The Effects of Failure of Anchor Rods on the Performance of the Tower of the New Bay Bridge

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Abstract

The single tower of the new Bay Bridge opened in 2013, is connected to its footing by 424, high-strength ASTM A354 BD, hot dip galvanized anchor rods. The bridge is expected to experience strong earthquakes from nearby Hayward or San Andreas faults. Bridge specifications do not recommend the use of ASTM A354 BD hot-dip galvanized anchor rods due to the high probability of “hydrogen embrittlement” in these anchor rods leading to their fracture. A few months before the bridge was opened, 32, A354 BD anchor rods fractured after tightening. This paper investigates the push-over behavior of the main tower of the SAS Bay Bridge in the likely event of fracture of the anchor rods at the base of the tower due to hydrogen embrittlement. At least two anchor rods have already fractured at this writing.

In this project, the push-over behavior of the main bridge tower without anchor rods connecting its base to the pile cap footing was analyzed through numerical simulation. A realistic non-linear model of the tower was created in ANSYS. The bridge tower, base plate, and the concrete-steel composite pile cap were modeled in detail. No anchor rods were included in the model to connect the tower to the footing. After applying the gravity load, the top of the tower was pushed in the transverse direction until it collapsed. The results of the realistic pushover analysis indicated that the lateral strength of the tower drops relatively fast after the peak due to local buckling of the legs of the main tower, yielding of the base plate and crushing of the concrete under the base plate. During late stages of the pushover, the Partial Joint Penetration (PJP) welds connecting the tower to the base plate also fractured. When the anchor rods are in place and tightened, these PJP welds are in compression, however, without the anchor rods, the welds will be directly subjected to tension and eventually fracture. The Bridge Design Team has indicated that the anchor rods are not needed. These studies, however, question the validity of the statement and suggest retrofit measures to restore the strength, stiffness, and ductility of the tower.

Introduction

The new East Spans of San Francisco-Oakland Bay Bridge is a “Self-Anchored-Suspension (SAS)”bridge with a single tower. The bridge opened to traffic in 2013, is located between

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two active seismic faults, the Hayward, and San Andreas faults. Figure 1 shows the elevation and plan view of the new self-anchored suspension Bay Bridge. More information on the properties of the bridge can be found in Astaneh-Asl and Qian [1] and Caltrans drawings [2].

