# Validating Innovative Design Solutions - Analysis of the Gerald Desmond Bridge Replacement

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## Abstract

*In 2011, the Port of Long-Beach, in collaboration with Caltrans and LA Metro, awarded the Design and Build contract for the replacement of the deteriorating Gerald Desmond Bridge to SFI Construction (Schimmick / FCC / Impregilo joint-venture). Arup had been lead designer for SFI's tender proposal, providing structural and geotechnical engineering, traffic operations analysis, lighting design and civil engineering services. Arup designed an elegant mono-pole stayed-cable solution that met all the project requirements while providing dramatic cost-savings to the Client. The team's innovative solution earned the judges' highest ratings for both technical design and price and ultimately won the job.*

*The deployment of advanced LS-DYNA® analysis capabilities was instrumental in assessing the structural options against the stringent project requirements. The extreme seismic demands of the 1000-year Safety Evaluation Event (SEE) could be addressed by isolating, by means of viscous dampers, the Main Bridge deck from the Towers and by introducing a ground-breaking approach for the design of the ductile hollow-section columns. These innovative solutions, among other particular features of the bridge, required detailed Finite Element modelling and validation through explicit nonlinear time-history analysis.*

*This paper presents some of the key modelling techniques and analyses results that contributed to the successful development of this new landmark.*

*Keywords: Viscous dampers, fibre section, soil-pile interaction, *MAT_CONCRETE_EC2, *MAT_HYSTERETIC_REINFORCEMENT*

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## Introduction

The Port of Long Beach in collaboration with the California Department of Transportation ("Caltrans") has awarded a design and construction contract for the replacement of the deteriorating Gerald Desmond Bridge to SFI Construction (Schimmick / FCC / Impregilo joint-venture).

As lead designer for SFI's tender proposal, Arup provided structural and geotechnical engineering, traffic operations analysis, lighting design and civil engineering services.
Arup designed an elegant mono-pole stayed-cable solution, including 6 traffic lanes, over 6,000ft of elevated approach viaducts and a major freeway interchange that met all the project requirements while providing dramatic cost-savings to the Client. The team's innovative approach earned the judges' highest ratings for both technical design and price and ultimately won the job.
The Main Span Bridge is a cable stayed bridge with a 1,000 ft main span which comprises a steel-concrete composite ladder beam superstructure supported by multi-strand stay cables from hollow reinforced concrete mono-pole towers.
The Approach Bridges are single cell prestressed concrete box girders with a maximum span up to 230 ft. Adjacent to the Main Span Bridge, the girders are cast in-situ using a Mobile Scaffold System (MSS) and are integral with ductile reinforced concrete hollow columns.
The Towers, with a unique cross section transforming from an octagon to a diamond, are bound to form a landmark for the Port of Long Beach.
The GDB Replacement is also set to become California's first long-span cable-stayed bridge.

## Seismic Design

Arup's design brought significant improvements to the cost, construction schedule, durability, aesthetics and seismic performance of the Towers compared to the Bid reference design. One of the key design features is the introduction of viscous dampers to seismically isolate the main deck and the Towers of the cable-stayed bridge. Under the 1000-year Safety Evaluation Event (SEE), relative movement at the deck-Tower junction is allowed and the seismic demands in the Towers are significantly reduced, making a traditional single shaft option viable.
The critical issue for the Approaches is the ductile behaviour of the tall -up to 150ft- hollow columns. Californian design practice is based on established concepts of ductility for confined concrete in plastic hinges which have originally been developed for solid cross sections. For such tall columns a solid section is not economical and self-weight itself would lead to seismic issues not covered by the standards. Arup design team extended beyond traditional code of practice to develop and implement acceptable ways of designing plastic hinges in hollow column sections.
These innovative solutions, among other particular features of the bridge, required detailed Finite Element modelling and validation through explicit nonlinear time-history analysis (NLTHA). The deployment of advanced LS-DYNA analysis capabilities was instrumental in assessing the structural options against the stringent project requirements.

## Analysis process

The objectives of the LS-DYNA analysis were:

- Assessment of all seismic effects on the Main Span Bridge between and inclusive of the end bents and the expansion joint thereon, to provide confirmation of the design - in particular a review of the demands on the towers and foundations, the dampers' strokes and forces
- Seismic analysis of seating of the Approach Bridges on the end bents
- Design validation for the structural elements and foundations of the Approach Bridges

The analysis was carried out in accordance with the requirements set out by Caltrans in the Basis of Design Report. Caltrans *Seismic Design Criteria [1]* and Caltrans *Guide Specifications for Seismic Design of Steel Bridges [2]*, augmented with pertinent provisions of ATC-32 [3], NCHRP 12-49 [4], AASHTO *LRFD Bridge Design Specifications*, *4th Edition*, with California Amendments *[5]*, AASHTO *Guide Specifications for Seismic Isolation Design [6]*, PTI *Recommendations for Cable Stay Design, Testing, and Installation [7]*, and Project specific criteria formed the basis of the seismic design of the bridge.

