SEISMIC TIME-HISTORY ANALYSIS AND STRAIN-BASED DESIGN OF CABLE-STAYED BRIDGES

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Abstract

Cable-stayed bridges are becoming increasingly popular in North America, with several recent designs in locations with relatively high seismicity. The general approach to the seismic design of these complex structures is discussed, and future improvements to the modeling techniques currently used are suggested.

Introduction

For a large complex structure, such as the cable-stayed bridge shown on Figure 1, a rigorous, yet not overly conservative, seismic design approach should be implemented. A non-linear time-history analysis, utilizing a global structural finite element bridge model and depth varying soil-structure interaction (SSI), is a powerful tool in this regard. The development of the structural model, from the geotechnical development of soil properties and seismic motions, to the non-linear structural modeling of the bridge structure, is discussed. The process of seismic strain-based design of the various structural elements is described. Lastly, potential future improvements to the implementation of non-linear SSI analysis are suggested.

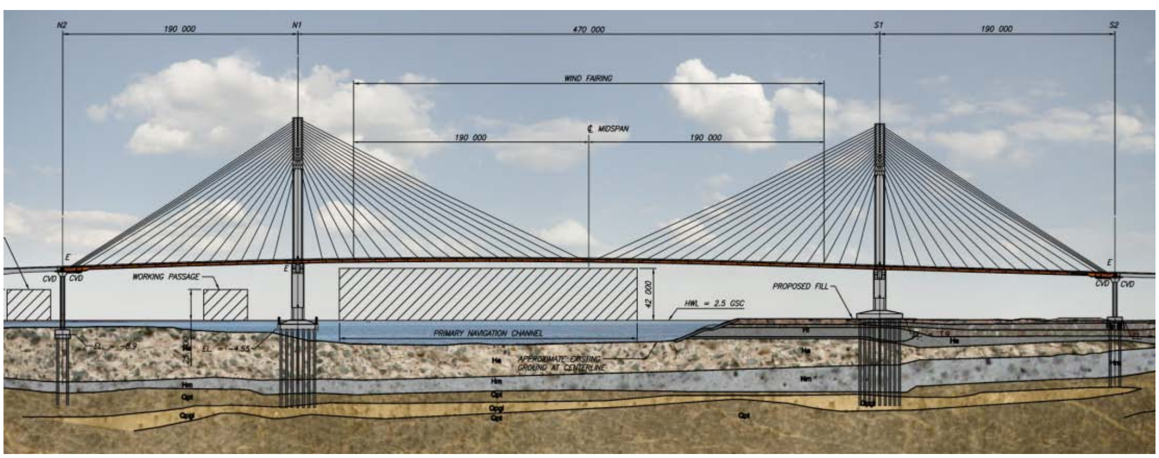

Figure 1: Cable-Stayed Bridge Layout

Seismic Design Philosophy

The objective of this analysis is to provide a realistic characterization of the soil, sub-structure, and superstructure response and interaction during strong ground shaking. Unique aspects of the response and interaction modeling described in this paper include:

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(1) variation of the soil-relative density and stiffness with depth and horizontal distance; (2) soil constitutive models that model soil behavior during strong ground shaking including liquefaction and lateral spreading; (3) two-dimensional site response of a long alignment to capture wave scattering and ground motion coherency between piers; and (4) non-linear dynamic time-history modeling with depth varying SSI and structural strain-based design.

The three-dimensional structural model of the entire bridge, including the individual piles/shafts at each pier, requires pier-specific soil response in the form of depth-varying ground motion time histories and depth-varying foundation load-deformation response curves (i.e., p-y, t-z, and Q-z curves). A one- or two-dimensional equivalent-linear or non-linear effective stress site response analysis provides the required depth-varying ground motion time histories.

The foundation load-deformation response curves are obtained from either explicit modeling of the piles and shafts in the two-dimensional site response model or closed-form solutions. The *p-y* curves simulate lateral soil resistance in the direction of lateral pile movement; *t-z* curves to represent the vertical frictional response along the sides of a pile/shaft; and *Q-z* curves to represent load-deformation of the pile/shaft tip.

These items are discussed in additional detail below.

Site Response Model Development

One- and two-dimensional equivalent linear or non-linear effective stress site response models can be used to evaluate site response due to strong ground shaking. Equivalent linear site response analysis is a total stress analysis and does not consider pore pressure development or estimates of permanent ground deformation. Alternatively, the non-linear effective stress analyses site response models consider pore pressure development and provide estimates of the permanent ground deformation and reduction in soil strength as pore pressure develops in the soil.

The equivalent linear site response analysis would be used to develop ground motions in soil that is not subject to liquefaction. Conversely, the non-linear effective stress analyses site response would be used to develop ground motions in soil that is subject to liquefaction and lateral spreading. The discussion in this paper is limited to non-linear effective stress analyses site response; however, equivalent linear methods are included in the site response model discussed below.

Typically, we select one- and two-dimensional models to evaluate the ground response transverse (perpendicular) and longitudinal (parallel) to a project alignment, respectively. A one-dimensional model assumes no lateral variation in the subsurface soil behavior and geometry. A two-dimensional model assumes that a plane strain condition exists in the area of interest; that is, displacements are expected to be in the plane of the model only.

Although other software exists for estimating site response, the software we use for these site response analyses is Fast Lagrangian Analysis of Continua (FLAC) (Itasca Consulting Group, 2008). FLAC provides a number of models to represent the stress-strain relationship of the soil along with the flexibility for the user to implement their own relationship. In addition, FLAC can perform fluid flow calculations that enable the user to model groundwater flow and pore water pressure changes within the soil mass. Dynamic excitation of the model is accomplished by providing an input time history stress.

The following data are necessary to perform the site response analysis and develop the required geotechnical input into the structural bridge model: (1) detailed subsurface profile along the bridge alignment, including soil stratigraphy, relative density, shear wave velocity, and other soil index parameters; (2) free-field firm ground or soft rock time histories for input at the base on the site response model; and (3) soil constitutive models to describe the stress-strain relationships of various soil types.