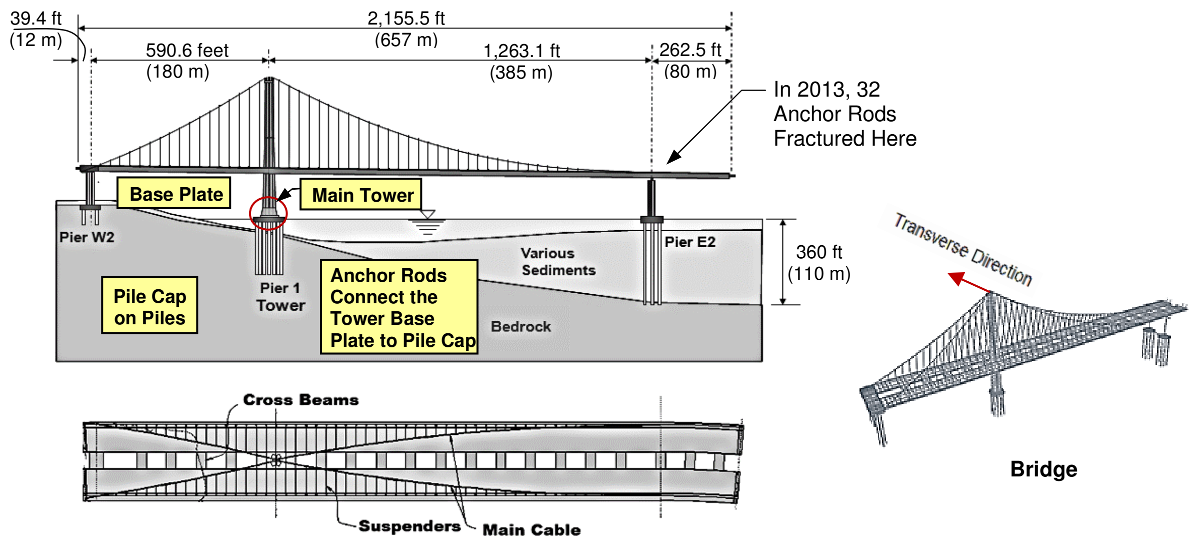


Figure 1. Elevation and Plan View of the SAS Bay Bridge

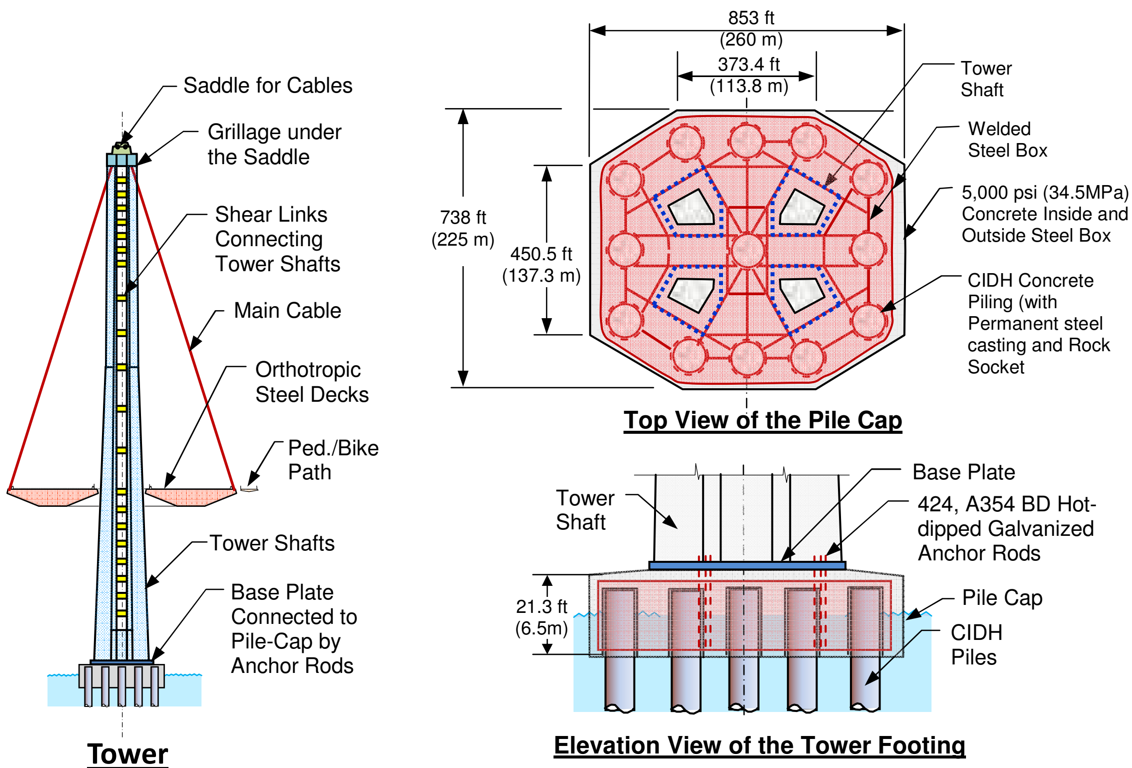


Figure 2: Elevation of the Tower (left) and Plan and Elevation of the Pile Cap Footing Supporting the Tower

As shown in Figure 2, the pile cap supporting the tower is a concrete-steel composite box. The shafts, as well as the base plate, are made of ASTM A709 Gr. 50 steel with a minimum specified yield stress of 50 ksi (345MPa) and an ultimate strength of 65 ksi (448 MPa). The shafts are connected to each other by steel I-shaped shear links along the height of the tower, by a saddle at the top, and by steel vertical shear plates at the base. For more information on the tower, see Astaneh-Asl & Qian [1].

