

Enhancing the Seismic Performance of Toll Road Bridges

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Caltrans and AASHTO highway bridge criteria focus on collapse prevention during an extreme seismic event, and allow or require substantial damage to occur as part of the mechanism resisting extreme earthquakes in high seismic zones. Such damage implies possible extended closure of the facility until repaired, and, in the case of toll roads, loss of revenue until traffic is restored. A special study was made to determine the feasibility of bridge design criteria which will obtain a higher degree of seismic reliability for the Orange County, California toll roads. The study included a seismic hazard analysis of the area, development of site specific response spectra with several different probabilities of exceedance, a review of Caltrans criteria, and evaluation of various strategies for improving the structural performance or controlling damage. The principal strategies considered were: a two level design approach; the use of energy dissipators at abutments only; seismic isolation; and lighter weight superstructures. Evaluations were made by preparing a range of bridge designs using a linear elastic analysis for strength design and a nonlinear analysis to assess performance. Cost estimates were prepared using the results of the analytical work. Criteria are proposed for implementation in the design of bridges in the Orange County, California toll road corridors.

The first U.S. toll roads in a high seismic area are being built by the Transportation Corridor Agencies (TCA) in Orange County, California. Funded in part by bonds, the reliability of the bridges is important to the revenue base of the project. Once opened to traffic these 65 miles of new roads will be turned over to the California Department of Transportation (Caltrans) for traffic operation and maintenance. Following the October 17, 1989 Loma Prieta earthquake, TCA sponsored a study to determine the feasibility of using seismic design criteria for bridges which would exceed the minimum requirements of Caltrans and thereby improve the performance of the structures during moderate earthquakes. An additional objective was less damage and shorter closure time to repair damage from an extreme event.

Computech Engineering Services, Inc. (CES), Imbsen & Associates (IAI) and Woodward/Clyde Consultants were contracted to develop such criteria. Woodward/Clyde Consultants performed a seismic hazard analysis of the area and developed site-specific response spectra with different probabilities of exceedance. CES and IAI were responsible for inves-

tigating the performance and cost implications of different design strategies that would achieve the basic objectives of the study, and then develop project-specific design criteria.

Elements of the study included a review of Caltrans design criteria, consideration of a two level design approach, the use of energy dissipators at abutments, seismic isolation and lighter weight superstructures. The cost and performance of these various strategies are calculated and compared to possible repair and closure costs.

The analytical phase of the study consisted of the design and analysis of two basic bridge configurations; a typical three-span bridge with 22-foot-high columns and a nine-span bridge with 59-foot-high columns. The three-span bridge was designed with both steel and concrete superstructures, providing a total of three primary structural configurations. The columns of each bridge were designed with various levels of strength (Z-Factors). As alternate design strategies, each of the above configurations, incorporated lead rubber isolation bearings throughout, and as an energy dissipation mechanism only at the abutments. An analysis matrix was developed to provide a range of bridge designs for performance assessment and criteria development. The matrix was separated into two distinct phases; linear elastic response spectrum analysis to establish adequate strength, and full nonlinear analysis to assess performance. Within the linear elastic phase, bridge designs were developed using standard Caltrans design strategies with varying damage and risk (Z-Factor) adjustment factors. Each of the designs were then modeled with all structural elements exhibiting their non-linear characteristics and subjected to a series of spectrum compatible time histories in order to establish their performance level.

The analytical results were used to develop cost estimates of initial construction, cost to repair column earthquake damage, and estimates of the closure costs while significant column damage is repaired. Abutment damage was not assessed.

Based on the cost and performance evaluation, the project-specific design criteria was developed to consist of a two level procedure. Although this is a departure from current Caltrans and AASHTO practice, it meets the objective of the study as it provides a higher and more uniform level of safety and reliability. Briefly, the design criteria requires two levels of analyses: one corresponding to a 72 year return period event and the other corresponding to a maximum credible design event. Design forces and displacements resulting from the lower level event will be used so that minimal damage will occur as a result of this event. The forces and displacements resulting from the Maximum Credible analysis will be used to

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protect against the structure collapse.

SEISMIC HAZARD STUDY

A map showing the location of the three transportation corridors is presented in Fig. 1. Included in this figure are faults in the area and seismic contours showing the acceleration levels developed by the California Department of Mines and Geology which vary from 0.6g to 0.3g. Woodward-Clyde performed a seismic hazard analysis of the three corridors which resulted in uniform risk site specific response spectra with varying return periods for five different sites (A to E) shown in Figure 1.

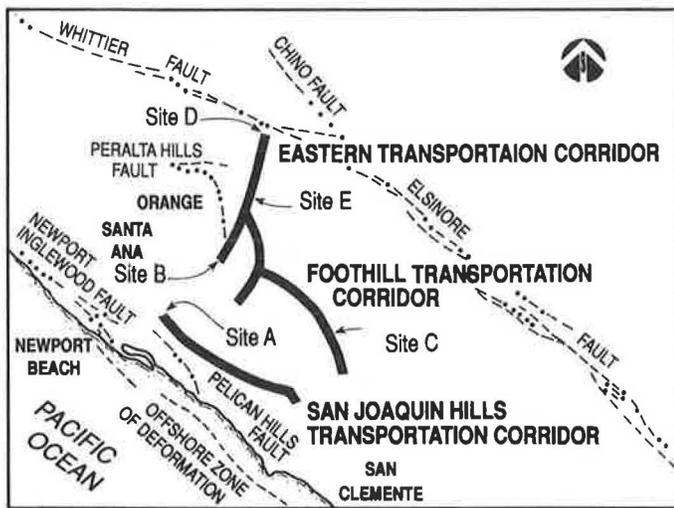


FIGURE 1 Location Map

In a seismic hazard study several terms are used which are defined for the purpose of clarity.

- a. Response Spectrum - a plot of the maximum earthquake response with respect to natural period or frequency of the structure. Response spectra can show acceleration, velocity or displacement. This paper will only include acceleration response spectra.
- b. Return Period - an appropriate response spectra with a return period of 72 years means that such an earthquake will occur approximately every 72 years. Because of the major uncertainties involved with earthquakes an event larger than the return period event may occur and therefore return periods are also expressed in a more meaningful term called probability of exceedance.
- c. Probability of Exceedance - this expresses the probability that a given event will be exceeded in a certain time frame. For example, a 72 year return period response spectra has a 50% chance of being exceeded every 50 years. See Table 1 for the relationship between return period and probability of exceedance.
- d. Maximum Credible Spectrum - a maximum credible response spectrum is a measure of the maximum amount of energy that can be released by a given fault.

