Seismic Design of the Gerald Desmond Bridge

Matt Carter, P.E.
ARUP
Director
matt.carter@arup.com
Long-Span Bridge Leader

Josh Mattheis, P.E
ARUP
Associate Director
josh.mattheis@arup.com
Civil Engineer

Andy Dodds, G.E.
ARUP
Associate Director
andy.dodds@arup.com
Civil Engineer

Kevin Acosta, P.E.
ARUP
Associate
kevin.acosta@arup.com
Civil Engineer

ABSTRACT

Gerald Desmond Bridge Replacement is a 610-meter cable-stayed bridge with 157-meter tall mono-pole towers and a steel ladder-deck main span acting compositely with precast lightweight concrete deck panels. The cable stayed main span bridge is the longest structure of its type on the highly seismic West Coast of the United States. This paper explores three of the seismic-oriented detailed design features of the ground-breaking cable stayed bridge: the use of fused fluid viscous dampers to dissipate seismic energy, seismic interaction with adjacent approach bridge structures and the performance design basis of deep foundations in liquefiable soil are detailed to explain how performance criteria were translated through the design process into a functioning structure.

KEYWORDS: cable stayed bridge, composite deck, seismic bridge design, liquefaction, viscous dampers

1. Introduction

Late in 2012, the Port of Long Beach in collaboration with the California Department of Transportation awarded a design and construction contract to Shimmick / FCC / Impregilo JV (SFI) for the replacement of the Port of Long Beach's obsolete and deteriorating Gerald Desmond Bridge. The Project consists of the design and construction of a 610-meter main span cable-stayed bridge and approximately two miles of cast-in-place box-girder approach structures. Final design services are performed by Arup North America Ltd. in association with Biggs Cardosa Associates.

The cable-stayed main span bridge is the longest structure of its type on the highly seismic West Coast of the United States. The critical nature of the roadway network created by the bridge manifests itself through the Port of Long Beach's performance criteria, which include a 100-year design life, a 1,000-year return period seismic event for safety verification, and a 100-year seismic event for functional performance evaluation.

This article explores the development of three of the technical solutions developed and implemented by Arup to address the regions exceptional seismicity and the Port of Long Beach's visionary performance requirements:

1. The use of fused fluid viscous dampers to dissipate seismic energy;
2. Management of displacement interaction between adjacent structures during seismic events;
3. Soil-structure interaction between deep foundations and variably liquifiable soil.

This paper explores the above topics to understand how regulatory guidance, project performance requirements and engineering judgement are combined for each to deliver a state-of-the-art, best-value structure.

Figure 1. Rendering of Gerald Desmond Bridge Replacement Project

2. Regional Seismicity

The site is located in an active seismic area of Southern California. Seismic shaking is anticipated during the life of the structure. Potential seismic hazards include seismic shaking, ground surface rupture, seismic compaction, liquefaction, lateral spreading, slope instability, ground lurching and tsunamis.

2.1 Seismic Sources

Figure 2. Fault Map

Figure 2.

2.2 Historical Seismicity

The largest recorded historical earthquake within the Basin was the 1933 Long Beach earthquake, which occurred on the Newport-Inglewood Fault. This earthquake had a measured moment magnitude (Mw) of 6.4 and recorded ground acceleration (amax) of 0.22g.

Larger earthquakes outside the Basin, including the 1971 San Fernando (Mw 6.6), the 1987 Whittier Narrows (Mw 5.9), and the 1994 Northridge (Mw 6.7) earthquakes did not cause any damage at the Port of Long Beach. However, the adjacent Port of Los Angeles did sustain light damage as a result of liquefaction during the 1994 Northridge earthquake. According to Schiff [3], surface manifestations of liquefaction were observed at Berth 121-126, and included sand boils penetrating through 7 inches of asphalt, lateral displacements of the wharf ranging from 2 to 3 inches, and 6 to 8 inches of pavement settlement. This damage required repairs to crane rails, expansion joints between adjacent wharves, utility lines, pavements, and asphalt [3].