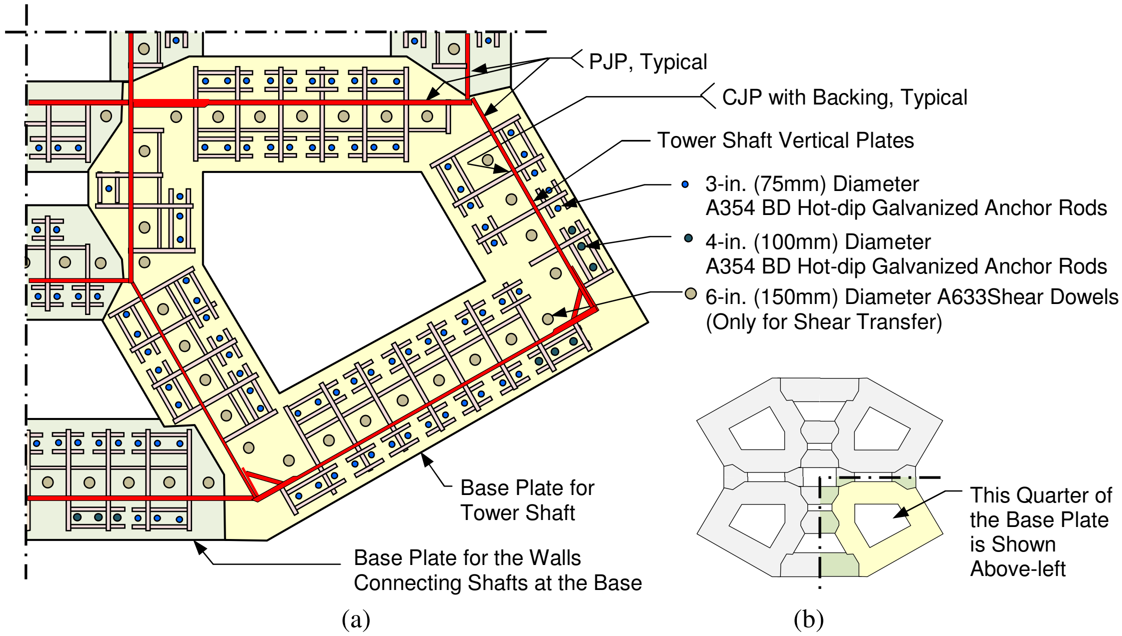


Figure 3: Plan of the Base Plate and Location of Anchor Rods

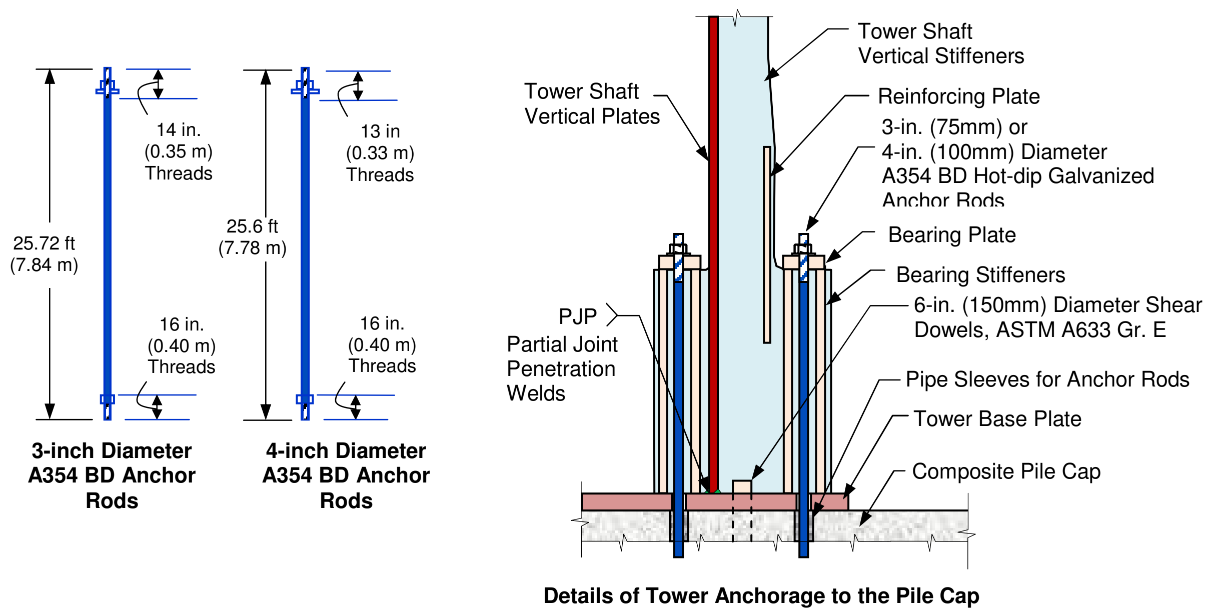


Figure 4: Anchor Rods and Details of Tower Anchorage to the Pile Cap

Figure 3 shows a plan view of a quarter of the base plate with the locations of 3in. (75mm) and 4-in. (100mm) diameter ASTM A354 BD hot-dip galvanized anchor rods, 6-in diameter ASTM A633 shear dowels, and the anchorage plates. The rods are made of SAE 4140 steel with a minimum yield strength of 115 ksi (793 MPa) and a minimum ultimate strength of 140 ksi (965 MPa) [3]. The base plate of the tower is made of 14 separate plates, Figure 3(b). Figure 4 shows typical details of the connection of the tower shafts to the base plate and anchorage of the tower base to the pile cap by anchor rods.

Investigation of the Fracture of the A354 BD Anchor Rods in the SAS Bay Bridge

The use of A354 BD hot-dip galvanized anchor rods in this important bridge created a serious undesirable behavior. In 2013, and a few months before the opening of the bridge, 32 of the 96 anchor rods connecting the seismic shear keys to the top of the Pier E2 on the east end of the SAS Bay Bridge fractured when tightened (see Figure 1 for location).

In 2015, M. Nader, (of the TYLI/Moffitt Joint Venture) the Chief Engineer and Engineer of the Record for the SAS Bay Bridge, presented an analysis of the bridge subjected to a selected number of ground motions. He concluded that even without any anchor rods, the response of the bridge to six ground motion records (that the Bridge Design Team has considered in the design of the bridge) will be almost the same as the response with all anchor rods present [4] & [5]. A critical review of the validity of this claim as well as the correctness of the analysis is not possible at this time since Refs. [4]& [5] do not provide much information on the analysis itself. However, the bridge model that the Bridge Design Team has used does not seem to include local buckling of the tower shaft plates, and fracture of the Partial Joint Penetration (PJP) welds that connect the base of the tower to the base plate, see Figure 4 earlier.

The Performance Criteria established for this "lifeline" bridge in [6] states that: "*The bridge shall have a clearly defined inelastic mechanism for response to lateral loads and inelastic behavior shall be restricted to piers, tower shear links, and hinge beam fuses.*" According to this statement in the Performance Criteria [6] for the bridge and publications by the Bridge Design Team, [7]& [8], local buckling, yielding or fracture of any other element of the tower, including plates, bolts, anchor rods and welds is not allowed. To access whether the Performance Criteria can be fulfilled with fractured anchor rods, this paper summarizes the results of realistic pushover analysis of the tower of the SAS Bay Bridge without anchor rods connecting the tower to the pile cap. Moreover, if the pushover behavior is not acceptable, suggest a measure of retrofit that can prevent premature failure of the tower during the future seismic events.

Objective

The main objective of the research summarized in this paper was to investigate pushover behavior of the main tower of the new SAS Bay Bridge with <u>no anchor rods connecting the base of the tower to the top of the pile cap.</u>

In 2013, it was discovered that two out of the 424 anchor rods connecting the tower to the pile cap were fractured. The cause of fracture was the hydrogen embrittlement of the A354 BD anchor rods. Since all 424 anchor rods are hydrogen embrittled [10] & [11], it is likely that more anchor rods will fracture during the service life of the bridge. The fracture of the remaining anchor rods due to the hydrogen embrittlement is the reason why we undertook this research to find out what will happen to the bridge tower during the future major seismic event without any anchor rods.

Realistic Push-Over Analysis of the Tower with No Anchor Rods at the Base

The remainder of the paper focuses on the realistic pushover analysis of the tower of the SAS Bay Bridge in the transverse direction (most critical) with no anchor rods connect the tower base plate to the pile cap.

Finite Element Modeling

ANSYS Workbench finite element nonlinear software was used to simulate the behavior of the main tower, base plate, and pile cap supporting the tower. All anchor rods were assumed to have fractured, and they were not included in the model. Proper gap elements were used to allow uplifting of the base plate.