Subsurface Profile

The site response models are developed from the results of subsurface explorations performed at each pier, field measurements of shear wave velocity, and engineering index properties. The primary input parameters for the equivalent-linear or non-linear effective stress site response includes stratigraphy, relative density in the form of the Standard Penetration Test (SPT) blow count and shear wave velocity. For the non-linear effective stress site response analysis used in many of our project-specific analyses, the normalized corrected Standard Penetration Test (SPT) blow count, $(N_1)_{60,CS}$, and modulus values (calculated from field-measured V_S) are developed with depth and horizontal distance are heavily relied upon in the soil constitutive model we utilize. The field-measured and converted $(N_1)_{60,CS}$ and shear wave velocity measured from the borings are interpolated over the extent of the model. The interpolated $(N_1)_{60,CS}$ and shear wave velocity are mapped onto the model mesh. An example of a final model mesh of interpolated $(N_1)_{60,CS}$ is shown on Figure 2. Additional input parameters are required for the site response models and will be briefly described below.

Input Free-field Ground Motion Time Histories

The discussion of the development of design ground motions is beyond the scope of this paper. However, we do provide some general background regarding the ground motions used in the site response models. Free-field rock time histories are input into the base of the model. These design ground motion time histories are typically spectrally matched or scaled to a target response spectrum that corresponds to the project ground motion criteria. The target response spectrum would consist of either a probabilistic uniform hazard spectrum or deterministic source-specific response spectrum. Reference ground motion time histories would be spectrally matched or scaled to the target response spectrum. Additionally these ground motions would be selected from recorded ground motions that were obtained at seismic source(s) that are similar to the project tectonic conditions (i.e. crustal, subduction zone, etc). Sets of the design ground motion time histories would be used in the site response analysis. The number of design ground motion time histories would depend upon the project design criteria, the methods used to develop the ground motions, and objectives of the site response analyses. Additional details regarding the selection and development of the input ground motion time histories can be found in Kramer (1996).

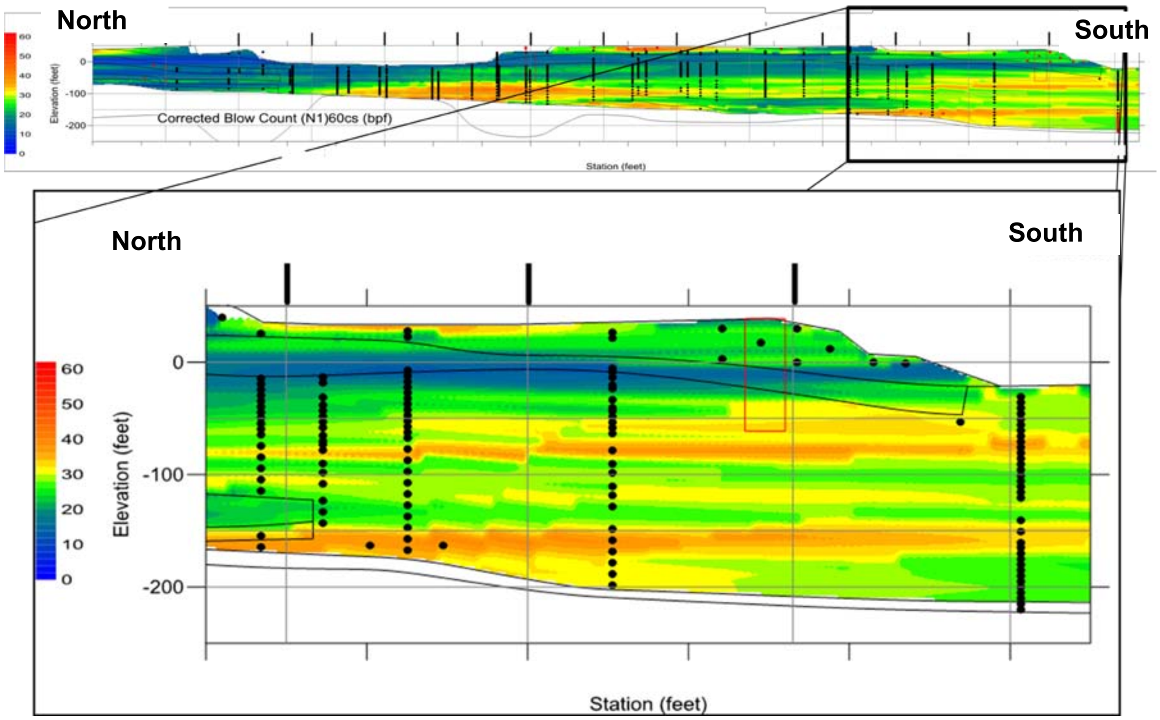


Figure 2: Mesh of interpolated normalized corrected Standard Penetration Test (SPT) blow count, $(N_1)_{60,CS}$,

Non-Linear Effective Stress Site Response

Evaluation of non-linear soil response including the effects of dynamic pore pressure generation is performed at sites to evaluate the soil liquefaction potential and lateral spreading effects. Since the input motions supplied are assumed to originate from upward propagating waves, FLAC's quiet (compliant) base formulation is applied to the bottom of the model so that downward propagating waves do not reflect off the model boundaries. The mesh zone sizes in a two-dimensional model are varied depending on the stiffness of the material. In a recent model, approximately 23,600 zones were required to model a 2 kilometer long alignment. Similar to the quiet base formulation, FLAC's free field formulation would be applied to the sides of the model to prevent reflection of waves back into the model. The free field formulation emulates the outermost zones of the model and performs calculations in small strain. The dynamic loading of both models is implemented in the same manner. A shear stress time history, required when modeling a compliant base, is applied to the bottom of the model. The input horizontal shear stress time history is calculated from the velocity time histories of the input ground motion time histories.

Constitutive Models

FLAC provides several internal constitutive models including the Mohr-Coulomb model and the hysteretic model. The Mohr-Coulomb combined with the hysteretic model is used to model soil that would not develop significant pore pressures during the design ground motions. In recent projects we utilized a user-defined constitutive model, UBCSAND, on soil where significant pore pressure changes are anticipated during dynamic loading of design ground motions. These models are utilized in one-dimensional and two-dimensional models. A brief discussion of these models is provided below.

