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TUBULAR ECCENTRICALLY BRACED FRAMES FOR THE SAN FRANCISCO OAKLAND BAY BRIDGE TEMPORARY WORKS

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ABSTRACT: Construction of the signature span of the self-anchored suspension span of the San Francisco-Oakland Bay Bridge required the entire deck to be in place prior to transferring its weight to the hangers and main cable. Consequently, the deck needed to be supported on an independent, temporary structure during the significant period of time required for the assembly of all the components and the load transfer. While supported by the temporary structure, the deck remains vulnerable to earthquakes. For the design of the temporary support structures Caltrans specified a response spectrum based on a 92-year return period, and required the steel support towers to have a minimum ductility of two. The erected portions of the permanent structure were to remain undamaged in the design earthquake. To meet this requirement Klohn Crippen Berger designed twin trusses supported on eccentrically-braced frame (EBF) towers constructed using tubular sections. Tubular EBF's, based on recent research and development at the University at Buffalo in New York, do not require lateral support at the yielding shear link, thus allowing the towers to be flexible in the longitudinal direction of the bridge and ductile in the lateral direction. This paper details the analysis and design of the Tubular EBF temporary towers for the erection and assembly of the bridge deck for towers ranging from shorter ones over land, to taller marine towers on piles sunk deep into the bay sediments. The interplay between design for wind and the design for earthquake is described.

1. Introduction

The east span of the San Francisco Oakland Bay Bridge includes the world's largest self-anchored suspension span (SAS). Figure 1 shows a rendering of the completed structure. The SAS is a 625 m long steel structure that extends from the west end W2 bent to the east end where it cantilevers approximately 50m over the E2 bent to its connection with the skyway structure. The deck weight is carried by hangers to the suspension cables to the T1 and the piers at each end. A single suspension cable runs from the east end of the SAS over the T1 tower to W2; wraps around the W2 cap beam and runs over the T1 tower again, and is then anchored at the east end. The bridge and the falsework are shown in Figure 2.

The erection sequence of the SAS was essentially opposite to that of a traditional suspension bridge. This is because the horizontal component of the suspension cable is resisted by compression in the deck; therefore, the deck and tower must be in place before the cable is installed. In a normal suspension bridge the cable is installed supported by the cable and the end anchor blocks and then the deck segments are lifted and supported by the main cable. Conversely, the deck of the deck of the self-anchored structure needed to be erected near its final position and supported until the cable was installed and then the deck load was transferred to the cable.

The SAS deck was supported on temporary works for about three years, an unusually long load duration for falsework. Because of the long exposure and high seismic zone, the owner had specified seismic performance requirements in excess of what is usually applied for falsework. The purpose of this paper is to describe the design of the TEBF towers and how they provided seismic protection.

The objective of the design was to prevent damage to the permanent structures, should a significant earthquake occur at any time during the construction. To satisfy these requirements Klohn Crippen Berger (KCB) utilized ductile tubular eccentrically braced frame (TEBF) towers based on research by Berman and Bruneau (2005, 2006(a), 2006(b)). The main advantage that TEBF's have over common EBF's is that they utilize box sections for the links and therefore do not require lateral bracing to stabilize the links during yielding. The TEBF's were central to the design as they provided the mechanism to dissipate seismic energy and limit the loads acting through the foundations, towers, trusses and deck segments.

Figure 1 - Artist's rendering of the completed SAS.

2. Temporary Works Design

The temporary support structures were arranged as shown in Figure 2. The twin steel orthotropic box girders (OBG's) were supported on twin trusses running the full length of the permanent bridge. At the shorter west span between T1 and W2 the temporary truss spans were shorter and the truss depth was 6.5 m and designed with single 400mmx400mm square tube chord members. East of T1 where the temporary truss spans of up to 130 m were used, the trusses transitioned to 8.5 m in depth with dual 400mmx400mm chord members. The vertical truss planes were set apart 10m, a separation that was determined by the Contractor to facilitate access to work on the permanent structures. To resist longitudinal forces, the trusses were attached to the east and west end permanent concrete bents using collars.

The temporary support structure included 7 towers:

- Tower A nearest W1;
- Tower B situated on the rock knoll on Yerba Buena Island;
- Tower C at the edge of the water just to the east of the historic Torpedo building;

- Tower D aligned with the permanent main tower T1 and partly supported by the permanent tower foundation;
- Towers F and G in the deep marine sediments between T1 and the east bent E2, supporting truss spans of 130m; and,
- Tower H situated on and surrounding the E2 foundation.

