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AN IMPROVED METHOD FOR SEISMIC DESIGN OF HIGHWAY BRIDGES

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ABSTRACT

The damage to highway bridges during the recent California earthquakes has highlighted the need for improved seismic analysis and design procedures. This paper is a study of one such approach that aims to enhance the seismic performance of bridges. Chosen as an application of this approach is a 12-span viaduct that is part of the San Joaquin Hills Toll Roads project in Orange County, CA.

Current California Department of Transportation (Caltrans) and AASHTO design criteria require bridges to be designed for a maximum credible earthquake to safeguard against collapse. Damage prevention under moderate earthquakes is not ensured. This damage can not only lead to costly repairs but the cost is magnified many folds due to the closure of the highway and the resulting loss to commuters and businesses until the facility is repaired.

To avoid such a situation, a two level design approach has been proposed. This is a more rational approach to design and provides a higher and more uniform level of safety and reliability. Briefly, the design criterion requires two levels of analysis: one corresponding to a 72-year return period event and the other corresponding to a maximum credible level design event. Design forces and displacements resulting from the lower level event are used to ensure that no damage occurs as a result of this event. The forces and displacements resulting from the maximum credible analysis are used to ensure that the structure will not collapse.

In the paper this two level approach is used to design the column steel based on the column flexural ductility. Displacement ductility checks are then performed on the columns to ensure that they can undergo the inelastic deformations produced by the upper level spectrum loads.

KEYWORDS

Response spectrum; viaduct; demand; capacity; ductility; plastic hinge; column; reinforcement.

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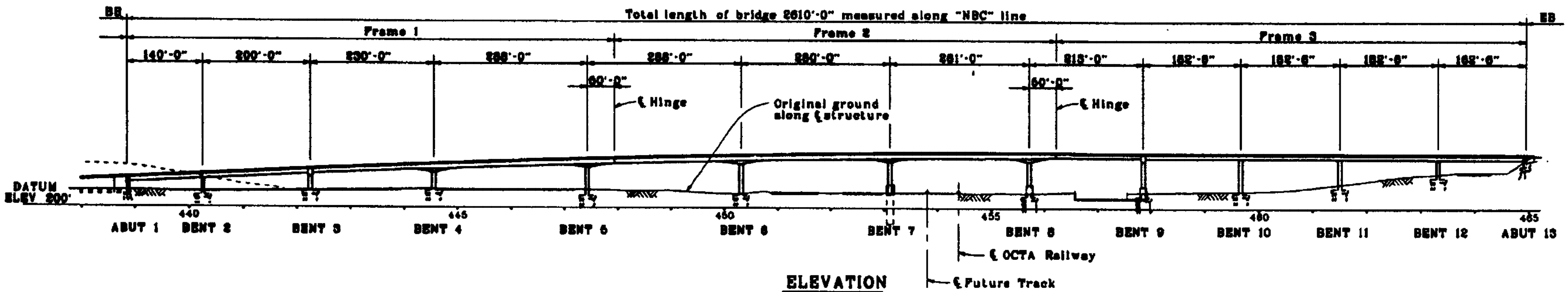
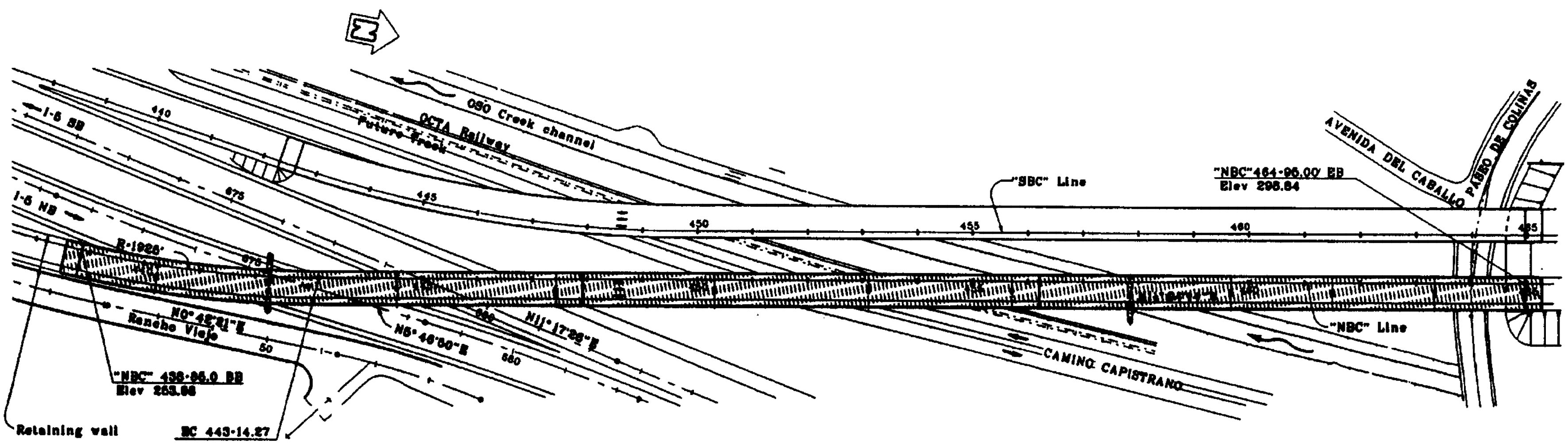


