

INNOVATIVE SOLUTIONS TO SEISMIC DESIGN CHALLENGES ON THE MOCOCO OVERHEAD PROJECT

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ABSTRACT

The Mococo Overhead, is one of several bridges that comprise the network of structures crossing the Carquinez Strait between the cities of Benicia and Martinez in the San Francisco Bay Area of California. This is one of only two sites with bridges designated as “Important” by the California Department of Transportation, the other being the San Francisco-Oakland Bay Bridge complex of structures. “Important” bridges are designed to a higher level of performance than “Ordinary” bridges, requiring essentially elastic performance under a Functional Evaluation Level Earthquakes (FEE), and limited damage that can be repaired with a minimum risk of loss of functionality and impacts to traffic under the Safety Evaluation Level Earthquake (SEE). The Mococo Overhead is located amid an array of roadways, utilities, and railroad lines and sits in a “bathtub” of soft mud overlaying bedrock of varying type, quality and orientation. This paper describes the numerous challenges associated with this project including the site conditions, complicated geometry, environmental restrictions, hazardous and contaminated soil, permit constraints, geotechnical complexity, heavy railroad traffic, and a high level of seismicity. This paper discusses a number of innovative solutions which were developed by the project development team in order to address these challenges while meeting the stringent project seismic design requirements.

INTRODUCTION

Owners may require higher levels of seismic performance at selected bridge sites in order to enhance emergency response capabilities and to limit impacts to local economies after a significant seismic event. The Mococo Overhead, on Interstate 680 in the Bay Area of northern California, was designated with other bridges crossing the Carquinez Strait between the cities of Benicia and Martinez as an “Important” bridge by the California Department of Transportation. This designation required the bridge to meet enhanced performance requirements at a site posing a number of seismic design challenges. These included the array of roadways, utilities, and railroad lines crossing the site as well as the complex geology with the bridge sitting in a “bathtub” of soft mud overlaying bedrock of varying type, quality and orientation. Other challenges included the complicated geometry, environmental restrictions, hazardous and contaminated soil, permit constraints, geotechnical complexity, heavy railroad traffic, and a high level of seismicity.

BRIDGE SITE LOCATION

The Mococo Overhead site is located adjacent to the Carquinez Strait at the north end of the San Francisco Bay on Interstate 680, connecting the Silicon Valley to Interstate 80 in Fairfield. The existing bridge carries six lanes of traffic with three lanes in both the northbound and southbound directions. The new bridge is located directly adjacent to the east side of the existing structure, and will ultimately carry northbound traffic to the Toll

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Plaza leading to the new Benicia-Martinez Bridge, with the existing bridge being modified and converted to carry southbound traffic exclusively.