Mohr-Coulomb and Hysteretic Model

The Mohr-Coulomb model treats a material as purely-elastic-purely-plastic. That is, the model behaves as a linearly elastic material at shear stresses less than the shear strength of the material and shows purely plastic behavior at shear stresses greater than or equal to the shear strength of the material. The properties required for the Mohr-Coulomb model, as implemented in FLAC, include mass density, cohesion, angle of internal friction, tension limit, dilation angle, bulk modulus, and shear modulus. Mass density represents the mass of the soil; cohesion, angle of internal friction, tension limit, and dilation angle describe the shear strength limit of the soil; and bulk and shear modulus describe the elastic behavior of the soil. These soil properties are obtained as part of the field exploration and laboratory testing program.

UBCSAND Model

The UBCSAND model was developed independently from FLAC by Professor Peter M. Byrne and his colleagues at the University of British Columbia, Vancouver, Canada. UBCSAND modifies the internal Mohr-Coulomb model in FLAC to better capture the plastic strain response of the soil at all stages of loading and unloading. In addition, the UBCSAND model uses a hyperbolic formulation to describe the shear and bulk modulus of the material as a function of the current effective stresses and any changes during loading. These additional features allow the model to approximate the non-linear hysteretic behavior which is observed in granular soil that develops significant pore pressures.

The UBCSAND model as implemented in FLAC requires twelve input and four calibration parameters. Most of the input parameters can be empirically related to other parameters, therefore only about seven parameters are required. The primary input parameters is the SPT blow count corrected to 60 percent hammer efficiency and 1 ton per square foot of overburden pressure, $(N_1)_{60}$. The four additional calibration parameters of the UBCSAND model are used to calibrate the pore pressure generation and post-liquefied soil behavior. As described in the previous sections, there are several model input parameters required for each constitutive model. Some input parameters remain constant over an entire soil unit. Alternatively, several other input parameters including $(N_1)_{60}$, modulus values (calculated from shear wave velocity) and shear strength are varied with depth and horizontal distance.

We calibrate UBCSAND to replicate the empirical liquefaction-triggering behavior described in Youd and others (2001). The number of cycles to reach liquefaction is assumed to occur at an excess pore pressure ratio equal to 0.9. The specific Youd and others (2001) empirical relationships targeted in our calibration include the cyclic resistance ratio (CRR) versus $(N_1)_{60}$ at 15 cycles, CRR versus effective confining stress, and CRR versus number of cycles based on site-specific cyclic direct simple shear testing.

Site Response Output

Several model parameters are monitored throughout the simulations, including maximum shear strain, maximum horizontal acceleration, lateral ground displacement, and excess pore pressure ratio at the end of shaking. At the end of shaking, residual strengths are calculated from the end of shaking excess pore pressure ratio and initial vertical effective stress for the liquefiable soil modeled with UBCSAND. Figure 3 presents a typical plot of the excess pore pressure ratio. Plots of lateral ground displacement, and time to an excess pore pressure ratio equal to 0.9 (liquefaction) are also obtained along the project alignment.

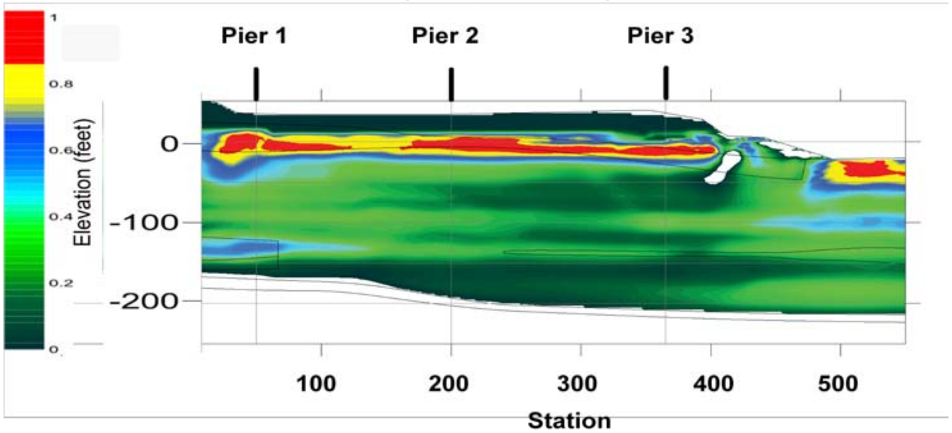


Figure 3: Variation of excess pore pressure ratio along an alignment for a typical input ground motion

The ground motions were estimated for depths along foundation elements. That is, acceleration, velocity, and displacement time histories are recorded at each pier as a function of depth for input into the structural model. Plots of the acceleration, velocity, and displacement time history are developed for each pier and at each depth increment along the foundation element (Figure 4).

Foundation Load-Deformation Response Curves

Depth-varying foundation load-deformation response curves for the individual piles/shafts at all bridge piers are directly incorporated into a three-dimensional nonlinear bridge model (Figure 4). The load-deformation response curves include p-y curves to simulate lateral soil resistance in the direction of lateral pile movement; t-z curves to represent the vertical frictional response along the sides of a pile/shaft; and Q-z curves to represent load-deformation of the pile/shaft tip.

The lateral p-y and vertical side t-z curves are applied along the length of all individual piles within a pier. To account for possible multi-directional lateral foundation behavior, uncoupled pairs of p-y curves are applied in the longitudinal and transverse load directions. Load deformation response curves are dependent on several factors including pile diameter, vertical effective stress, soil strength and stiffness, and soil layering. P-y curves for sands are based on Reese and others (1974). Reese and others (1974) formulation of the P-y curves for sand are used