ELEVATION
1" = 100'



PLAN
1" = 100'

FIGURE 1 - GENERAL PLAN OF N6-N73 VIADUCT

LEGEND

Indicates limits of Northbound Connector

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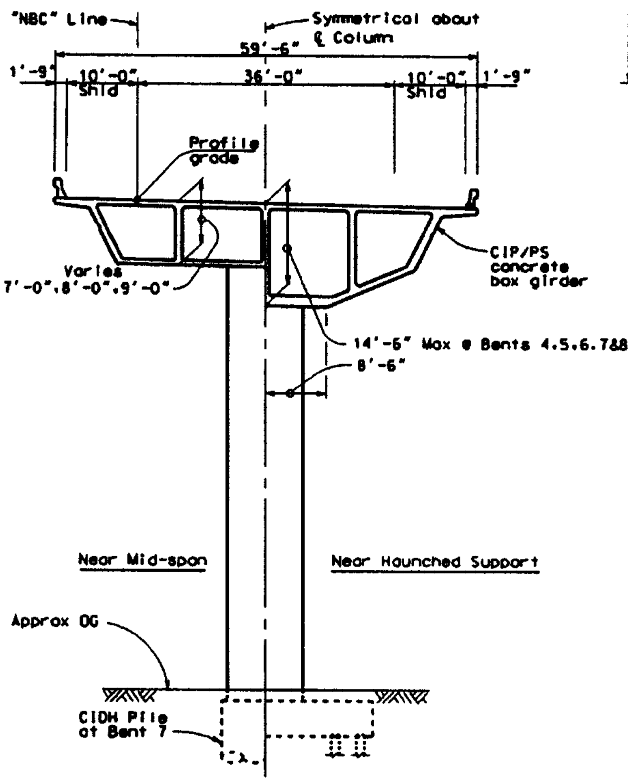