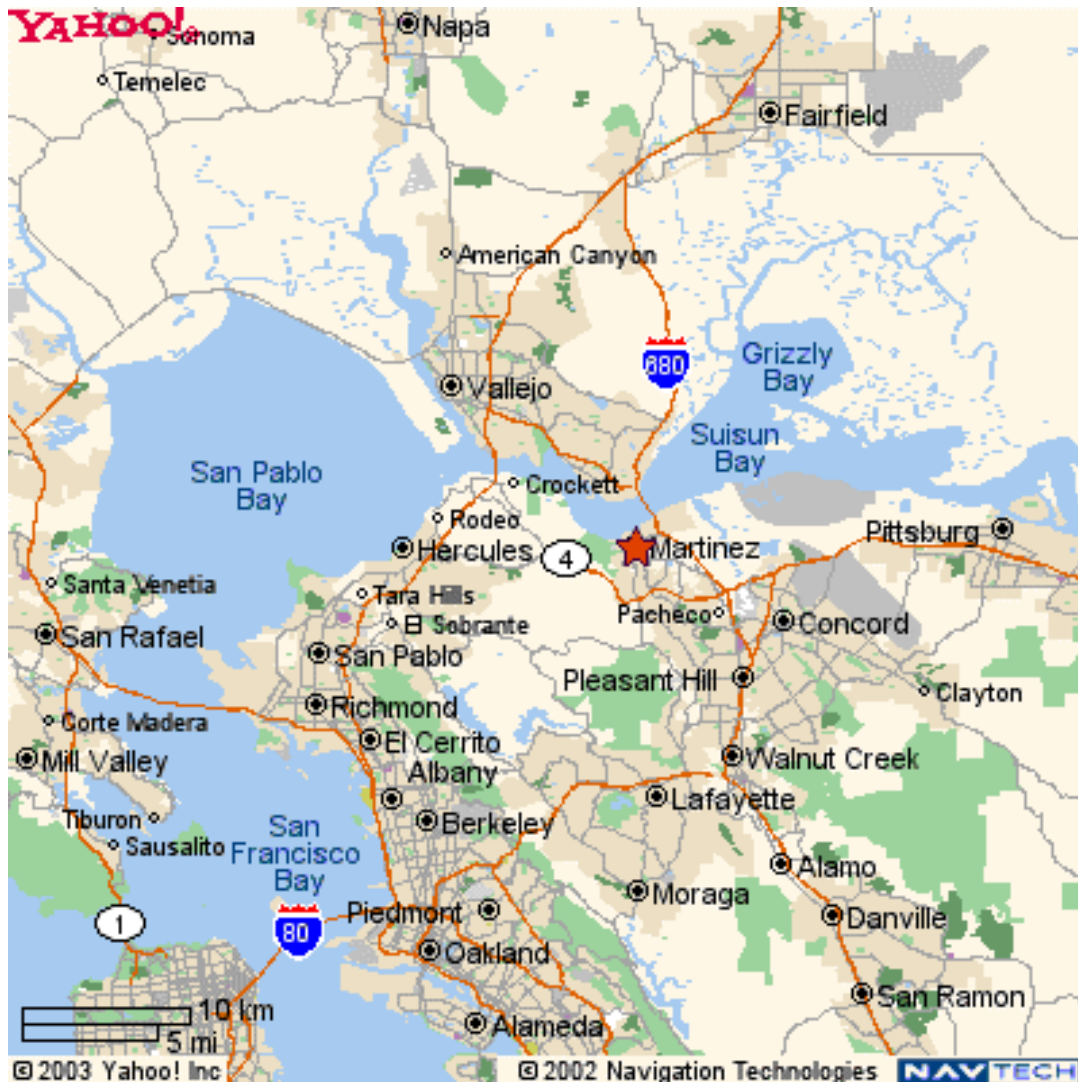


FIG 1 BRIDGE SITE LOCATION

Existing conditions at the site imposed constraints affecting the structural configuration, and led to constructability considerations that affected decisions on foundation and superstructure types selected for the project. Local two-lane roads and multiple highly active railroad lines with both commuter and industrial traffic cross underneath the bridge site at varying alignments. Both the roads and the rail lines were required to remain in service during construction, affecting the windows of time available for construction activities. Multiple public and private utilities, including high pressure gas lines, aviation fuel, and oil lines were located both above and below grade at multiple locations throughout the site. In addition a sanitary sewer main and other utilities, some very difficult to locate, criss-crossed the site. This array of infrastructure underneath the bridge put constraints and limitations on locations to place bridge columns and foundations.

The site is located in a brackish marshland, which resulted in environmental restrictions and mitigation requirements designed to protect the wetlands and native flora and fauna. Juxtaposed with the environmental requirements, the numerous oil pipelines and other utilities at the site have led to ground water contamination and hazardous soil conditions requiring special construction practices. Seasonal flooding of the marshland along with tidal action keeps the site alternatively wet or underwater, with a water table just above or just below the ground surface. These conditions made construction access requirements difficult to predict prior to the start of construction, requiring a design that provided flexibility to the contractor depending on the actual site conditions encountered and preferred methods to address those conditions.

GEOLOGICAL CONDITIONS AND SEISMICITY

The geology of the site varies dramatically along the length of the bridge. The bedrock, which consists of fresh to intensely weathered and fractured claystone, siltstone, sandstone and shale, creates a “bathtub”, high at the surrounding edges near the north and south ends of the bridge. The bedrock is as shallow as 4m below the ground surface at one edge of Abutment 1, and is as deep as 20m at Bent 5. The characteristics of the rock varied along the dip, which was as steep as 50-70 degrees and was not uniform. At some locations weaker fractured rock was located below fresher harder material. Due to the high dipping angles of the bedding planes, the type of rock could vary significantly within a short lateral distance. The great amount of variation and unpredictability required an in depth very extensive geotechnical investigation at the site. The soil layers above the bedrock consist of very soft to soft highly organic silty clay and silty clay with sand and peat lenses which becomes stiffer and denser with depth. The bearing capacity at the ground surface varied from 0.015 to 0.075 MPa. The soil was prone to caving and sloughing during drilling. Groundwater was located at elevation 0.0m and varied with the tide. The upper layers of the soil are considered hazardous or contaminated. The site was tested and found to be corrosive due to low pH, high sulfate and chloride concentrations