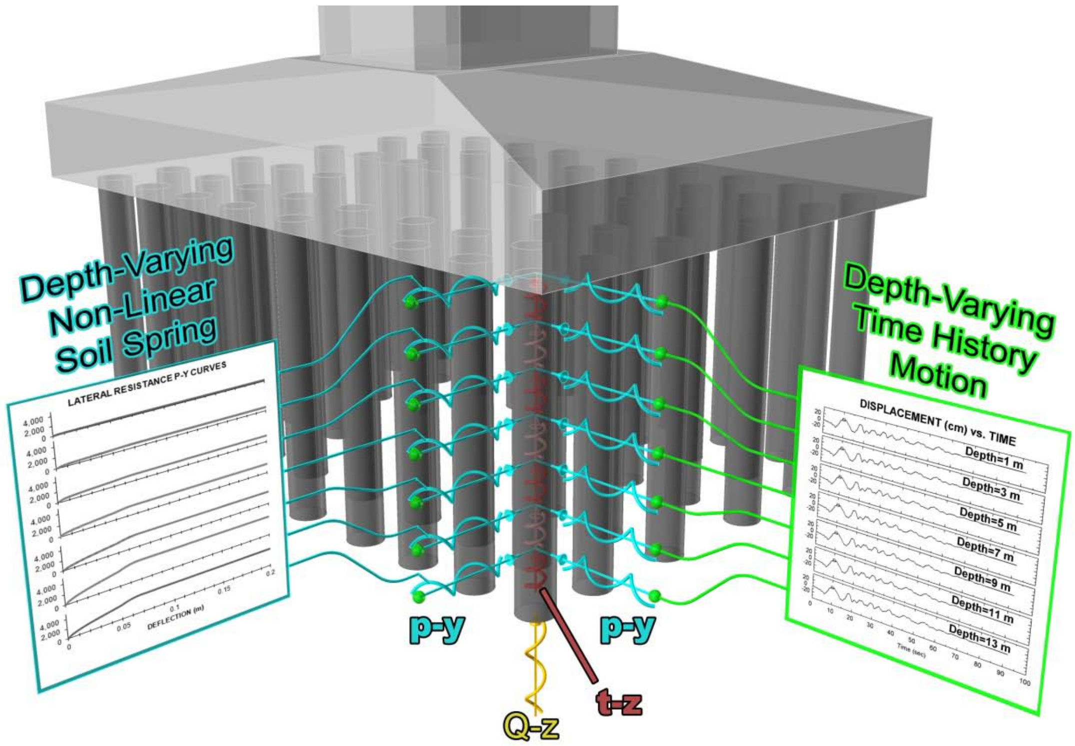


Figure 4: SSI developed for each pier and as a function of depth along pile

for the site response analysis that considers liquefaction, with soil input properties reflecting the residual strength consistent with the estimated pore pressure accumulation. P-y curves for soft and stiff clays are based on work by Matlock (1970) and Reese and others (1975). All p-y curves are generated based on a single pile analysis. T-z and Q-z curves for frictional and cohesive soil are developed using the general form outlined by Vijayvergiya (1977).

We generate p-y and t-z curves at intervals along the full length of a foundation element at all pier locations for the various analyses including seismic (no liquefaction), liquefaction, no scour, and half scour. Q-z curves for the same cases are provided for the estimated pile/shaft tip elevations. P-y and t-z load deformation response curves are also provided for the pier caps, where applicable, based on an equivalent diameter approach. Single pile/shaft p-y curves are multiplied by the reduction factors described below to account for lateral group behavior. No group reduction factors are applied to the t-z, Q-z, or pier cap curves.

Group Reduction Factors

The p-y curves represent soil resistance for a single pile. For groups of piles, the lateral soil resistance mobilized is dependent on the number and proximity of adjacent piles. At spacings less than 7 pile diameters, pile group efficiency is reduced. For a recent project the reduction factors based on Reese and Van Impe (2001) were applied to the p-y curves.

Nonlinear Structural Global Model

The depth-varying ground motions and soil springs, developed under the geotechnical effort described above, are applied to a nonlinear structural global finite-element model in order to evaluate the seismic demands of the various bridge components. A typical global bridge model developed in the commercially available finite element program ADINA (ADINA, 2009) is shown in Figure 5.

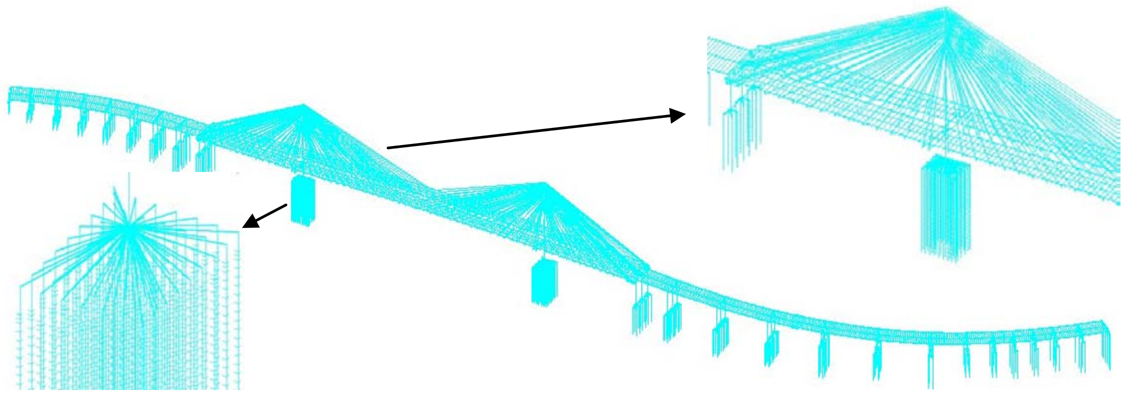


Figure 5: Global ADINA model with cable-stayed unit and approach structures

The cable-stayed unit as well as the approach structures is modeled together; this allows for a direct evaluation of the dynamic seismic interaction between the units. The following description of the ADINA model development is focused on the more complex cable-stayed unit.

Deck Modeling

The composite deck with concrete slab and steel edge girders and floorbeams is modeled with a drop-down grid model as shown in Figure 6. The edge girders and floorbeams are modeled with elastic beam elements, reflecting the appropriate composite structural stiffness. The lateral stiffness of the beam elements is tuned, by parametric plate analysis, to reflect the concrete deck slab lateral diaphragm stiffness. Classical Rayleigh damping is applied to the elastic structure groups, anchored at appropriate periods to get a reasonable percent damping.

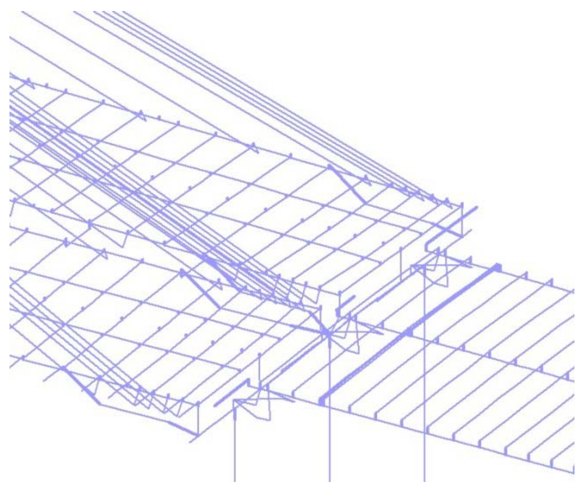


Figure 6: ADINA Deck Modeling

The cables are modeled with elastic truss elements with a modified E-modulus to reflect the catenary cable stiffness for the dead load stressed condition. In order to model the initial dead load condition (both element forces and initial cambered geometry), a simplified staged analysis, cable shortenings and general element initial strains are implemented. If the stay cable gets close to dynamically off-loading (around

10% GUTS), the cable is discretized to directly reflect the non-linear catenary stiffness at low stress levels.

