as the tower of the SAS Bay Bridge.

Realistic Pushover Analysis of the Tower of the SAS Bay Bridge

Below is a summary of a realistic pushover analysis of the tower. The analysis used inelastic shell elements capable of yielding and local buckling to represent all steel plates in the tower. The only exception was the vertical stiffeners in the tower shafts, which were modeled as beam elements.

Finite Element Modeling of Tower

The general purpose finite element software ANSYS R15.0 was used to determine pushover behavior of the main tower. As shown in Figure 6, all components of the main tower, except the vertical stiffeners of the tower shaft, were modeled with the SHELL181 element. This is a 4-node shell element that is suitable for linear, large deflection, and large strain nonlinear applications. The BEAM188 element of ANSYS, which is a 2-noded linear, quadratic, or cubic 3D beam element based on Timoshenko beam theory, was used to model the vertical stiffeners of the tower shafts.

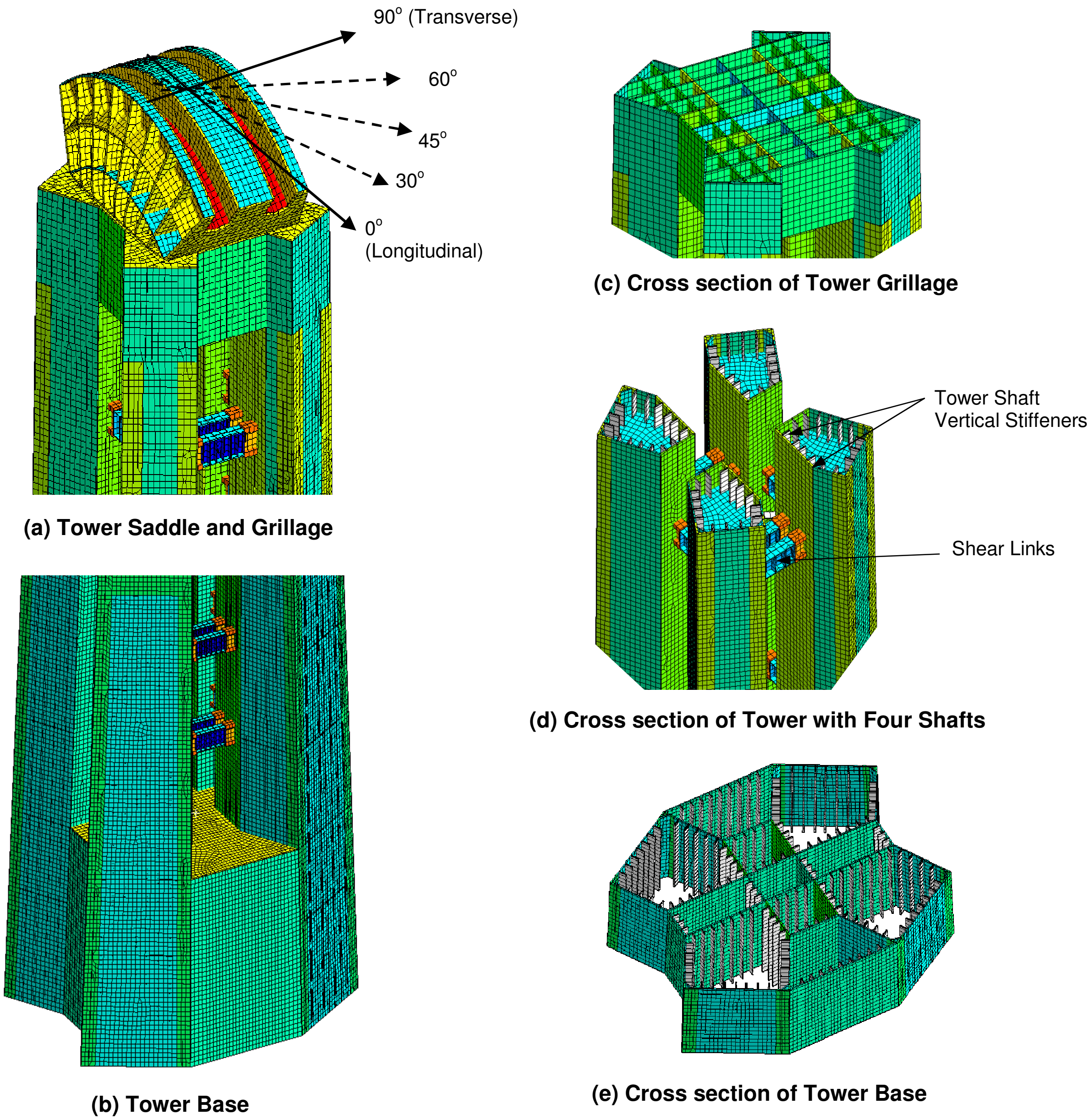


Figure 6: Finite Element Model of the SAS Bay Bridge Tower with Details of the Cross Sections

The tower shaft stiffeners were attached to the tower shaft plates with bonded contact [7]. Figure 6 show ANSYS finite element model of the tower. The geometric features of the tower were modeled in detail based on the construction drawings [8].
All critical structural components of the main tower including all three types of shear links, vertical stiffeners of the tower shafts, and horizontal diaphragms inside the tower shafts were modeled. Non-structural and architectural features, e.g., the tower skirt at the base of the tower, were not included in the model.

Since pushover analysis of the tower serves to establish the capacity of the tower only, the tower base is assumed to be fixed to prevent both displacement and rotation. The impact of the decks on the tower shafts were not included considering the gap between the main tower and the bridge decks; see Figure 2.