The seismic response of the soil-pile-pilecap-pier-bridge superstructure to the strong motion earthquake excitation was simulated by the nonlinear response history analysis method in LS-DYNA (971 R6.1.0) [8].

Maximum seismic force and deformation demands during the entire response time-history were to be compared with the structure's capacities on a component by-component basis.

## Earthquake motion Time-histories

In accordance with the project requirements, the ground motions for use in dynamic seismic analysis of the bridge structures were taken from the *Project Seismic Ground Motion Report information* provided by Caltrans which documents the project-specific acceleration response spectrum (ARS) design curves and spectrum-compatible ground motion time histories for the Safety Evaluation Earthquake (SEE).

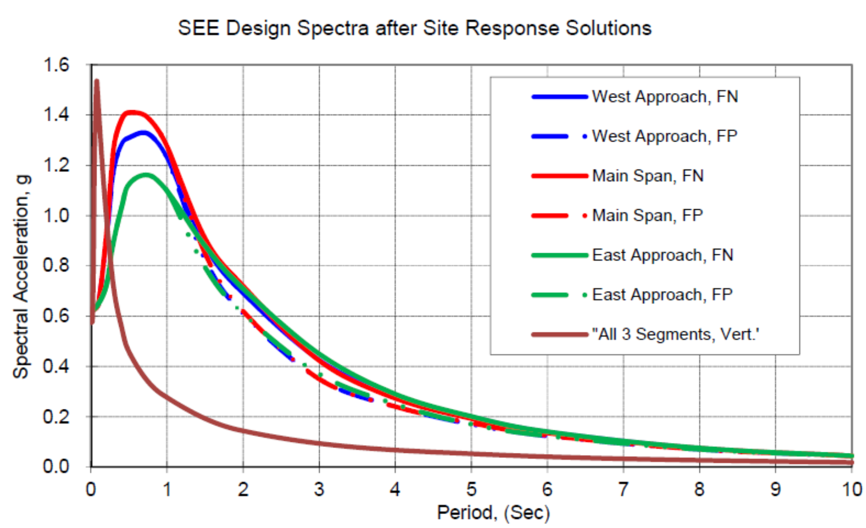


**Figure 1: SEE ARS curves for the three GDB segments**

The Project site was characterized into three soil zones: West Approach, Main Span, and East Approach. For each soil zone, the ARS design spectra and earthquake motion time histories spectrally matched to the ARS were provided by the Client to be applied to the FE model foundations.

Each set of ground motions consisted of three motion time histories, two horizontal orthogonal and one vertical components.

Figures 2 below shows an example of SEE Main Span excitations.

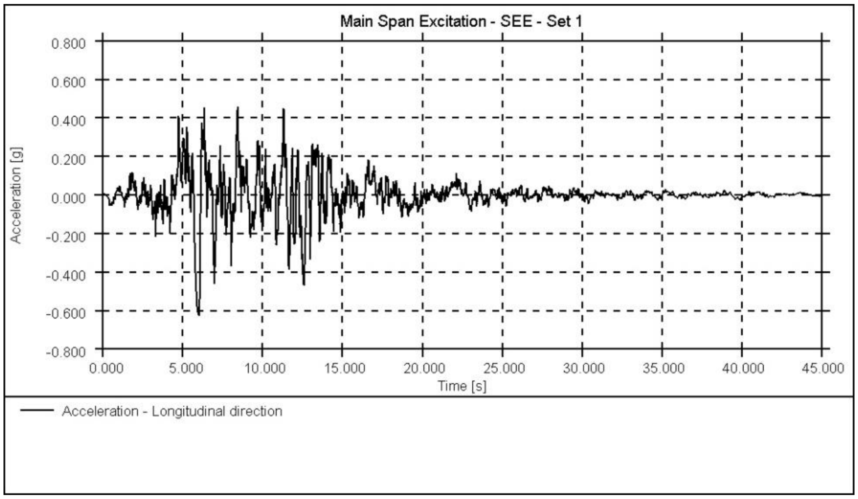


**Figure 2: SEE Main Span excitation – Set 1 – x (longitudinal) direction**

## Ground motion input mechanism

The design specification required the seismic excitation time histories to be applied over the entire depth of the foundation zones, with no consideration given to the variations of motion that might be expected with depth. Therefore, at each support, all the local soil to pile springs received the same excitation. Figure 3 below shows the concurrent ground motion input as applied in the analysis model.