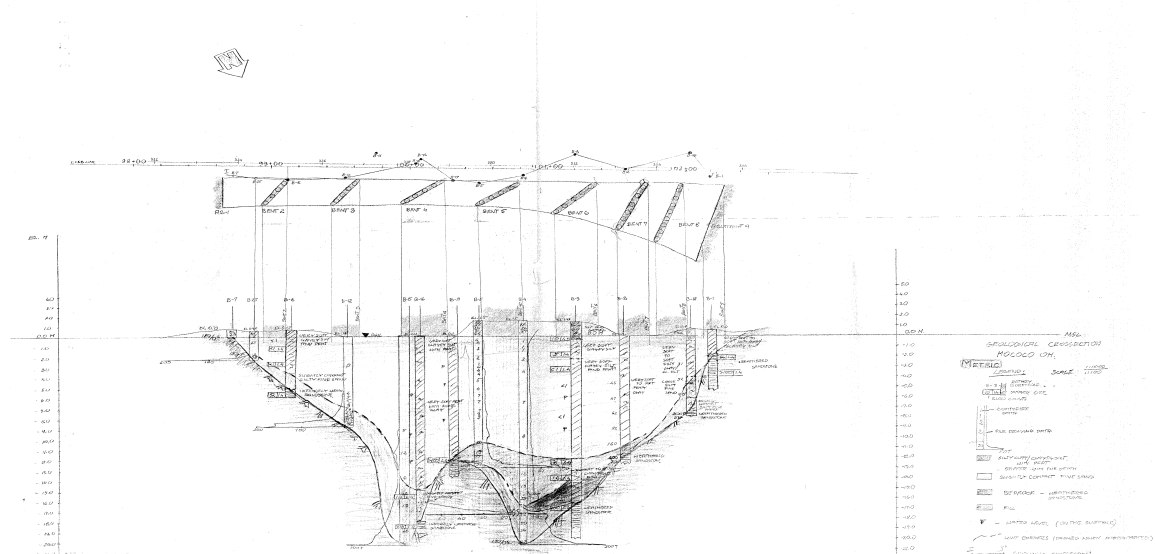


FIG. 2 BEDROCK PROFILE

The site has a high seismicity with the controlling Green Valley fault located 2.5 km away with a Moment Magnitude of 6.75. The soft soil will amplify structural response thru soil-structure interaction effects, requiring consideration using enhanced seismic analysis techniques. The “bathtub” complicates the effects because different soil layer depths at each support tends to drive diverse seismic response.

SEISMIC DESIGN CRITERIA REQUIREMENTS

The Mococo Overhead is located at one of only two sites with bridges designated as “Important” by the California Department of Transportation, the other being the San Francisco-Oakland Bay Bridge complex of structures. “Important” bridges are designed to a higher level of performance than “Ordinary” bridges, requiring essentially elastic performance under a Functional Evaluation Level Earthquakes (FEE), and limited damage that can be repaired with a minimum risk of loss of functionality and impacts to traffic under the Safety Evaluation Level Earthquake (SEE). This is summarized in Table 1, taken from Caltrans guidance material.

TABLE 1

Seismic Performance Criteria		
Ground Motion at Site	Level of Damage and Post Earthquake Service	
	Ordinary Bridge	Important Bridge
Functional – Evaluation Ground Motion	Service: Immediate Damage: Repairable	Service: Immediate Damage: Minimal
Safety-Evaluation Ground Motion	Service: Limited Damage: Significant	Service: Immediate Damage: Repairable

Definitions:

Functional - Evaluation Ground Motion: This ground motion may be assessed either deterministically or probabilistically. The determination of this event is to be reviewed by a Caltrans-approved consensus group.

Safety - Evaluation Ground Motion: This ground motion may be assessed either deterministically or probabilistically. The deterministic assessment corresponds to the Maximum Credible Earthquake (MCE). The probabilistic ground motion for the safety evaluation typically has a long return period (approximately 1000-2000 years).

MCE: The largest earthquake that is capable of occurring along an earthquake fault, based on current geologic information as defined by the 1996 Caltrans Seismic Hazard Map.

Service Levels:

- Immediate: Full access to normal traffic is available almost immediately following the earthquake.

- Limited: Limited access (e.g. reduced lanes, light emergency traffic) is possible within days of the earthquake. Full service is restorable within months.

Damage Levels:

- Minimal: Essentially elastic performance.
- Repairable: Damage that can be repaired with a minimum risk of losing functionality.
- Significant: A minimum risk of collapse, but damage that would require closure to repair.

The Safety-Evaluation Level Earthquake (SEE) is the Maximum Credible Earthquake for the given site. The earthquake level represents an approximately 1000-year return period. Given the SEE, the criteria require immediate full access to normal traffic and repairable damage after the event. Thus, demands on the structural sections are limited to 67% of the calculated concrete ultimate compressive strain and/or 67% of the rebar steel ultimate tensile strain.

The Functional-Evaluation Level Earthquake (FEE) criteria require immediate full access to normal traffic and repairable damage after the event. Thus, demands on the structural sections are limited to 33% of the concrete ultimate compressive strain and/or 33% of the rebar steel ultimate tensile strain.

Consistent with Caltrans seismic design philosophy, the project specific Mococo Overhead seismic design criteria adopts a deformation ductility approach to seismic design of bridges [1]. The design philosophy prescribes columns/piles to deform inelastically at predesignated locations without significant degradation of strength or stiffness under earthquake demands. The superstructure is designed to remain elastic during the seismic event. The project specific Mococo Overhead seismic design criteria included the following fundamental concepts taken from the Caltrans Seismic Design Criteria (SDC).

The column displacement capacity is calculated based on the section curvature capacity determined by moment-curvature analysis. For this “Important” structure, the curvature capacity is determined by the limited strain requirements assigned for the SEE and FEE events.

