



SEISMIC PERFORMANCE OF THE NEW TACOMA NARROWS BRIDGE

Serafim ARZOUMANIDIS¹, Ayman SHAMA² and Shawn MARLOW³

SUMMARY

The new Tacoma Narrows is a suspension bridge with a main span of 2,800 feet and a total length of 5,400 feet. It consists of two concrete towers, supported on deep rectangular caissons at 216 feet below the water level, and a steel truss superstructure with an orthotropic deck. Provisions for a future lower level deck to carry road or light rail traffic have been made that will include a modified superstructure and a secondary cable system to carry the additional loads.

The three-dimensional finite element model for analysis of the bridge incorporated soil-structure interaction of the deep caissons, geometric non-linearity as well as material non-linearity of the concrete towers. The seismic analysis was performed using a multi-support excitation time-history approach of the entire structure including the caissons. The bridge was analyzed for three 2,500-year return period earthquakes (two subduction zone and one near field earthquake) and one 100-year return period earthquake. The analysis was carried out for the initial upper deck and also for the future lower level configuration

This paper presents key elements of the seismic analysis and design of the bridge including the design criteria, aspects of modeling, observations regarding the response of the bridge as well as certain design issues.

INTRODUCTION

The new Tacoma Narrows Bridge, Figure 1, is parallel to the existing bridge and is being constructed 115 feet to the south. The structure is a 5,400-foot long suspension bridge with a main span of 2,800 feet and side spans of 1,200 feet on the Tacoma side and 1,400 feet on the Gig Harbor side. The superstructure is a 23.5 feet deep steel-stiffening truss with an orthotropic deck integral with the top chord, thus serving as roadway and top lateral system. The main cables are connected with the stiffening truss at the center of the main span by a center-tie, which provides a means for resisting longitudinal forces acting on the truss. Eight rocker links, two at each tower and two at each anchorage, support the superstructure. Sliding bearings support the stiffening truss laterally at the towers and at the anchorages.

¹ Technical Director, Ph.D., P.E., Parsons, New York, NY. Email: serafim.g.Arzoumanidis@parsons.com

² Bridge Engineer, Ph.D., P.E, Parsons, New York, NY. Email: ayman.shama@parsons.com

³ Project Engineer, P.E., Parsons, San Francisco, CA. Email: shawn.marlow@parsons.com

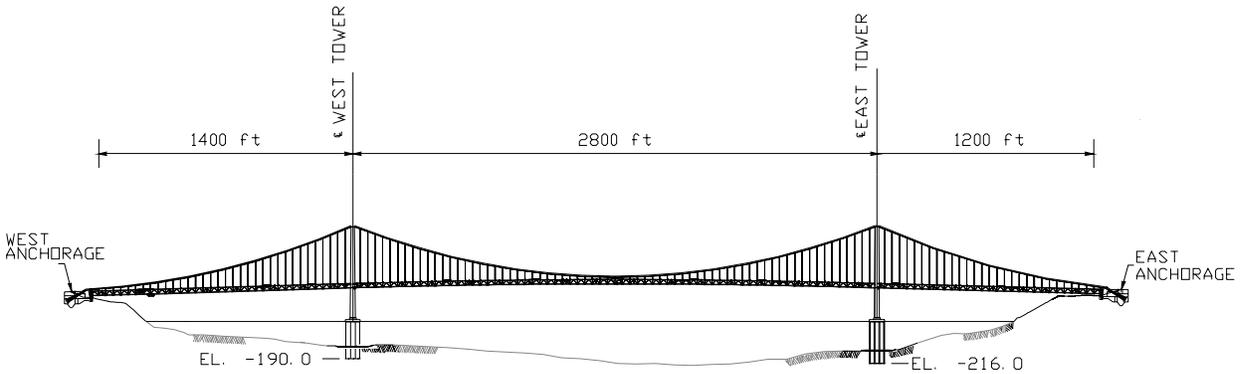


Figure 1. Elevation of Tacoma Narrows Bridge

Two 505-foot tall reinforced concrete towers consist of constant 14 feet wide shafts with depths varying from 29 feet at the base to 19 feet at the tower top. The thickness of the walls is 2 feet, except for the 14 feet wide sides from the base of the towers to the level of the top chord of the truss, which are 4 feet thick. The towers are supported on 216 feet deep reinforced concrete caissons, 130 x 80 feet in cross section. The main cable anchorages are gravity-type structures with plan dimensions 116 x 151 feet.