2.3 Earthquake geotechnical challenges

Earthquake geotechnical challenges at the site include proximity to major faults capable of generating large magnitude earthquakes, coupled with some soils susceptible to liquefaction or cyclic softening. The seismically susceptible soils are seated in Holocene sediments comprising near-shore, marine and non-marine strata, including beach, estuary, tidal flat, lagoon, shallow water bay sediments and shoreline terrace deposits. The result is a highly stratified ground where alternating layers or lenses of sands, silts and clays are typically present.

3. Fused viscous dampers

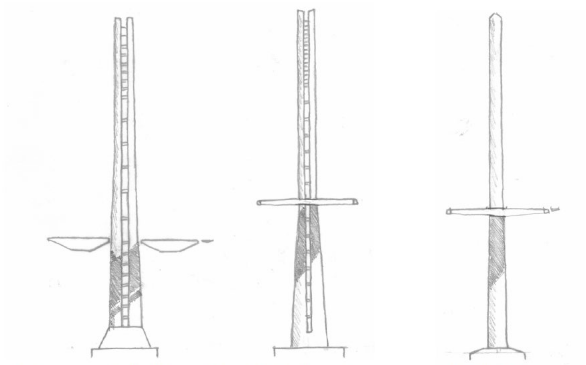


Figure 3 Evolution of Gerald Desmond Bridge Replacement Project Tower design

The request for Design-Build proposal documents included a Reference Design of a twin shaft tower (**Figure 3**) and twin shaft end bents, both with shear links. Project specifications required shear link design and testing requirements should shear links be adopted in the final design. Early in the preparation of the proposal design, these testing requirements flagged the twin shaft tower as a significant cost and schedule risk. The mono-pole tower was preferred by the team because the shaft verticality, the absence of portal beams and the absence of shear links facilitate an efficient climbing formwork construction process.

A parametric study was carried out using multi-modal response spectrum analysis which established the feasibility of adopting a Type 3 AASHTO global seismic design strategy [4]. This means that towers, end bents and superstructure remain essentially elastic, and a damping mechanism is introduced between superstructure and substructure. Viscous dampers are the damping mechanism.

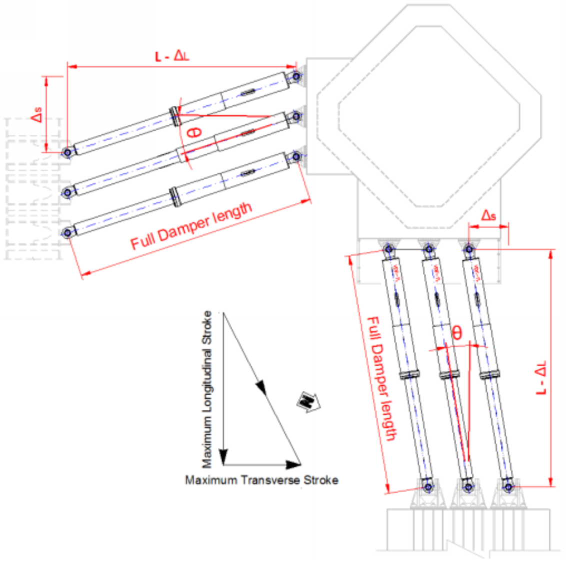
3.1 Performance criteria and selection

In sizing the viscous dampers, the design team focused on reliability, ease of maintenance, redundancy and future-proofing if replacement is required. To achieve this, multiple dampers arranged in sets are prescribed linking the superstructure to the towers and the end-bents. Each end-bent has two longitudinal and two transverse dampers. Each tower has six longitudinal dampers and three transverse dampers, for a project total of thirty-four dampers. The overall quantity makes the damper size manageable for installation, maintenance or replacement, and provides redundancy.