The towers consisted of tubular eccentric braces between rectangular box columns that were slanted inward at a slope of approximately 16:1. A photograph of one of the towers taken during the construction of the temporary works is shown in Figure 3.

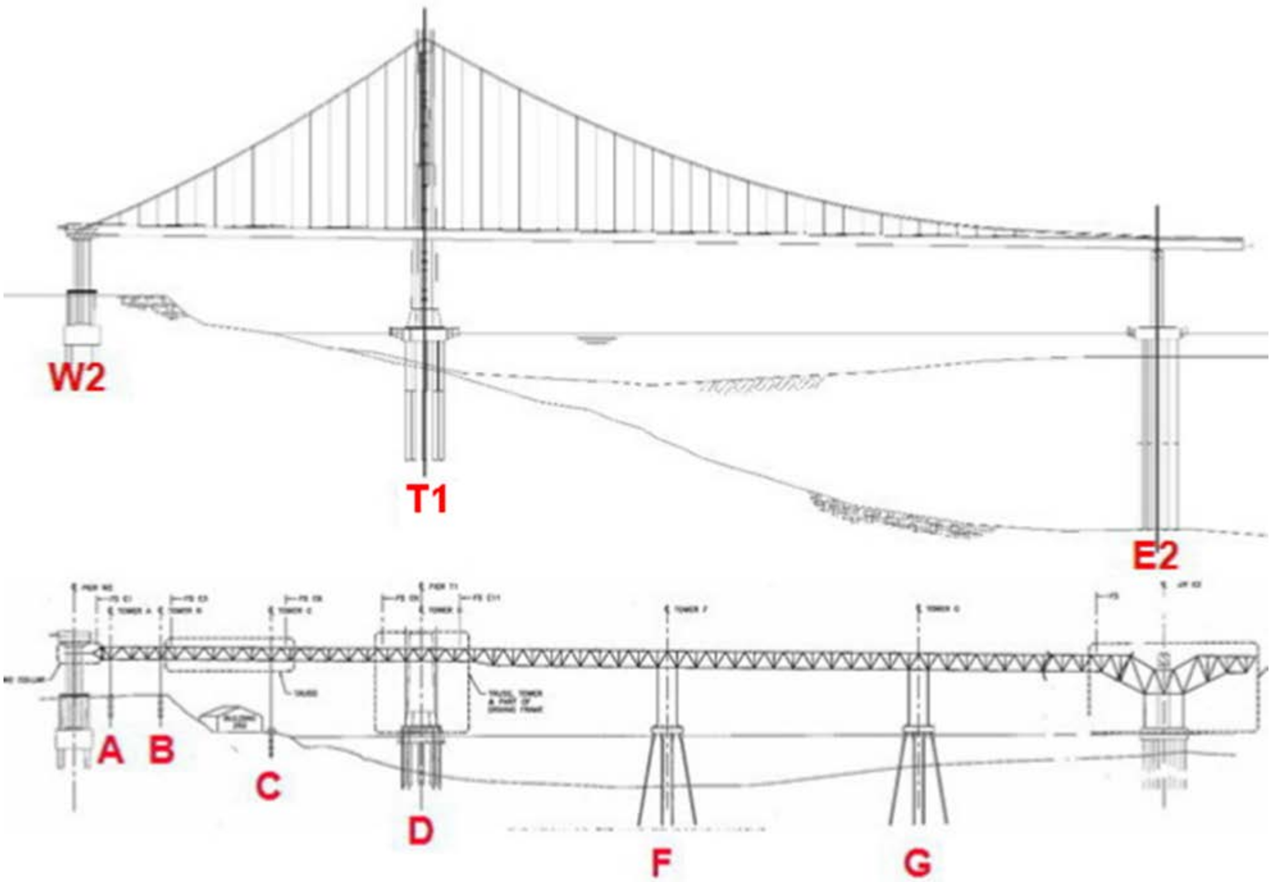

Figure 2 - Location and configuration the permanent structure, above. Below, the temporary towers and truss.

3. Seismic Performance Design Criteria

Caltrans developed a design spectra specifically for the temporary structures based on a 92-year return period event, deemed appropriate for the exposure time. The spectrum is shown in Figure 5. The seismic accelerations used to design the SAS temporary works were much greater than standard practice for falsework. The SAS design specification states that "The seismic performance of the temporary towers shall be such that the bridge structure is undamaged and not stressed excessively."

The spectra was based on 5% structural damping which is representative of a structure with capability of developing significant hysteresis (energy dissipation due to inelastic structural action) during the design earthquake.

