Seismic Analysis and Design of the New Tacoma Narrows Suspension Bridge

SERAFIM ARZOUMANIDIS, Ph.D., P.E., AYMAN A. SHAMA, Ph.D., P.E., Parsons Corporation, New York, NY; SHAWN MARLOW, P.E., Parsons Corporation, San Francisco, CA; AND FARHANG OSTADAN, Ph.D., P.E., Bechtel Group of Companies, San Francisco, CA

KEYWORDS: Suspension Bridge, Seismic Design, Concrete Towers, Non-linear analysis, Multi-support excitation.

ABSTRACT

The new Tacoma Narrows Suspension Bridge is a 5400 feet structure including a 2800 feet main span. The bridge consists of two concrete towers, supported on deep rectangular caissons, and a steel truss superstructure with an orthotropic deck. The seismically active location of the bridge mandated an extensive investigation of its performance under earthquake loads. The paper presents key elements of the seismic analysis including the multi support excitation, soil-structure interaction, and the inelastic behavior of towers.

INTRODUCTION

The existing Tacoma Narrows Bridge in the State of Washington is a well-known structure because of the disastrous collapse of its predecessor due to wind instability. A new parallel suspension bridge is under construction that will provide relief to traffic congestion. It is located 115 feet south of the existing bridge across the Puget Sound at the Tacoma Narrows. The new bridge is only the second major suspension bridge to be built with reinforced concrete towers in the US. It will provide the solution to traffic congestions on the State Route (SR) 16 corridor between the Nalley Valley viaduct in Tacoma and the Olympic Drive interchange in Gig Harbor by reducing delays across the Tacoma Narrows.

The bridge is located in a potentially active seismic region that may produce significant earthquakes (magnitude 6 or more). Hence, WSDOT established performance-based seismic design criteria for the bridge to ensure its structural integrity when exposed to ground motions. Design criteria were developed for a high level Safety Evaluation Earthquake (SEE) and a lower level Functional Evaluation Earthquake (FEE).

The joint venture of Parsons/HNTB has provided engineering design services for the project. During the design advancement of the new bridge, two engineering teams were formed, one dedicated to the design advancement and the other dedicated to independent checking– a requirement of the project. This paper presents the work of the engineering design team with regard to seismic performance of the completed bridge design.

BRIDGE DESCRIPTION

The new Tacoma Narrows Bridge, Figure 1, is a 5400-feet long suspension bridge with a main span of 2800 feet and side spans of 1200 feet on the Tacoma side and 1400 feet on the Gig Harbor side.

The superstructure, Figure 2, is a 23.5 feet deep steel truss with a panel length of 20 feet and an orthotropic deck integral with the top chord. The deck serves as a top lateral system as well. The lower lateral system has an x-shaped configuration and its members consist of box-shaped and I-beams. The top and bottom chords are welded box-shaped members and the diagonals and verticals are I-shaped members. Cross frame diagonals fabricated from 8″ and 12″ diameter structural pipes are provided for cross section torsional rigidity.

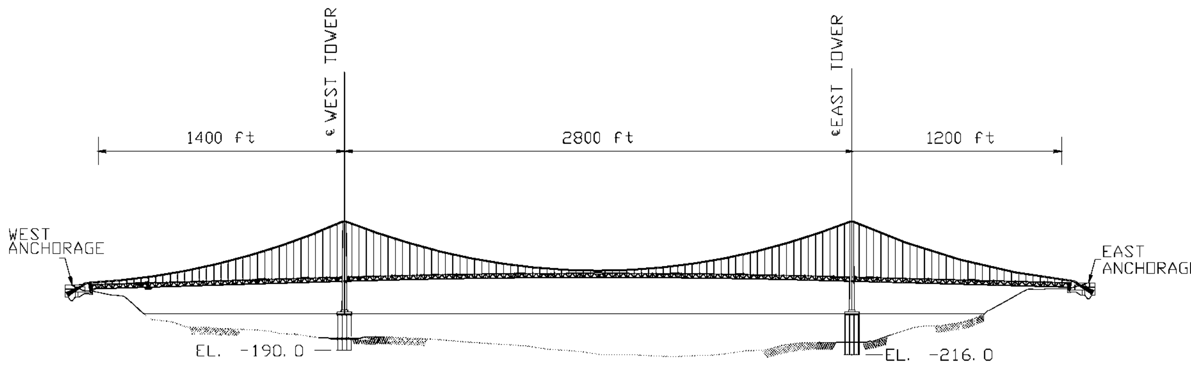


FIGURE 1: Tacoma Narrows Bridge Elevation

The main cables, 20½″ in diameter, consist of 19 strands of 464 wires each for a total of 8,816 wires. Each wire is 0.196 inches in diameter. One hundred thirty two 1 5/8″ and 1 7/8″ diameter structural wire rope suspenders support the structure from the main cables. The main cables are connected with the stiffening truss at the center of the main span by a center-tie, which provides a means of resisting longitudinal forces acting on the truss. The center tie transfers longitudinal loads with the help of two diagonal struts.