Pushover of the tower is achieved by nonlinear large deflection multi-step static analysis with both material and geometric nonlinearities included. In addition to the self-weight of the tower itself, concentrated forces were applied at the tower tip saddle to represent the vertical and horizontal components of the cable forces acting on the saddle. The cable forces were obtained from gravity analysis of the whole bridge model in SAP2000 shown in Figure 1. With the pre-stress from gravity effects in place, incremental horizontal displacements were then applied using the displacement controlled iteration algorithm. The horizontal displacements were applied at the cable saddle groove location.

This research project conducted pushover analyses in five different directions - longitudinal (0 degrees), transverse (90 degrees), and 30, 45, and 60 degrees from the longitudinal axis of the bridge, as shown in Figure 6(a). This paper focuses on the behavior in the transverse direction, which is normally the most critical direction.

The material for all components of the steel tower was Gr.50 steel with a yield stress of 50 ksi (345MPa). The only exception was the rigid connection plates of the shear links to the tower shafts, which were Gr.70 steel with a yield stress of 70 ksi (485 MPa). The steel was modeled using a bilinear kinematic hardening material model with an initial elastic modulus equal to 29,000 ksi (200 GPa), a Poisson ratio of 0.3, and a strain-hardening ratio of 1%.

Results of Transverse Direction Pushover Analysis

Pushover behavior of the tower in its transverse direction is normally more critical than the other directions. Figure 7 shows the pushover curves in the transverse direction, which indicate that the tower yielded gradually and then the lateral load resistance dropped relatively quickly after the applied pushover force reached its maximum value. There was no pronounced yield plateau on the pushover curve. Based on the analysis results, several phases of behavior were observed, and three important points are identified as Points A, B, and C in Figure 7. They are explained as follows:

Point "Y" on the push-over curve represents the "yield point" of the tower. For large and complex structures, such as the main tower of the SAS Bay Bridge, local yielding occurs at relatively small displacements due primarily to stress concentrations; such small local yielding cannot be considered the yield point of the tower. Therefore, a "yield point" must be defined for such structures. Two definitions are shown in Figure 8. Figure 8(a) is the definition of yield point when there is relatively clear initial elastic linear behavior as well as a linear second branch of the force-displacement curve. In this case, the yield point can be defined as the point of intersection of the initial stiffness line and the secondary stiffness line; see Point Y in Figure 8(a). However, in many cases, the initial behavior and the secondary branch of the force-displacement curve is not a straight line, which is the case for the push-over curve of the tower shown in Figure 8. In these cases, the "yield point" can be defined as the point where the displacement of the structure deviates from the initial stiffness line [line "ob" in Figure 8(b)], with an amount equal to 10% of the elastic displacement. In other words, the Yield Point Y is a point where distance "bY" in Figure 8(b) is equal to 10% of distance "ab". This paper has followed the definition of the yield point as shown in Figure 8(b), using the "10% deviation rule."

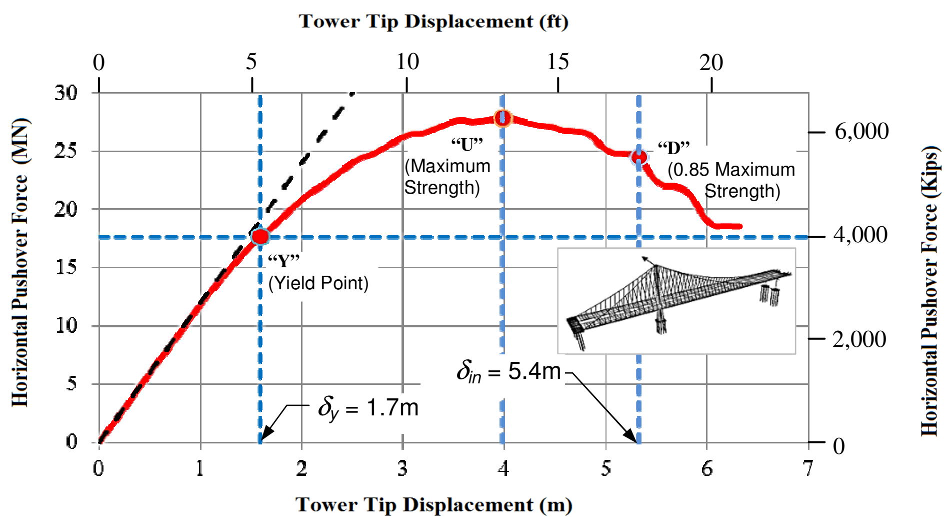


Figure 7: Transverse Pushover Curve of the Tower

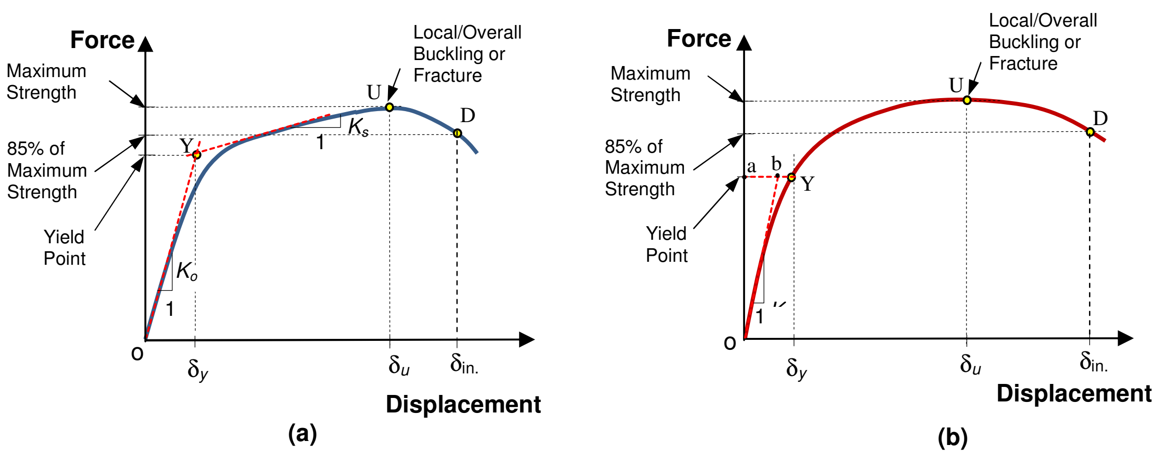
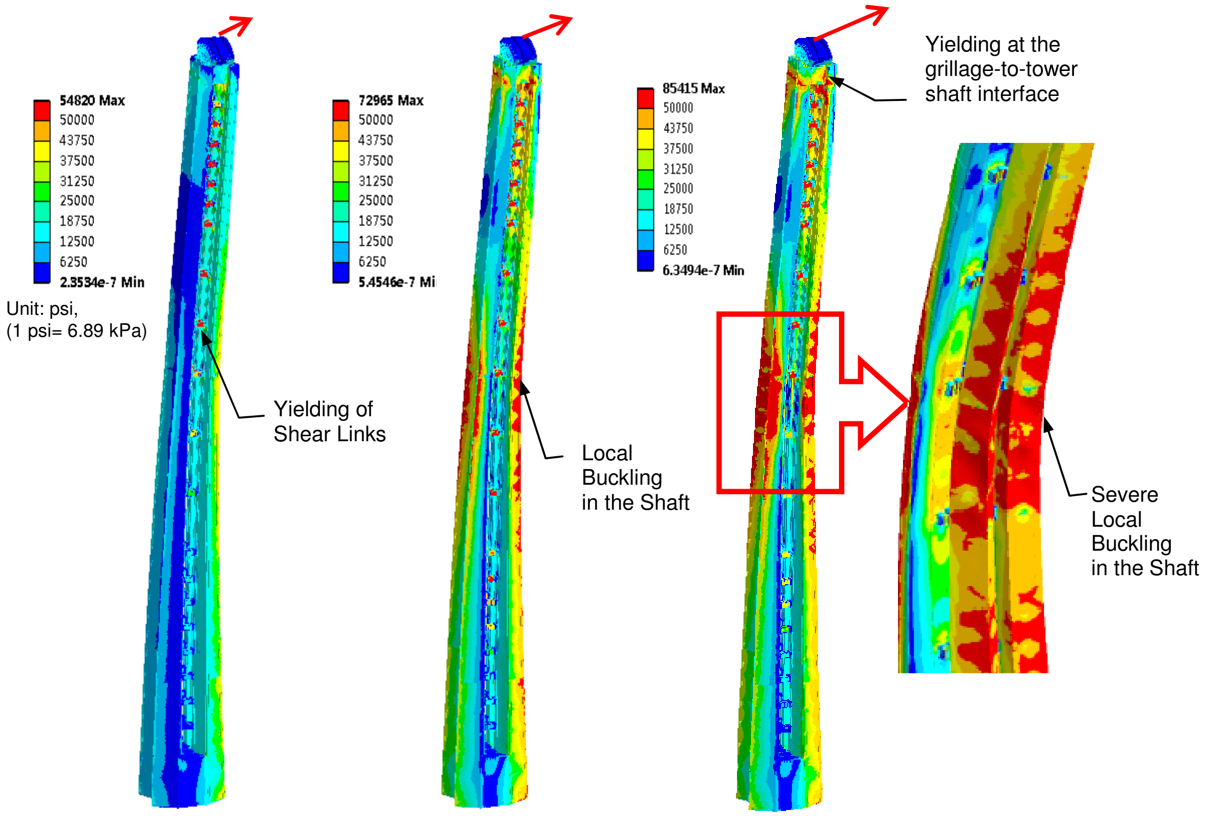


Figure 8: Definition of Yield Point for (a) a Structure with Clear Linear Hardening Slope; and, (b) a Structure with no Clear Hardening Slope

Point "U" on the pushover curve of Figure 7 corresponds to the point where maximum push-over strength was reached. Point "D" corresponds to a point where the pushover strength has dropped to 85% of the maximum strength at Point U. Point "D" is an important point, since it is used to calculate the ductility of a system. The displacement δ_{in} in Figure 8 is considered the maximum inelastic displacement. The ductility of a system is defined as this displacement δ_{in} at 85% maximum strength divided by the displacement at yield point, δ_y in Figure 8. Considering the pushover curve shown in Figure 7, ductility of the tower in the transverse direction pushover is δ_{in}/δ_y =5.4m/1.7m = 3.2, where 5.4m and 1.7m are δ_{in} and δ_y, respectively, from Figure 7 earlier.