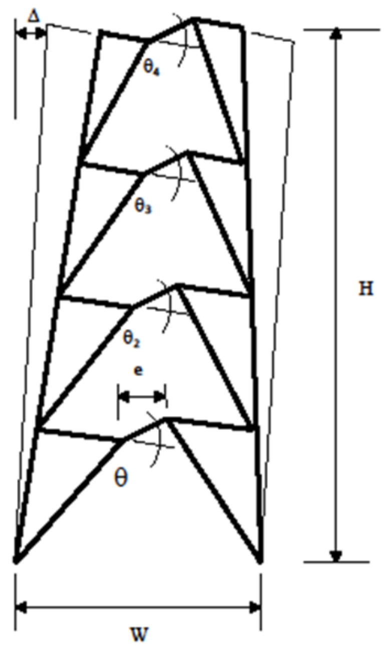

Figure 3 - TEBF tower under construction.

Figure 4 - Initial design assumptions for TEBF tower.

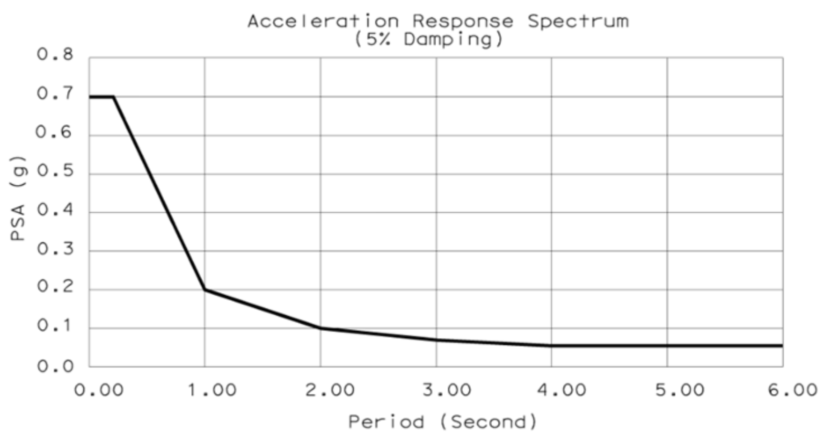

Figure 5 - Temporary Works Design Spectra.

Structures in the San-Francisco Bay are also subjected to potentially high winds. The 3-second gust load given by Caltrans for the design of the temporary structures was 100 miles per hour at the deck elevation. The temporary structures have to remain elastic for wind loads; however, for earthquake, it is desirable for the structure to yield and dissipate energy. If the structure remains elastic during the design earthquake and energy dissipation mechanisms are not present, the damping will not be as high as the 5% assumed in the definition of the design response spectra and seismic demands will be under-predicted.

The design criteria also specified that the temporary towers should have a minimum ductility of 2. This means the top of the structure must be capable of deflecting at least 2 x the deflection which cause the initial yield of a tower component. The intent of this was to provide additional capacity reserve for a larger seismic event. As the design progressed this was found to be impractical to achieve for the taller more flexible towers.

The Owner, Permanent works designer, Contractor and KCB collaborated at the beginning of the project to adjust the criteria to achieve the Owner's intent while making it practical to design the towers.

Substantial differences in height and foundation conditions for the various towers required a range of criteria.

The agreed upon criteria was as follows:

> *Structural System Ductility (μ) $\geq$ 2.0, i.e. $\Delta_{ult} \geq 2.0\ \Delta y$, where Δy = effective yield point of the structural system and Δ_{ult} = the total nominal displacement capacity of the structure. However, at locations where the design wind displacement exceeds the design seismic displacement, the following criteria may be used:*
>
> *Structural System Ductility (μ) $\geq$ 1.5, i.e. $\Delta_{ult} \geq 1.5\ \Delta y$; and Deformation Capacity (Δ_{ult}) $\geq 3.0\ \Delta_{sa}$, where Δ_{sa} = peak seismic displacement demand.*

The intent of this latter requirement was to provide an alternate but conservative requirement for the taller, flexible towers that were shown by analysis to remain elastic for the design earthquake. The criteria recognises that if the wind loads exceed the seismic loads as was the case for Towers F and G, the structure can be designed to have a lower system ductility providing the ultimate displacement capacity substantially exceeds the maximum displacement expected in the design earthquake. The result of applying these additional requirements was that the structure could withstand a much higher return period earthquake. Fortunately, the TEBF tower design was able to achieve these additional design requirements with tolerable cost impact.