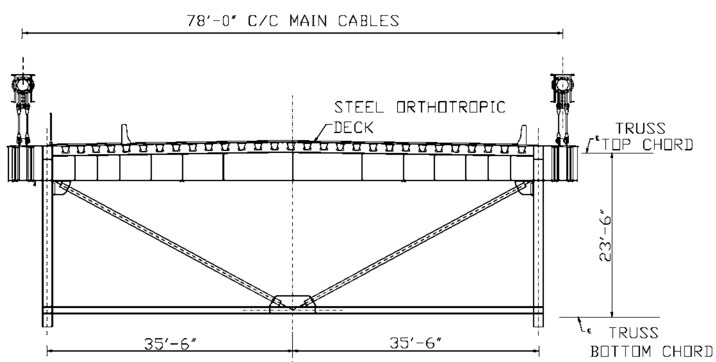


Figure 2: Cross Section of the Bridge

Eight rocker links, two at each tower and two at each anchorage, provide vertical support to the truss superstructure at the respective locations. Lateral support of the stiffening truss at the towers is provided with two self-lubricating sliding bearings at either side of the truss (north, south) and the respective tower legs. On each side, one of the bearings is at the top chord level and the other at the bottom chord level. The bearings consist of a bronze substrate that comes in contact with a stainless-steel plate under compression.

Two 505 feet tall concrete towers, Figure 3, support the main cables and also resist lateral forces from wind and seismic loads.

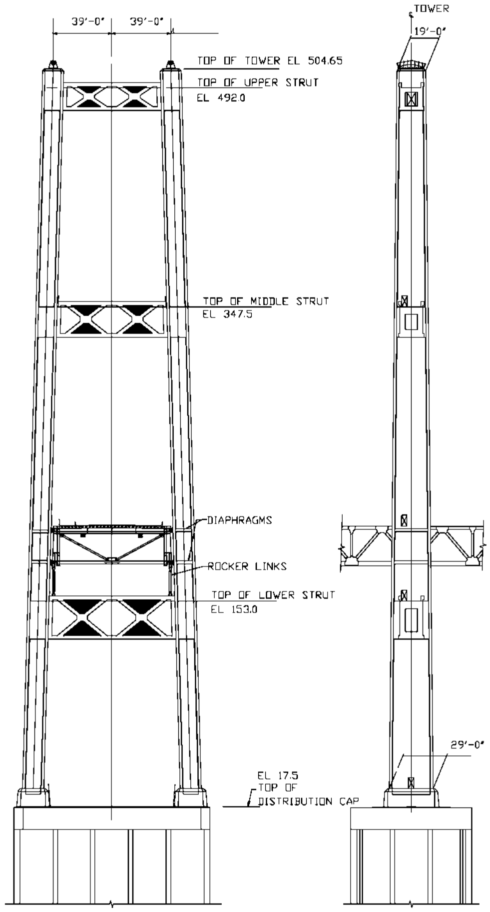


Figure 3: Tower Elevation and Side View

As shown in the side view (longitudinal direction of the bridge), the towers consist of constant 14'-0" wide shafts with depths varying from 29'-0" at the base to 19'-0" at the tower top. Three horizontal prestressed struts with depths of 25, 20 and 15 feet (Lower, Middle and Upper Struts) connect the two tower legs at elevations 153.0, 347.5, and 492.0 feet respectively. The thickness of the tower walls is 2 feet, except for the 14'-0" wide sides of the shafts from the base of the towers to the level of the top chord of the truss, which are 4 feet thick. Other than an opening for access inside the tower shafts, the connections of the struts with the shafts are of solid concrete.

The towers are supported on deep-water reinforced concrete caissons 130'-0" x 80'-0" in cross section. The west tower caisson is founded at elevation -190.0 feet and the east caisson at elevation -216.0 feet. The caissons include 15 dredge wells with 4 feet thick exterior walls, 3 feet interior walls and corner fillets. The dredge wells provide access for excavating soil from the bottom of the Tacoma sound during the caisson sinking operation. The bottom 78 feet of the caissons are enclosed in steel shells with the lower 18'-0" forming the cutting edge that cuts into the soil during the caisson sinking operation. Three typical cross sections are used along the height of the caisson: Exterior square corners with no interior fillets to accommodate false bottoms above the cutting edge; exterior square corners with 3'-8" interior fillets below mud line; and exterior chamfered corners with interior 3'-8" fillets above the mud line. A photo of the west tower caisson during construction is depicted in Figure 4.