The SDC prescribes a balanced period practice in which stiffness and mass distribution of each bent and frame are relatively similar. The practice reduces out-of-phase and complex nonlinear response between columns, bents, and frames that are difficult to predict by analysis.

Force demands on components not designed for ductile seismic response are determined using capacity design principles. Thus demands placed on capacity protected components such as girders, bent caps and foundations can be designed to remain essentially elastic. In addition, expected shear demands placed on the columns or pile shafts can be more reliably determined.

BRIDGE DESCRIPTION

The structure geometry was controlled by the alignment of the existing roadway, rail lines, and numerous utilities in place, with a length of 364m and span lengths varying from 40 to 59m. A cantilever span of 18m is located between two separate bridge

frames. The infrastructure complexity restricts the location available to place foundations and thus results in variable bent skew angles. The variation in bent skew angles creates significant differences in span lengths as well as girder lengths within the same span. The bridge deck surface widens significantly along the length of the bridge, increasing from 25 to 80 meters. The large flare at the end of the bridge is due to alignment requirements associated with the distribution of traffic leading to the Toll Plaza.

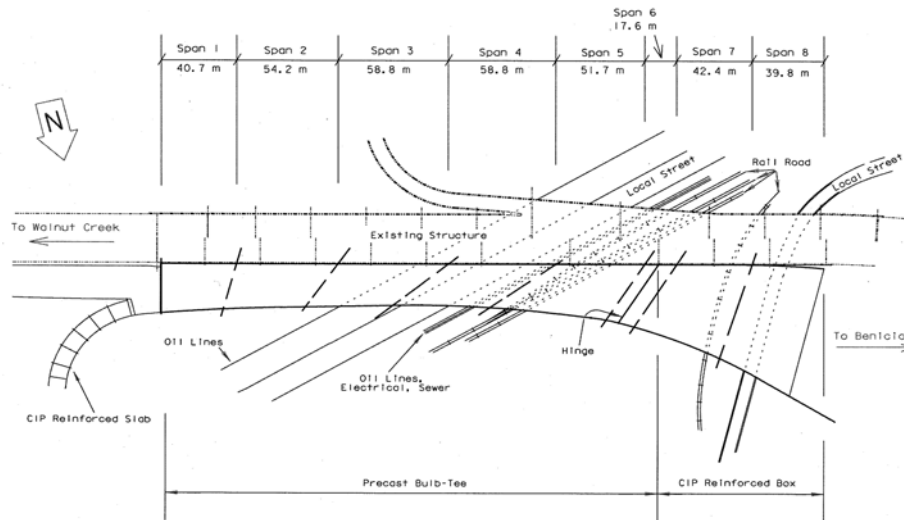


FIG 3 BRIDGE LAYOUT

The superstructure was originally designed as a precast Bulb-T girder for most of the length of the bridge. This superstructure type was selected due to concerns about permit restrictions that had not yet been resolved during the design phase of the project, restricting the use of falsework at the site. There were also constructability concerns associated with cast-in-place superstructure construction at the site. Following extensive contractor outreach efforts, it was determined that while site constraints imposed by the soft soil conditions, traffic requirements, railroad schedule, and protective measures required for the extensive utilities at the site, cast-in-place construction was feasible if the permits could be obtained. Therefore preliminary calculations were made for both precast and cast-in-place superstructure types. The properties of the precast Bulb-T superstructure were found to have a mass that was within the range of mass expected for a cast-in-place concrete box girder bridge. In lieu of developing alternative designs, to maximize the number of bidders and to obtain the most cost-effective bids, the contract was written to allow the contractor to modify the superstructure type as long as the mass was within 10% of the original design. Due to the complex seismic analysis required for the project, it was determined that the design should be developed such that if a substitute superstructure were selected by the contractor, no additional post-bid seismic analysis should be required by the contractor. This was intended to avoid lengthy impacts to the construction schedule. Ultimately the permit issues were resolved after the completion of the original design. During the construction phase the Contractor elected to modify the superstructure type to a cast-in-place prestress box girder in Frame 1, with Frame 2 remaining as a reinforced concrete box girder as originally designed.