4. Design of TEBF

The Tubular Eccentrically Braced Frame towers were designed to the requirements of AISC 341-05 Section 15 with guidance from Berman and Bruneau (2005, 2006(a), 2006(b)). The link beam is the component in which the yielding is induced by the eccentric bracing. The yielding in the link beam can be of two types: flexural, or shear. A shear link is controlled by the yielding of the webs in shear while a flexural link is controlled by the plastic yielding at the ends of the link. The yielding of a shear link is distributed along the full length of the link section, while a flexural link has the yield zone concentrated at the ends of the link. For a given cross-section, there is a range of lengths within which the link transitions from a pure shear link to a pure flexural link. The transition occurs between $1.6M_p/V_p<e<2.6M_p/V_p$. For $e<1.6M_p/V_p$ the link is a shear link and for $e>2.6M_p/V_p$ the link is a pure flexural link, where e is the link length, M_p is the plastic moment capacity and V_p is the shear yield capacity of the section. Pure shear links have a rotational capacity of 0.08 radians while a flexural link has a plastic rotation capacity of 0.02 radians.

4.1. Design of Link Beams

Studies of deformation capacities of shear and flexural links led to the decision to design for shear links to maximize the tower deformation capacity. An example displacement capacity vs. link length plot is shown in Figure 4. The plot illustrates that for short link lengths, the maximum rotation capacity of 0.08 radians leads to greater displacement capacity than a much longer flexural link having the same cross section with maximum rotation capacity of 0.02 radians. It is not desirable to be in the transition range between a shear and a flexural link. Optimal performance was found to be with a link length of $1.6M_p/V_p$. Optimization of the links led to the selection of the effective link length of 1.4 m, as indicated by the vertical line in Figure 6.

Link beams were designed as built-up members constructed of ASTM A572 Gr. 50 steel. Welding of the link beams was done as shown in Figure 7, consistent with the recommendations of Berman and Bruneau (2006(b)). Complete joint penetration welds through the flange uses a greater amount of weld than would welding the webs to the flanges; however, there is a benefit gained by eliminating potential lamellar tearing of the thick steel flange plates, and the resulting continuity of the web steel is less likely to introduce a weak zone at the connections between the webs and the flanges.

AISC 341-05 Section 15 contains provisions for the addition of stiffener plates to stabilize the top flange of the yielded cross-section. Plate thicknesses were selected to avoid having to add stiffener plates.

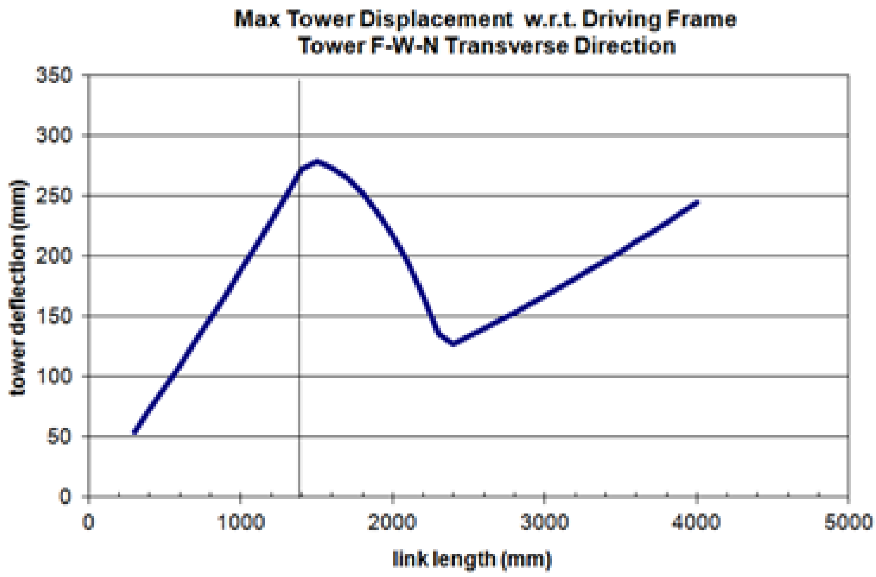

Figure 6 - Maximum tower deflection: shear vs. flexural links.

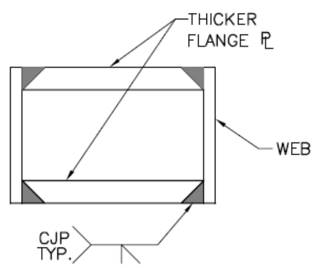

Figure 7 - Welding of link beam built-up section.