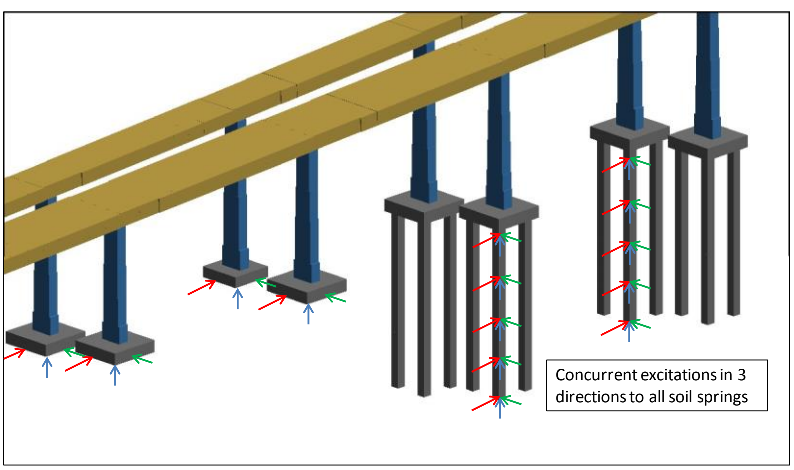


**Figure 3: Ground motion input mechanism**

## Model description

The geometry and properties of the Approach, Main Span superstructure and foundations reflected the final GDB design. All dead loads (concrete and steel) and superimposed dead loads (surfacing, barriers, etc) were accounted for and accurately represented in the model as mass density or added lumped mass elements.

1- Superstructure

For the Main Span superstructure, the longitudinal steel edge box girders, transverse floor beams and precast panel deck were modelled by a grillage of linear elastic beam elements.

Approach superstructures were modeled by a series of linear elastic beam elements at the centers of mass of the deck cross section. To better represent the mass distribution over the width of the decks, lump masses were offset from the deck beam axis by means of rigid outriggers and calibrated to achieve the correct mass moment of inertia.

Figure 4 illustrates the bridge superstructure modeling details.

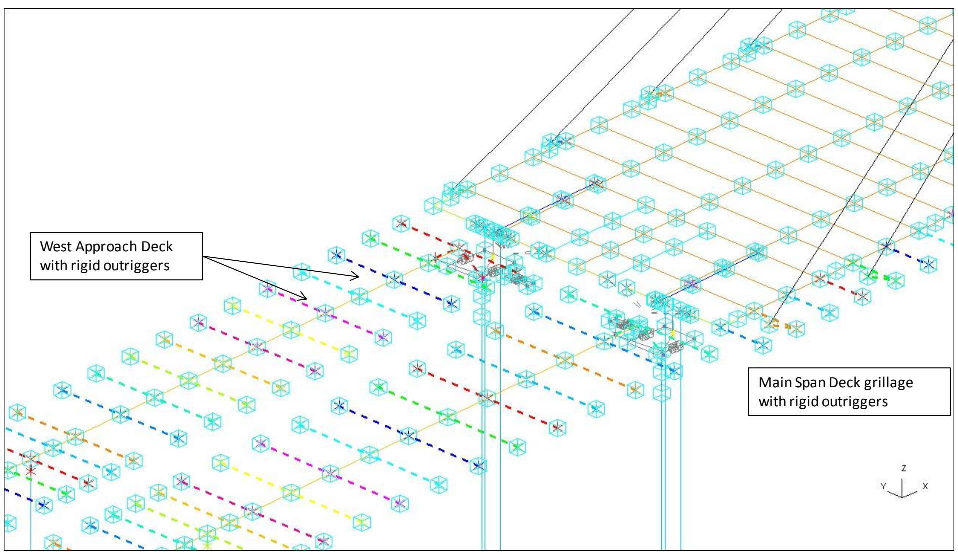


**Figure 4: LS-DYNA model – Bridge superstructure**

2- Modelling of the columns

The Towers, End Bents and Approach Columns were all modeled by one dimensional beam elements.

In particular, the tapered cross sections of the columns were accurately represented in the LS-DYNA model.

A fiber beam approach was adopted for the plastic hinge zones at the top and bottom of the Approach Bridge columns as well as for the entire length of the End Bents and Towers in order to more accurately capture non-linear cyclic moment-curvature behavior. Figure 5 describes the damage formation zones on the Approach columns and Figure 6 illustrates the extent of section fiber modeling in the End-Bents and Towers.

Plastic hinge lengths were calculated following [2].