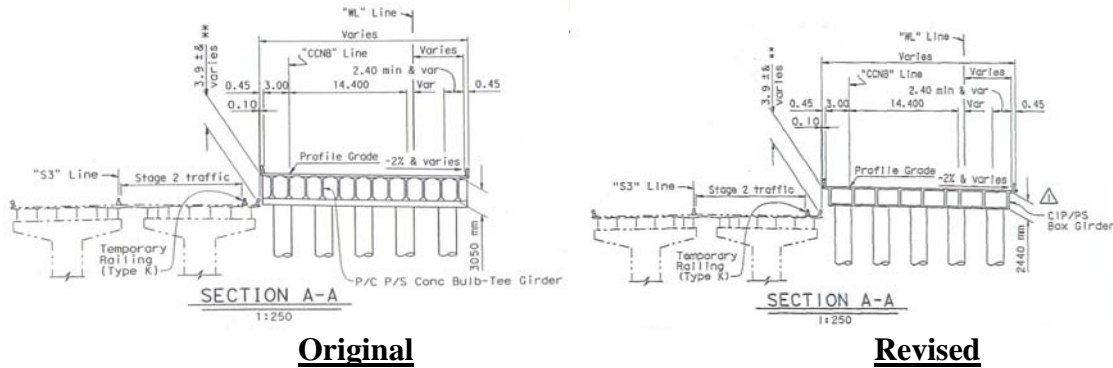


FIG 4 SUPERSTRUCTURE CROSS-SECTIONS

To ensure the seismic performance would not be significantly affected as long as the superstructure mass remained essentially constant, the superstructure and substructure were decoupled through the use of telescoping steel pipe pins at the tops of the columns. This type of pin was used in lieu of conventional clustered rebar pins in order to achieve a performance more closely approximating a true zero moment connection. This also limited joint shear demands transferred into the bent cap, which had details that would vary significantly for the two different superstructure types. This detail was also successful in controlling the loads transferred from the column into the superstructure under seismic loading.

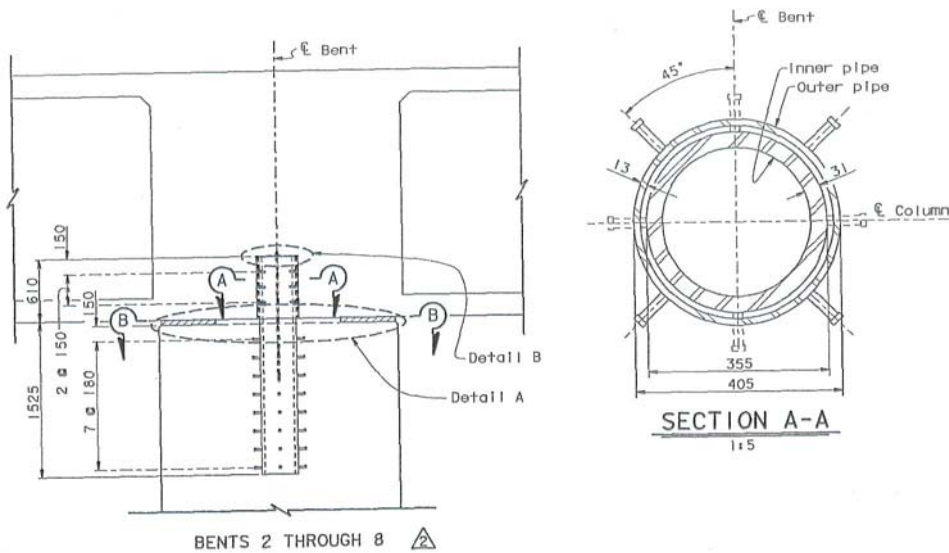


FIG 5 TELESCOPING PIPE PIN DETAIL

Pileshafts were selected due to the limited foundation footprint available due to the utilities crossing the site and the ability to cost-effectively tune the period of the structure to achieve a balanced mass/stiffness distribution. The extended length of the column/pileshaft rebar cages required splices in the main reinforcement. Based on potential plastic hinge locations identified by the designer, and a sensitivity analysis to

determine potential alternative plastic hinge locations, the plans identified allowable column reinforcing steel splice regions. These splices were required to meet “ultimate” splice specifications which require necking of the rebar away from splice under production testing.

DESIGN MEASURES

The soft soil conditions and the “bathtub” rock profile create complex seismic soil-structure interactions and structural responses. Without special design measures, the greatly varying bedrock levels would force columns in each bent to form plastic hinges at vastly different depths and thus have different effective column lengths. This variation would cause a disparity in column lateral stiffness and capacities that translate into unbalanced and complex seismic response. This would tend to lead to inconsistencies in the level of damage associated with each column and a general degradation in overall performance. To reduce seismic response complexities and to ensure balanced response among columns and bents, it was necessary to use several innovative design techniques.