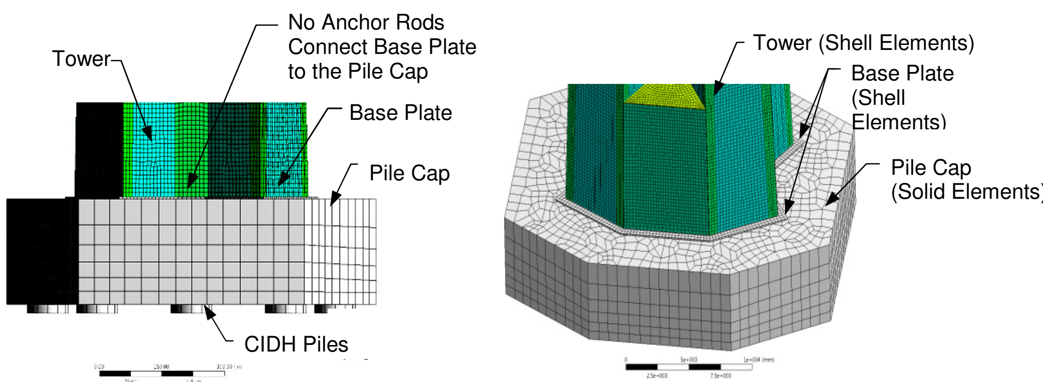


Figure 5: Finite Element Model of the SAS Bay Bridge Tower, Base Plate, and Pile Cap

The detailed model of the base plate and pile cap was added to be able to investigate the effects of failure modes associated with the connection of the tower to the base plate, the base plate, and the pile cap, while there is no anchor rods present.

Two types of steel material properties were used in the modeling of the tower structure. Almost all components of the main tower and the tower base plate were ASTM A709 Gr. 50 with a minimum specified yield stress of 50 ksi (345MPa). The connection plates of the shear links to the tower shafts were modeled with Gr. 70 steel with a minimum specified yield stress of 70 ksi (485MPa). Bi-linear kinematic hardening model was considered for the steel with an elastic modulus of 29,000 ksi (200 GPa), Poisson ratio of 0.3, and strain hardening ratio of 1%. The concrete used in this analysis to model the pile cap and the piles has an elastic modulus of 4,350 ksi (30 GPa), Poisson ratio of 0.18 and compressive strength of f'_c=5.073 ksi (35 MPa) which was obtained from the construction drawings [2] as the specified values. The concrete inside the steel box was modeled as confined concrete. More detailed information on modeling of the tower itself is in Astaneh-Asl & Qian [1]. After applying the gravity force to the tower, incremental horizontal displacements were applied to the cable saddle groove location under ANSYS displacement controlled iteration algorithm [9].

Results of Pushover Analysis of SAS Bay Bridge Tower with no Anchor Rods

Figure 6 shows the pushover curve of the tower in the transverse direction regarding horizontal force on the vertical axis versus applied horizontal displacement of the cable saddle on top of the tower on the horizontal axis. As the figure shows, the tower does not have a clear yield plateau, which would be a desirable characteristic of structures in resisting seismic effects. The tower behaves elastically from origin to the point of “initial yielding”, see Figure 6, and continues to yield more elements until it reaches the defined “yield point” at Point Y. In large structures, due to local yielding of very small areas, the pushover curve starts deviating from the initial elastic stiffness line very early, see Point Y in Figure 6. This point cannot be considered the yield point since the structure is essentially elastic. For these cases, we have defined a “yield point” where the pushover curve is deviated from the initial elastic line a horizontal distance equal to 10% of the horizontal elastic deformation. For more information on this definition of yield point see Ref [1].

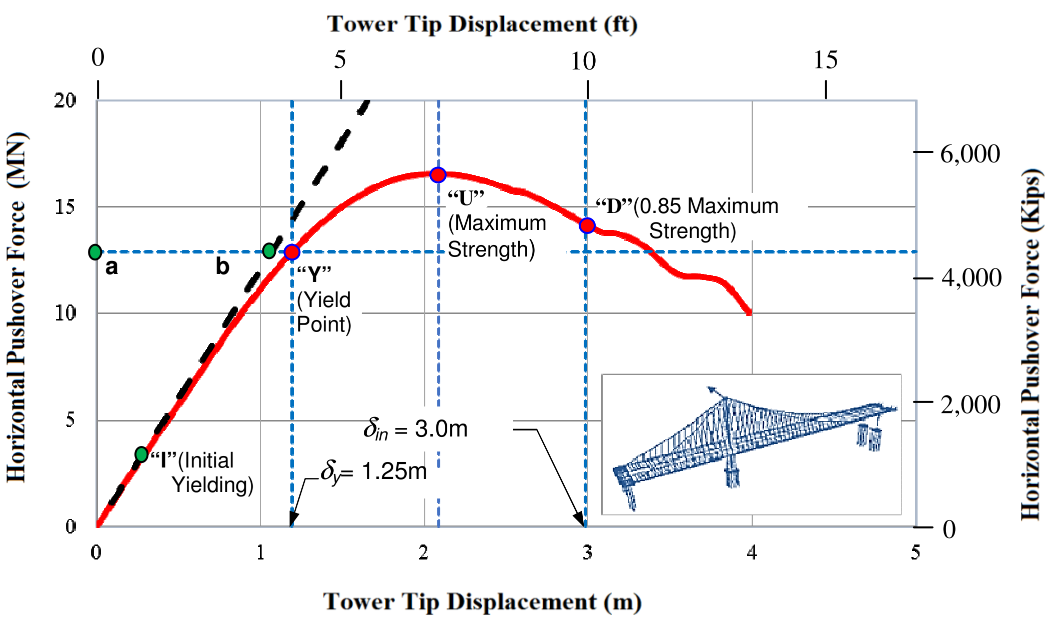


Figure 6: Transverse Pushover Curve When There Are No Anchor Rods at the Base

As pushover continued beyond the yield point, Point Y in Figure 6, the tower continued to accept larger forces due to strain hardening while the stiffness continues to decrease due to further yielding. During the pushover, shear links yielded first, then, as pushover continued, yielding of shear links continued, but the base of the tower started uplifting on the tension side and yielding on the compression side of the base plate. As the tower reached its maximum strength at Point U, see Figure 6, significant yielding and local buckling of the compression side of the base of the tower had already occurred.