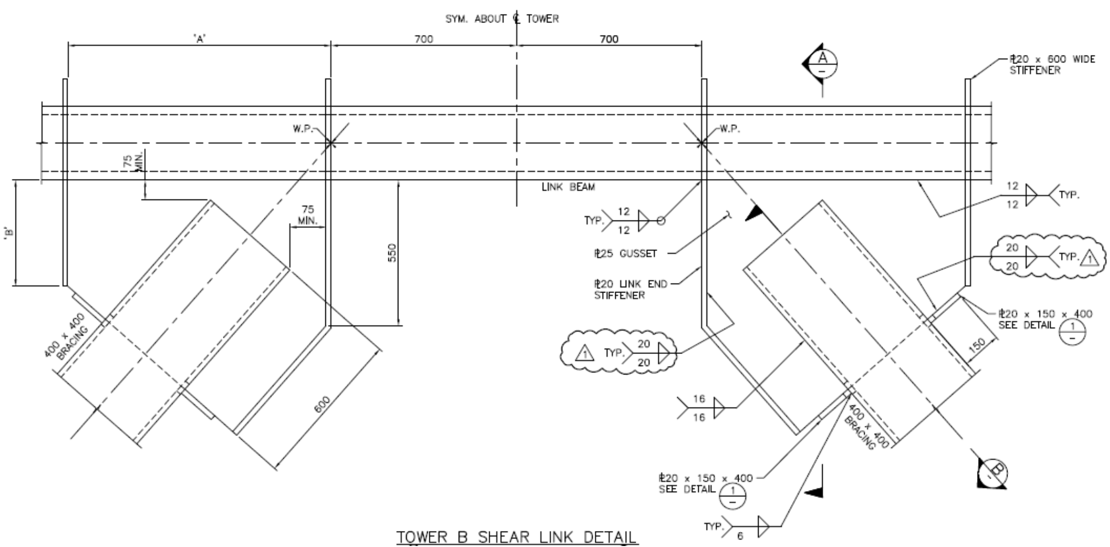

Figure 8 - Typical shear link and connection detail.

4.2. Link Beam End Connections

The connection detail at the ends of link beams was adapted from the research by Berman and Bruneau. Figure 8 shows a typical link connection. The link-end stiffener plates were designed to distribute shear forces equally to the webs and ensure the top and bottom flanges contribute equally to the shear deformation. By utilising the above detail, similar to the one considered by Berman and Bruneau in their tests, the shear link behaviour was expected to correlate with their experimental results.

4.3. Strain Hardening

One of the key aspects of the design of the shear links is the characterization of the strain hardening. AISC 341 provides a strain hardening factor to establish the maximum link shear. The remainder of the structures are designed to accommodate a force that is greater than the strain-hardened link.

Design for the minimum capacity of the shear links was based on nominal material strengths for Gr. 50 (345 MPa) steel. Material test certificates for the supplied material were provided by the fabricator showing the yield and ultimate strength of the material. To avoid excessive forces on the rest of the structure resulting from link beams yielding, the following additional provisions were added for the link beam steel:

- material testing certificates were required to be approved by the engineer prior to fabrication of the link beams;
- maximum tensile strength of the steel was set at 540 MPa (i.e., steel was rejected if testing certificates showed that its ultimate strength was higher than 540 MPa);
- sulfur content of the steel was limited to a maximum of 0.01%.

4.4. Tower Configuration

The TEBF towers were designed to be stiff in the transverse direction and flexible in the longitudinal direction of the bridge. The longitudinal forces during construction were designed to be carried through the collars to the permanent works concrete piers. The TEBF towers were designed to carry the transverse loads during construction.

The TEBF towers were designed with slanted legs, which provided a wider base to reduce the column tension and compression forces due to the force couple caused by the lateral shears at the 10 m wide tower tops. The slanted legs introduced a complication in the design as the loads introduced into the tower legs varied along the height leading to a variation in the plastic rotations achieved in each of the links.

5. Modelling of Links Using ADINA

Frame displacement capacities were initially estimated using the provisions in AICS 341-05 Section 15. However, the code formulae were developed base on simplified frame behaviour that assumes that all links rotate equally. The use of slanted columns leads to a non-uniform distribution of link rotations along the height of the frame. In order to evaluate the frame force-deflection relationships and to find the maximum deflection capacities, a nonlinear finite element analysis was conducted using the program ADINA. The program is capable of incorporating the behavior of non-linear materials to independently evaluate of the behaviour of the towers.

Tower frames were modeled using beam-column elements. The shear links were modelled using shell elements with non-linear material properties, as shown in Figure 9. The stress-strain curve assigned to the steel is shown in Figure 10 based on nominal values of yield and tensile strength. Load was applied to the top of the tower where the bulk of the effective mass of the permanent bridge deck was located. The structure was then pushed to failure, where failure was defined as the first exceedance of a rotation of 0.08 radians in any of the shear links.