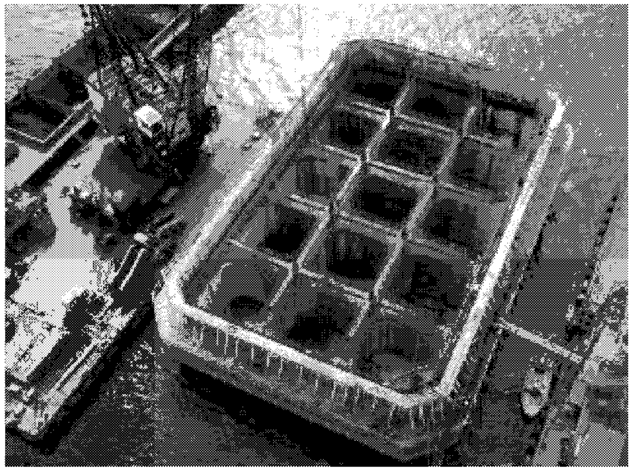


Figure 4: West Tower Caisson under Construction

The gravity-type main cable anchorages shown under construction in Figure 5 have plan dimensions 116'-0" x 151'-0". The anchorages serve not only as gravity structures to anchor the cables but also as abutments to support the ends of the truss superstructure.

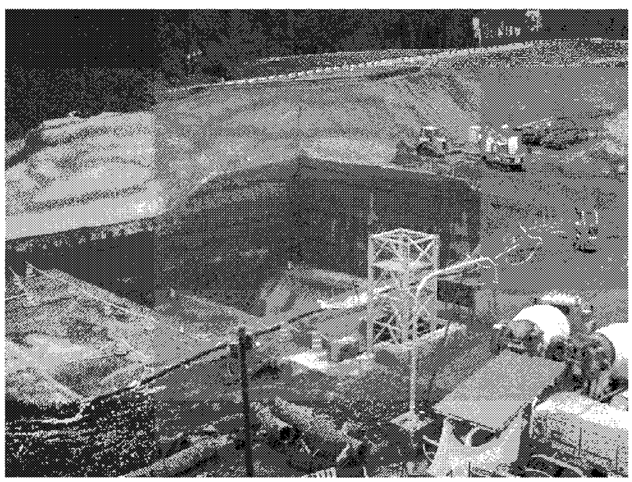


Figure 5: West Anchorage under Construction

The bridge is designed to accommodate a future lower level for highway or light rail transit traffic. Major bridge components such as the caissons, towers, anchorages, and trusses were designed for the current upper level configuration (ULC) as well as the additional loads imposed on the structure from the future lower level configuration (FLLC). Conceptual details were developed to accommodate the FLLC. This includes the incorporation of a secondary main cable along with its suspenders that would carry the additional dead and live loads. Other provisions include modification of the bridge cross section by removing the cross frame diagonals and incorporating new lower level floorbeams and orthotropic deck.

PERFORMANCE-BASED DESIGN CRITERIA

WSDOT [1] established performance-based design criteria for the bridge components to ensure the structural integrity of the bridge under seismic loads. According to the criteria, four states of damage were defined for the bridge: no damage, minimal (slight) damage, repairable (moderate) damage, and significant (extensive). *"No Damage"* is defined in accordance with the nominal capacity (full serviceability) of structural members as described in AASTO LRFD [2]. *"Minimal Damage"* is defined as minor inelastic response with post-earthquake damage limited to narrow cracking in concrete and inconsequential yielding of secondary steel members. Permanent offsets of the foundations may occur if the strain limits are not exceeded and they do not prevent immediate use of the bridge. *"Repairable Damage"* is defined as inelastic response resulting in concrete cracking, reinforcement yielding, minor spalling of cover concrete, and minor yielding of structural steel. The extent of damage must be limited so that the structure can be restored to its pre-earthquake condition without replacement of reinforcement or structural components, except secondary stiffening truss elements. Repair or replacement must not require closure. *"Significant Damage"* is defined as concrete cracking and major spalling, reinforcement yielding, and deformations in minor bridge components that may require closure to repair. Partial or complete replacement of secondary elements may be required. Permanent offsets may occur in elements other than the foundations.

The design criteria required that the bridge be analyzed under two levels of ground motions. The lower level Functional Evaluation Earthquake (FEE) corresponds to an event with a mean return period of 100 years. The overall bridge performance objective for this earthquake is "No Damage". The upper level Safety Evaluation Earthquake (SEE) corresponds to a mean return period of 2,500 years. The overall bridge performance objective is determined by the performance of different bridge components as shown in Table 1.

Table 1: Bridge Components Performance Levels for the SEE Earthquake

Bridge Component	Damage States
Tower Caissons	Minimal Damage
Anchorage Blocks	No Damage
Towers above Caissons	Repairable Damage
Stiffness Truss (except Sec. Elements)	No Damage
Secondary Stiffening Truss Elements	Repairable Damage
Bearings	Repairable Damage
Expansion Joints	Significant Damage
Cable System Structural Elements	No Damage

Further, for each damage state the design criteria specified limiting values for stresses, strains curvatures, residual drifts, and foundation settlement. At minimal damage, concrete strain is limited to 0.004 and reinforcing steel strain is limited to 0.015. At the repairable damage, concrete strain is limited to 75% of the ultimate strain as determined by the confined concrete model (Mander et al [3]). Reinforcing steel strains were limited for this damage state to 55% and 40% of the ultimate strain for #11 and #5 bars used for the longitudinal and confinement of the towers. The criteria also stipulated limiting values for the permanent drifts at the top of tower. These values were set for the SEE to 2 feet and 3 feet with respect to the top and bottom of the supporting caissons in the transverse direction. In the longitudinal direction, these values were set to 1ft and 2ft respectively. No permanent drifts for any member is allowed for the FEE.