One of the advantages of a fully non-linear global model is the ability to dynamically monitor force and displacement demands across expansion joints between approach units and the cable-stayed main span, as well as approach unit midspan hinges (Figure 7).

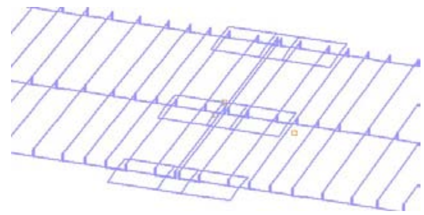


Figure 7: Midspan hinge

Cable-Stayed Unit Tower Modeling

The main towers of the cable-stayed unit are modeled with moment-curvature elements throughout, from the top of the pile cap to the bottom of the cable anchor box zone; see Figure 8. This approach allows for the non-linear tower stiffness to reflect the seismic demands at any location in the tower. An identical approach is taken for end piers and approach span columns. The development of these moment-curvature elements is discussed later.

Pile Foundations

The foundation modeling is fundamental to the non-linear soil structure interaction. Piles are modeled with moment curvature elements throughout, from the soffit of the pile cap to the pile tip; refer to Figure 8. Orthogonal non-linear soil springs (py, tz and Qz), derived under the geotechnical work described earlier, are connected to the pile moment-curvature elements at appropriate vertical intervals, typically around 1-2 pile diameters, depending on local soil layers and expected structural response. The far end node of the soil springs is the "ground node" and the location at which the time-history ground motions are applied. The soil springs as well as the ground motions vary with depth, and both are uniquely derived under the geotechnical effort for each foundation location. Near-field effects, such as radiation damping, can be modeled with dash-pot elements if deemed necessary; for pile foundations of the type described here, radial damping is not modeled explicitly (Ingham et. al., 2009) . It should be noted that for a non-linear soil spring, the orthogonal modeling technique results in a too large soil stiffness if displacement demands are large in both longitudinal and transverse directions at the same instant in the time-history analysis. In the limit, the soil stiffness will be a square-root two factor too large. This issue can be addressed by further discretizing the soil springs from two bi-directional springs to a rosette of springs (say 8 to 16) with the appropriate spring values, thereby creating a combined stiffness that is independent of the direction of loading.

Dampers and Seismic Transmission Units

A fully non-linear time-history analysis allows for explicit modeling and evaluation of dampers and seismic transmission units (STUs), both in terms of seismic force demands and seismic displacement demands. We have found that preliminary sizing of such devices using a linear response-spectrum analysis (RSA) approach can result in demands/sizing significantly different than those based on time-history analyses.