Figure 9 shows a contour plot of the typical link stresses obtained from the non-linear model. Blue and green bands represent low stresses while red and pink bands indicate yielding. The webs are entirely red indicating that they reach plastic yield uniformly along the length of the shear link. Axial forces in the flange plates build from zero to the link maximum at the ends of the link, but remain essentially below yield up to the ultimate link rotation. The red bands at each end of the link beam flange results primarily from local bending of the flange plates.

Typical results are shown for short and tall towers in Figures 11 and 12, respectively. The transverse displacement demands on Tower A due to wind was computed to be 25mm. For the design earthquake, this displacement demand was found to be 60 mm. By the design criteria, the structure needed to have a displacement ductility of 2. After designing the shear link and modelling the tower in ADINA, the frame yield displacement was found to be 48mm, and the ultimate displacement capacity determined when the uppermost link beam rotation reached 0.08 radians was found to be 183mm, with a resulting ductility of 3.8. At a deflection of 96mm , corresponding to the required ductility of 2, the base shear in the frame was found to be approximately 1700 kN (382 kips) and the tower and foundation were designed to carry this force.

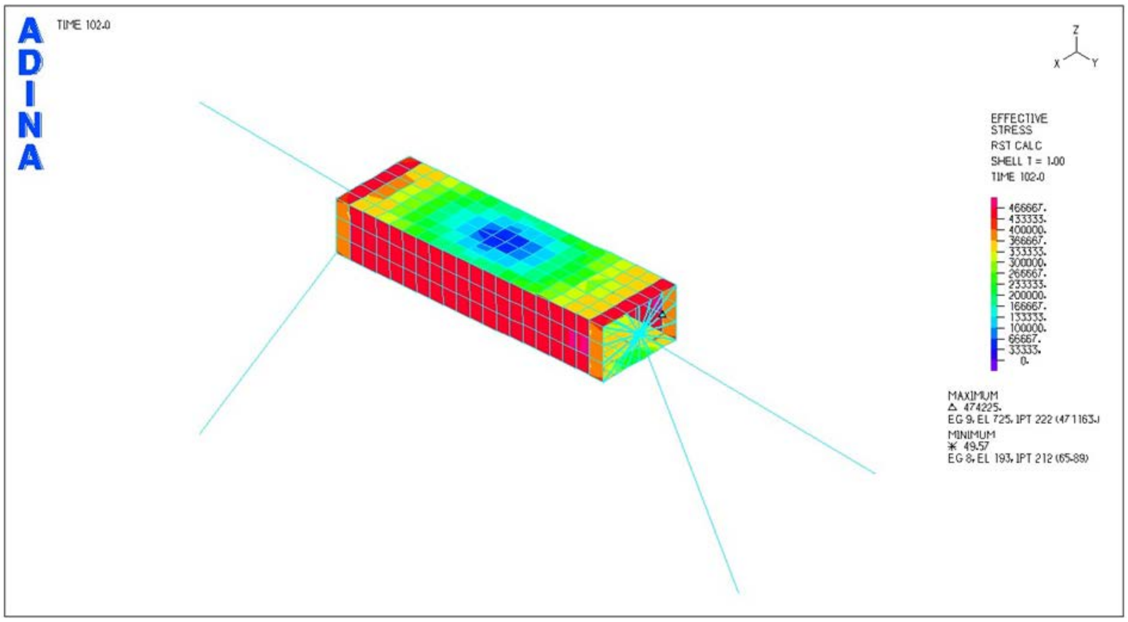

Figure 9 - Shell model of shear link in its stressed state.