TABLE 1 Return Period & Probability of Exceedance

Return Period	Probability of Exceedance	
	in 50 Years	in 100 Years
25 Years	86%	98%
50 Years	63%	86%
72 Years	50%	75%
150 Years	28%	49%
250 Years	18%	33%
475 Years	10%	19%
2500 Years	2%	4%

Fig. 2 is an example of a response spectra plot at a typical site, showing the relative magnitude of spectra with return periods of 25, 50, 72, 150, 475, and 2500 years as well as the appropriate Caltrans Maximum Credible Spectra. Fig. 3 compares the 475 year return period spectra at the five different sites. Note there is a factor of 2 or more difference between extreme sites. Site A is different from the other 4 sites in that it has a soft ground condition.

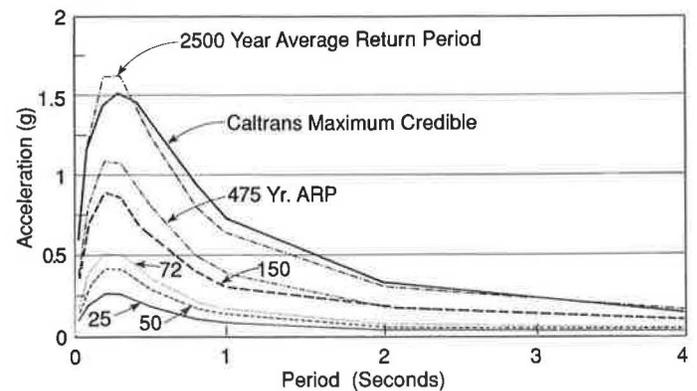


FIGURE 2 Site Specific Response Spectra - Site E

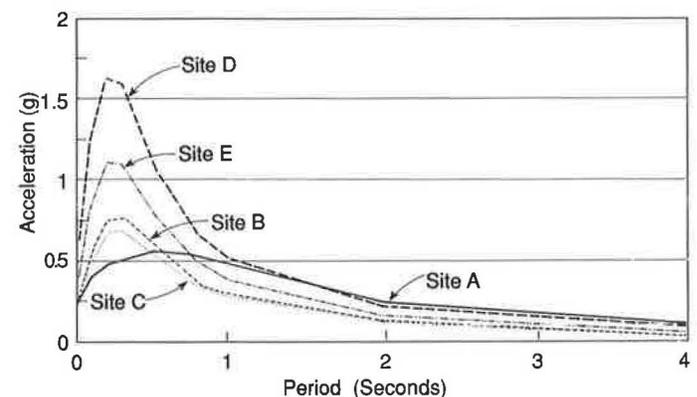


FIGURE 3 Site Specific Response Spectra - 475 Yrs ARP

CURRENT CALTRANS DESIGN PHILOSOPHY

Both current U.S. seismic highway bridge design codes (AASHTO and Caltrans) are one-level design approaches, i.e.,

an analysis is performed for only one level of response spectra and all design forces and displacements are derived from this analysis. The primary advantages of a one-level design approach is its simplicity. The disadvantages are that the elastic capacity of the columns, shear keys, etc. correspond to a lower-level design event that has an unknown return period or probability of exceedance. The primary differences between the Caltrans design requirements and the AASHTO requirements are that Caltrans uses a maximum credible response spectrum for the analysis, whereas the AASHTO Specifications use a 475-year return period spectrum. In addition, the reduction factors used for column design Z-Factor in Caltrans, a R-factor in the AASHTO Specifications) are also different. For Caltrans, the Z-Factor (Fig. 4) varies between 8 and 4, depending on period, for multi-column bents, and between 6 and 3 for single columns. In the AASHTO Specifications, the R-factors are 5 and 3 for multi-and single-column bents, respectively.

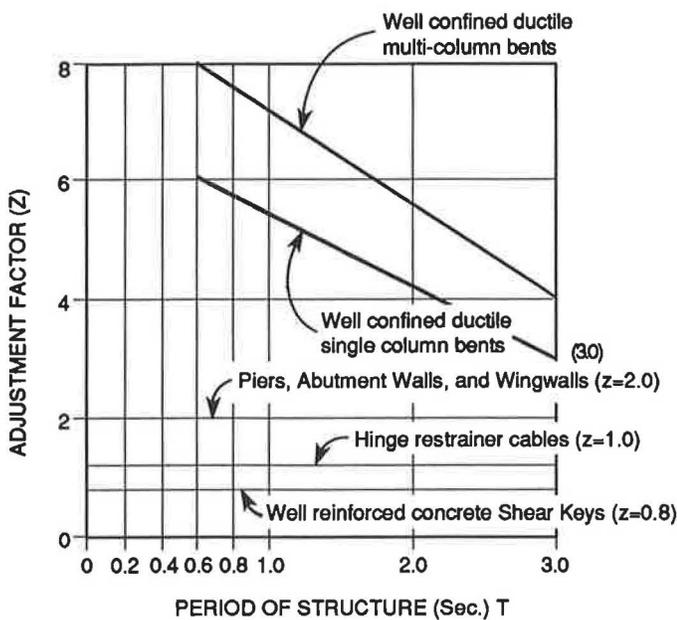


FIGURE 4 Adjustment of Ductility and Risk (Caltrans)

Design Strategies

For monolithic bridges with multi-column bents, the current Caltrans design philosophy is to provide a fully pinned connection at the base of the column. This evolved as a cost-saving measure due to requirements embodied in AASHTO Specifications and the Caltrans design criteria. The current design codes state that the foundations must be designed for the maximum forces developed by the formation of column plastic hinges. This design requirement was developed to ensure that any damage that does occur in a bridge is readily detectable and repairable. The disadvantage of the pinned column base design philosophy is that redundancy is reduced and the ductility demand on the plastic hinge that will form at the top of the column will be increased. The redundancy reduction also eliminates an energy-dissipating mechanism, i.e., column plastic

hinge; thus, if one joint should fail, there are fewer joints to provide a resisting mechanism.