By contract, several specific features are included with the damper design:

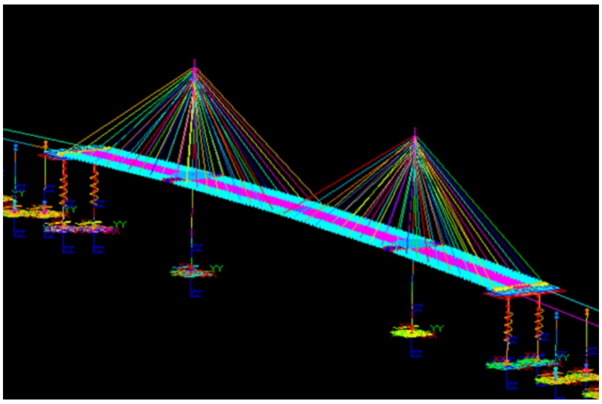
- A port for recharging the fluid inside the piston chamber;
- An additional port for bleeding air from the chamber;
- A glass window to observe the fluid level inside the chamber or an instrument to measure the volume of fluid in the piston chamber;
- Three pressure gages for measuring fluid pressure inside the piston chamber;
- A force transducer for measurement of actual as well as the functional earthquake event.

Dampers and fuses are provided by Taylor Devices, Inc. Testing of the full-scale dampers is performed in house and at the University of California, San Diego (UCSD) laboratory.



Dampers are designed for the maximum number of cycles, velocity and displacement predicted by the dynamic model, including angular distortion (**Figure 4**).

3.2 Dynamic modeling



LS-DYNA is used for the nonlinear time-history analysis of the main span structure with consideration of the two approach bridge frames at each end bent. Ground motions for use in dynamic seismic analysis of the bridge structures are taken from the Project Seismic Ground Motion Report information which documents the project-specific acceleration response spectrum (ARS) design curves and spectrum-compatible ground motion time histories for the Safety Evaluation Earthquake (SEE) and the lower level Functional Evaluation Earthquake (FEE) events.

$$F = CV^{\alpha} \qquad (1)$$

Within LS-DYNA, the viscous dampers are explicitly modeled using non-linear damper elements defined by the constitutive law presented in Equation 1. In Equation 1, the C parameter is tuned to achieve a specified 'trigger' force at V = 0.0394 in / s-1. The trigger force replicates the action of the damper fuse, so that the damper acts as a pinned rigid link until the trigger force is achieved.

3.3 Integrated fuse

Structural fuses are provided to isolate the dampers from ambient cyclic motion such as that from thermal, wind and traffic sources. The objective is to prevent movement-sensitive damper components, such as the piston seals, from experiencing premature failure due to the aggregate high number of cycles from ambient sources. Fuses take the form of structural steel tube damper encasements with a shear ring designed to withstand forces corresponding to the demand predicted from a seismic event corresponding to a one in one-hundred-year return period seismic event (FEE event).

Should the fuses break, horizontal stability of the superstructure is maintained by the cable stays and end-bent bearing friction. End bent bearing friction is augmented by end-bent deck tie-downs. Shear key stops are provided should bearing friction be overcome by concurrent extreme lateral forces, such as the extreme wind event. Fuses are designed to be replaced.

The decision to integrate the fuses directly with the dampers resulted from lengthy discussions with SFI and the owner. Criteria such as structural congestion, access, maintenance and force distribution were considered. Separate and parallel fuses located in-line with dampers brought conceptual simplicity, at the expense of increasing the number of structural connections, and the associated maintenance and installation complications. Integrating the fuse with the damper resolved many issues while ensuring that the line of action of the fuse force is perfectly aligned with the associated damper.

3.4 Integration into project

The connection of the dampers to the deck was coordinated in a way that would least adversely affect the deck structure. The ideal location for the dampers is for them to be within the plane of the deck which is effectively a flat diaphragm, however this would introduce large openings in the deck and require and even wider deck to accommodate the traffic lanes. For practical purposes, the end bent dampers were attached to the underside of the End Girder, the tower longitudinal dampers were attached to a large concrete anchor that mobilized three of the steel floor beams beneath, while the tower transverse dampers were accommodated in the plane of the below deck steelwork, through the use local deck stiffening with shallow steel beams. All these positions introduce an eccentricity between the deck and the dampers, however the resulting moments were addressed in the design of the structure and connection, including additional longitudinal framing beams between the End Girder and the last three Floor Beams at either end of the bridge.