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.

DESIGN CRITERIA

Two levels of ground motions were used in the design. For the lower level Functional Evaluation Earthquake (FEE), corresponding to an event with a mean return period of 100 years, the target post earthquake level of service is full serviceability. The bridge must resist this earthquake with no damage or permanent offset of any structural component. For the upper level Safety Evaluation Earthquake (SEE), corresponding to a mean return period of 2,500 years, the desirable post earthquake level of service is immediate serviceability except for partial closure as required to repair potentially damaged expansion joints. A detailed description of no damage, minimal, repairable and significant damage specified the damage that may be sustained by each type of bridge component.

For each ground motion level, design criteria were established corresponding to the desired performance objectives for the two ground motion levels. These minimum acceptance criteria ascertain that the performance objectives will be accomplished. The criteria are set in terms of Demand to Capacity ratios (D/C) or absolute limit values of stresses, strains, curvatures, residual drifts and foundation settlements as stated below.

Functional Evaluation Earthquake

Seismic demand was limited to the seismic capacity of all structural components as determined by the AASHTO LRFD [1]. The strain limit for reinforced concrete members was limited to 0.004 and for the steel reinforcement was limited to 0.015. No residual drift and foundation settlement was allowed.

Safety Evaluation Earthquake

Demand was limited to the seismic capacity of all steel members except for the bottom lateral system members, where the demand could exceed the nominal capacity by 50%. For repairable reinforced and prestressed concrete members like those of the towers, the allowable concrete strain was limited to 75% of the ultimate strain for confined concrete, Mander [2]. The allowable strains of the tower shaft reinforcement were limited to 0.05 for #11 and #5 bars corresponding to 56% and 42% of their ultimate strain respectively. A minimum curvature ductility of four (4) was specified at all sections of the plastic zones.

Residual tower top transverse drifts were limited to 2 feet and 3 feet relative to the top and bottom of the supporting caisson respectively. For the longitudinal direction, the tower top drifts were limited to 1 foot and 2 feet. Residual drift at the top of the caissons was limited to 1 foot at either direction plus a 0.5 feet settlement. The cable anchorages were allowed to move 0.5 feet transversely or 1 foot longitudinally.

GLOBAL ANALYSIS 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, Figure 2. 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 [3], a program that models the behavior of confined concrete using Mander's 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. Several small length elements were used to capture the rapid change of curvature at the plastic hinge zones occurring above and below the horizontal struts.