For larger bridges, a thermal separation is generally required at the abutment. An earlier Caltrans design concept was to permit the box girder to impact the backwall of the abutment, thereby mobilizing the soil backfill. Empirical stiffness relationships were developed for the abutment-soil-pile interaction in order to estimate the displacements that occurred. The problem with this concept is that significant damage may occur to the abutment backwall and abutment piles in addition to potential damage to the wing walls. This concept was modified to provide a knock-off or release mechanism at the bottom of the abutment backwall (see Fig. 5). This concept has the advantage of permitting the soil backfill to be mobilized as a resisting mechanism. The disadvantage is that it is very difficult to inspect the damage that may occur at the knock-off plane. Damage may still occur in the wing walls and the abutment backwall. In the transverse direction, shear keys are generally provided at the abutments. If their ultimate capacity is below the ultimate capacity of the piles in the transverse direction, the transverse shear keys will act as a force-limiting mechanism to protect the piles.

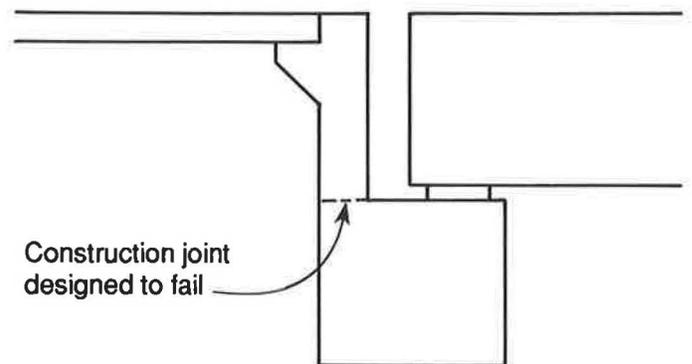


FIGURE 5 Bottom of Backwall Knockoff Detail

Return Period Implicit in Column Z-Factors

In the Caltrans design criteria, the Z-Factor is a reduction factor applied to the forces of a given structural component from an elastic analysis using the maximum credible earthquake. These reduced forces are then used for the design of the component. The Z-Factor thus becomes a measure of the ductility demand required in columns or other members critical to seismic loading. As shown in Fig. 4 the Caltrans Z-Factor also adjusts for risk by requiring greater strength for longer period (tall column) structures.

Even though the Caltrans procedures use the Maximum Credible Spectra for design, it is possible to estimate the return period for which such a design produces a purely elastic response in the columns. Two methods can be used:

1. Divide the Caltrans maximum credible spectra by the col-

umn Z-Factor and compare this reduced spectra with various site specific return period spectra. Fig. 6 is such a plot for Site E.

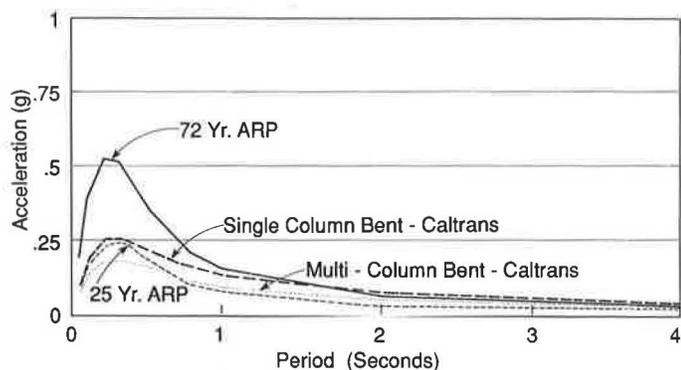


FIGURE 6 Maximum Credible Spectra Divided by Z-Factor - Site E

2. Divide the Caltrans maximum credible spectra at a given period by a given return period site specific spectra at the same period to obtain the Z-Factor implicit in the given return period spectra. Fig. 7 plots the results for Site E for the 72 Year event.

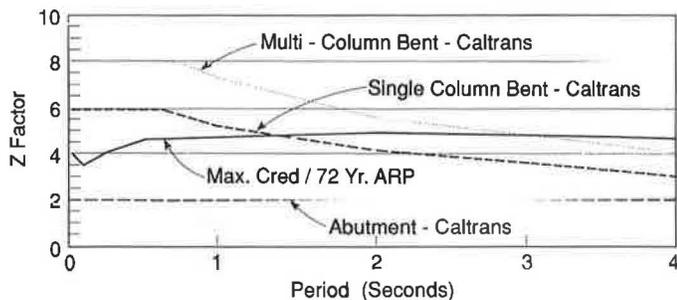


FIGURE 7 Comparison of Z-Factors - Site E (72 Yrs ARP)

This analysis of implicit Z-Factors shows the approximate range of elastic behavior which results from a design using Caltrans minimum standards. For multi-column bents current Caltrans minimum elastic design corresponds to less than a 25 year return period for bridges with a natural period less than 0.8 seconds and a 50 year return period for natural periods between 0.8 to 2.0 seconds. For single column construction the current Caltrans minimum elastic design corresponds to a 50 year return period for bridge periods less than 0.8 seconds and a 72 year return period for bridge periods between 0.8 and 2.0 seconds. By means of comparison Department of Defense essential facilities including hospitals require elastic design for a 72 year return period event.

ALTERNATE DESIGN STRATEGIES

Several different design strategies were evaluated as part the investigation to develop design criteria with a higher level of safety and reliability. These were as follows:

1. While retaining the Caltrans Maximum Credible design earthquake and a one level design approach use a lower Z-Factor in the design of the column. This will increase column cost but reduce ductility demand. The threshold of damage will be increased, and the severity of damage at extreme events reduced.
2. Return to the use of moment-resisting connections at the column bases of multiple column bents. This adds more energy dissipating mechanisms to the structure but increases the cost of the foundations.
3. At abutments, use a knock-off detail at the top of backwall to limit damage, rather than at the base of the backwall which is current Caltrans practice. A detail such as shown in Fig. 8 has been tested and used extensively in New Zealand.