Rotation of the dampers is allowed through spherical bearings at the ends of the dampers. For the tower longitudinal dampers, which are arranged in two rows, an additional balancing assembly was provided at the deck end linking vertically dampers to reduce load differences due to their vertical eccentricity.

The End Bent dampers are accessible through the top of the end bents and a suspended walkway structure, while the tower dampers are accessible from the deck and a suspended maintenance platform.

4. Dynamic Frame Balancing

An approach span frame on the Gerald Desmond Bridge Replacement Project typically consists of three spans and three columns. Maximum column height is 60 meters at the transition to the main span bridge, and a typical span length is approximately 70 meters. Frames are connected by seated hinges located approximately 13 meters from the final column of a frame. Typical longitudinal grade is 5%, meaning that columns may vary in height by more than six meters within the same frame.

Under the potentially high seismic loading of the site, this geometric configuration requires a design that balances stiffness between frame columns so that base shear is shared between them with reasonable evenness, and movements between adjacent frames are compatible.

4.1 Codified requirements

The SDC [5] stipulates limiting ranges for stiffness, stiffness-mass and fundamental period of vibration ratios between smaller and larger frames or columns, with the aim of reducing the probability of out-of-phase dynamic displacement between frames and unbalanced distribution of dynamic loading within the frame.

$$2 \geq \frac{k_i^e}{m_i} \Big/ \frac{k_j^e}{m_j} \geq 0.5 \qquad (2)$$

$$1.33 \geq \frac{k_i^e}{m_i} \Big/ \frac{k_j^e}{m_j} \geq 0.75 \qquad (3)$$

k_i^e = Smaller effective bent or column stiffnes
k_j^e = Larger effective bent or column stiffnes
m_j = Tributary mass of column or bent j
m_i = Tributary mass of column or bent i

For variable-width frames, limiting stiffness ratios between any two columns in a bent or any two bents in a frame are described by **Equation 2**. Limiting stiffness ratios between any two adjacent columns in a bent or any two adjacent bents in a frame are described by **Equation 3**, illustrating a more restrictive control over adjacent structural elements.

$$T_i / T_j \geq 0.7 \quad (4)$$

T_i = Natural period of less flexible frame
T_j = Natural period of more flexible frame

Equation 4 limits the difference between the natural periods of adjacent frames in either the longitudinal or the transverse orientation. The intent is to reduce the risk of out-of-phase dynamic displacements. Out-of-phase dynamic behavior of adjacent bents can result in collision of adjacent bents longitudinally at the expansion joints. The same phenomena may also result in the transfer of lateral forces from one bent to another through the joint seat shear keys, thereby reducing the anticipated frame capacity for dissipating the seismic energy of the frame's own mass.

4.2 Structural Impact

Balancing columns within frames and frames with adjacent frames in the Gerald Desmond Bridge Project, in consideration of the 5% longitudinal grade and variable deck width, resulted in some piles caps being constructed below the finished ground level. Isolation casing was specified to prevent passive earth pressure against the length of column below the finished grade level from affecting the global frame stiffness. Coordination between structural design teams as each frame progressed was essential, so that design evolutions in frame "N" did not create dynamic incompatibilities with frame "N+1". Closer to abutments, where the relative difference in column heights is greater, column dimensions were adjusted, or multiple column bents introduced to mitigate the required depth of isolation casing.

Constructing pile caps several meters below grade presented construction challenges, considering the high local water table and a dewatering moratorium due to the presence of a ground water contaminant plume. Braced cofferdams and tremie-concrete plugs were required to allow for the sum of the length of column below ground, the 3-meter-thick pile caps, and the space required to trim and prepare piles ends. Soil conditions and artesian ground water made sheet pile installation and extraction difficult.