(a) **At Point Y (Yield Point)**

(b) at Point U (Maximum Strength)

(c) at Point D (Point where Ductility is measured)

Figure 9: Equivalent (i.e., von Mises) Stresses at the Yield Point, Maximum Strength and 85% Maximum Strength Points for Transverse Pushover of Tower

The equivalent stress plot of the tower at three important points (i.e., yield, maximum strength, and 0.85 maximum strength points) is shown in Figure 9 for pushover in the transverse direction. Here, points "Y", "U", and "D" correspond to the same points on Figure 7. At the yield point (Point Y), the top seven and middle three pairs of shear links yield first, while all other parts of the tower remain essentially elastic. Then, as the tower is pushed beyond the yield point, gradually all the shear links, except the four at the bottom of the tower, yield as the system strain hardens before reaching the peak strength at Point U.

From Point Y to U, some yielding of the tower shafts also occurs. Such yielding of tower shaft plates occurred at the mid-height portion of the tower where there is a slight change of the slope of the tower shaft. The connection interface between the tower grillage directly below the saddle also yielded. After passing the Point U, strength of the tower drops relatively fast, and local buckling of the yielded mid-height portions of the tower shafts becomes more pronounced. The “local buckling” of the tower structure is essentially the “overall buckling” of the stiffened vertical shaft plates between the horizontal diaphragms.

During this stage, from Point U to Point D, larger regions of the tower grillage-to-tower shaft interface yielded. The yielding is likely to be due to the difference in rigidity of the grillage and the four separate tower shafts – the four tower shafts tend to deform independently (mechanism for the shear links to work) while the rigid grillage is trying to hold them together and remain flat.

Figure 10 shows shear link rotation versus tower tip displacement for two specific transverse direction shear links, which are the shear links at an elevation of 173ft-10in. (53m) and 357ft-7in. (109m). The shear link rotations plotted in Figure 10 are calculated using the same method described in McDaniel et al. [9], who performed tests of actual shear links representing the shear links in the SAS Bay Bridge.

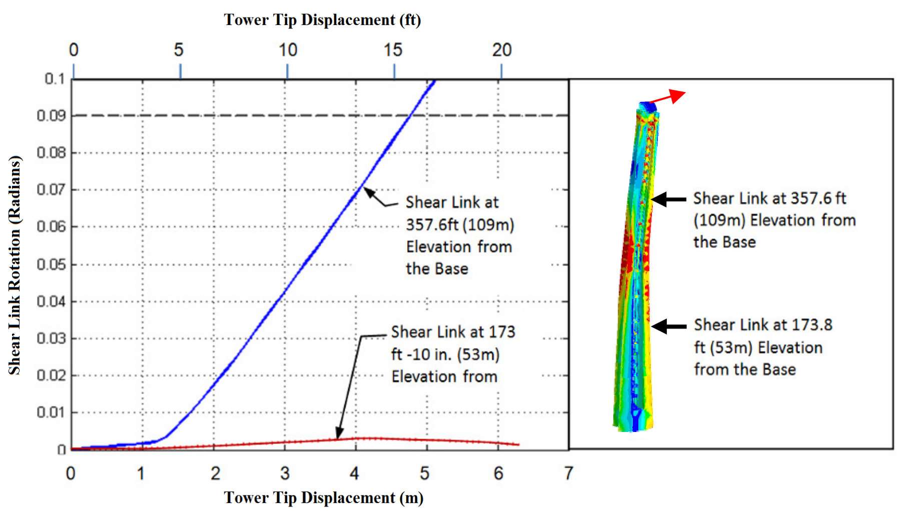


Figure 10: Shear Link Rotation vs. Tower Tip Displacement for Transverse Pushover Analysis