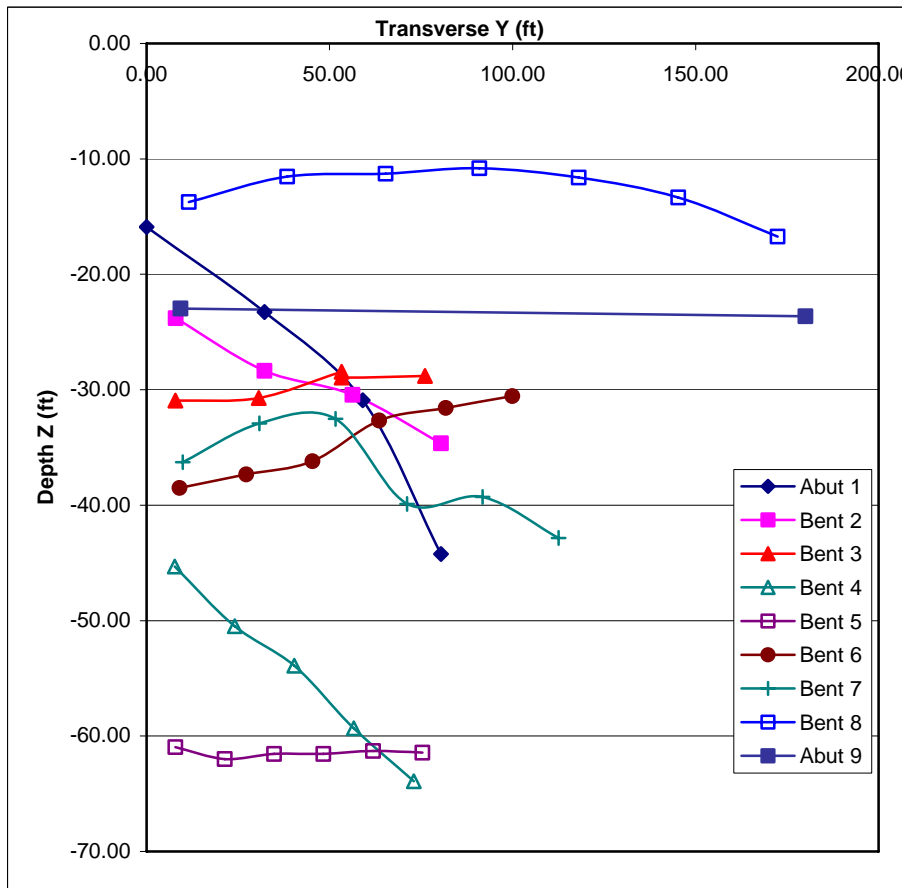


FIG. 6 ABUTMENT AND BENT BEDROCK PROFILE

Isolation casings were used at selected locations in order to increase the effective length of columns in shallow bedrock. In addition, the thickness of the permanent casings, adjusted for an assumed sacrificial thickness due to corrosive soil effects, vary at each bent from 25mm to 57mm to balance the structure stiffness and required strength.

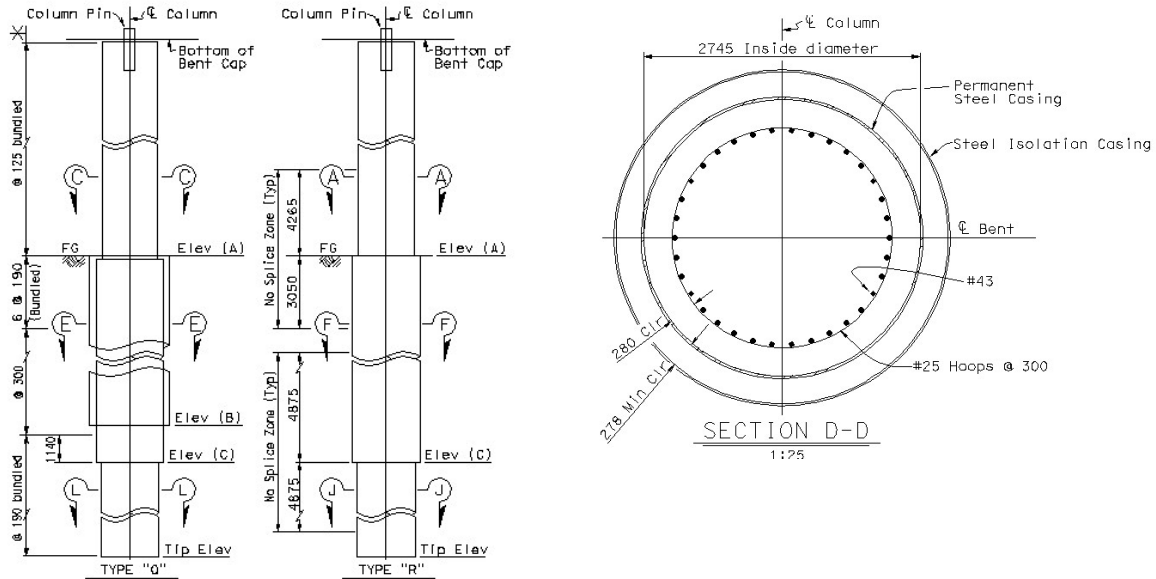


FIG 7 COLUMN/PILESHAFT

Isolation bearings were used at the abutments to further redistribute the inertial mass of the superstructure under seismic loads to achieve the desired performance. It was desired to minimize the force transfer at Abutment 1, thus PTFE spherical bearing bearings were used. However, due to the large amount of mass at the end of the bridge, lead-rubber isolation bearings were tuned to transfer some level of inertial mass at Abutment 9. No transverse shear keys were used at the abutments to maintain the mass/stiffness balance. Shear keys are typically used at bridge abutments to limit damage under moderate seismic events, however this will be achieved in part by the performance of the abutment isolation bearings, which will tend to be self-centering under an FEE seismic event.