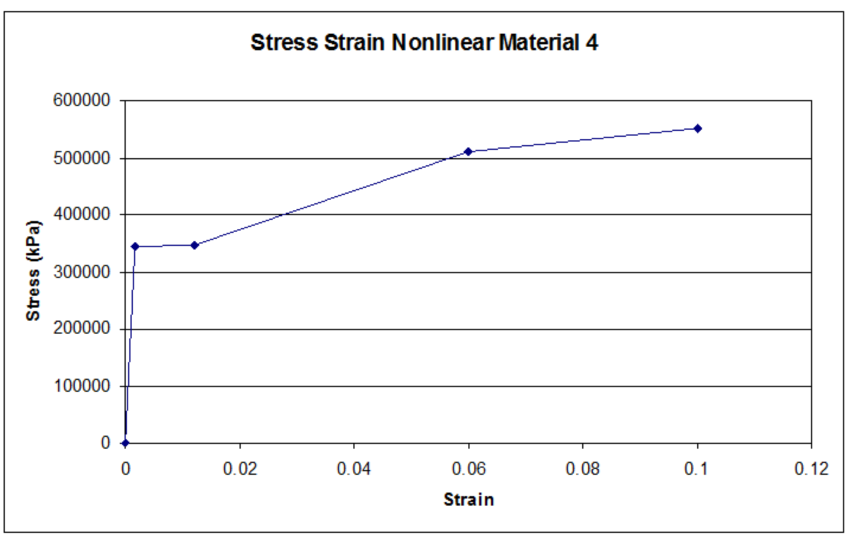

Figure 10 - Nonlinear material model used for shear link material.

For Tower C, the displacement under factored wind load was found to be 130mm while the earthquake displacement was found to be 100mm. The link beams were designed to remain elastic under factored wind load. The TEBF links were designed for a yield displacement of 133mm. The ultimate displacement capacity of the frame was determined to be 335mm, which exceeded the 300 mm corresponding to 3 times the seismic displacement and 200 mm corresponding to a ductility of 1.5. In the case of Tower C, it was the second link from the top that controlled the frame capacity. The base shear of the frame corresponding to 3 times seismic displacement was 3,800kN (855 kips). The tower and the foundation were designed to carry this load.

Unlike Tower A, Tower C was found to be elastic under the earthquake displacements computed using the 5% damped spectrum. Therefore, the damping in Tower C is likely to be lower than 5%. With lower damping, it is expected that the displacements induced in Tower C by the design earthquake would be larger than that computed using the 5% damped spectrum. Using 3 x seismic displacement instead of 2 x provides additional capacity to accommodate potential larger displacements thus satisfying the intent of the Owner.

Both Towers A and C show a variation in the rotation demand along the height of the tower with the upper links experiencing the greatest demands. The variation, influenced by the slant of the tower legs reduces the efficiency of the system as the upper links reach their rotation limit before the lower links. This was a deliberate trade-off in the design.

6. Conclusions

TEBF frames were chosen as the seismic load resisting system for the temporary works used to erect and assemble the San Francisco Oakland Bay Bridge self-anchored suspension bridge. A modified design criteria was introduced to provide a rational approach to the design of towers considering both wind and earthquake lateral loading. The TEBF's were designed according to the rules in AISC 341 and detailed per recommendation by Berman and Bruneau. Pushover analyses undertaken using ADINA with non-linear materials were used to verify that the performance of the TEBF satisfied the design criteria. Slanted tower legs were used in the design resulting in varying demands in the shear links along the heights of the towers. This limited the displacement capacity of the towers, but was an acceptable trade-off in the design.

Quality control of the link beams utilized an upper limit on the ultimate strength, and enforcement of the minimum elongation requirements of the steel.

The TEBF proved to provide significant ductility and deformation capacity reserve when compared to expected demands. The design provided a high level of confidence in the protection of the structures from seismic loading during construction.