Due to the assumed fixed tower base boundary condition, the shear link at 53m experienced less than 0.01-radian rotation during the entire pushover analysis. However, the shear links located at the height 109m experienced much higher rotation demand; the rotation reached 0.03 at tower tip displacement of about 2.5m, which is halfway between yield point Y and ultimate Point U in Figure 7. The rotation reached the ultimate rotation capacity of 0.09 radians at the displacement of 4.8m on the pushover curve. Since the fracture of the material is not included in the model and considering that the top group of shear links reached their ultimate rotation capacity, the degradation of base shear capacity could be even worse than that shown in Figure 7, in addition to the local buckling of the tower shafts.

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Conclusion

This paper presents the pushover analysis of the single tower of the SAS Bay Bridge performed by the bridge design team [3-6] and compares these results with a pushover analysis performed by the authors. The analysis model of the tower used by the bridge design team in their pushover analysis consisted of members such as the tower shafts, represented by "beam" elements with no "shell" elements. The "beam" elements cannot capture the most important failure mode of steel plates in compression, which is local buckling of plates, unless a more accurate nonlinear force-displacement relation is incorporated. As presented here, all plates except the vertical stiffeners were modeled, including the vertical shaft plates, horizontal diaphragms, and the shear links using nonlinear "shell" elements capable of developing local buckling. Thus, a more robust prediction of the actual behavior of the tower was achieved. Figure 11 compares the two approaches: the tower pushover curves obtained using a very simplistic model of the tower and one that used a realistic, detailed finite element model.

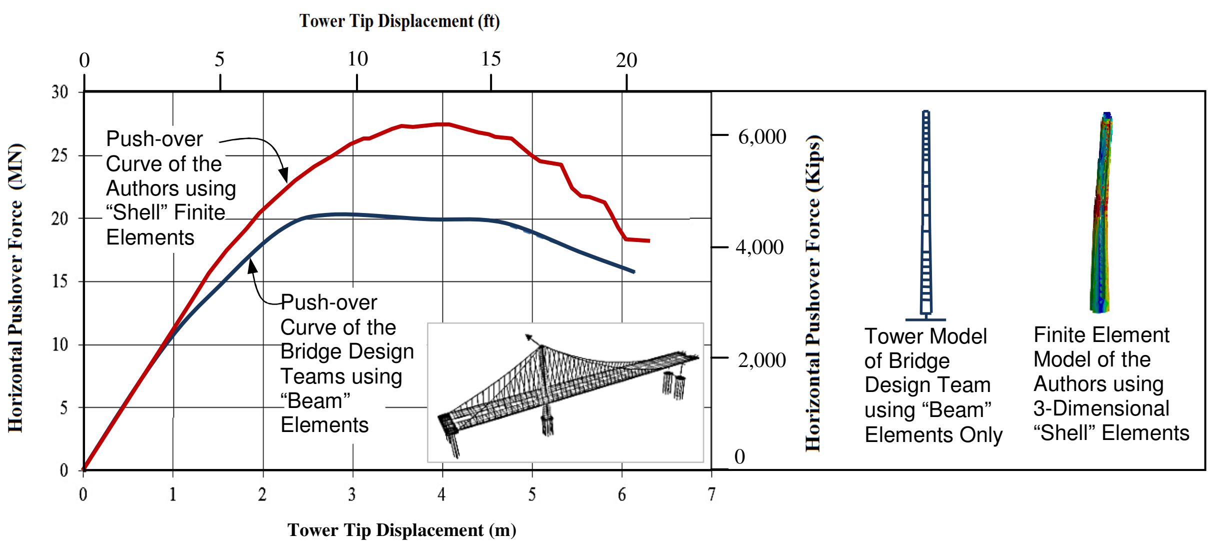

Figure 11: Comparison of Push-Over Curves by the Designers Using Simplistic "Beam" Elements to the Curves by the Authors Using Realistic "Shell" Elements