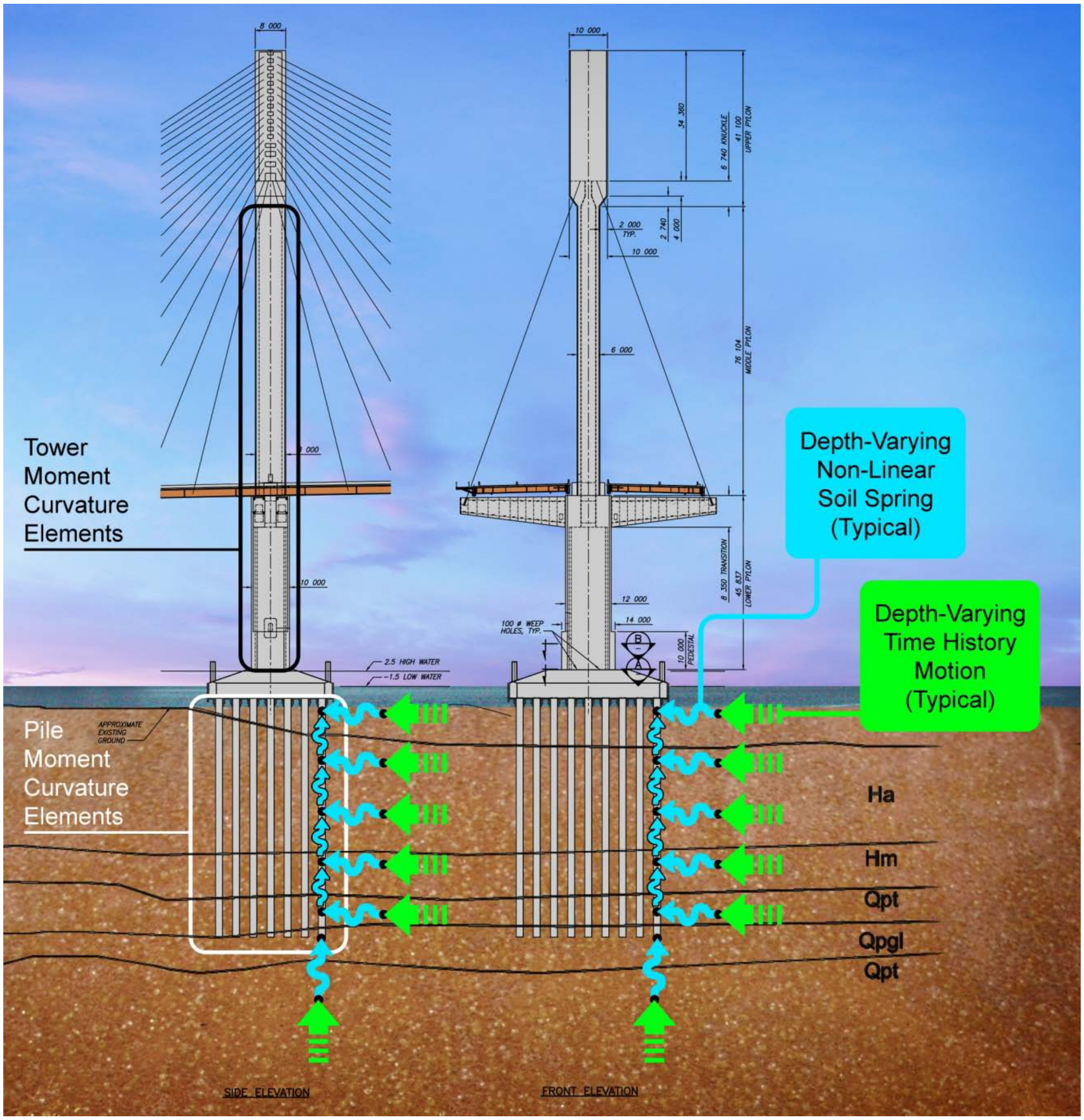


Figure 8: Cable-Stayed tower and foundation modeling

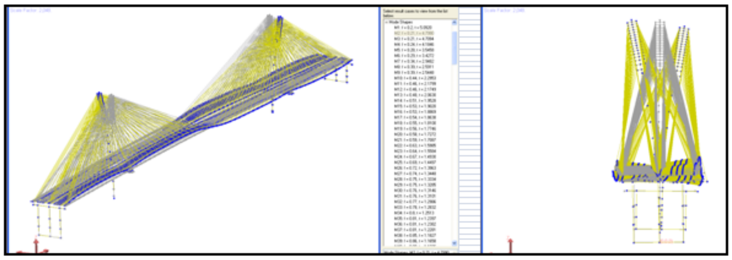
The non-linear time-history analysis allows for a parametric study of damper and STU devices for seismic connectivity between the deck and the substructure. Generally speaking, very stiff STU devices tend to develop large force demands that can be challenging in design. A better alternative can be a damper, tuned to have a limited displacement demand of a few inches. This will often significantly reduce the force demands in the damper, resulting in less expensive units and reduced demands into connections and the overall superstructure.

Model Verification

The global ADINA non-linear time-history model is only one of several global models developed in the bridge design process. Comparison of fundamental characteristics, between the rather complicated ADINA model and a classical elastic model, provides a good opportunity to

Mode	Frequency (Hz)	Period (s)	Description Larsa
Mode 1	0.20	5.09	Transverse, Towers same direction
Mode 2	0.21	4.80	Transverse, Towers Opposite Direction
Mode 3	0.21	4.71	Longitudinal, Deck (Floating at N1 and S2)
Mode 4	0.24	4.18	Longitudinal, N1 Tower
Mode 5	0.28	3.55	Transverse, Deck
Mode 6	0.29	3.43	Vert, Deck
Mode 7	0.34	2.95	Transverse, S1 Tower
Mode 8	0.39	2.59	Longitudinal, S2 Bent
Mode 9	0.39	2.54	Vertical, Deck NB and SB Opposite
Mode 10	0.44	2.30	Vertical (Deck) and Trans Rocking N2
Mode 11	0.46	2.18	Vertical (Deck) and Trans Rocking N2
Mode 12	0.46	2.17	Vert, Deck 2n mode

Mode	Frequency (Hz)	Period (s)	Description ADINA
Mode 1	0.19	5.14	Trans, Towers, S1 larger than N1
Mode 2	0.21	4.70	Transverse, Towers Opposite Direction - more deck sag than Larsa
Mode 3	0.21	4.69	Trans, Towers Opposite Direction
Mode 4	0.24	4.25	Long, Deck Floating N1 and S2
Mode 5	0.27	3.72	Trans, Deck and Tower S1
Mode 6	0.29	3.50	Vert, Deck
Mode 7	0.34	2.97	Trans, S1 Tower
Mode 8	0.38	2.62	Long, S2 Bent
Mode 9	0.39	2.59	Vert, Deck NB and SB Opposite
Mode 10	0.43	2.31	Vert, Deck (plus Trans Bent N2), SB and NB Opposite
Mode 11	0.45	2.23	Vert, Deck (plus Trans Bent N2), SB and NB same Direction
Mode 12	0.45	2.20	Vert, Deck 2nd Mode (N2 Trans Rocking)



Location	Larsa Node	ADINA Node	ADINA RZ (MN)	Larsa RZ (MN)	RZ ratio (1/1)
End Pier N2	600203	600204	48.7	47.9	1.02
Tower N1	10330	10331	476.4	478.9	0.99
Tower S1	20330	20331	470.0	472.4	0.99
End Pier S2	700203	700204	35.0	34.6	1.01

Figure 9: Non-linear time-history model verification

verify the detailed ADINA modeling such as moment curvature elements to non-linear soil springs varying with depth, etc. Figure 9 shows such a comparison between the seismic ADINA model and an elastic LARSA model (LARSA, 2009) of the same cable-stayed bridge. As can be seen, fundamental modes compare well, considering that the bending stiffness is that of classical beam theory in the LARSA model, versus the tangent stiffness of the non-linear moment-curvature elements in the ADINA model. Further checks include dead load reactions at all supports and a graphical comparison of the individual modes.

<u>Strain-Based Design Implementation</u>

The performance specification is typically formulated in terms of maximum allowable plastic strain in the various structural bridge components. These strain limits are project-specific and reflect the desired bridge performance for the various ground motion return periods. An example of unique strain limits for various bridge components is shown in Figure 10.

In order to discuss the application of project-specific strain limits, we will need to describe the detailed development of the moment curvature elements for the ADINA model. A computer program, such as Xtract or similar, is used to define a given structural cross section. Structural steel, concrete, reinforcement, and post-tensioning are all uniquely defined component of the cross section. Steel elements are typically defined using a bi-linear material model with parabolic strain-hardening. Concrete elements are usually defined using Mander's Confined Concrete Model (Manders, 1988).

For a variation of axial loads (enveloping the expected seismic axial force demand range), a classical plane-sections-remain-plane analysis is performed by Xtract. For a range of section curvatures, the individual material fiber stresses and strains are calculated based on the material definitions provided. The fiber forces are lastly integrated to achieve the overall section moment.