4.3 Integration into the Project

While the design technology behind dynamic frame balancing uses well-known and vetted calculation methods, the key to employing the solution successfully through construction is understanding the implication on all construction activities. In the case of the Gerald Desmond Bridge Replacement Project, cofferdam installation and extraction activities consequential to dynamic frame response balancing became significant due to a combination of several specific site conditions. The challenge is to consider the effect of all these elements combined when planning the activity.

5. Performance-Based Design of Deep Foundations

The new bridge is supported on groups of "cast-in-drilled-hole"(CIDH) pile foundations pressure grouted at their tip to maximize efficiency.

As mentioned in section 2.3, the geotechnical classification is highly stratified. Categorizing this variable ground for design purposes was first guided by geologic considerations and site investigation data to establish idealized layers of predominantly cohesionless and cohesive soil. Evaluation of SPT and CPT site investigation data according to Youd et al. [6], Boulanger and Idriss [7], Bray and Sancio [8] and Boulanger and Idriss [9] was then used to identify layers of sand-like material susceptible to liquefaction and clay-like material susceptible to cyclic softening. **Figure 6** illustrates a generalized account of these idealized ground conditions together with pile configurations east and west of the Back Channel, noting there are no over-water foundations. The extent of liquefaction reported refers to the extreme SEE (1,000 year return period) design earthquake event.

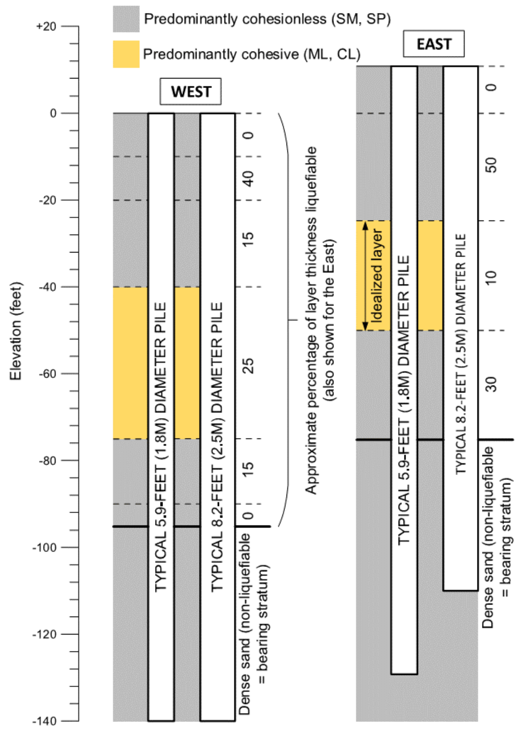


Figure 6 Generalized ground and pile configurations

The intent of **Figure 6** is twofold. Firstly, the varying percentages of liquefiable thickness apparent within each idealized soil layer serve to illustrate the highly stratified nature of the ground where liquefiable layers will be sandwiched between non-liquefiable layers and vice versa, within both predominantly cohesionless and cohesive idealized soil layers. Rationalizing such detail with reasonable accuracy was possible from CPT data that allowed subdividing the ground into 300mm (1-foot) thick sub-layers for liquefaction evaluation purposes [10].

Secondly, the generalized extent of the pile foundations shown in **Figure 6** (drawn to the same scale as the generalized ground profile) serves to illustrate the relative scale of the problem, which in turn provides a sense of how the possible loss of ground strength and stiffness due to liquefaction and cyclic softening could have an impact on pile performance. This aspect of the design was especially important given that the pile foundations are designed to accommodate potential earthquake-induced loss of ground strength and stiffness. This is certainly a performance-based design consideration requiring the designer to assess the impact on the pile foundations from both lateral and axial loading standpoints.

5.1 Lateral loading

Designing piles to withstand lateral loading is a soil-structure interaction problem and typically considered in practice using a structural "stick" model to represent the pile, and p-y curves to represent the lateral soil resistance mobilized against the pile.