The most important finding of this study is that the bridge design team adopted a simplistic modeling approach of the tower shafts by using only "beam" elements; premature local buckling of the tower shafts is not predicted in their pushover analysis and was not considered in the design of the tower. The main reason for premature local buckling of the tower plates is that the vertical stiffeners used in the tower are flat plates instead of geometries, i.e., "T", or "U", which can be more effective in stiffening steel plates and thus prevent their local buckling. Such stiffeners are used in most steel bridges, including in the orthotropic deck of the SAS Bay Bridge. Qian and Astaneh-Asl [10] studied the effects of various geometries and locations of the vertical stiffeners in steel bridge towers and piers. One of the important findings was that flat plate stiffeners spaced equally are the least effective stiffeners in preventing local buckling of plates. Based on these findings, it is necessary that the tower be retrofitted to prevent premature local buckling of the tower of the

SAS Bay Bridge during a major earthquake. We propose a retrofit measure shown in Figure 12, where bolted “T” sections or welded pipes and channels are added to the vertical stiffeners over about three-fourths of the height of the tower, where local buckling of vertical plates of the tower can occur. If a welded option is selected, the traffic on the bridge needs to be reduced or halted during welding. However, to avoid welding in the field, the pipe and the channel in Optios 2 and 3, can be shop-welded to a plate and the plate field-bolted to the vertical stiffeners.

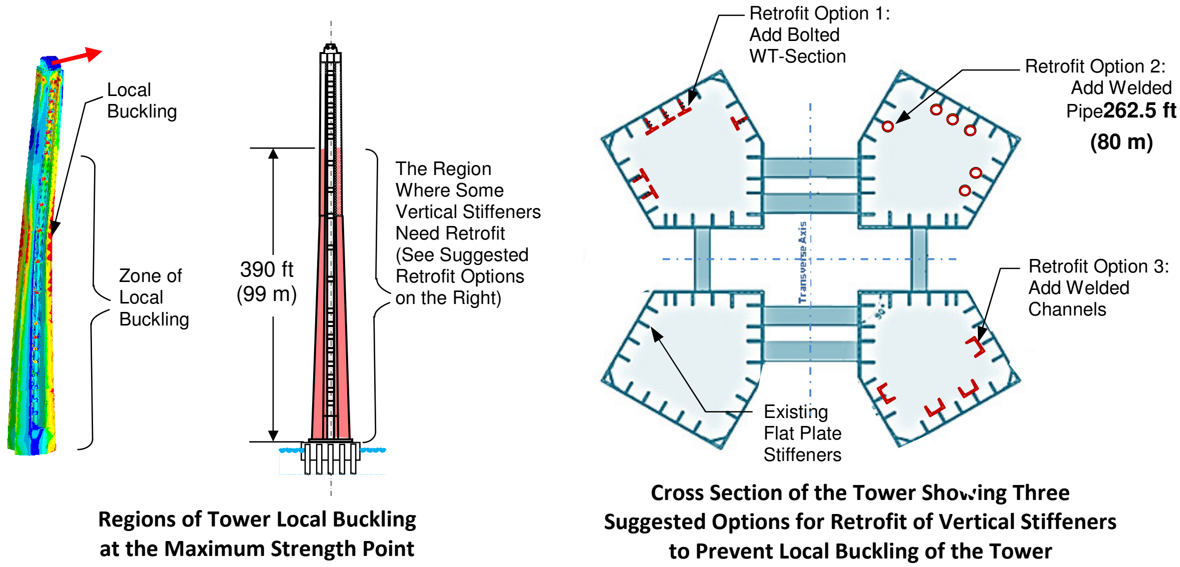

Figure 12: Suggested Retrofit for Vertical Stiffeners of the SAS Bay Bridge to Prevent Local Buckling of the Tower