7. References

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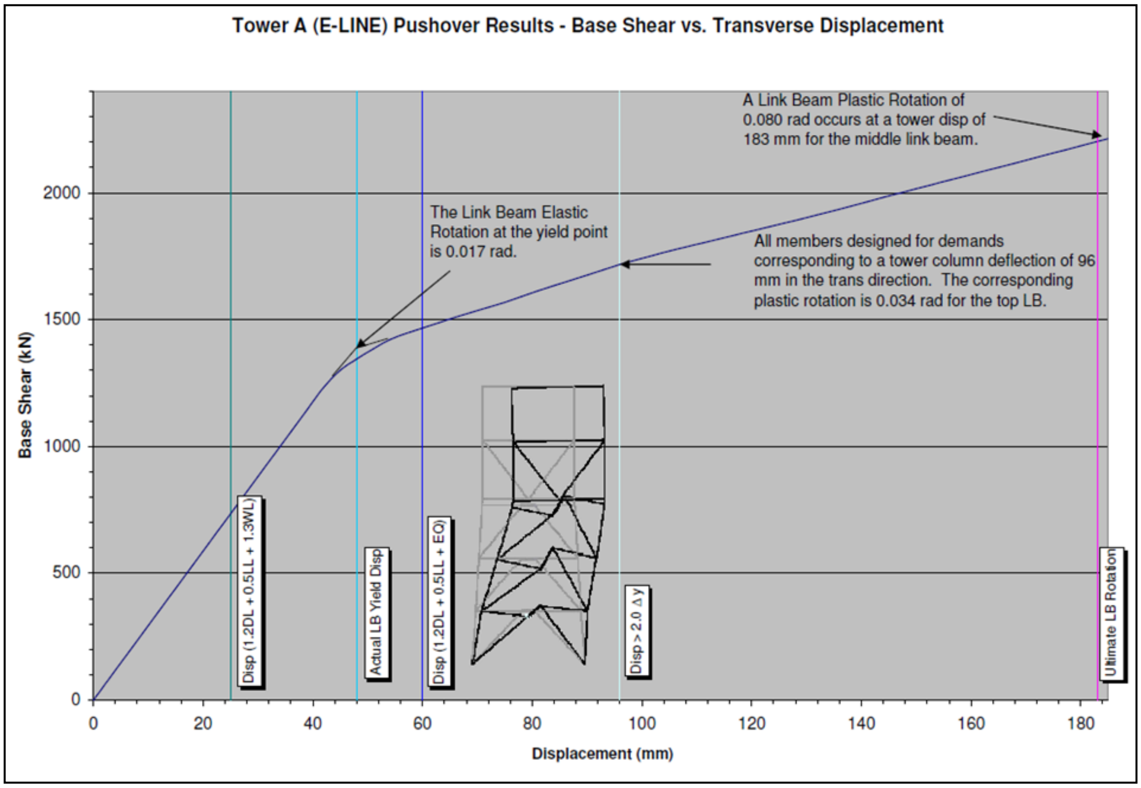

Figure 11 - Pushover analysis results: Tower A.

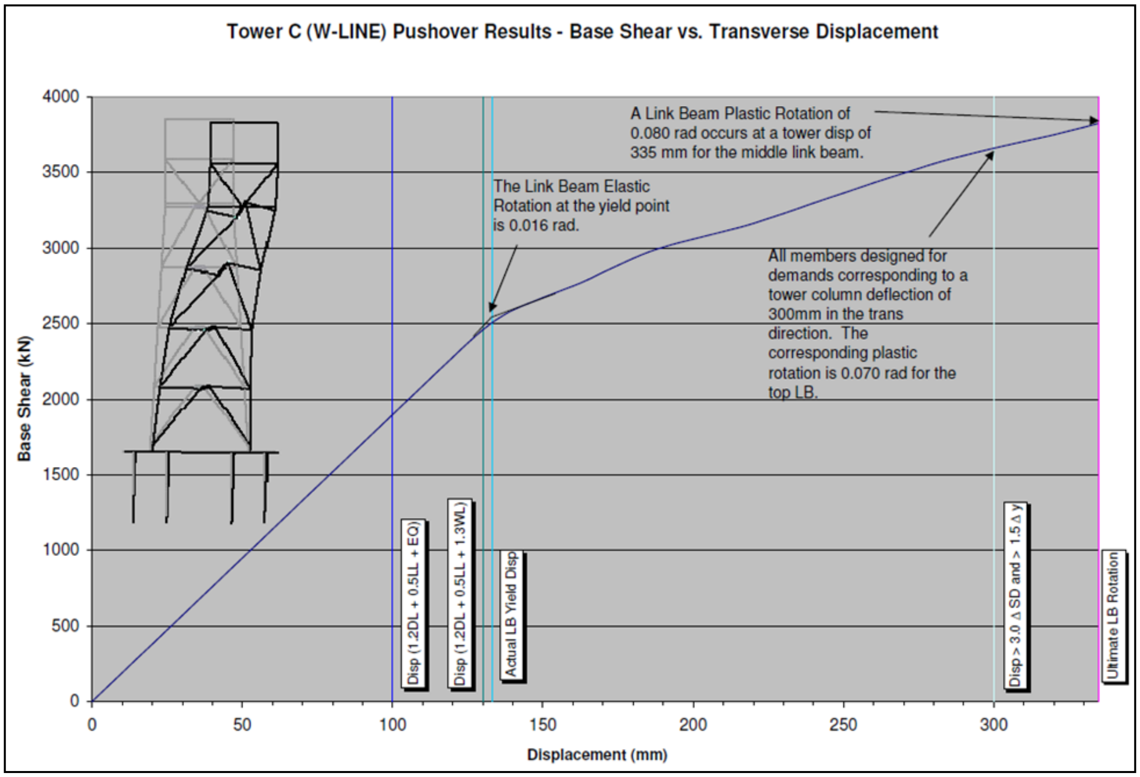

Figure 12 - Pushover analysis results: Tower C.