Figure 11 shows a typical resulting moment-curvature definition, where the individual material fiber strain and stresses can be tracked, graphically or in tabular format. We note that the

Event	Component	Concrete (1/1)	Rebar (1/1)	PT (1/1)	Steel Shell (1/1)
0475/subduction	Towers	0.004	0.010	NA	NA
	Piers and Link Slabs	0.004	0.010	0.008	NA
	CISS Piles, general composite	NA	0.002	NA	0.002
	CISS Piles, non-composite top section	NA	0.002	NA	NA
	Drilled Shafts	0.003	0.002	NA	NA
0975	Towers	0.004	0.010	NA	NA
	Piers and Link Slabs	0.007	0.025	0.015	NA
	CISS Piles, general composite	NA	0.010	NA	0.010
	CISS Piles, non-composite top section	NA	0.010	NA	NA
	Drilled Shafts	0.004	0.010	NA	NA
2475	Towers	0.004	0.010	NA	NA
	Piers and Link Slabs	0.011	0.090	0.045	NA
	CISS Piles, general composite	NA	0.025	NA	0.025
	CISS Piles, non-composite top section	NA	0.025	NA	NA
	Drilled Shafts	0.007	0.025	NA	NA

Figure 10: Bridge component strain limits

resulting moment-curvature is around a given axis. In principle, this analysis could be performed for any given direction of bending. However, the ADINA model is restricted to two orthogonal moment-curvature definitions around the principle axis of the ADINA beam element, so typically the Xtract analysis is restricted to orthogonal principle axis moment-curvatures. This limitation is accepted as current state-of-the-art modeling technique, and if this limitation is of concern for particular design elements, non-linear push-over analysis at a skew can be performed with the ADINA elements rotated and defined appropriate through a rotated moment-curvature definition. For symmetrical sections such as piles, this is of course not an issue.

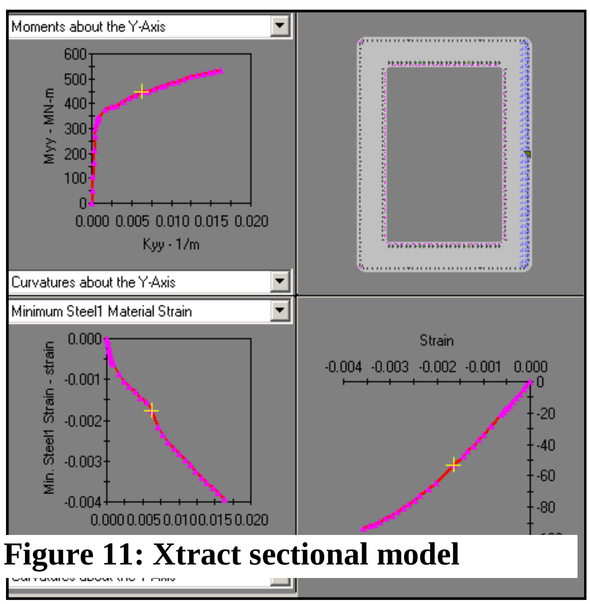


Figure 11: Xtract sectional model

For a non-linear indeterminate structural system, overly conservative stiffness modeling in one region can prove non-conservative for other critical areas of the system. Therefore, the Xtract material definitions are developed to reflect the best estimated structural stiffness. Expected over-strength factors are applied to the nominal material properties, typically around a 1.3 factor for concrete and a 1.1-1.2 factor for structural steel and rebar (dependent on the governing codes and the project-specific criteria).

The non-linear moment-curvature definitions described above are used to define orthogonal moment-curvature elements in ADINA for a range of axial loads (typically around 10-20 unique axial load levels are defined). The dynamic ADINA time-history analysis is next run for each time-history ground motion as developed during the geotechnical scope as described earlier. Typically, these analyses include several performance levels (operating, repairable, no-collapse, subduction) and several ground motions for each performance level, so the array of separate analyses can be quite large.

Post-Processing / Strain Demand Evaluation

After completing the dynamic non-linear time-history analyses in ADINA, a large amount of data needs to be extracted and evaluated against the project-specific strain limits described earlier. Material strains are not explicitly part of the ADINA moment-curvature definition so the ADINA section demands, in the form of concurrent axial force and orthogonal curvatures in each element, need to be related back to the Xtract sectional analysis results. This is performed for each and every moment-curvature element in the ADINA model, scanning all the dynamic time steps for concurrent minimum and maximum axial forces and curvatures.

An example of this post-processing is shown in Figure 12. Similar plots are developed for each structural element and for each material within that element. For circular sections like piles,