FIGURE 2 - TYPICAL SECTION

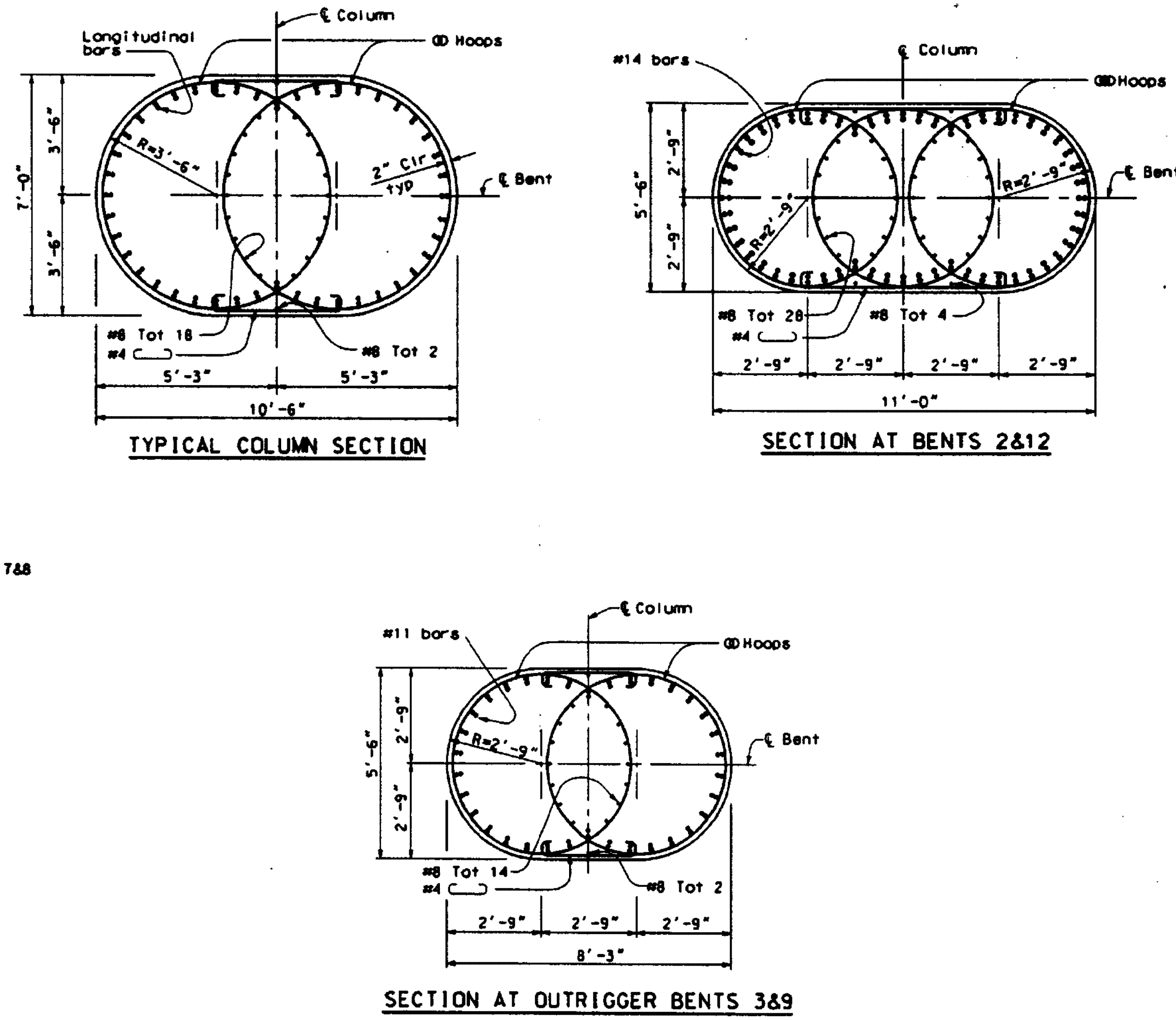

FIGURE 3 - COLUMN SECTIONS

CASE STUDY - "N5-N73 VIADUCT"

The bridge chosen for this study is located at the southern end of the San Joaquin Hills Toll Road (SR-73) where it connects to the I-5 in Orange County, CA. It is a 12-span viaduct traversing over local roads, railroad tracks and a water channel. Most bents have single columns except Bents 3 & 9 where outrigger beams are used to clear roadways and channels. A Plan and an Elevation of the viaduct are shown in Fig. 1.

The bridge superstructure comprises of post-tensioned concrete box girders that vary in depth from 7'-0" to 9'-0" depending upon their span lengths. Haunches are also provided at Bents 4, 5, 6, 7 & 8 with the box girder depth increasing to 14'-6" at the supports (Fig. 2). Two intermediate hinges are provided along the bridge dividing it into three longitudinal frames.

The columns vary in height from 29 feet to 67 feet at different bents reflecting the original topography along the bridge. The column sections are all oblong as shown in Fig. 3. They are all detailed to be fixed top and bottom except at bents with outrigger beams where the columns are detailed to provide a pin connection at the top.

Footings typically consist of driven piles of 100 ton capacity. However, at Bent 7 to satisfy clearance requirements to the railroad track, a 12'-0" diameter cast-in-drilled-hole (CIDH) pile is used.

DESIGN SPECTRUM

A two level spectrum approach is used to design the column reinforcement of the 12-span viaduct. The two spectra used in the design are shown in Fig. 4 and described briefly here.