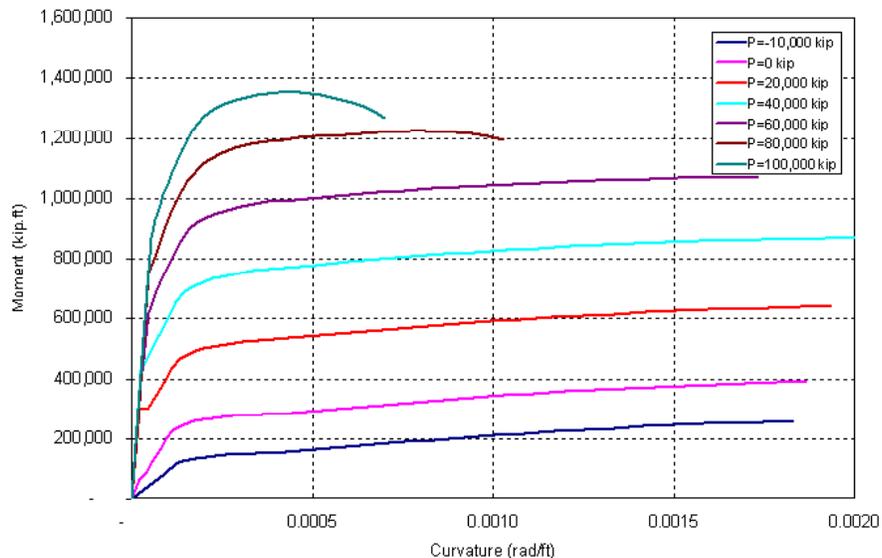


Figure 2. Moment-Curvature Relationships for various Levels of Axial Loads

The caisson model, Figure 3, consists of elastic beam elements representing the spine of the caissons, rigid links and truss 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) truss 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 truss elements represent the interaction between soils and caissons. Similarly, two traction truss 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.

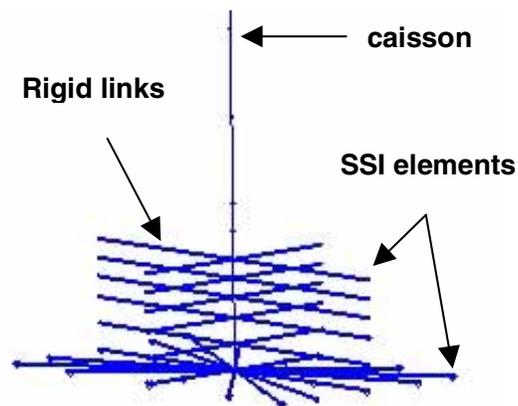


Figure 2. Caisson representation in global analysis model

The properties of the soil-interaction elements were determined by EMI [4], 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 value of approximately 4.5% was selected for the important tower modes. Damping was not used in the plastic hinge zones. The energy dissipation characteristics were explicitly modeled through the M-C relationships. An average Rayleigh damping of approximately 3% was selected for the important superstructure modes. The average Rayleigh damping of the caissons was taken as 4%. This depends on 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. The surrounding soils play a secondary role, since potential gaps forming between them and the sidewalls reduce their impact. The Rayleigh damping represents the energy dissipating during soil-structure interaction and consists of the near field effects attributable to the response of the soils near the caissons (plasticity of soils, gapping underneath the caisson base). The far field effects (radiation damping) attributable to propagation of the waves away from the caissons are small and have been conservatively neglected.

BRIDGE RESPONSE

The seismic demands were determined by non-linear multi-support dynamic time history analysis. 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, and two structural configurations (ULC and FLLC) producing sixteen sets of demand data.

Caissons

Figure 4 demonstrates that most of the 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 5 shows envelope values of instantaneous displacements. The almost straight-line shape of the curves 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 6, 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 7. The depicted permanent settlement results in a residual drift at the top of the caissons that must remain within the design criteria limits.