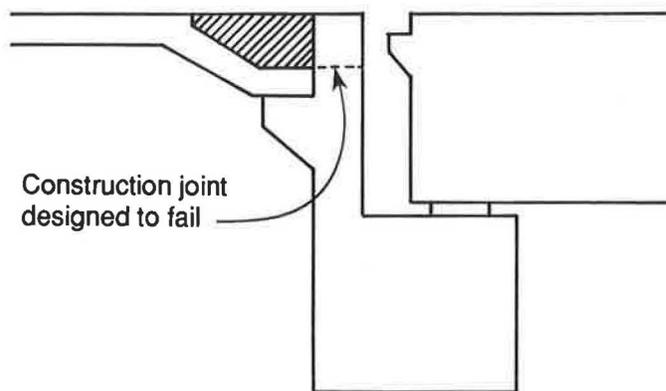


FIGURE 8 Top of Backwall Knockoff Detail

4. At abutments design transverse shear keys to release before damage to wing-walls or abutment piling can occur.
5. Energy Dissipation Devices - In Caltrans current design philosophy low height elastomeric bearings are generally used at the abutments. An alternative consideration is to use energy dissipating devices or bearings at the abutment to increase the amount of damping at this location. Significant additional damping will decrease the column forces, superstructure displacements and column ductility demands by 30 to 50%.
6. Seismic Isolation - This design concept works by reducing the seismic forces that the column and abutments must resist. The isolation bearings lengthen the period of the bridge and add a significant amount of damping. The

concept is quite different from current Caltrans practice in that a bearing must be introduced at either the top or bottom of all columns and at the abutments. The actual displacements that result from the use of seismic isolation are not very different from those that result from a non-linear analysis of a current Caltrans design. The advantages of the seismic isolation design concept are that damage to the columns can be eliminated and forces on the abutments can be significantly reduced.

7. Two level Design Strategy-as previously noted, current U.S. Bridge design codes use a one level design approach. For a two level design approach which is used in some other seismic codes, an analysis is performed for an upper and lower level design event. The upper level response spectra can be, for example, either a maximum credible or 2500 year return period event and a lower level response spectra is a 72 year or 150 year return period event. The lower level results would be used together with design requirements to ensure that there is no significant damage for that event. Columns for example would be designed using a strength design approach for forces that resulted from this lower level analysis. The upper level results would be used to ensure that there is no collapse potential under a severe earthquake. The primary advantage of the one level design approach is that it is similar to current Caltrans practice and there is only one analysis to be performed. The disadvantages are:
 - a. The return period at which onset of damage would occur is difficult to determine.
 - b. There is no logical basis to determine what the longitudinal abutment gaps should be before the knock-off device at the abutment is activated.
 - c. There are problems in deciding whether to include or exclude the transverse abutment shear keys in the analytical model.

The philosophical advantage of the two level approach is that it directly considers the behavior of the structure during events that are almost surely to occur in its lifetime as well as safeguarding against collapse during an extreme event. The disadvantage with this approach is that two dynamic analyses are required. The two level design approach is currently required on all Department of Defense essential facilities with the lower level event specified as a 72 year return period event. For a design criteria with a higher degree of reliability the advantages of a two level approach are:

- i. Design forces for all critical components can be used to ensure that the bridge will have no significant damage for the lower return period.
- ii. Transverse shear keys can be included in the analytical model for the lower level design event. The forces imposed on the key can be used to design its capacity as well as ensure that the capacity of the piles and the wing walls exceed the capacity of the shear keys.
- iii. 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 this lower

level event.

- iv. For the upper level design event there would be no confusion on what to do with the transverse shear key in the analytical model, since it would no longer be effective for this event.

PERFORMANCE AND COST STUDY

Coupled with the investigation of different design philosophies it was also necessary to determine what increase in performance (i.e., reduction in damage) was achieved with the different design strategies and how this related to both cost of column repair and initial cost of construction. In order to achieve this objective a number of different designs were performed on a 3 and 9 span bridge configuration. Each design was analyzed incorporating a full non-linear analytical model of the structure to evaluate its performance for varying return period events. Bridge configurations used in the study were:

1. A 3-span 155'-190'-155' bridge with 22' high columns and 83-½ ft. deck width.
2. A 9-span (7 at 140' and ends spans of 150' and 140') bridge with 59' high columns and 71 ft. deck width.
3. Super structure types were:
 - a) Concrete box girder monolithic with columns for both the 3- and 9-span configurations. The concrete box girder had two columns fixed at the superstructure and fully pinned at the foundation. Columns with fixed based were also included in the investigation.
 - b) Steel Girder connected with pins to the cap beam for the 3-span configuration. The weight of the steel superstructure was approximately ⅓ of the concrete box girder. The steel superstructure had a cap beam connecting the two columns. The columns were fixed at the foundation level.
4. Energy Dissipators at Abutments - The 3-span concrete configuration was designed and analyzed with the columns in their conventional configuration, but with energy dissipating bearings at the abutments.
5. Seismic Isolation Designs - Each of three primary configurations were designed and analyzed in a fully isolated and conventional configuration. The isolated concrete box girder superstructure had the isolators at the abutments and at the base of the columns. Similar results would be obtained if they were at the top of the columns. The steel superstructure had the isolators under the girders.
6. Column Design - The columns were designed for the Caltrans maximum credible response spectra for Site E. The following Z-Factors were used with octagonal columns.
 - a) 3-span Concrete Box Girder - Z = 7.5, 5 and 3
 - b) 3-span Steel Superstructure - Z = 5 and 3.25
 - c) 9-span Concrete Box Girder - Z = 4.5, 2.75 and 1.0

The 3-span concrete box girder bridge was also analyzed with a rectangular column with a Z-Factor of 7.75.

It was determined from preliminary studies that a bridge designed for a given Z-Factor for one site and subjected to a 2500 year return period seismic input for that site produced results that were reasonably similar to using the same Z-Factor at a different site and subjecting the bridge to the 2500 year return period for that site. As a consequence, this simplified the number of analysis and enabled the study to focus on the design and seismic input for just one Site. Site E was selected.

The seismic input used for the nonlinear time history analysis for Site E used the following return period events 2500; 475, 150, and 72 years. For each return period six spectrum compatible time histories were developed. Each set of six time histories included three orthogonal components (two horizontal, 1 vertical). Each set was frequency scaled such that two horizontal components were compatible with the appropriate site specific spectra and the vertical component was compatible with two-thirds of the horizontal component. All analyses were performed with 100% of each of the two horizontal components and the full vertical ($\frac{2}{3}$ horizontal) component.

SUMMARY OF ANALYTICAL RESULTS

A small selection of the analytical results are shown in Figs. 9, 10, and 11 for the conventional 3-span bridge with a concrete superstructure with varying Z-Factors 7.5, 5 and 3 with octagonal columns and for a 7.75 Z-Factor for a rectangular column. Figs. 12, 13, and 14 are similar results for all 3 global design options, conventional, with a Z-Factor of 5, seismic isolation and energy dissipators at the abutments. For the 9-span bridge with the conventional concrete superstructure a selection of results are shown in Figs. 15 and 16 with variations in Z-Factors. Although isolation was included in the study of the 9-span bridge it is not a good seismic isolation candidate, because the tall (59 ft. high) piers provide a flexible bridge without the incorporation of seismic isolation.