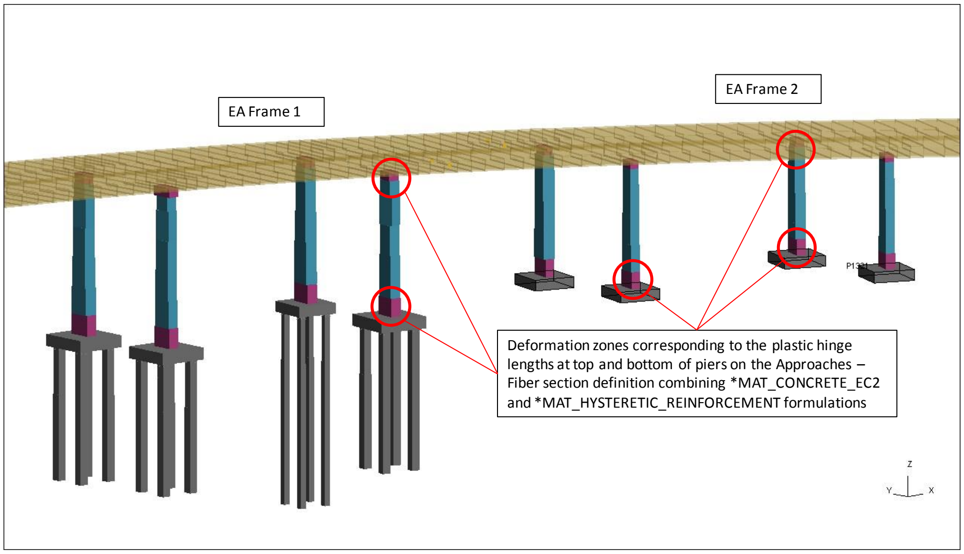


**Figure 5: Plastic hinge zones and fiber modeling on the Approach piers**

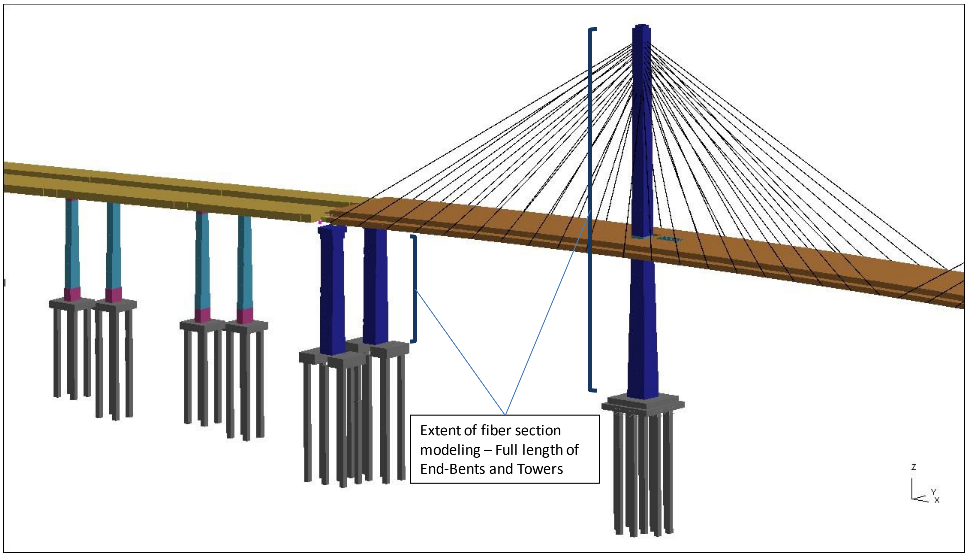


**Figure 6: End-Bent and Tower fiber sections**

The mid section of Frames 1 and 2 piers on the Approaches (cf. Figure 12) were represented with linear elastic material. More sophisticated material models were assigned to the fiber section regions.
**MAT_CONCRETE_EC2* and **MAT_HYSTERETIC_REINFORCEMENT* are particularly well suited to simulate the nonlinear hysteretic behavior of respectively confined/unconfined concrete and steel reinforcement in LS-DYNA.

**MAT_CONCRETE_EC2 (MAT_172)* includes concrete cracking in tension and crushing in compression. Non-thermally sensitive stress-strain relationship options have been selected (Parameter TYPEC=3 or 6) to match the various material properties (respectively parabolic-rectangular stress-strain or Mander's equations). A typical aggregate size of 20mm was taken into account to model interlock and to allow cracked concrete fibers to carry shear loading.

Tension capacity (from 3.2 to 4.48MPa depending on concrete grade) was taken into account to prevent premature loss of shear capacity.

**MAT_HYSTERETIC_REINFORCEMENT (MAT_203)* can be used in conjunction with **MAT_CONCRETE_EC2* in a fiber defined section in order to accurately model the nonlinear hysteretic behavior of steel rebars. Stress-strain curves for reinforcement steel in the LS-DYNA model accurately replicated the Caltrans Seismic Design Criteria Park Model for reinforcement. Extensive validation of the fiber section modelling in conjunction with these materials was performed before deployment on the full bridge analysis.

3- Modelling of pilecaps and piles

The pile groups at the End Bents and Towers as well as Approach Frames immediately adjacent to the Main Span Bridge were modeled explicitly. Pile group geometry, pile spacing, section and material properties reflected the GDB final design.