GROUND MOTIONS

Site-specific seismic design ground motions, including spectra and three component time histories, were developed for the 100 and 2,500 year return periods. One time history set (two horizontal and one vertical component) was developed for the 100-year return period motion, and has been designated as ground motion Set 3; three time history sets were developed for the 2,500-year motions and have been designated as ground motion Sets 1, 2 and 4. Ground motion Sets 1 and 2 represent subduction zone earthquakes while as Set 4 represents a crustal fault earthquake. The basis of these ground motions was a probabilistic seismic hazard analysis (PSHA) in which the Cascadia Subduction Zone (CSZ) and shallow crustal sources were modeled (Shannon and Wilson [4]. The PSHA provided uniform hazard spectra for both the 2500 year SEE and the 100 year FEE.

The response of the soil column at the anchorages and caissons was evaluated by one-dimensional equivalent linear analysis. Site response deconvolution analyses were conducted to provide depth-varying ground motions along caissons depths using the developed multiple-support ground motions as input. These analyses produced multiple time histories for various elevations of the east and west caissons above the caissons base consistent with the location of elements representing the soil-structure interaction effects. The depth-varying motions constitute the seismic input in the global time history analyses.

COMPUTATIONAL MODEL

A three-dimensional model in ADINA was used for global analysis. The tower shafts were modeled with a class of beam elements that incorporate the inelastic behavior due to concrete cracking and potential yielding of the reinforcement. These elements feature non-linear bending moment-curvature (M-C) and torque-angle of twist (T-Φ) relationships. Two sets of M-C curves, one for the transverse and another for the longitudinal response of the towers are necessary. The relationships vary with the level of applied axial forces. The varying axial forces in combination with the family of moment-curvature curves define an interaction surface for each element. Rupture occurs when the accumulated plastic curvature reaches the ultimate curvature. The M-C and T-Φ curves are calculated with XTRACT, Imbsen [5], a program that models the behavior of confined concrete using Mander's universal stress-strain relationship for confined concrete. This type of elements allows the calculation of the varying stiffness of the tower shafts due to concrete cracking and formation of plastic zones that gradually expand as the applied loads increase.

The caisson model, Figure 6, consists of elastic beam elements along the centerline of the caissons, rigid links, and soil spring elements with elasto-plastic material properties and gapping features. The rigid links at the caisson base have a spider-like configuration that incorporates twenty-five (25) soil spring elements connected at one end with the caisson and at the other end with a rigid boundary surface, which is excited by the ground motions. The soil spring represent the interaction between soils and caissons. Similarly, two traction soil spring elements, one for each horizontal direction, simulate the friction behavior between the caisson base and the underlying soils. The interaction between the caisson sidewalls and surrounding soils was modeled in a similar fashion as the base of the caissons using outrigger rigid link elements and soil-structure interaction elements similar to those at the base. Traction elements at each outrigger represent friction in the tangential and vertical directions. This type of modeling captures rocking and sliding of the caissons due to soil compression or settlement and/or separation between the caisson base and the underlying soils. It also captures the potential formation of gaps along the sidewalls.

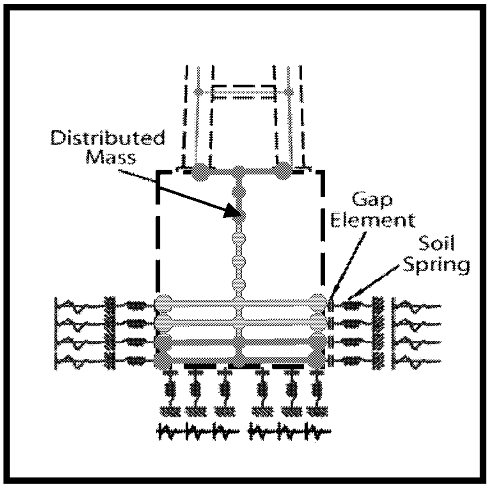


Figure 6: Caisson Representation in Global Analysis Model

The properties of the soil-interaction elements were determined by EMI [6], through benchmark pushover analysis with three-dimensional finite element models consisting of a caisson and the surrounding soils. Brick elements were used to model both caisson and soils. The soil behavior was modeled by elasto-plastic constitutive relationships with gap elements representing the potential separation between caisson and soils. Back analyses were conducted with the soils represented by lumped soil-structure interaction elements.