Three Span Bridge

Three global design options have been examined in terms of their impact on the response of the 3-span bridge. The improvement in seismic performance has been assessed in terms of both the columns and the abutments. From a design strategy perspective the results are summarized as follows:

The column performance is assessed in terms of column ductility demands.

1. Z-Factor - the column ductility demand reduces as the Z-Factor decreases. Up to a 50% reduction in ductility demand was achieved as Z decreased from 7.5 to 3. Thus increasing the column strength improves the performance of the columns.
2. Use of Seismic Isolation - this provided the most dramatic improvement in column performance in that the column ductility demand was eliminated.

3. Use of Energy Dissipators at Abutments - this provided up to a 40% reduction in column ductility demand with more significant reductions achieved with higher Z-Factors.

The abutment performance is more difficult to assess, since it is dependent upon the strategy adopted for the design. Displacements at the superstructure level are used to assess the potential abutment performance.

1. Z-Factor - in general the abutment performance will improve, although not significantly, with decreasing Z-Factors, since at higher return period events displacements are not reduced significantly. In terms of an abutment design strategy, changes in the column Z-Factor will not have a significant impact on the design forces or displacements.
2. Use of Energy Dissipators - this design option has the most dramatic impact in terms of abutment performance since it provides up to a 50% decrease in the superstructure displacements. It also provides a 40% reduction in the forces the abutments are required to resist.
3. Use of Seismic Isolation - compared to conventional Caltrans abutment design the primary benefit provided by the use of seismic isolation are the reduced forces for the abutment design. Design displacements are similar to the conventional design and therefore the performance in terms of displacements will be dependent on the design strategy and detailing (knock-off detail, engagement of backwall, etc.).

Nine Span Bridge

The 9-span bridge was designed with the conventional Caltrans configuration with 2 intermediate hinge joints and three different Z-Factors (1.0, 2.75 and 4.5). The bridge had reasonably tall columns (59 ft. high) and thus was quite flexible. Consequently, it would not be considered to be a good candidate for seismic isolation or the use of energy dissipators at the abutments.

The column ductility demands, as with the 3-span bridge, decrease as the Z-Factor reduces. There is no actual ductility demand for either the Z=2.75 or Z=4.5 designs for the 72 Year event and there is approximately a 25% reduction in ductility demand as Z decreases from 4.75 to 2.75 for the 2500 Year event.

In assessing the abutment performance greater clearance will be required for the 72 Year event, compared to the 3-span bridge with shorter columns, if abutment backwall engagement is to be avoided. The displacements in the longitudinal direction are reduced by approximately 30% as Z decreases from 4.5 to 2.75 for the higher level events. Depending upon the abutment design strategy this would improve the abutment performance. In the transverse direction the displacements are also reduced, but not as significantly.