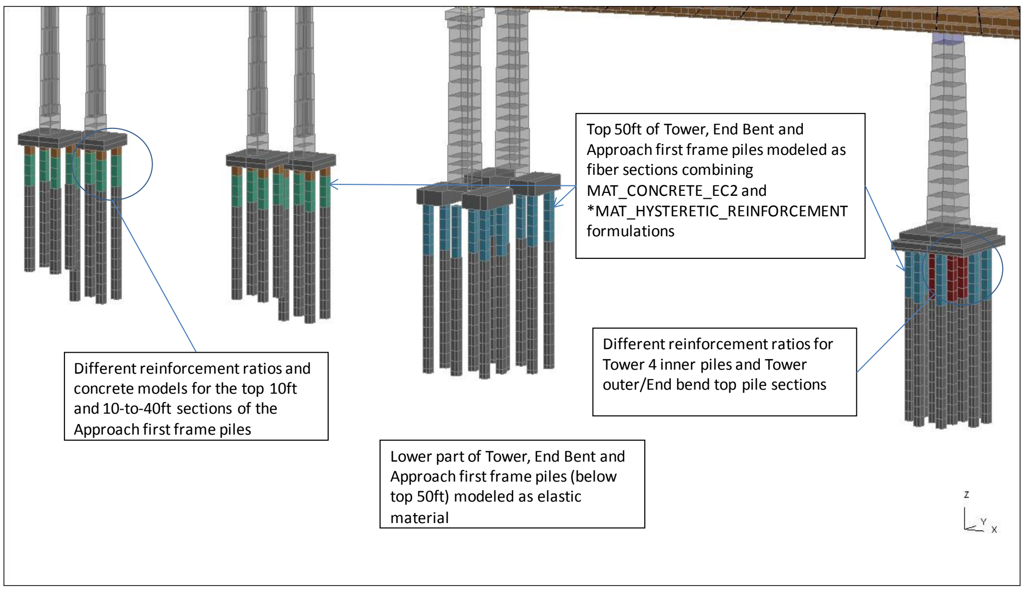


**Figure 7: Explicit pile modeling – Tower, End Bent and Approach foundation details**

Mass compensation and buoyancy effects for the buried piles and pilecaps were considered. The top 50 feet of the Tower, End Bent piles and the top 40 feet of the first Approach frame piles were also defined as fiber sections. A combination of **MAT_CONCRETE_EC2* and **MAT_HYSTERETIC_REINFORCEMENT* was adopted to represent accurately the moment-curvature behavior and to monitor strain demands.

4- Modelling of pilecaps and piles

As required by [2], the interaction between soil and pile elements was modeled by elasto-plastic springs. As illustrated in Figure 8, each node of an explicit pile beam-column element was connected to spring elements modeling the horizontal (p-y elements) and the vertical (t-z springs, skin friction) interactions between the piles and the foundation soil. The free-end of the each foundation spring received the ground motion velocity excitations. For each soil layer, the elasto-plastic spring stiffness and yielding force (passive soil resistance) were modeled in accordance with the results of geotechnical surveys and estimates.

Different pile group effects, for axial and vertical displacements, were taken into account in the definition of the soil nonlinear springs:

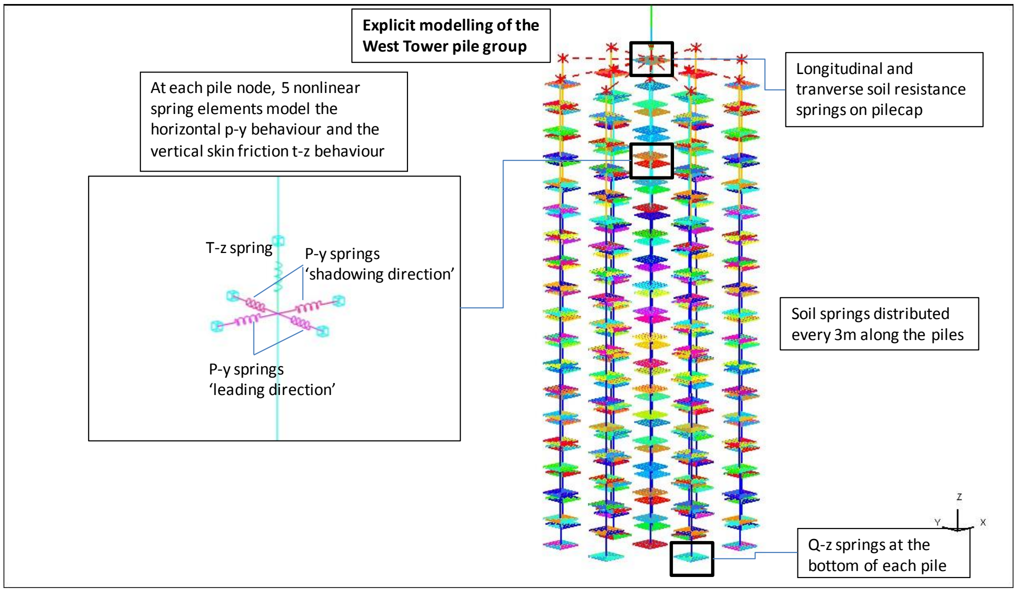


**Figure 8: Modelling of soil springs**

The soil resistance effect on buried or partially buried pilecaps was explicitly represented by means of elastic-plastic springs in both transverse and longitudinal directions.

5- Modelling of viscous dampers

The viscous dampers that control the relative displacements between Tower/Main Span deck and between End Bent/Main Span deck were modeled by nonlinear damper elements in LS-DYNA. Viscous damper elements are rigidly connected to the structure.

Their constitutive law is $F=CV^{\alpha}$ with $\alpha=0.3$ based on supplier's data

Where F: force [kip]

V: velocity [in/s]

C= Constant $[kip/(in/s)^{0.3}]$

Figure 9 gives an example of F=f(V) characteristics.

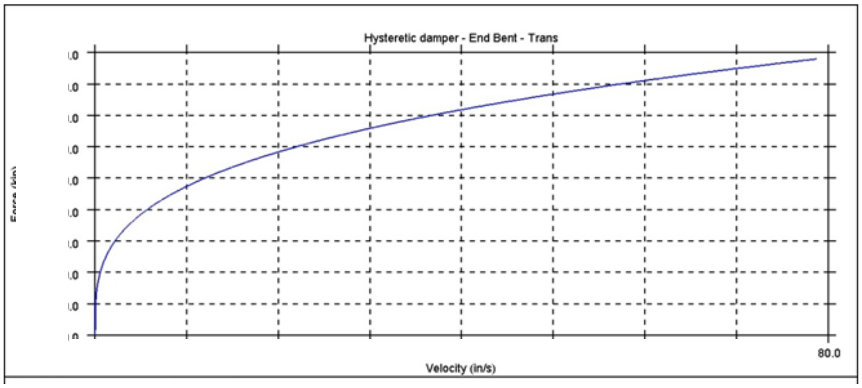


**Figure 9: End Bent transverse damper properties**

Figures 10 and 11 show the damper arrangements at the Main Tower and End Bent.