An average Rayleigh damping ratio of approximately 4.5% was selected for the important tower modes. According to design criteria, Rayleigh damping was not assigned to elements in the plastic hinge regions because energy dissipation characteristics of these elements were explicitly modeled through their inelastic hysteretic behavior. An average Rayleigh damping ratio of approximately 3% was selected for the dominant superstructure modes. Caisson damping is attributable to two sources, the near field effects (material damping) due to plasticity of soils in the vicinity of the caissons walls and base; and the far field effects (radiation damping) attributable to propagation of the waves away from the caissons. Radiation damping is usually small and has been conservatively neglected in this study.

The average Rayleigh us damping ratio of the caissons was taken as 4%. This value reflects the characteristics of the caisson rocking motion, which in turn is greatly influenced by the underlying soils and has a considerable effect on the overall response of the bridge due to the large mass of the caissons.

RESPONSE ANALYSIS RESULTS

The seismic demands were determined by non-linear multi-support dynamic time history analyses. The protocol for every analysis consisted of a geometrically non-linear dead load analysis, which was conducted first. The state of the structure at the end of this analysis was used as the initial condition for a successor non-linear seismic time history analysis. To signify a seismic physical scenario, the three components of ground motions were applied simultaneously at each bridge support. One time history set representing the FEE and three time history sets representing the SEE were used. The SEE ground motion Sets 1 and 2 represent subduction zone earthquakes while as Set 4 represents a shallow crustal fault earthquake. Seismic demands were calculated for the conditions of no scour and half of the maximum anticipated scour.

CAISSONS

Figure 7 shows that most of the caisson seismic shear forces are due to the inertia forces from the mass of the caissons. On the contrary, the bending moments are influenced by the oscillations of the towers, (especially in the transverse direction). Figure 8 shows envelope values of displacements. The almost straight-line shape of the plots confirms that the caissons are very rigid structures displacing as rigid bodies. The curves also show that the caisson vibrations are due to rocking in the longitudinal and transverse directions.

The performance of the underlain soils is demonstrated in Figure 9, which shows the force-displacement relationship of a soil-interaction element at the edge of the base along the centerline of the bridge. Time history of the displacements for this element and the progression of settlements with time are shown in Figure 10. The depicted permanent settlements produce a residual drift at the top of the caissons.

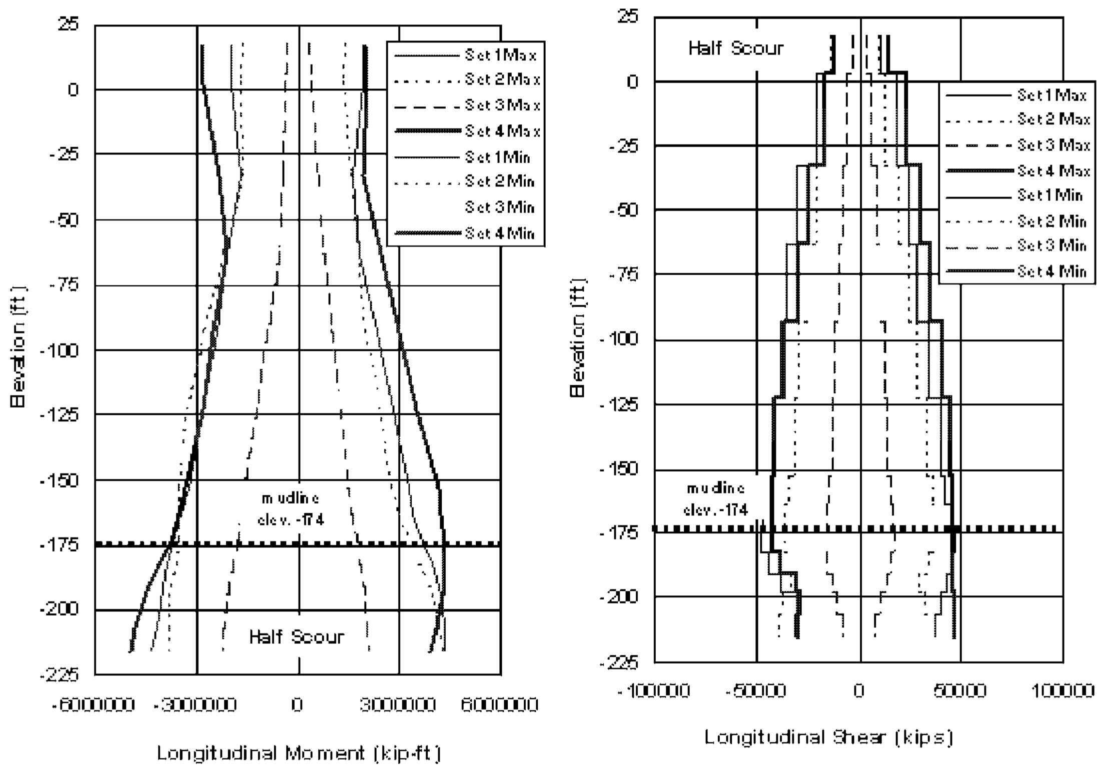
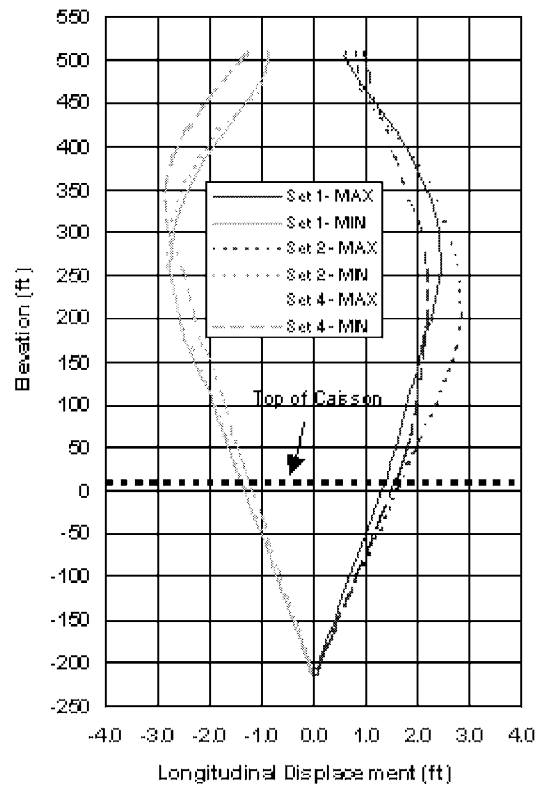