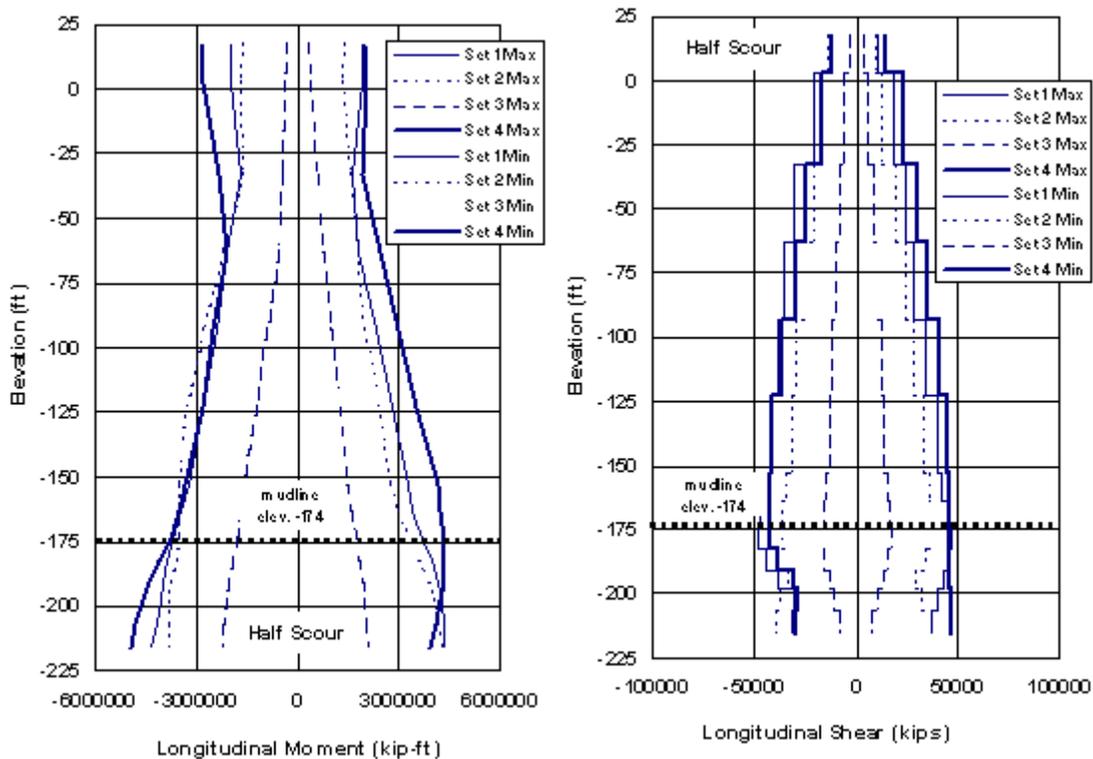


Figure 4. Envelope Values of Forces for the East Caisson

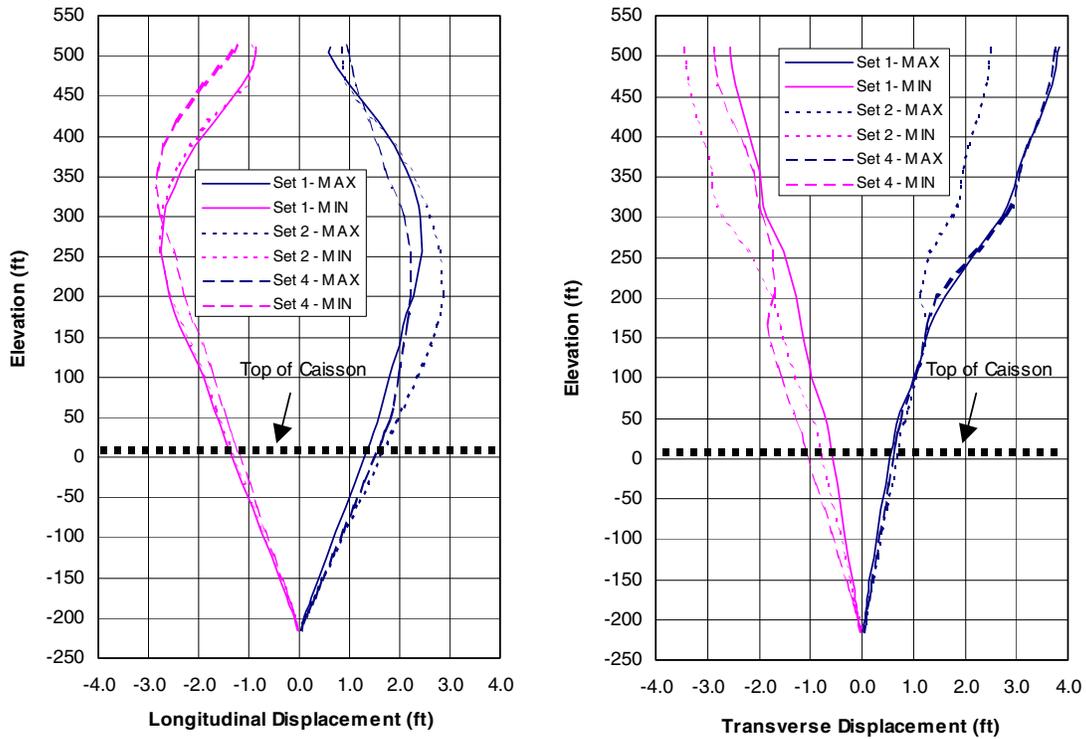


Figure 5. Envelope Values of Displacements for East Tower and Caisson

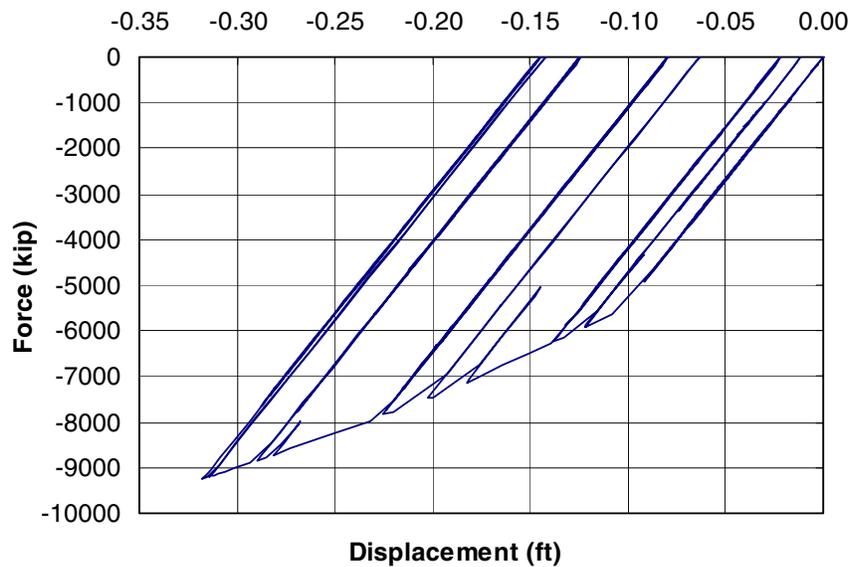


Figure 6. Force-Displacement Relationship for a Soil-Structure Interaction Element at the East Edge of the West Caisson Base (Center Line of Bridge)

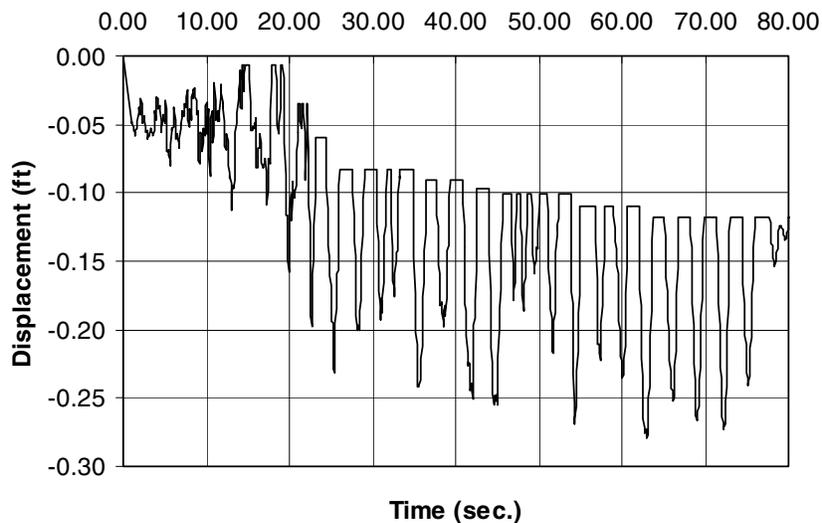


Figure 7. Time History of a Soil-Structure Interaction Element at the East Edge of the West Caisson Base (Center Line of Bridge)

The passive soil pressures on the sidewalls below the mud line and the bearing pressures on the base of the caissons were determined by a three-dimensional soil-structure interaction analysis using the program FLAC and were found to be considerably higher at the caisson base edges than the center. The passive pressures on the sidewalls varied from a maximum of 30 ksf at the top of the 15 feet thick seal slab to 5 ksf near the mud line.

Towers

Envelope values for the east tower displacements are shown in Figure 8. The maximum longitudinal displacements occur near the middle strut, and the magnitude is not influenced by the scour conditions.