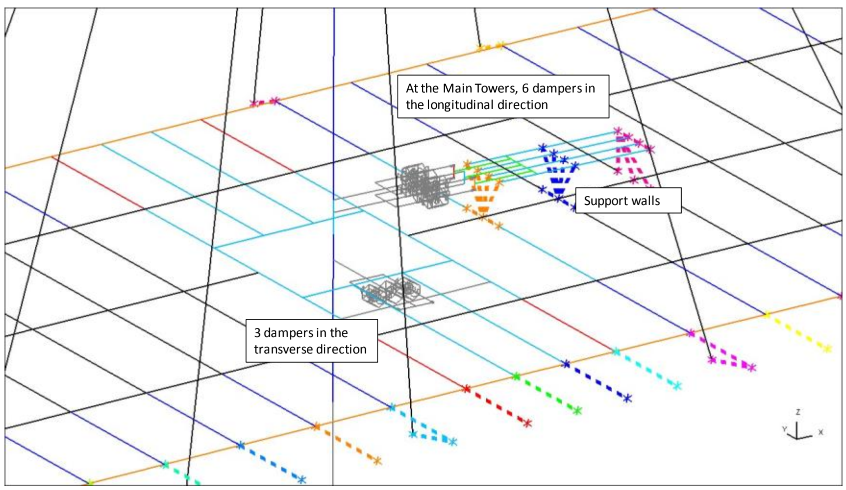


**Figure 10: Damper arrangement at the Towers**

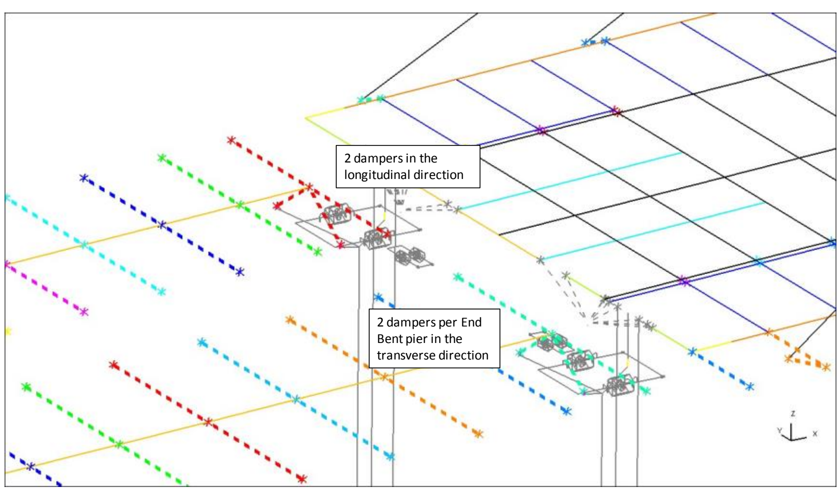


**Figure 11: Damper arrangement at the End Bents**

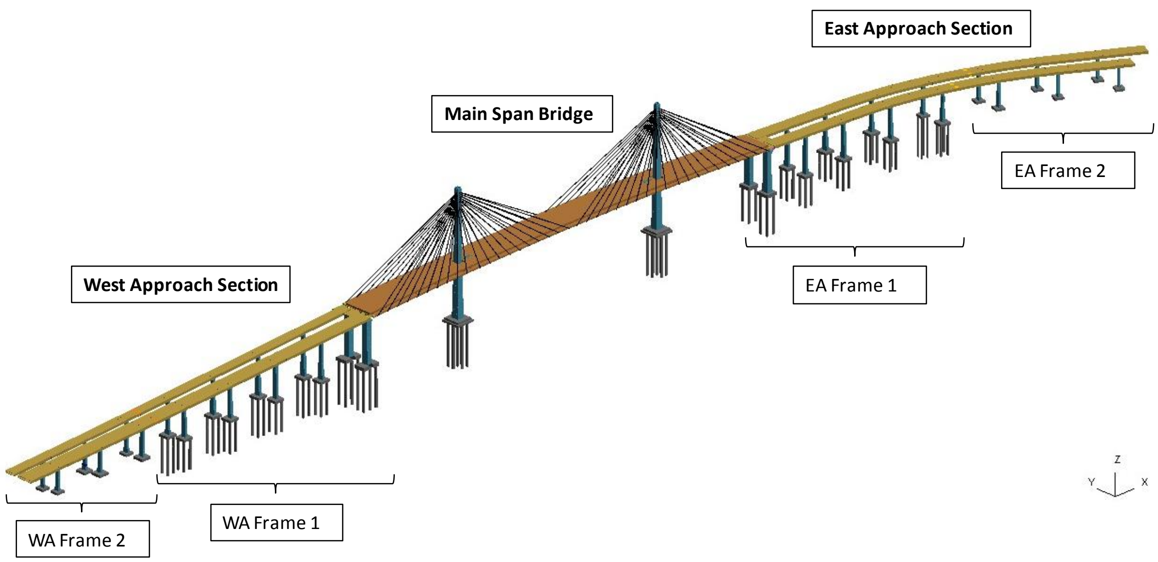


**Figure 12: Global analysis model**

## Analysis procedure and typical results

1- Prestress and damping

Prestresses in the cable elements to achieve the desired permanent load condition were incorporated at the start of the seismic analysis. Element forces and node displacements due to gravity and preloads formed the initial state for the nonlinear seismic response history analysis.

As prescribed in [2], a Rayleigh damping method was applied to the LS-DYNA model.

For the superstructure elements, where the degree of inelastic behavior is very limited, a damping level of 1.5% of critical was adopted.

In the 'Moderate Damage' zones, concrete cracking, rebar yielding hardening and hysteretic energy dissipation was automatically represented by the material formulation. Only the damping required in the small deformation range – before cracking and yielding – needed to be externally applied, by means of 1.5% Rayleigh damping also.

2- Results

Seismic demand envelopes for all critical components were calculated. A selection of typical results is given below:

Main span Bridge-Approach maximum relative displacement
For the optimized final design, the longitudinal displacements between Main Span Bridge and Approach superstructures were all within the 6.0' limit which is set by the gap between the structures (no pounding predicted).

| MSB to Approach relative displacement | | | | |
|---|---|---|---|---|
| | Envelope [ft] | | | |
| | Longitudinal direction | Transverse direction | Vertical direction | Resultant |
| **SW** | 4' 6'' | 4' 11'' | 6'' | 5' 5'' |
| **NW** | 4' 10'' | 5' 5'' | 5'' | 5' 5'' |
| **SE** | 3' 4'' | 5'' | 5'' | 5' 5'' |
| **NE** | 3' 3'' | 4' 10'' | 5'' | 5' 5'' |
| **Max** | 4' 10'' | 5' 5'' | 6'' | 5' 5'' |
| **Target** | 6' | N/A | N/A | N/A |

**Table 1: MSB-Approach relative displacements – SEE – Envelope results**

Viscous damper strokes

Simulation results for damper strokes and total displacements were also used to derive specifications. Table 2 summarizes Tower and End Bent bearing demands.