Figure 7: Envelope Values of Bending Moments (Top) and Shear Forces (Bottom) for the East Caisson



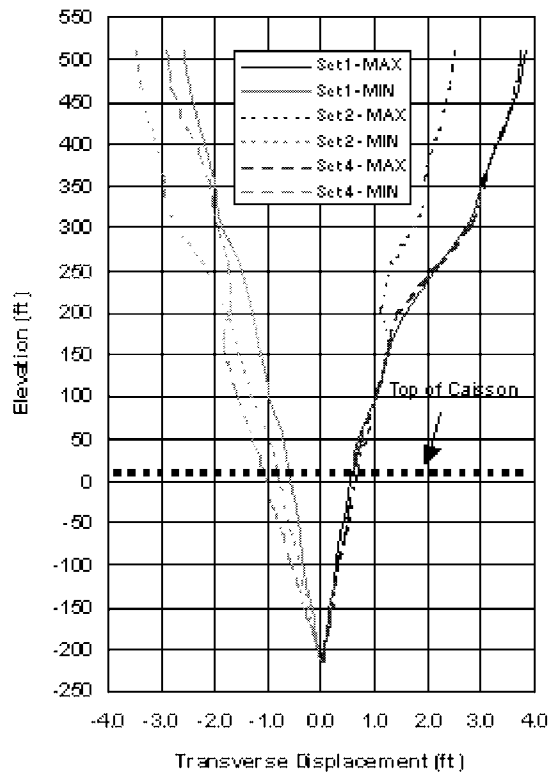


Figure 8: Envelope Values of Displacements for East Tower and Caisson

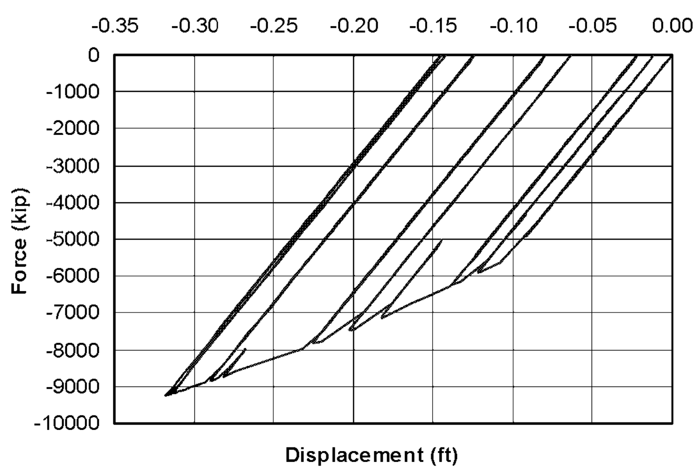


Figure 9: Force-Displacement Relationship for a Soil Spring Element at the East Edge of the West Caisson Base (Center Line of Bridge)

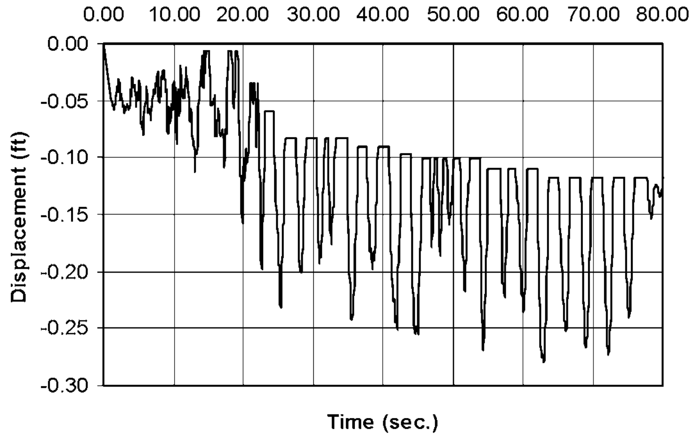


Figure 10: Time History of a Soil-Spring Element at the East Edge of the West Caisson Base (Center Line of Bridge)

TOWERS

Envelope values for the east tower displacements are shown in Figure 8. This figure shows that the maximum longitudinal displacements occur near the middle strut. It was determined that the magnitude of this displacement was not influenced by the scour conditions.