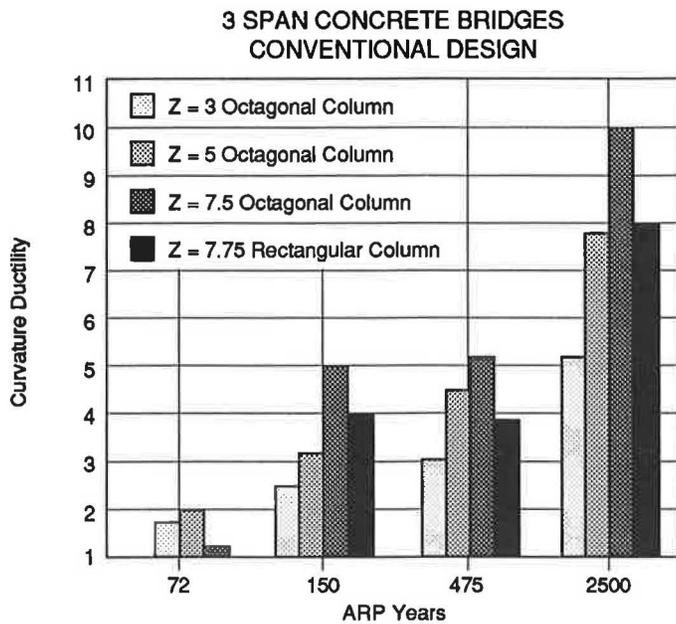


FIGURE 9 Curvature Ductility Demand

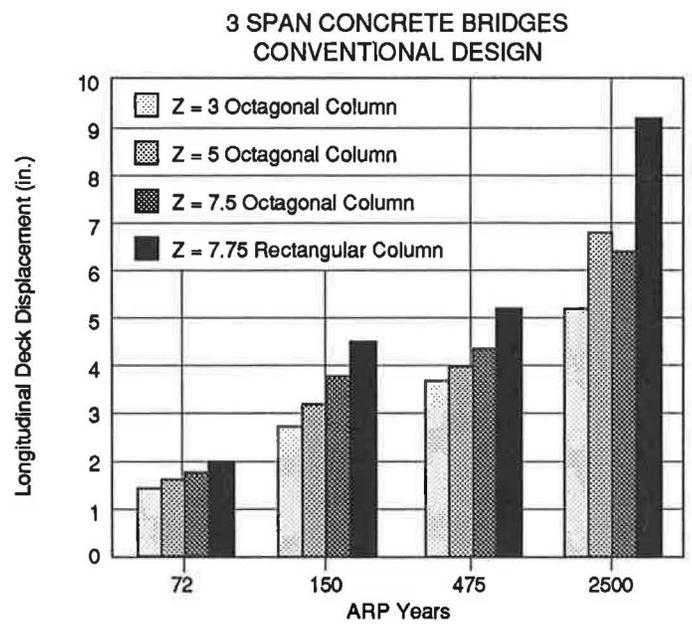


FIGURE 10 Longitudinal Deck Displacement

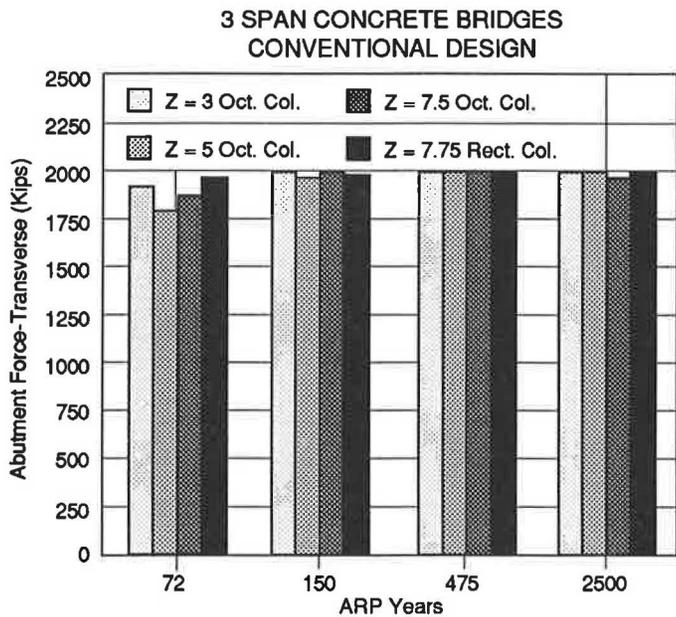


FIGURE 11 Transverse Abutment Force

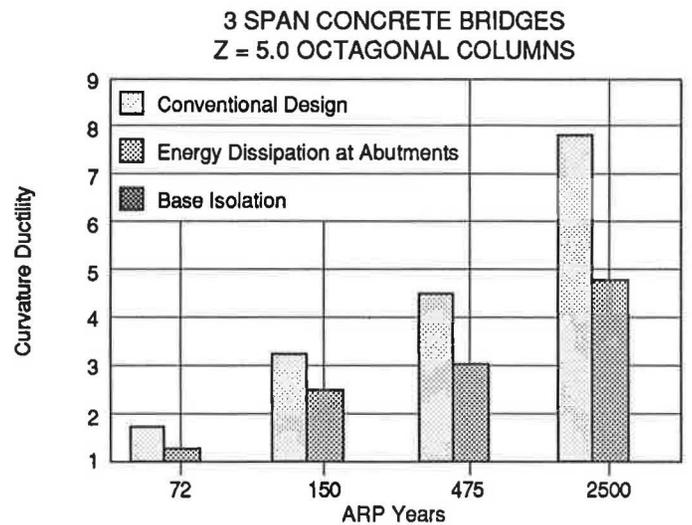


FIGURE 12 Curvature Ductility Demand

Effects of Vertical Accelerations

To assess the impact of vertical acceleration one of the models (the three span concrete bridge) was selected and analyzed both with and without the vertical earthquake component of ground motion. The analyses show that the vertical accelerations have a marked effect on the vertical deck displacements and bending moments. The difference in moments in each span between the analyses with and without vertical earthquakes is equivalent to the moments caused by a uniform load

approximately equal to the dead load.

The conclusion from this limited study of vertical effects is that bridge girders may be in the period range where vertical amplification may occur and so the vertical component of motion should be included in the response spectrum analysis. The added forces from this component may affect the design of the girders and column. The vertical earthquake effects are less likely to influence the ductility demands on the columns.