The observations and conclusions based on the results of realistic pushover analysis of the SAS Bay Bridge tower performed using shell elements for the steel plates are presented below:

1. This study shows that using “beam” elements in modeling large and complex structures instead of realistic “shell” elements may result in an incorrect prediction of the behavior of the structure. In this case, “local buckling” failure mode of the steel plates, a critical failure mode of the bridge tower, are captured correctly by “beam” elements used by the bridge design team;

2. The bridge design team’s model used “beam” elements for the shafts, which resulted in underestimating the stiffness and ultimate strength before significant yielding. The consequence of underestimating stiffness and strength is that the inertia forces generated in the structure during a seismic event will be significantly larger than the time-history analysis of the structure predicted by the designers. As a result, the bridge was designed for smaller forces than it will actually experience when subjected to the seven earthquake records [11] that the bridge design team used in their design of the bridge;

3. As shown in Figure 11, because the bridge design team used “beam” elements in their pushover analysis, yielding of the tower occurs under much smaller forces than it would if the tower was realistically modeled using “shell” elements. This unrealistic early yielding resulted in a ductility of about 5.5 for the tower by the bridge design team, which is incorrect, compared to the realistic value of 3.2, resulting from a pushover analysis of the tower modeled using “shell” elements. The lower, but more realistic ductility, can result in less than desirable performance of the tower during major earthquakes;

4. The statement by bridge design team that: “*The shear links between the tower shafts will be the only inelastic elements in the tower and will act as fuses to protect the tower shafts from yielding”* is inaccurate. Pushover analysis of the tower incorporating steel plates modeled as shell elements demonstrates that, in fact, yielding and local buckling of the tower shafts occurs relatively early in the pushover analysis;

5. The analysis results summarized herein also pose a question on the effectiveness of using a shear-link coupling system as a seismic fuse with a rigid restraint at the top in the form of the saddle and its supporting grillage. The shear link yielding depends on the relative displacement of the tower shafts in the vertical direction. However, with a rigid saddle restraining the top of the tower shafts, such a yielding mechanism is disrupted because of the saddle and its supporting grillage; The results also show that the change of the slope of the tower shafts at about mid-height results in stress concentration at that location and may cause local yielding and local buckling to initiate at that location. A constant slope for the tower shafts could prevent such stress concentrations;

6. Local buckling of the tower shafts occurs relatively early in the pushover analysis. Since the tower will be pushed beyond the design level earthquake during major earthquakes, it is critical to prevent local buckling of the tower shafts;

7. Based on the findings of Reference [10] detailing the behavior of stiffeners in steel bridge towers and piers, we propose efficient and economical retrofit measures , as shown in Figure 12, for vertical stiffeners of the tower to prevent local buckling of tower shafts.

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Biographies

ABOLHASSAN ASTANEH-ASL, Ph.D., P.E., Professor, Dept. of Civil and Env. Engineering, Univ. of California, Berkeley has over 48 years of experience in design, research and teaching courses in structural and earthquake engineering and design of buildings, bridges, and other structures as well as blast protection of structures. He has testified as an expert before the policy making and judicial bodies, and has completed more than 50 major research projects and has published more than 300 technical publications. Dr. Astaneh-Asl may be reached at Astaneh@berkeley.edu.

XIN QIAN is currently a Ph.D. candidate at the University of California at Berkeley. She earned her B.S. degree at the Hong Kong Polytechnic University and M.S. in Civil

Engineering from the University of California, Berkeley. Her research interests include steel and composite structures and earthquake engineering. She is the recipient of the 1st Professor T. Y. Lin Fellowship in Structural Engineering at UC Berkeley and is active in student activities in earthquake engineering as the President of the EERI Berkeley student chapter. Ms. Qian may be reached at cindyqian@berkeley.edu.