The analysis showed that plastic zones might develop adjacent to the horizontal struts of the tower shafts and the tower base during an SEE seismic event. The maximum residual drift was 1.21 feet and occurred at the top of the west tower for the ground motion SEE Set 1 in half scour conditions. The maximum D/C ratio for the plastic curvature was also observed under the same conditions and occurred below the middle strut.

Figure 11 depicts the relationship of axial force-curvature for a typical plastic zone. Envelope curves representing the yield level of the reinforcement and design limit value of the section are also illustrated. It is observed that longitudinal reinforcement may experience some yielding. Nevertheless, the section will not reach its limit design value.

The seismic design of the towers was based on the philosophy of elastic struts, and shafts that might develop inelastic behavior that could be controlled by specific residual drift and strain limits. Additionally, ductility criteria safeguarded the integrity of the concrete core. Thus, the design goal was to provide well-confined cross sections that maintain their integrity under compressive loads.

The design goal was achieved by using low strain limits for the lateral tie reinforcement and accordingly the concrete core. The tower cross sections have highly confined walls with #5 ties at 6 inches on center, and corners with #5 closed hoops and four cross ties at 6 inches on center. This corner reinforcing provides a confining pressure that is approximately equivalent to a #6 spiral at a 4-inch pitch. This reinforcement provides a minimum curvature ductility capacity ratio $\mu_c=\phi_u/\phi_y$ of 8.5, well within the project design criteria requirements. Tower cross-sections details near its base and top are shown in Figure 12.

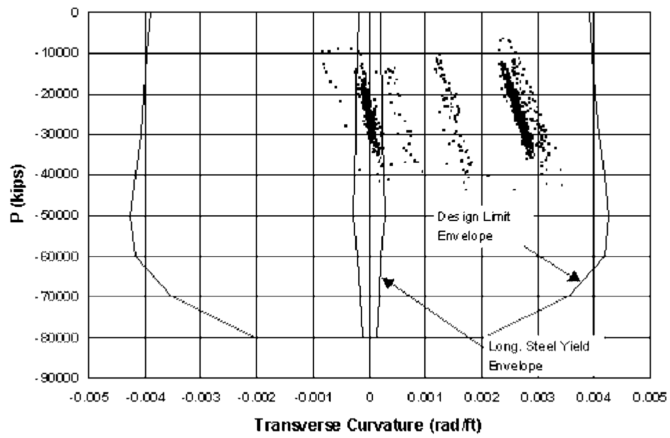


Figure 11: Axial Force-Curvature Interaction Diagram for North Leg of East Tower, Bottom Face of Middle Strut, Half Scour, Motion Set 1, ULC

The capacity of the tower shafts of the Tacoma Narrows Bridge was further checked using local models STD&A [7] and [8]. Two detailed models from elastic and inelastic concrete elements with reinforcing bar sub-elements were developed. The models represented tower shaft sections from the base of the pedestal to mid-height between the pedestal and the lower strut; and from the middle strut down to the inflection point. The modeled elements were subjected to cyclic loading as well as monotonic pushes in the transverse, longitudinal and an oblique direction assuming varying axial loads.

The detailed studies of the tower shafts response concluded as follows: (i) The plastic hinge length varies considerably depending on the axial load, direction of loading and ductility demand on the section; (ii) for all directions of application of the seismic forces, the curvature ductility ratio capacity is greater than 4, the minimum required by the design criteria; (iii) the deformation and load demands do not exceed the material (concrete and reinforcement) strains and stress limits of the design criteria; (iv) the potential damage is limited to spalling of cover concrete and cracking, which represent conditions of repairable damage. These conclusions confirmed that the tower shafts satisfy the design criteria and that the damage due to plastic hinging is repairable. The latter conclusion provided evidence that the stress and deformation design criteria lead to tower shafts that satisfy the performance objectives.