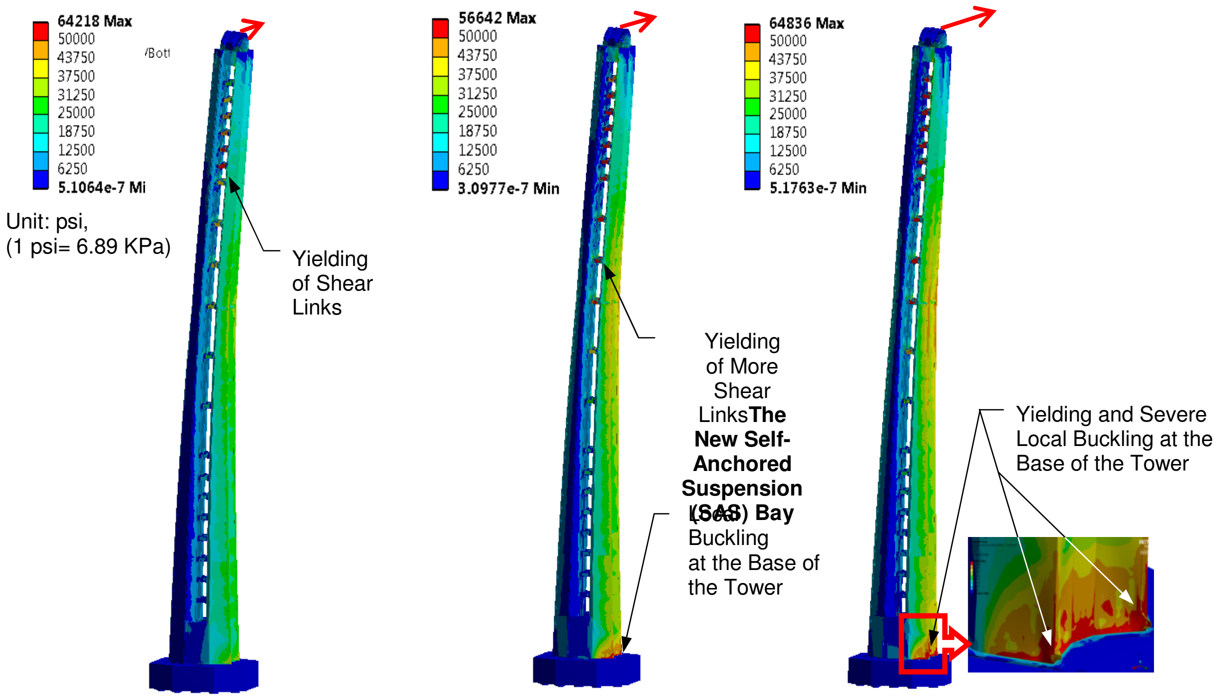
Point D on the pushover curve in Figures 6 is an important point on the behavior since it represents the point where the ductility of the system is measured. For a relatively large structural system such as the SAS Bay Bridge tower, the ductility is defined as the ratio of the displacement at a point where the strength is 85% of the maximum strength (δ_{in} in Figure 6) and the displacement at the "yield point" (δ_y in Figure 6). As shown in Figure 6, the δ_{in} and δ_y are 9.8 ft (3.0m) and 4.1ft (1.25m) respectively. Therefore, the pushover ductility of the tower is 9.8/4.1 = 2.5.

Performance of the Main Elements of the Tower during the Pushover

This section discusses the results of the behavior of main elements of the tower during the pushover when all anchor rods are assumed to have fractured due to a combination of hydrogen embrittlement and seismic forces.

Tower Behavior under Pushover- The equivalent Von-Mises stresses in the tower at Points "Y", "U" and "D" during the pushover in the transverse direction are shown in Figure 7. The red color represents yielding in the shell elements (i.e. plates). In this case, yielding is defined as equivalent Von Mises stress reaching the specified minimum yield stress of the steel plates, 50 ksi, (345 MPa) for all steel plates including the shear links, and 70 ksi (483 MPa) for the end connection plates of the shear links to the tower shafts. The Points Y, U, and D correspond to the same points as in Figure 6, i.e. yield, maximum strength, and 0.85 maximum strength points on the pushover curve.

At yield point, Figure 7(a), two out of seven pairs of shear links at the top portion of the tower yield, while the other parts mainly remain elastic. High stresses are generated on the compression side of the tower in the middle portion and at the base of the tower. At the point of maximum strength (Point U) in Figure 6, shear links in the upper part, as well as in the middle part of the tower have yielded in shear, Figure 7(b). There is also yielding and some local buckling in the compression side of the tower in the middle portion. However, the compression side of the tower at the base shows severe yielding and local buckling. After reaching the maximum strength, pushover strength of the tower drops relatively fast due to local buckling in several areas of the tower base and corner stiffeners. At the point (Point D in Figure 6) where the applied force has dropped to 85% of the maximum strength, and where ductility is measured, more yielding of the shear links at the top and middle portions of the tower occurs, Figure 7(c). Moreover, the base of the tower shows widespread yielding and severe local buckling on the compression side.