Guidance on assessing the lateral resistance of piles in liquefied soil is available [1] where it is usual to employ the same framework as for non-liquefied conditions except the p-y curves are modified to reflect much lesser stiffness and ultimate resistance (strength) associated with liquefied conditions. Decreased stiffness and strength will also apply to clay-like layers that are softened during seismic shaking, but in either a liquefied or softened layer case the guidance is limited to such layers being distinct. For the highly stratified ground at GDBRP, assigning each and every liquefied layer its own liquefied p-y curve would require a ridiculous level of detail and result in the design being unwieldy.

The approach taken on GDBRP was to employ a modeling economy utilizing the idealized layers and CPT-derived liquefaction evaluation previously noted. If more than 50% of an idealized layer was liquefiable, then the entire layer was considered liquefied and liquefied p-y curves assigned throughout. If the percentage liquefiable was less than 50% then a weighted average of the non-liquefied and liquefied p-y curves (i.e. a "partially-liquefied" p-y curve) was adopted, as illustrated in **Figure 7**. In this way a workable and defendable design approach for lateral pile resistance for liquefied conditions was achieved based on a rational and reasonable adjustment that recognized the collective contribution of both liquefied and non-liquefied layers in resisting pile movement.

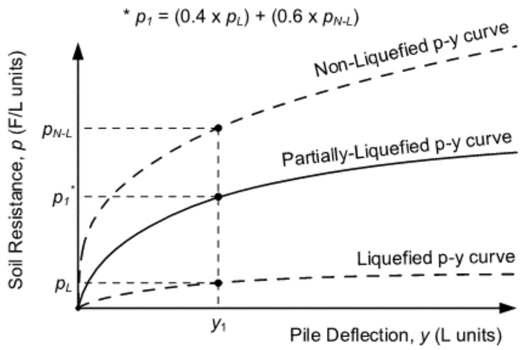


Figure 7 Partially liquefied p-y curve determination, based on 40% liquefiable and 60% non-liquefiable for idealized layer

5.2 Axial loading

Axial loading design of the piles for the GDBRP project followed AASHTO [11] as amended by Caltrans [12], in accordance with specified project design criteria. This is a load and resistance factor design (LRFD) methodology employing a force-based procedure to assess the axial resistance of the piles for the strength and extreme event (earthquake) limit states. In terms of earthquake loading, Article 10.5.4.2 of AASHTO directs the designer to evaluate the effects of potential liquefaction on the foundations and Article 10.7.4 provides more detail regarding liquefaction-induced downdrag. It is in regards to the interpretation of Article 10.7.4 that attention is required, namely consideration of the following sentence: "For seismic design, all soil within and above the liquefiable zone, if the soil is liquefiable, shall not be considered to contribute axial compressive resistance".

On GDBRP this sentence was interpreted in two different, opposing ways when assessing axial pile resistance during earthquake shaking, as illustrated in **Figure 8**. Whether or not the non-liquefiable layers located above the lowest liquefiable layer can be relied on is embroiled in the timing of liquefaction. The timing of liquefaction is a topic now receiving serious attention [13], but in any bridge design the interplay between inertial loading imposed on piles during shaking and the triggering of liquefaction and its manifestation in the form of ground settlement is by no means precisely defined. While such uncertainty invites disagreement, it is not the intent of this paper to discuss this further, but rather highlight that the force-based approach is mostly to blame for the opposing interpretations indicated in **Figure 8**.

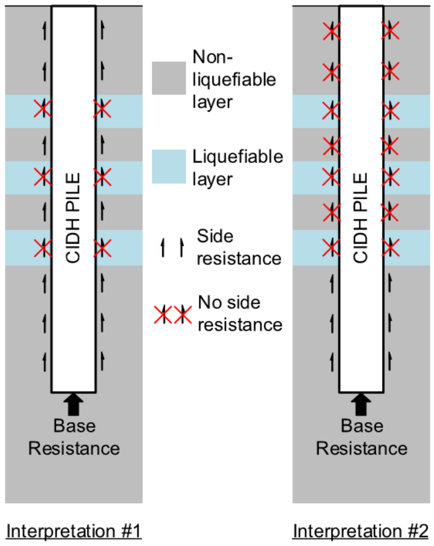


Figure 8 Different interpretations of Article 10.7.4 in AASHTO [12]