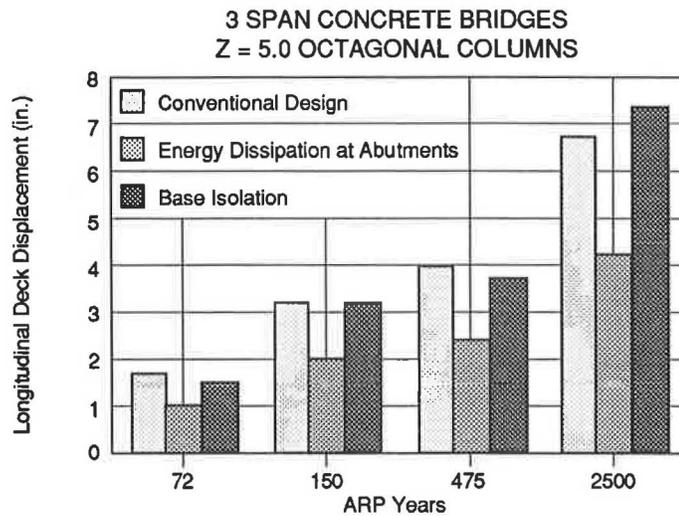


FIGURE 13 Longitudinal Deck Displacement

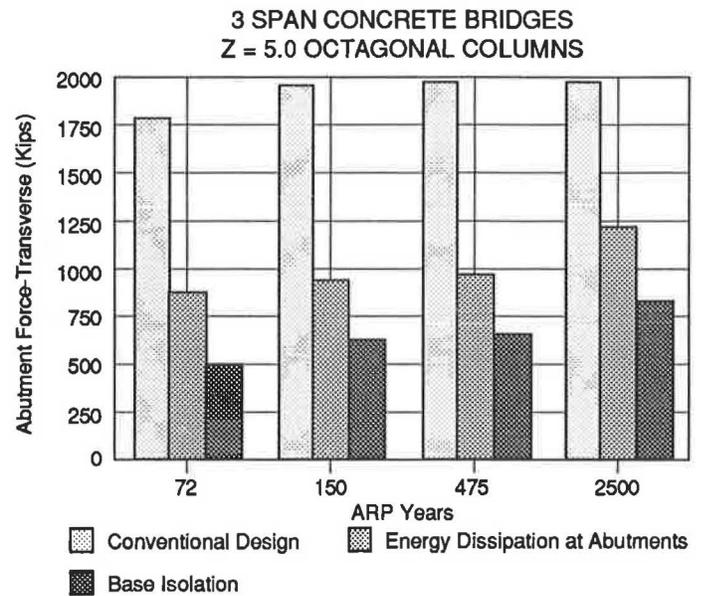


FIGURE 14 Transverse Abutment Force

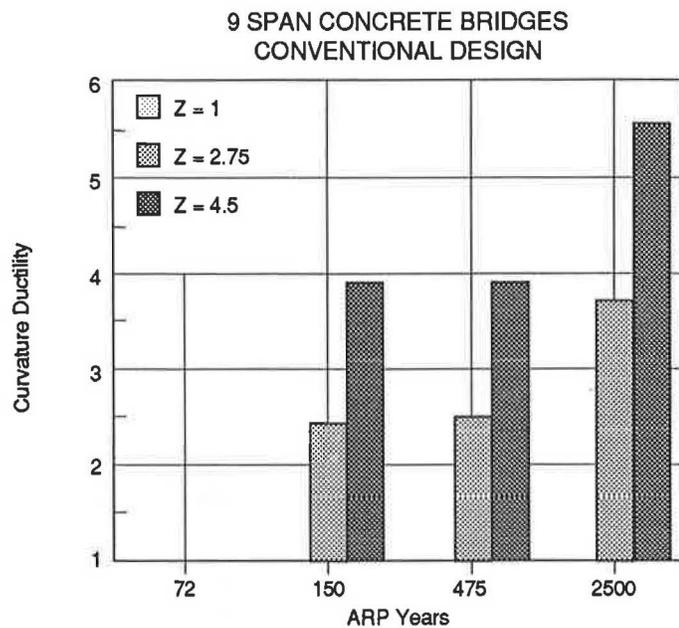


FIGURE 15 Curvature Ductility Demand

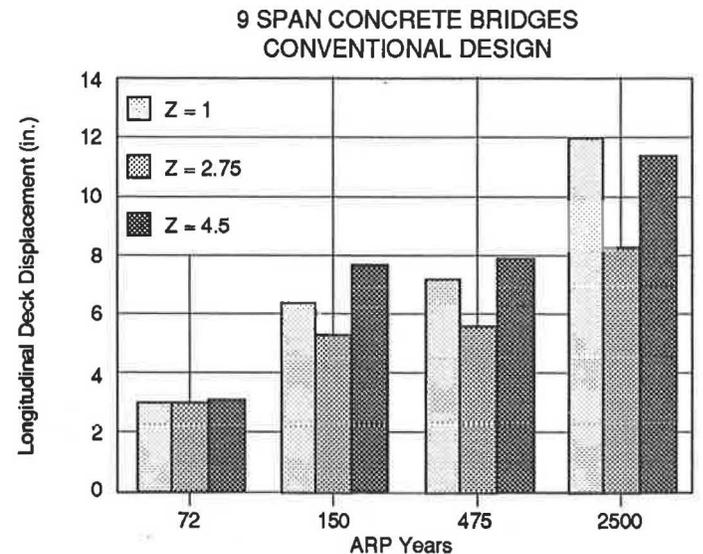


FIGURE 16 Longitudinal Deck Displacement

Summary

A significant number of analyses were performed on a range of design configurations and design options. The results are useful in providing trends of the impact of the different variables considered. Because of the limited range of configurations utilized some generalizations on the impact of the different design options are provided as an aid to designers.

1. Increasing the column strength will decrease the column

ductility demand and provide some small decrease in the deck displacements.

2. The use of energy dissipators at the abutments will decrease column ductility demands and deck displacements. These reductions were significant (up to 40%) in the three-span example. These trends would probably occur for continuous bridges in the 2-to 5-span range. For bridges with larger numbers of spans and with intermediate hinges, abutment energy dissipators would only impact the

response of the segment adjacent to the abutment.

3. Seismic isolation provides the ability to eliminate the ductility demand on the columns provided the period of the bridge in the isolated configuration is less than approximately 1.5 seconds. Therefore bridges with tall columns are not good seismic isolation candidates. The deck displacements resulting from the use of seismic isolation are similar to those of conventional construction when analyzed in a non-linear configuration.
4. The use of light weight superstructures does not have an economic advantage in designing for greater seismic reliability.

ECONOMIC ASSESSMENT OF RESULTS

For TCA the economic impact of bridge seismic design and performance is a combination of the following: the cost of initial construction, the cost of earthquake insurance, and the cost of closure of the system during repairs. For Caltrans the impact is the cost to maintain the bridge and repair damage that occurs. In addition, the seismic performance of a bridge as a component of a transportation system will have an economic impact on the surrounding community if the bridge must be closed. The cost components considered in the economic assessment are as follows:

Construction Costs

The baseline cost assumed is for a standard Caltrans type of design. The costs, without construction engineering, are assumed as follows:

Concrete Box Girder Bridge	\$65.00/sq. ft.
Steel Girder Bridge	\$65.00/sq. ft.

A cost of \$65.00/sq ft. was assumed for both girder types to show only the relative effect of the impact of seismic design strategies. It is acknowledged that there maybe an initial cost differential between steel and concrete and that this will vary with time and location. The costs include all standard details including abutments and joints. No attempt was made to include cost increases in any of the designs for changes that may result in the abutments and joints. The cost increases in initial construction only included the increase in column costs and the cost of isolators and energy dissipators where appropriate.

Other items in the cost tables are expressed as a percentage of this initial cost. If it is assumed that all bridges have the same percentage increase, then the project bridge cost also increases by this percentage. A 1% increase in bridge cost represents a 0.17% increase in total project cost (estimated total project cost \$2.1 billion, estimated bridge cost \$0.36 billion). All costs are present value, assuming both the cost of money and the cost of inflation over time are equal. Additional assumptions are as follows:

1. The cost of abutment damage has not been included in any of the designs. The anticipated amount of damage is dependent on the design strategy used for the abutment (e.g. gap provided, backwall engagement, knock-off detail, wing wall capacity etc.).
2. For the steel alternate (fixed base columns) changing the Z-Factor changes the foundation cost. For the concrete alternate (pinned base columns), changing the Z-Factor does not change the foundation cost. It was assumed that the dead and live load requirements govern the pile design and that this provides sufficient lateral capacity to resist the overstrength column shear of 1.3M/L. In some cases, such as poor soil conditions, this assumption may not be accurate, so foundation costs would increase.

Closure Cost

The estimated annual revenue of the three systems is \$43 million per year for 1992-93, \$100 million per year for 1996-97, and \$147 million per year for 1999-2020.

Closure costs for the system would range form \$118,000 to \$402,000 per day. Assuming the year 2,000 revenues, the cost per week of closure expressed as a percentage of bridge cost is:

$$\frac{147 \times 100}{52 \times 360} = 0.8\% \text{ per week of closure}$$

The estimate of closure time is based on the level of inelastic behavior in the bridge columns. Significant abutment damage may also cause closure but abutment damage has not been included in this economic assessment.

<i>Column Displacement Ductility</i>	<i>Weeks Closed</i>
<3	0
3-4	1
4-5	1-3

Repair Costs

The repair costs are based on the level of the inelastic behavior of the columns. No abutment repair costs have been included. The repair costs do not provide for shoring or difficult access to the repair location.