(a) At Yield (Point Y) **(b) at Max. Strength (Point U)** **(c) at Point where Ductility is Measured (Point D)**

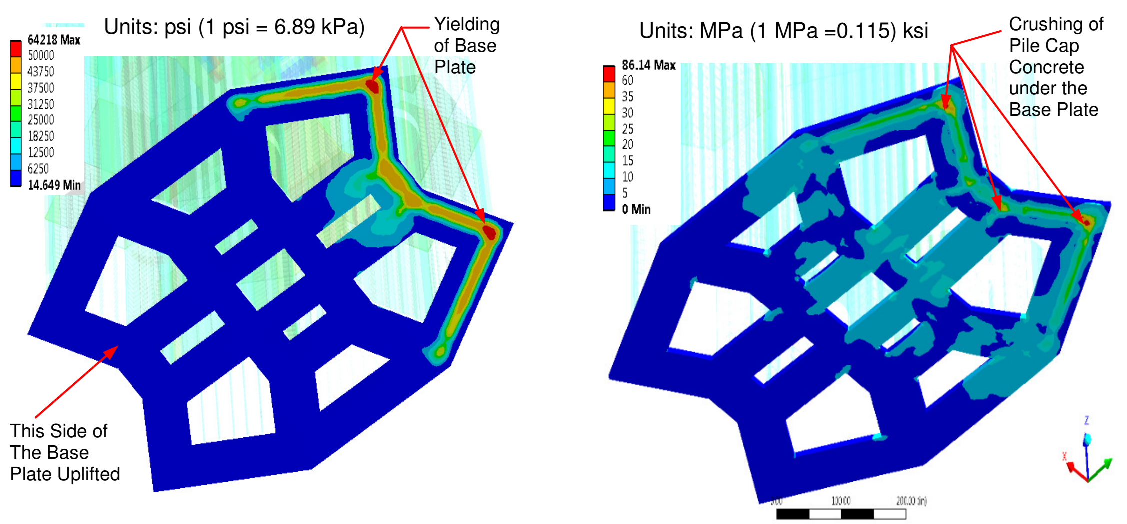
Figure 7: Equivalent (i.e. von Mises) Stresses at the Yield Point, Maximum Strength and 85% Maximum Strength Points for Transverse Pushover of Tower

The Behavior of PJP Welds Connecting the Tower to the Base Plate- Partial Joint Penetration welds connect the tower legs to the base plates, see Figure 4 earlier.
Notice that when anchor rods are present and tightened to the specified amount of pre-tensioning (i.e. 70% of their tensile strength), the base of the tower and the entire area of the base plate are compressed down on top of the pile cap. The tower pressing on the base plate results in the PJP welds connecting the base of the tower shafts to the base plate to be in compression. However, when there are no anchor rods in place, or if the anchor rods are not tightened to 70% of their tensile strength, during the push over, part of the base plate can uplift resulting in subjecting the PJP welds to tension. Therefore, it is important to investigate if the welds connecting the tower's vertical plates to the base plate have the strength to transfer the tensile uplift stresses from the vertical plates of the tower to the base plate.

The pushover analysis indicated that when the pushover horizontal displacement reaches 7.6 ft (2.34 m), the PJP welds at the base of the tower will fracture. Obviously, this is an unacceptable performance for a lifeline bridge that according to its performance criteria, the only damage allowed in the tower is the ductile shear yielding of the shear links, while no fracture of welds is allowed.

The Behavior of the Base Plate- Figure 8(a) shows the equivalent Von-Mises stresses on the bottom surface of the tower base plate when the tower reaches its maximum strength (i.e. Point U in Figure 6 given earlier). At this point, some areas of the base plate have yielded. Such yielding of the base plate during the pushover is in violation of the Performance Criteria for this bridge, which allows only yielding of the shear links while the rest of the tower remains elastic.

Stresses on the Pile Cap- The vertical pressures that the bottom surface of the base plate exerts on the top of the pile cap at Point "U" during the pushover are shown in Figure 8(b). Since the concrete under the base plate is confined, the maximum compression strength on it can reach $1.7f'c$, where f'_c is the specified compressive strength of the concrete measured using cylinder specimens. Therefore, in Figure 8, the red corresponds to the locations with pressure on the concrete exceeding $1.7f'_c$, which indicates compressive crushing of concrete under the base plate.



(a) Yielding of the Base Plate **(b) Crushing of the Concrete under the Base Plate**

Figure 8: Equivalent (i.e. von Mises) Stresses at (a) the Bottom Surface of the Base Plate and (b) at the Top Surface of the Pile Cap, When the Tower, Without Anchor Rods, Reaches its Maximum Strength during the Pushover

Comparison of Tower Behavior With and Without Anchor Rods

Figure 9 compares pushover behavior of the tower with and without anchor rods connecting it to the pile cap. The curve for the tower pushover with the anchor rods is extracted from Astaneh-Asl & Qian [1]. On the curve for the tower with the anchor rods, points Y, U, and D correspond to points of yield, maximum strength and 85% of maximum strength (i.e. the point where ductility is measured). Points Y', U', and D' are similar points on the pushover curve for the tower without any anchor rods. The initial elastic stiffness for both curves is almost identical, which means, before significant yielding, under dynamic loading during the

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earthquakes, the tower with or without anchor rods connecting it to the pile cap, will be subjected to almost the same seismic inertia forces. However, comparing force at Points Y and Y', the tower without the anchor rods will yield the shear links at about 70% of the force that will yield the tower with the anchor rods.