The analysis showed that plastic hinge zones might develop in certain locations of the tower shafts during an SEE seismic event. The maximum residual drift was 1.21 foot 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

The seismic design of the towers was based on the philosophy of elastic struts and shafts that may develop inelastic behavior, which is controlled by specific residual drift and strain limits. Additionally, ductility criteria safeguard 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 provided a minimum curvature ductility capacity ratio $\mu_c = \phi_u / \phi_y$ of 8.5, well within the project design criteria requirements.

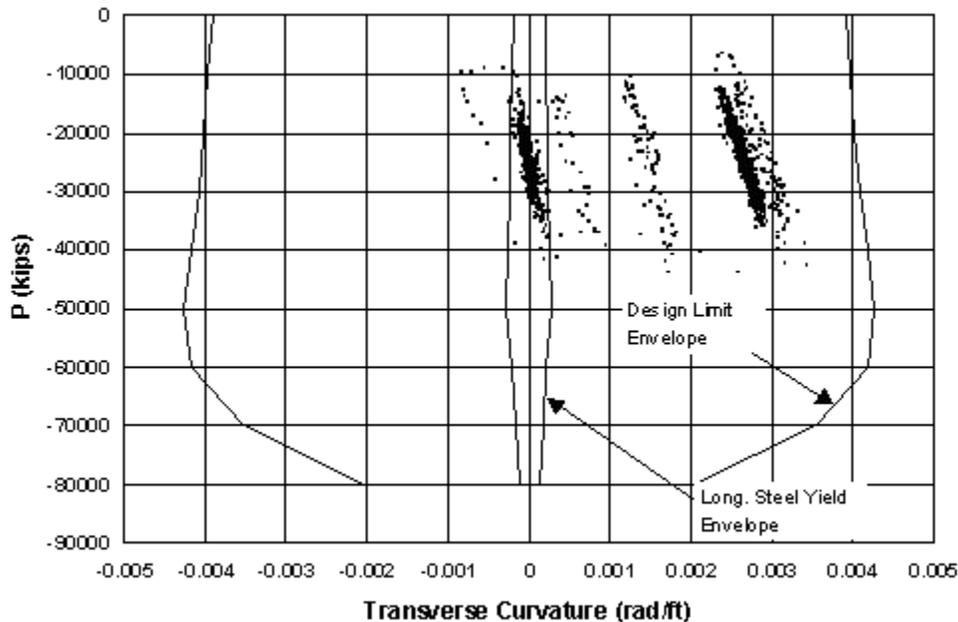


Figure 8. Axial Force-Curvature Interaction Diagram for North Leg of East Tower, Bottom Face of Middle Strut, Half Scour Condition, Motion Set 1, ULC

The capacity of the tower shafts of the Tacoma Narrows Bridge was further checked using local models STD&A [5] and [6]. 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 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.

CONCLUSIONS

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.

The stress, deformation and energy dissipation criteria are significant parameters that control the design results. In our view, greater experience with physical structural performance and experimental data are needed 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-built contract. TNC retained a 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. Professor J. Penzien provided peer review of the structural analysis of the caissons, the soil-structure interaction and the overall bridge.

REFERENCES

1. AASHTO, LRFD Bridge Design Specifications, Washington, D.C., 1998.
2. Mander, J.B., Priestley, M.J.N., and Park, R.R. "Theoretical stress-strain model for confined concrete" *Journal of Structural Engineering*, ASCE 1988; 114(8), 1804-1826.
3. Imbsen Software Systems, "XTRACT, Version 2.6.2, 2002, Cross-sectional structural analysis of components" Sacramento, CA.
4. EMI project No. 02-142, "Soil-Structure Interaction for Tacoma Narrows Bridge Caisson Response" April 7, 2003.
5. STD&A, "Detailed Analysis of Tower Base of Tacoma Narrows Bridge" April 11, 2003.
6. STD&A, "Tacoma Narrows Tower Legs – Supplemental Tower Seismic Performance Analysis" August 15, 2003.