The width of the bridge flares out dramatically at the end of the bridge. To limit the interaction between the Frame 1 (Abutment 1 to Bent 6) and Frame 2 (Bent 7 to Abutment 9) a double cantilever span was used creating a “seatless hinge” with neither frame dependent on the other for vertical stability. A hinge pipe restrainer was used to maintain vertical alignment under live load. The free end of the pipe is supported by a horizontally slotted formed hole to allow independent transverse movement between the two frames under seismic displacements.

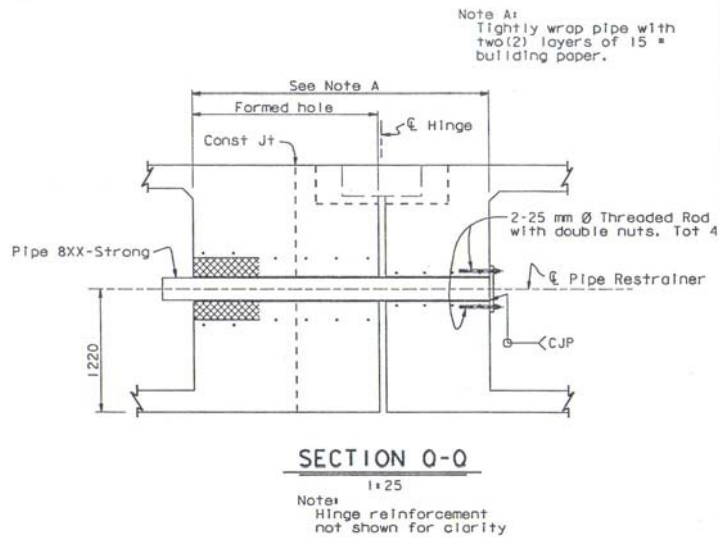
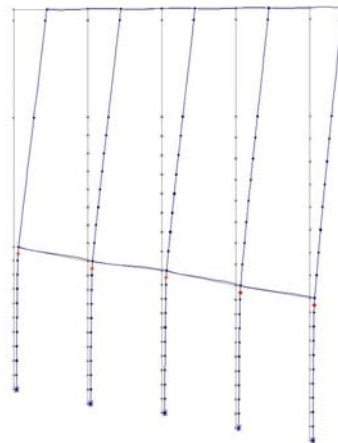


FIG 8 SPAN 6 SEATLESS HINGE CROSS-SECTION

ANALYSIS

For seismic evaluation, both simple and more complex analyses were used to validate the seismic design to ensure it met project performance requirements.

Nonlinear static pushover analyses were conducted on local frame models to study column plastic-hinging behaviors and capacities. The pushover analysis used plane-frame transverse model built with inelastic column hinges and nonlinear inelastic p-y soil springs. Each bent model was pushed until all column/pile hinges and subsequently reach the curvature limits set by FEE and SEE criteria. The subsequent design was



analyzed to ensure the balance-structure behavior.

FIG 9 NONLINEAR STATIC PUSHOVER – PLAN FRAME MODEL

Both linear response spectrum as well as nonlinear dynamic time-history analyses using three component ground motions were conducted on the global model to determine seismic demands on the structure and to verify that the structure satisfies the performance

criteria. The three-dimensional global bridge model consisted of a spine of linear frame elements representing the superstructure. Each column and pile was modelled with frame elements that use cracked section properties. The FEE seismic demands and criteria, requiring essentially elastic performance ultimately controlled the design. The time history model used elastic column and pile elements based on the assumption that seismic displacement demands would be comparable using elastic or inelastic structural properties.

Soil-structure interactions were modelled by connecting piles to nonlinear inelastic links simulating lateral (p-y) soil springs. In addition, the seatless hinge was connected by gap (compression only) springs that captured pounding effects and the isolation bearings were modelled by elasto-plastic/friction elements.

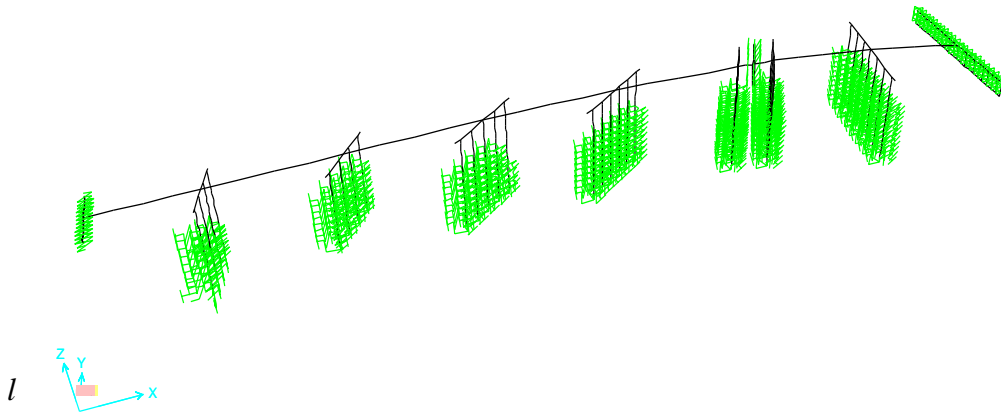


FIG. 10 NONLINEAR TIME-HISTORY DYNAMIC 3-D FRAME MODEL

Three separate FEE and SEE events were generated and ground motions from each event were specific based on location, soil depth, and wave direction. In this multi-support excitation condition, independent time displacement motions were forced into p-y springs along the length of each bent to determine seismic displacement demands.