As for the maximum strength, comparing points U and U', the tower without the anchor rods reaches its maximum and drops the load at about 63% of what the tower with anchor rods could take. To compare ductility of these two cases of the tower with and without anchor rods, we need to compare the ratio of displacements at D and Y to the ratio of displacements at D' and Y'. This process indicates that ductility of the tower without the anchor rods is reduced to 2.5 compared to 3.2 which is the ductility of the tower with the anchor rods.

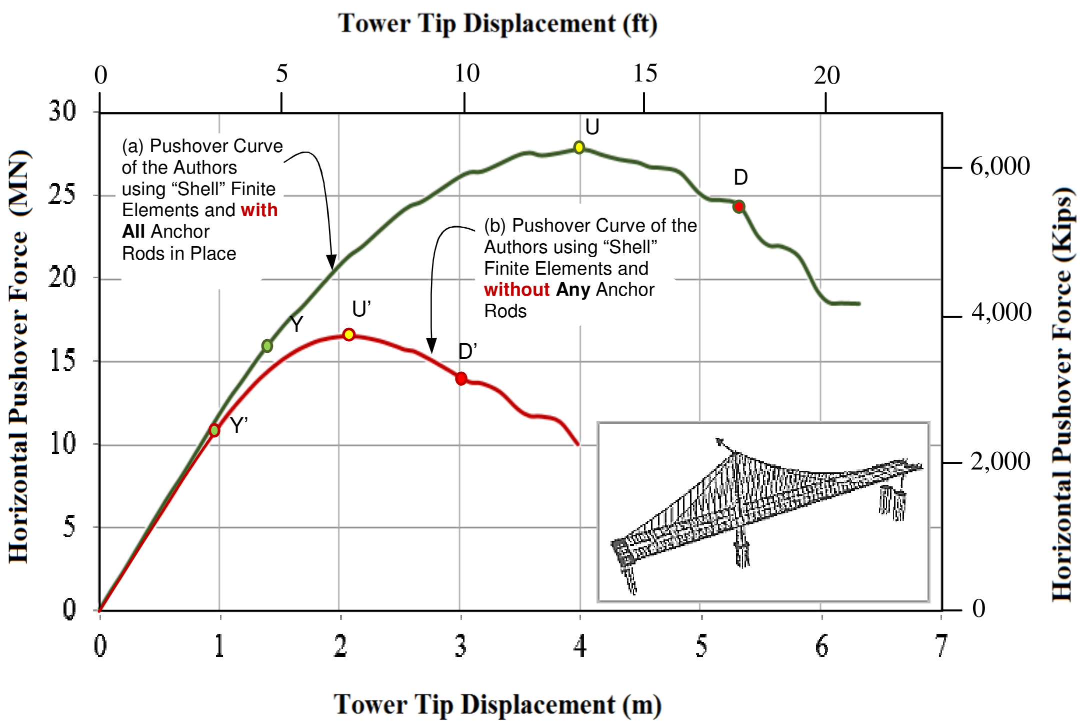

Figure 9: Comparison of Push-Over Curves of the tower with and without anchor rods

Suggested Retrofit to Mitigate the Hazard Posed by Brittle Tower Anchor Rods

The problem of SAS Bay Bridge anchor rods is directly related to the following aspects:

(a) the design decision to use ASTM A354 BD hot-dip galvanized anchor rods in a very corrosive offshore environment;
(b) the failure during construction in leaving the anchor rods in the open environment unprotected for more than two years;

(c) embedding the anchor rods in the pile cap concrete inside sleeves and not filling the sleeves with grout. The lack of protective grout resulted in the seawater seeping into the pile cap to collect around the anchor rods and caused further stress corrosion in them.

In this paper, it was shown that not having the anchor rods results in the undesirable behavior of the main tower. To mitigate the problem, following is the summary of proposed retrofit measures:

1. The tower needs to be retrofitted to prevent yielding and local buckling of the tower legs. The problem of the local buckling of the tower legs is also an issue even when anchor rods have no problem as shown in Astaneh-Asl & Qian [1]. Hence, the vertical stiffeners inside the tower legs need to be strengthened by adding stiffening material to them as shown in [1]. The satisfactory performance of such stiffeners was established in [13];
2. The Partial Joint Penetration Welds (PJP) connecting the tower legs to the base plate need to be strengthened to develop the yield strength of the tower leg plates;
3. Many of the existing 3-inch (7.6 cm) diameter unacceptable brittle A354 BD anchor rods that are not above the piles can be replaced, albeit with very high cost, with 3.5-inch (8.9 cm) ductile "upset" A354 BC anchor rods. To do so will require boring through the existing anchor rods through the entire 19.7 ft (6m) depth of the composite pile cap, attaching a steel reaction frame to the bottom of the pile cap, and installing the new A354 BC upset anchor rods connecting the tower base plate to the new steel structure at the bottom of the pile cap;
4. Water sealed caisson around the pile cap needs to be constructed to prevent seawater from reaching pile cap. The seawater currently is causing not only corrosion of the anchor rods but corrosion of the steel plate box of the composite pile cap. This latter problem is not part of this paper but also needs to be solved. So, this retrofit step can protect the pile cap as well.

Summary of the Anchor Rod Issue & Conclusions of the Pushover Analysis

In 2013, a few months before the SAS Bay Bridge was opened, 32, three-inch diameter A354 BD hot-dipped galvanized anchor rods connecting the shear keys on the east end of the bridge fractured after tightening. The cause of the fracture was established to be hydrogen embrittlement [10] & [11]. Bridge specifications do not recommend the use of A354 BD high-strength anchor rods and bolts and specifically prohibit hot-dip galvanizing them because of the possibility of hydrogen embrittlement as happened in this bridge.