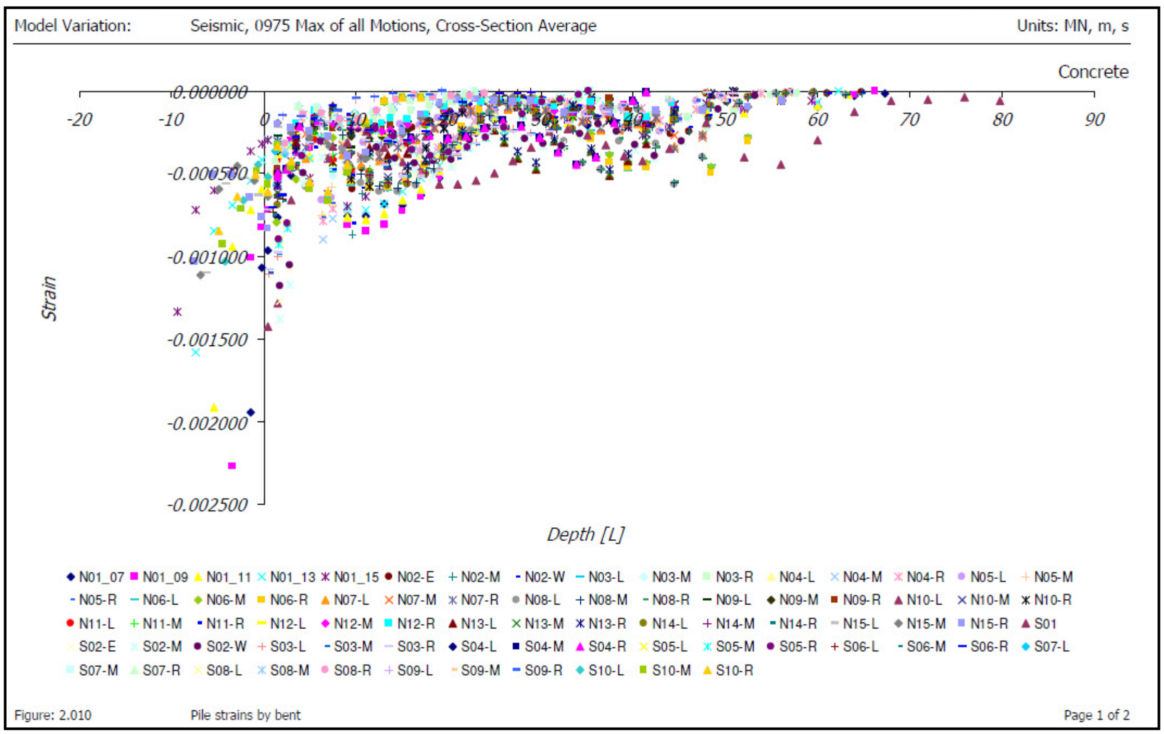


Figure 12: Pile concrete strain demands with depth and location

the moment-curvature is constant around any axis, and the ADINA results can be scanned for the maximum 2D moment; the extreme strain (concrete or rebar) is based on this 2D moment. In the general case, the section corner strain is conservatively calculated by adding the strain demands for the individual orthogonal curvature demands.

The strain demands on Figure 12 are for a particular ground motion. If the project criteria require the strain evaluation to be based on the maximum strain demand from any of the ground motions, the post-processing is simplified to scanning for the maximum demand. However, certain project-specific criteria are based on the average strain from all ground motion at a particular performance level. This “averaging across ground motions” is cumbersome to implement and conceptually flawed for non-linear elements; it will result in inconsistent force and strain demands that cannot be uniquely related back to the section analysis. A more consistent and practical approach might be to provide a reduction factor to be applied to the actual ground motions if some sort of averaging is desired.

Structural displacement demands are extracted in a similar fashion from the dynamic analyses. An example of drift demands is shown in Figure 13, where the maximum pile drift demands are extracted for all foundation location and for all 475 year return period motions. Drift demands can be absolute, but the drift relative to a reference location (pile tip for piles, pile cap for tower, etc.) is typically of more interest as it reflects actual structural element deformations.

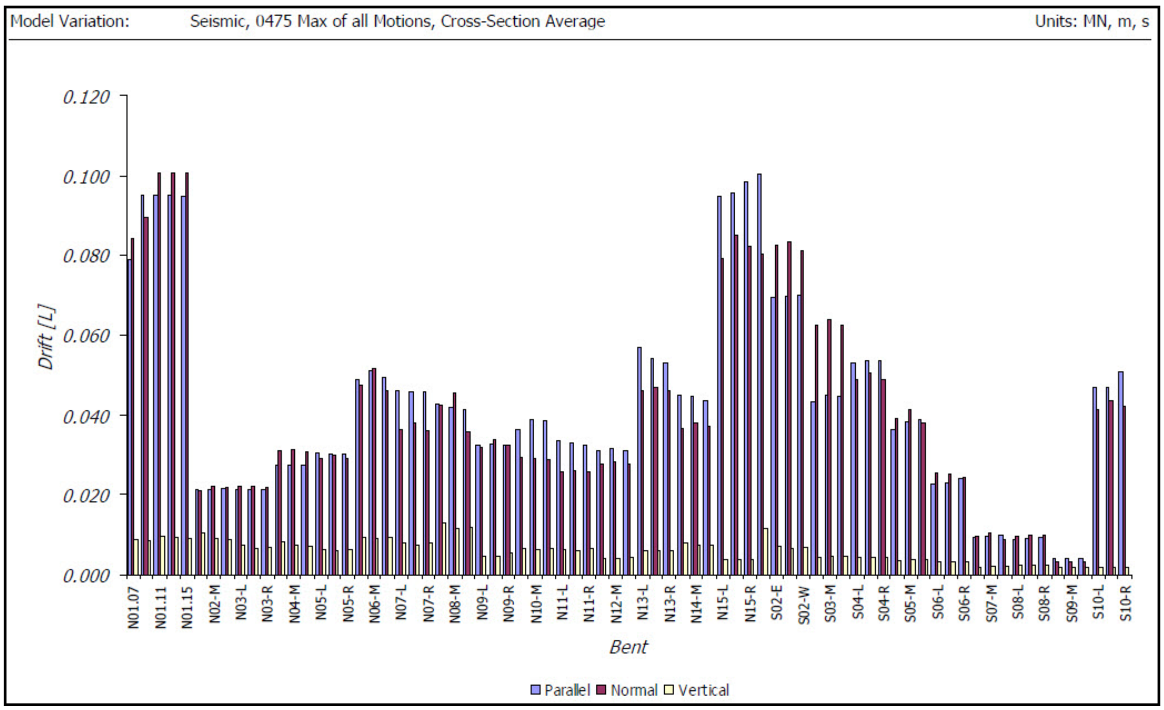


Figure 13: Maximum top of pile drift relative to pile tip, all foundation locations

Based on the above-described structural demands (forces, strains, displacements, etc), the structural layout/design can be modified to keep the bridge performance within codes and project specific criteria, yet avoid an overly conservative design. For example, on a recent bridge design, a structural solution to lateral spreading conditions was found to be more cost-effective than the alternative of soil ground improvement (the lateral spreading soil deformations were explicitly modeled in the SSI analyses).

<u>Conclusions</u>

For cable-stayed bridges under high seismic loading, non-linear time-history analyses with SSI, as described in this paper, is perhaps the best tool we have today to evaluate the structural bridge behavior and ultimately optimize the bridge and foundation design. These analyses allow for the explicit modeling of depth-varying soil properties and depth-varying ground motions. Material strain demands in bridge super- and substructure components are predicted through moment-curvature modeling. Damper and STU devices can be tuned for optimal performance, and seismic displacement demands at expansion joints and hinges can be directly determined.

Future development of these models might include 3D soil springs and 3D moment-curvature definitions. Fully integrated 3D SSI with non-linear structural and soil modeling has been used sporadically in the industry, but significant improvements in computer hardware and software are required for this to become a practical approach in the design industry.

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The discussed approach to non-linear time-history analysis of cable-stayed bridges has lately been implemented for several large design-build bridge projects in North America. The authors would like to acknowledge owners, contractors, co-workers and peer review teams for their outstanding commitment to deliver successful design-build cable-stayed bridge projects.

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