The AASHTO [12] treatment of liquefaction-induced downdrag (settlement) as a load and resistance problem is simply a code-based approach that does not reflect actual axial response of a pile during liquefaction. Indeed, Fellenius and Siegel [14] considered the AASHTO approach fundamentally flawed and advised to treat the problem as a settlement issue instead**. Figure 8** conveys a suitable approach along these lines, incorporating a structural "stick" model of the pile with appropriate t-z springs and a q-z spring representing side and base resistance behavior respectively. Indicative liquefaction-induced settlement is applied to ground nodes of affected t-z springs with 100mm and 25mm values called out to illustrate.

Axial loads are then applied at the pile head to reflect inertial demands from the bridge.

The approach depicted in **Figure 9** is displacement-based and simulates axial pile response to liquefaction-induced drag load and inertial loading in a far more realistic manner. Moreover, it brings to attention the fact that seismic loading results in imposed deformation (additional pile settlement in the present case) that the bridge structure is designed to tolerate. This is the spirit of PBD and looking at in this way avoids any of the aforementioned disagreement associated with the force-based approach. Designers must therefore be wary of force-based simplifications when entertaining a PBD approach as codified requirements such as Article 10.7.4 can thwart the design process.

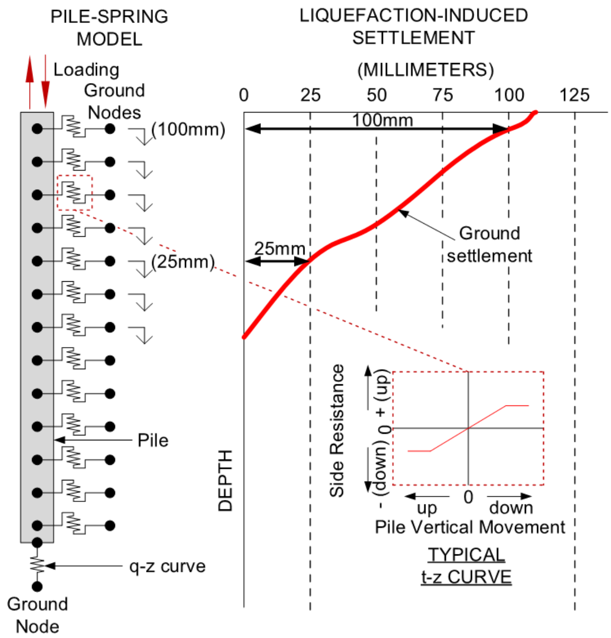


Figure 9 Axial soil-structure interaction model for displacement-based seismic downdrag assessment

6. Conclusions

The challenge of integrating extreme seismic loading, regulatory requirements and owner performance objectives for a best-value and iconic structure requires engineering judgement. This theme is recurrent across all three topics discussed in this paper, and is a hallmark of both performance based design and design contexts that push the boundaries of regulatory advice and standards. Through the development of the three themes of this paper, we see some of what the process of engineering judgment consists of.

Successfully designing and implementing dynamic frame balancing, integrally-fused viscous dampers and hundreds of tip-grouted CIDH foundations in highly stratified liquefiable conditions in the context of a fixed-price, fixed schedule procurement required engaging all stakeholders. Each stakeholder has a specific set of demands for the design, be it constructability, durability, regulatory compliance, cost or ease of maintenance. The authors contend that engineering judgment is the process of rationalizing all of these elements into a final and balanced product.

7. Acknowledgments

The Authors would like to recognize the Port of Long Beach for its vision in pursuing a structure that delivers technological innovation and the quantitative benefits of a performance-based design. The Port of Long Beach, also known as the "Green Port" for its progressive environmental priorities and policies, is a leader in recognizing that efficient design and construction is evolving into an environmental obligation.

The Authors would also like to recognize the significant contribution of the California Department of Transportation for its progressive approach in embracing the innovative approaches outlined in this article.

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