The fracture of the 32 anchor rods in the shear keys after tightening, and then fracture of at least one anchor rod at the base of the tower, which was not even fully tightened, created a serious concern about the safety of the bridge [12]. In addition to hydrogen embrittlement of the anchor rods, when one of the anchor rods at the base of the tower also failed because of "thread stripping", the safety of the entire bridge came under question. More than 2,200 A354 BD hot-dip galvanized bolts and anchor rods are used in the most critical connections of the bridge superstructure. In particular, the single tower of the bridge is anchored to its pile cap support by 424 A354BD hot-dip galvanized anchor rods. These are the anchor rods that have the problem of hydrogen embrittlement and thread stripping.

After fracture of the first anchor at the base of the tower, the transportation officials in charge of the bridge initiated an investigation of the case to establish the cause and to develop a repair and retrofit strategy. As the investigation continued, it became apparent that the cause of fracture of the anchor rod at the base of the tower was hydrogen embrittlement, exacerbated by the presence of the salty ocean water inside the pipe sleeves around the anchor rods which in many cases were not filled with the grout as specified in the drawings. The fact that the threads on the anchor rods were also suspect to stripping made it very likely that in the long term, the anchor rods at the base of the tower are susceptible to fracture during or even before a major earthquake. Since the bottom ends of the anchor rods were embedded in the pile cap, removal of the existing brittle anchor rods and replacing them with sound anchor rods was almost impossible and cost prohibitive. In 2015, the Bridge Design Team in a presentation to the Toll Bridge Program Oversight Committee (TBPOC) provided results of their analysis. They concluded that the anchor rods at the base of the tower are not needed, and the seismic performance of bridge with or without anchor rods at the base of the tower will be almost the same during major earthquakes. They only recommended some limited repair, maintenance, and monitoring activities for the anchor rods and leaving the anchor rods in their place.

The main objective of the investigation summarized in this paper was to establish the pushover behavior of the tower without the anchor rods connecting it to the pile cap. Also, the two pushover curves with and without anchor rods were compared. In the model used in this study, summarized in this paper and Astaneh-Asl & Qian [1], all plates in the shafts were modeled as "inelastic "shell" elements capable of yielding and buckling.

Based on the results of realistic push-over of the SAS Bay Bridge tower, using shell elements for the plates, the following observation were made and conclusions reached:

1. This study shows that the pushover strength and ductility of the tower without anchor rods connecting it to the pile cap is only about 60% and 80% of that for the case with the anchor rods respectively.
2. There is a need for seismic retrofit of the tower itself and its base anchored to the pile cap by exiting brittle A354 BD hot dip galvanized anchor rods. Retrofit plans are proposed, which, if implemented, can mitigate the problems and bring the performance of this bridge to the level of "lifeline" bridge and satisf the corresponding Performance Criteria. The lifeline performance is for the bridge to open to traffic almost immediately after a major earthquake, with limited damage in specifically designated areas.

Lessons Learned and How This Problem Could be Avoided?

The following is a list of important lessons learned from this case study, which, also applies to the design and construction of other structures with similar details:

a. As many bridge design codes recommend, the A354 BD high-strength anchor rods should not have been used in this structure in the corrosive environment over the seawater, where the anchor rods are embedded in the pile cap, and pile cap is submerged in seawater.
b. The A354 BD anchor rods and bolts should not have been "hot-dip" galvanized, which is well known to cause hydrogen embrittlement in high-strength steel. Instead, they should have been mechanically galvanized with zinc or aluminum, which is a very common procedure.
c. The anchor rods should not have been left at the site in the open environment for more than three years to be exposed to rain and seawater environment.
d. The number of threads per inch and the depth of the thread should have been specified correctly to avoid thread stripping under tension.
e. The anchor rods should not have been embedded in the concrete of the pile cap. Instead, the following solutions were recommended in this case. First: the anchor rods should have been passed through the pile cap and have the bottom nut under the pile cap to ease replacement. Second: the anchor rods should have had a small segment at the top weaker than the main body of the anchor rods to ensure that the small portion at the top acts as a fuse and prevents damage to the main body of the anchor rod embedded in the concrete. The top segment could be replaced easily in case of corrosion during the service life or damage after a seismic event.
f. Instead of using A354 BD high-strength anchor rods, not allowed to be hot dip galvanized, "upset" A3254 BC anchor rods should have been used. The A354 BC has a specified minimum yield stress of 99 ksi (683 MPa) [3]. The A354 BC bolts can be galvanized either by hot-dip galvanizing, without developing hydrogen embrittlement, or by mechanical galvanizing.
g. In regular "straight" anchor rods and bolts, threads are cut into the cross section of the shanks, resulting in the fracture of the under-thread area of the threaded part to be the tensile failure mode. This failure mode is quite brittle and is not desirable in high-seismic applications, where ductile failure modes need to govern. In "upset" bolts and anchor rods, the shank cross-sectional area is smaller than the under-thread area such that the *yielding of the gross area of the shank* is the governing failure mode instead of the *fracture of the under-thread area.* If instead of problematic 3-inch A354 BD bolts, A365 BC upset anchor rods were used in this bridge, the diameter of the anchor rods (outside upset portion) would be 3.5 inches (8.9 cm) instead of 3-inch (7.6 cm) for the currently used anchor rods. With this relatively small but a very important design decision, there would be no problem of hydrogen embrittlement and thread-stripping for bolts and anchor rods as occurred in the new $6.5 billion SAS Bay Bridge.

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