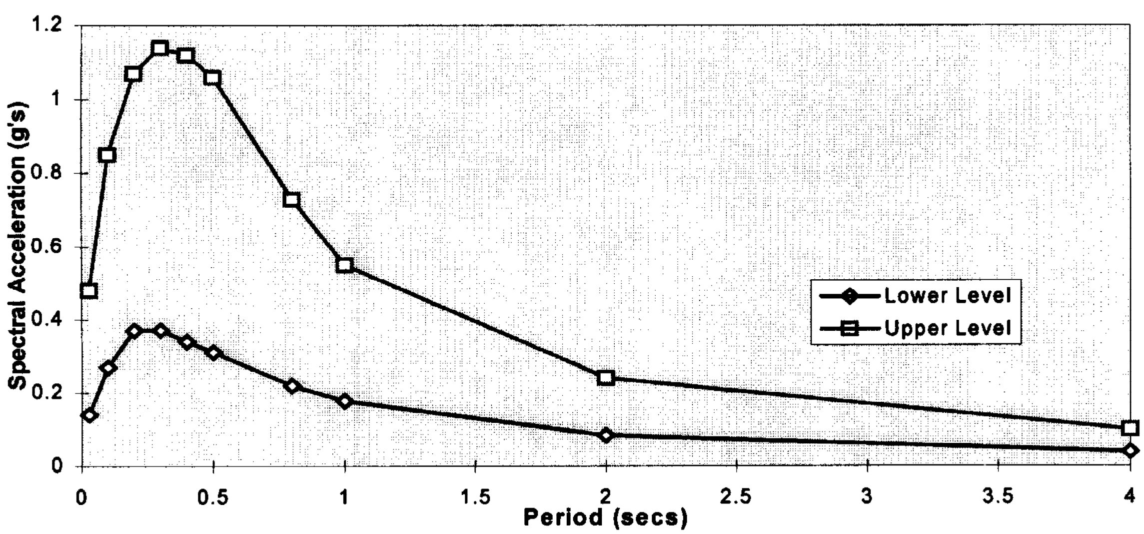

Fig. 4. Design Response Spectrum

1. Lower Level Spectrum- Also called the "functional evaluation earthquake" spectrum, this is a site specific response spectrum with an average return period of 72 years which also corresponds to a 50% chance of being exceeded every 50 years. Forces obtained under these excitations are used for the design of all critical components to ensure that no significant damage occurs to the bridge for this lower earthquake and the structure behaves elastically.

2. Upper Level Spectrum- Also called the "safety evaluation earthquake" spectrum, this can either be a 2500 years return period event based on a probabilistic approach or a maximum credible event based on a deterministic approach. For this study the maximum credible response spectrum is used which represents the maximum amount of energy that can be released by a fault system at the site. Forces obtained under these excitations are used to ensure that there is no collapse or catastrophic failure of the bridge while allowing for inelastic behavior.

DISPLACEMENT DUCTILITY

Displacement ductility checks on column design are carried out in addition to the initial design based on strength ductility. "Displacement demands" are obtained from the elastic dynamic modal analysis for the upper level spectrum using effective section properties of the columns.

"Displacement capacity" of the columns is the sum of the elastic and the plastic component of the displacements. Elastic displacement is calculated as the displacement at the onset of yield. Plastic displacement is due to the rotation in the plastic hinge zone and is calculated based on the inelastic behavior of concrete or reinforcing steel within this region.

For a satisfactory performance, the "displacement capacity" of columns should exceed the "displacement demand". A comparison of the displacement demand and capacity of the columns is presented in Table 1. Also a "displacement ductility capacity" (μ_c) of four or higher is required for all columns. Here, μ_c = displacement capacity (Δ_c) / displacement at onset of yield (Δ_y).

The displacement based design is more rational to use under strong earthquakes and inelastic behavior. In this approach, distinct elements of the primary lateral force resisting system are chosen and suitably detailed for energy dissipation under severe imposed deformations. The critical regions of these members, often termed "plastic hinges", are detailed for inelastic flexural actions, and shear failure is inhibited by a suitable strength differential. Based on a capacity design, all other structural elements are then designed to have sufficient strength capacity to form the "plastic hinges" in the critical regions only. An example of this approach is the Caltrans criterion of designing the superstructure and footings for forces greater than those required to form column "plastic hinges".