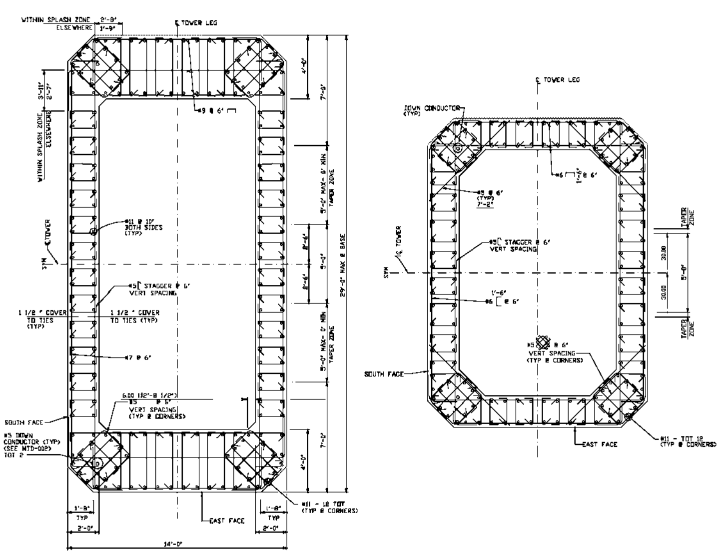


Figure 12: Cross-Sections at Tower Base and Top

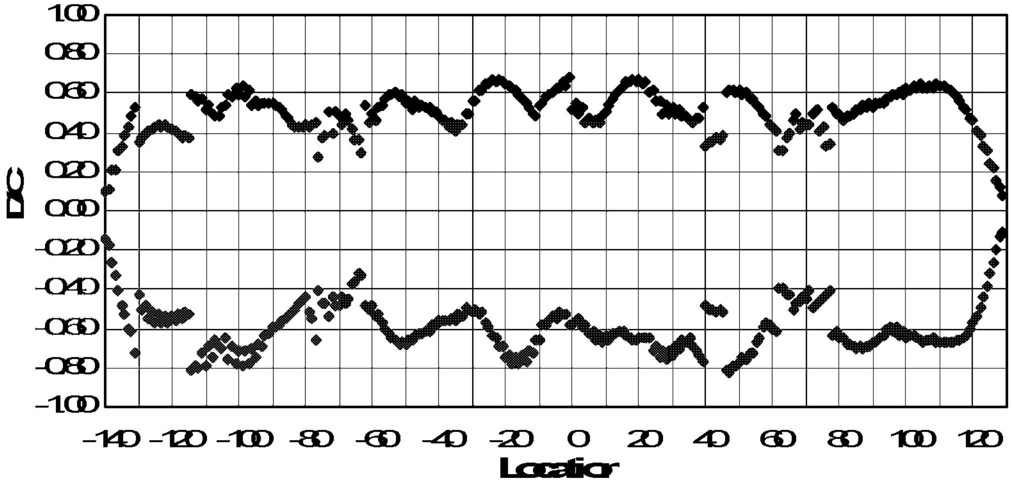


Figure 13: Axial Force D/C Envelope Ratios for the Stiffening Truss Lower chord. Top Curve is Tension; and Bottom Curve is Compression.

SUPERSTRUCTURE AND CABLES

Superstructure and cables were modeled using linear material properties. Accordingly, their performance was calculated by calculating their D/C (demand to capacity) ratio. As an example, Figure 13 depicts axial force D/C envelope ratios for the stiffening truss lower chord. All superstructure components were designed to stay elastic and, thus, this ratio remained below one.

CONCLUSIONS

This paper presented the procedure involved in the seismic analysis and design of the New Tacoma Narrows Suspension Bridge. Performance objectives of the bridge were first established and the bridge design criteria were defined accordingly. Next, the seismic analysis and design of the bridge was performed by means of a three-dimensional finite element model, which included rocking behavior of caissons and inelastic behavior of the reinforced concrete towers. Based on the study presented, the following conclusions are drawn:

- The seismic analysis and design of large structures such as the Tacoma Narrows Bridge requires detailed and complex procedures that are consistent with the specified limit values of the design criteria.
- Performance based design methodology can be employed successfully for a design-build project, such as the Tacoma Narrows Bridge, in a manner that satisfies both the owner and the contractor.
- Rocking of deep caissons was modeled in a manner that included soil-structure interaction and gapping at the base and sidewalls as well as depth-varying ground motions. This study demonstrated that such behavior should not be overlooked in future analyses of bridges with caisson foundations.
- The stress, deformation and energy dissipation criteria are significant parameters that control the design results. In our view, experience with physical structural performance and experimental data can be used to fine-tune the acceptable limit values that correspond to various performance objectives.

ACKNOWLEDGEMENTS

The Washington State Department of Transportation (WSDOT) is the owner of the Tacoma Narrows Bridge and is providing oversight during the design and construction. Tacoma Narrows Constructors (TNC), a joint venture of Bechtel and Kiewit is constructing the bridge under a design-build contract. TNC retained the joint venture of Parsons and HNTB to perform engineering services for the design and construction of the bridge. Shannon and Wilson, Earth Mechanics Inc. and STD&A provided specialized engineering services relating to seismic design. The authors are indebted to Professor Joseph Penzien for his invaluable discussions during the course of this study. He also, provided peer review of the structural analysis of the caissons, the soil-structure interaction and the overall bridge.

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