| Damper strokes and displacements | | | |
|---|---|---|---|
| | Envelope [ft] | | |
| | Direction | Stroke | Displacement along main axis |
| Tower longitudinal | W | 1' 9'' | 1' 9'' |
| | E | 2' 1'' | 2' 1'' |
| | Max | 2' 1'' | 2' 1'' |
| | Limit | **2' 8''** | **2' 8''** |
| Tower transverse | W | 2' 2'' | 2' 2'' |
| | E | 2' 1'' | 2' 1'' |
| | Max | 2' 2'' | 2' 2'' |
| | Limit | **2' 8''** | **2' 8''** |
| End Bent longitudinal | W | 2' 9'' | 2' 9'' |
| | E | 2' 6'' | 2' 6'' |
| | Max | 2' 6'' | 2' 9'' |
| | Limit | **3' 4''** | **3' 4''** |
| End Bent transverse | W | 1' 3'' | 1' 3'' |
| | E | 1' 2'' | 1' 1'' |
| | Max | 1' 3'' | 1' 3'' |
| | Limit | **1' 8''** | **1' 8''** |

**Table 2: Damper strokes and displacements – SEE – Envelope results**

Seismic demands on the End-Bents at specific elevations

The capacities of the structural elements were thoroughly checked against envelope demands. Table 3 gives an example of demand envelopes on End Bent column elements at critical elevations. For the optimized final design, these demands are within component capacity limits.

| **West End Bents** | **Axial force [kip]** | | **Shear forces [kip]** | | **Bending moments [kip.ft]** | | **Torsion [kip.ft]** |
|---|---|---|---|---|---|---|---|
| **Tower elevations** | **Min C** | **Max C** | **Fy** | **Fz** | **Myy** | **Mzz** | **Mxx** |
| **West EB North** | | | | | | | |
| Top of Pile Cap | 4131 | -15598 | 7012 | 6218 | 421169 | 457560 | 39282 |
| Top of Plastic Region | 1621 | -16243 | 3802 | 4562 | 318884 | 413602 | 28537 |
| Change of Wall T | 1304 | -15165 | 3410 | 3452 | 245261 | 327440 | 23765 |
| Change of Bar A, B & D | 767 | -13549 | 3476 | 3941 | 160139 | 195579 | 15169 |
| Bottom of Pier Head | 164 | -12120 | 3391 | 3851 | 76787 | 105331 | 11864 |
| Top of Pier Slab | -12 | -11677 | 2635 | 3596 | 76950 | 78860 | 10037 |
| **West EB South** | | | | | | | |
| Top of Pile Cap | 3351 | -17767 | 5212 | 7327 | 421221 | 424665 | 31360 |
| Top of Plastic Region | -73 | -18963 | 3552 | 4675 | 320617 | 362791 | 23597 |
| Change of Wall T | -1003 | -17277 | 2990 | 3251 | 271208 | 296388 | 19575 |
| Change of Bar A, B & D | -1438 | -15861 | 3212 | 3705 | 172840 | 209415 | 16293 |
| Bottom of Pier Head | -1692 | -14595 | 3100 | 3387 | 108488 | 124662 | 11515 |
| Top of Pier Slab | -2125 | -13846 | 2616 | 3077 | 113274 | 117405 | 9731 |

**Table 3: Seismic demands – SEE envelope – West End Bent elements**

Plastic strains in Tower and End-Bent pile sections

Strain predictions in key components were compared to the design limits set by [2]. Table 4 illustrates the computed strain peak levels in concrete and steel rebar elements both in the Towers and End Bent columns. Peak strains levels in all section fibers meet the Design Basis Criteria.

| Pile fiber sections | Maximum concrete strain | Maximum reinforcement strain |
|---|---|---|
| West Tower | 0.002 | 0.005 |
| East Tower | 0.002 | 0.006 |
| West End Bent N | 0.002 | 0.006 |
| West End Bent S | 0.002 | 0.006 |
| East End Bent N | 0.002 | 0.004 |
| East End Bent S | 0.002 | 0.004 |

**Table 4: Plastic strain - SEE envelope – Tower and End Bent pile sections**

## Conclusions

The deployment of advanced LS-DYNA capabilities was instrumental in assessing the design of the Gerald Desmond Replacement Bridge and ultimately passing the Expert Panel Reviews.

The detailed LS-DYNA model of the structure allowed the design team to accurately represent, analyse and validate innovative solutions:

- The fuse-viscous damper system effectively isolates the Main Bridge Deck during the SEE seismic event.
- The adopted beyond-the-code approach allows for the design of the ductile hollow sections of the Towers and End-Bent and Approach columns.

Automated model building and postprocessing tools were developed and successfully employed to treat the multiple loadcases and successive design iterations in the most productive way.

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