Thus by choosing suitable locations for the formation of "plastic hinges" a designer can control the structure behavior and can maximize energy absorption before a catastrophic collapse of the structure. Therefore in this approach the designer "tells the structure what to do" and can desensitize the structure to the characteristics of the earthquake loading, which is, after all, unknown.

SEISMIC FORCES FOR COLUMN DESIGN

The bridge is designed for Caltrans defined Group VII load combination, i.e., forces due to combined effects of Dead Load, Prestressing and Earthquake. Other load combinations due to Live Load, Temperature etc. may govern the column design but have not been investigated in this paper as it aims to study only the effects of different seismic loadings on bridges.

Seismic forces are determined along the longitudinal and transverse axes of the bridge. In order to account for directional uncertainty of earthquake motions, the forces and moments from the two perpendicular seismic loadings are combined into two load cases as follows:
Seismic Load Case 1: Forces and moments resulting from the transverse loading are combined with 30% of the corresponding forces and moments from the longitudinal loading.
Seismic Load Case 2: Forces and moments resulting from the longitudinal loading are combined with 30% of the corresponding forces and moments from the transverse loading.

DISCUSSION OF RESULTS

The compression model and the tension model of the bridge are each subjected to loading generated by the lower level and upper level spectra to form a total of four loadings. The results of this design are presented in Table 1. In the table:

M_{demand} = Flexural demand obtained as a resultant of the longitudinal and transverse forces on the column due to Dead Loads, Prestressing and Seismic Loads. M_{demand} value for the governing load case only is presented in the table.

Z = Adjustment factor for flexural ductility and risk assessment. A "Z" value of one is chosen for lower level spectrum and three for upper level spectrum.

% Steel required is obtained from the ratio of the area of longitudinal column steel required to the gross area of column cross-section. A minimum of 1% steel is provided.

From the results presented in Table 1 the following observations can be made:

1. Lower level spectrum forces predominantly govern the design of most columns except the very stiff columns next to abutments, design for which is governed by upper level spectrum loading.
2. It is important to consider both the compression and tension models for design of bridges with multiple frames. The tension model is able to capture the seismic behavior of individual frames connected at the hinges only by transverse shear keys and horizontal restrainers. Design of columns for the stiff Frame 1 is governed by compression model. In the more flexible Frames 2 & 3 the design of most columns is governed by the tension model.
3. Caltrans requires all inelastic deformations to occur in the column "plastic hinge" zone. Thus, footings and the superstructure have to be designed to develop the column plastic moment capacity M_P. The two level approach results in a larger amount of column steel and a larger M_p and thus a more expensive structure.
4. Columns designed on the basis of flexural ductility criteria also satisfy the displacement criteria, i.e., "displacement capacity" exceeds the "displacement demand" for all columns. Also "displacement ductility capacity" (μ_c) is greater than four for all columns.

Table - 1. Percent Steel Required and Displacement based Evaluation of Columns

Compression Model

Bent	M_{demand}/Z (k-ft)		% Steel required		Displ	Displ	Displ
Location	Lower Level(Z=1)	Upper Level(Z=3)	Lower Level	Upper Level	Demand (in)	Capacity (in)	Ductility μ_c
Bent 2(Top)	26,780	32,700	2.21	2.96	2.9	11.3	5.08
Bent 2(Bot)	35,190	42,930	3.15	3.85			
Bent 3(Top)	Pinned				2.3	6.6	5.68
Bent 3(Bot)	18,540	12,200	2.04	1.00			
Bent 4(Top)	42,220	40,070	2.28	2.11	3.2	14.1	4.96
Bent 4(Bot)	48,150	44,150	2.77	2.41			
Bent 5(Top)	26,330	27,530	1.00	1.00	6.4	26.5	4.42
Bent 5(Bot)	24,510	24,360	1.00	1.00			
Bent 6(Top)	33,120	21,510	1.43	1.00	10.3	30.9	4.40
Bent 6(Bot)	43,260	25,630	1.39	1.00			
Bent 7(Top)	17,830	14,890	1.00	1.00	14.1	37.6	4.63
Bent 7(Bot)	33,200	22,770	1.00	1.00			
Bent 8(Top)	25,020	19,480	1.00	1.00	14.0	37.2	4.00
Bent 8(Bot)	43,890	30,460	1.57	1.00			
Bent 9(Top)	Pinned				9.4	31.5	5.14
Bent 9(bot)	12,910	7,010	1.00	1.00			
Bent 10(Top)	15,850	11,840	1.00	1.00	6.2	41.6	4.93
Bent 10(Bot)	25,180	16,360	1.00	1.00			
Bent 11(Top)	23,840	18,710	1.00	1.00	4.0	31.2	4.90
Bent 11(Bot)	26,540	19,480	1.14	1.00			
Bent 12(Top)	25,800	27,480	2.08	2.33	3.4	12.9	4.98
Bent 12(Bot)	31,250	32,400	2.68	2.83			