After extensive trials, the final design yielded results that satisfied all the design requirements under the prescribed seismic events. Figure 11 compares the column capacity and demand displacements under the controlling FEE event. Analysis results indicated that all capacities exceeded the seismic demands and thus validated the seismic design.

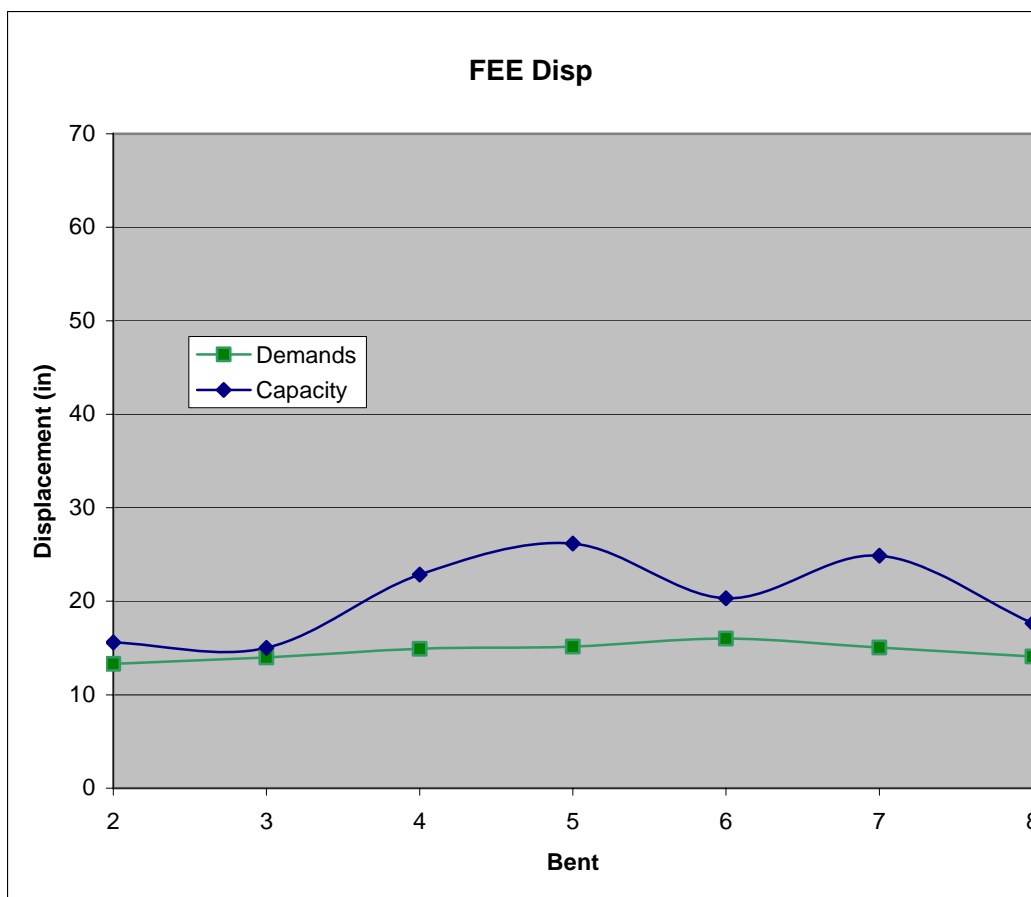


FIG. 11 FUNCTIONAL EVALUATION EARTHQUAKE DEMAND/CAPACITY

CONCLUSION AND ACKNOWLEDGEMENT

This project demonstrated the feasibility of utilizing nonlinear seismic analysis and innovative measures to design a bridge founded on soft soils with complex geotechnical and site conditions to meet the enhanced performance criteria required of an “Important” bridge as defined by the California Department of Transportation.

The author wishes to acknowledge and thank Paul Chung, Kevin Ross, Traci Holden, Kambiz Kouchesfahani and other Caltrans engineering and support staff for their invaluable work invested on this project. In addition, Bob Morris of Computers and Structures, Inc. was very helpful in assisting with the nonlinear analysis, and his efforts are greatly appreciated.

REFERENCES

- [1] Seismic Design Criteria- Version 1.2, Caltrans, Sacramento, California, 2001.
 - [2] Edward L. Wilson, Three Dimensional Static and Dynamic Analysis of Structures, Computers and Structures, Inc., Berkeley, California, 2000, 22 pp.
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