Tension Model

Bent	M_{demand}/Z (k-ft)		% Steel required		Displ	Displ	Displ
Location	Lower Level(Z=1)	Upper Level(Z=3)	Lower Level	Upper Level	Demand (in)	Capacity (in)	Ductility μ_c
Bent 2(Top)	22,460	26,570	1.66	2.15	2.9	11.4	5.14
Bent 2(Bot)	28,290	36,120	2.28	3.08			
Bent 3(Top)	Pinned				2.3	7.3	6.63
Bent 3(Bot)	17,870	11,610	1.35	1.00			
Bent 4(Top)	36,300	36,790	1.80	1.82	3.4	13.1	4.81
Bent 4(Bot)	39,150	38,880	1.94	1.89			
Bent 5(Top)	18,010	16,510	1.00	1.00	7.4	26.5	4.42
Bent 5(Bot)	35,170	27,000	1.00	1.00			
Bent 6(Top)	32,440	26,550	1.37	1.00	11.3	29.6	4.01
Bent 6(Bot)	48,470	29,920	1.90	1.00			
Bent 7(Top)	26,520	19,340	1.00	1.00	14.4	37.6	4.63
Bent 7(Bot)	35,290	24,490	1.00	1.00			
Bent 8(Top)	27,145	23,930	1.00	1.00	13.4	36.7	4.00
Bent 8(Bot)	44,350	32,430	1.66	1.00			
Bent 9(Top)	Pinned				9.4	31.8	5.43
Bent 9(bot)	12,200	7,420	1.00	1.00			
Bent 10(Top)	18,890	13,570	1.00	1.00	6.5	41.6	4.93
Bent 10(Bot)	25,910	17,530	1.00	1.00			
Bent 11(Top)	26,720	19,210	1.23	1.00	4.2	29.4	4.65
Bent 11(Bot)	30,690	21,920	1.45	1.00			
Bent 12(Top)	29,170	28,430	2.52	2.45	3.5	13.5	4.99
Bent 12(Bot)	35,890	33,940	3.29	2.95			

CONCLUSIONS

There is an increasing application of the dual-level method in the U.S. not only in the design of new bridges but also in the seismic retrofit of some monumental bridges. The primary advantages of the two level approach are:

1. In this method "no significant damage" can be ensured to the bridge under seismic events that are very likely to occur during its lifetime. The structure is also checked to safeguard against collapse during an extreme seismic event. This design criterion thus provides a higher degree of reliability than a design based only on a maximum credible earthquake.
2. The designer has the option of choosing the return period of the lower level spectra depending upon the importance of the structure. A higher return period spectra, with a performance criterion of "no damage", provides a higher level of safety, but results in a costlier structure.
3. As can be seen from Table 1, lower level spectrum governs the design of most columns. Thus a bridge designed based on the current criterion of designing for a maximum credible earthquake only, would have experienced inelastic deformations and suffered damage in a lower level event.
4. Restraining or releasing the transverse degree-of-freedom at the abutments is clearly defined in this approach. Abutment transverse shear keys are included in the analytical model for the lower level design event. The forces imposed on the key are used to design its capacity as well as to ensure no damage to the piles and wingwalls. The breakaway keys are no longer effective for upper level design event.
5. The displacements that occur in the longitudinal direction can be used to size the abutment gaps such that the knock-off detail is not activated for the lower level event.

The disadvantage of the two level approach is the increase in the design effort. There is also an increase in the cost of the bridge if a higher level of safety is desired. The site specific response spectra for different return periods are currently not available for all sites in California. Generating these may not be economically justified for bridges not classified as "important". The emphasis, however, is shifting toward designing all bridges and buildings using the two level approach to minimize damage during earthquakes.

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