Cost Comparison

Tables 2 and 3 illustrate the nominal costs of designing a bridge for a higher level of seismic performance. In general, performance of all alternatives is reasonable; for the 72 year ARP event, all alternates remain open assuming no abutment damage occurs. For only one alternative (steel superstructure, Z=5) is closure anticipated for a 475 year return period event again assuming no abutment damage occurs. For the 2500 year ARP event, more severe column damage, requiring closure and increased repair cost occurs for the conventional alternates, except for the Z=3 conventional concrete, where

closure is not required. There is improved performance as the design Z-Factor decreases. Only the fully isolated structure remains damage-free for the 2500 year return period event.

The steel structure's performance is marginally worse than the concrete alternative.

TABLE 2 Three-Span Bridge. Costs of Alternate Design Strategies (All Percent Values Are Expressed as a Percentage of Standard Caltrans Design)

Design Alternate	Construction Cost		Column Repair Cost	Lost Revenue	Downtime (weeks)	Total TCA Cost Increase %
	\$/sq. ft.	% Incr. of Bridge Cost	% of Initial Cost			% of Initial Cost
<i>Concrete Box Girder - Minimum Design (Z=7.75)</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$67	3	1 1-2 3-5	0 0 1	0 0 1	4 4-5 6-8
<i>Concrete Box Girder - Minimum Design (Z=7.5)</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$65	-	1 1-2 6-8	0 0 1-3	0 0 1-3	1 1-2 7-11
<i>Concrete Box Girder - Reduce Z-Factor from 7.5 to 5</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$66	1	1 1-2 3-5	0 0 1	0 0 1	2 2-3 5-7
<i>Concrete Box Girder - Reduce Z-Factor from 7.5 to 3</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$67	3	0 1 1-3	0 0 0	0 0 0	3 4 4-6
<i>Concrete Box Girder - Isolation at All Supports</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$68-\$69	4-6	- - -	0 0 0	0 0 0	4-6 4-6 4-6
<i>Concrete Box Girder - Energy Dissipation Devices at Abutments (Z=7.5)</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$67-\$68	3-5	1 1-2 3-5	0 0 1	0 0 1	4-6 4-6 7-11
<i>Concrete Box Girder - Energy Dissipation Devices at Abutments (Z=5)</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$68-\$69	4-6	1 1 1-2	0 0 0	0 0 0	5-7 5-7 5-8
<i>Lightweight Superstructure - Steel, Conventional Z=5</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$66	1	1 3-5 6-8	0 1 1-3	0 1 1-3	2 6-8 9-15
<i>Lightweight Superstructure Steel, Conventional Z=3.25</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$67-\$69	3-6	0 1-2 6-8	0 0 1-3	0 0 1-3	3-6 4-8 11-20
<i>Lightweight Superstructure - Steel, Fully Isolated</i> 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)	\$67-\$69	3-6	0 0 0	0 0 0	0 0 0	3-6 3-6 3-6

Note: Z = 7.75 is the 4'x8' oblong column. All other columns are octagonal.

TABLE 3 Nine-Span Bridge. Costs of Alternate Design Strategies (All Percent Values Are Expressed as a Percentage of Standard Caltrans Design)

Design Alternate	Construction Cost		Column Repair Cost	Lost Revenue	Downtime (weeks)	Total TCA Cost Increase %
	\$/sq. ft.	% Incr. of Bridge Cost	% of Initial Cost			% of Initial Cost
Concrete Box Girder with Fixed-Base Design (Z=4.5) 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)			0	0	0	0
	\$65		3-4	0	0	3-4
		-	5-7	0	0	5-7
Concrete Box Girder with Fixed-Base Design Reduce Z-Factor from 4.5 to 2.75 75% Prob. of Exceedance (72 Year Return Period) 20% Prob. of Exceedance (475 Year Return Period) 4% Prob. of Exceedance (2500 Year Return Period)			0	0	0	1-3
	\$66-\$67		3-4	0	0	4-7
		1-3	3-4	0	0	4-7

RECOMMENDED PROJECT SPECIFIC CRITERIA

The intent of the recommended seismic design criteria for all bridges on the TCA project is to provide a higher level of safety and reliability than the current Caltrans design criteria. The criteria are as follows:

2. The selection of optimal column cross section;
3. For multiple column bents, whether to pin columns at the base to minimize foundation costs or to fix column bases to achieve more redundancy under seismic loads and smaller moment magnifications under live loads.

Design Response Spectra

The design response spectra for a bridge shall be obtained by using the appropriate site specific spectra for both the maximum credible and 72 year return period earthquake. The spectral ordinates for vertical earthquake ground motions are obtained by multiply the horizontal values by two-thirds.

Column Design

Column Design forces are based on the forces obtained from the lower level design earthquake. This will ensure that there is no ductility demand on the columns for this design event. A second analysis is required to ensure that the ductility demand on the columns is limited to acceptable levels for the upper level design event ($\mu < 3$ for single columns and $\mu < 4$ for multi-column bents). For this design check it is assumed that there will be no resistance provided by the abutments. This will ensure that the columns will provide adequate resistance and acceptable performance regardless of what damage may occur at the abutments.

Methods of Analysis

With the exception of single-span bridges, multi-mode response spectra analysis shall be used for all bridges. The maximum response (displacement or component force) shall be estimated based on the CQC (Complete Quadratic Combination) modal combination procedure. If discontinuities or other sources of nonlinearity exist, use an equivalent linear solution procedure with iteration as required to satisfy equilibrium of forces and displacements. Non linear analysis may be used.

Abutment Design

Design displacements and forces are obtained from an analysis using the lower level design spectra and appropriate stiffnesses of columns and abutments. There should be no engagement of the abutment backwall and there should be no failure of any fuses.

Preliminary Design

During the preliminary design stage, due considerations should be given to the following items:

1. The selection of span configuration. Unbalanced dead load moments in the columns should be minimized as much as possible;

For the upper level design event an analysis is required to determine the maximum displacements and forces that may occur. This analysis is performed assuming any fuses or keys have failed. If the abutment backwall is engaged, limits on the displacement are provided to avoid excessive damage.

If energy dissipation or seismic isolation bearings are used the site specific response spectra should be modified to incorporate

the additional damping provided by these devices.

Design Forces

Load Case 1: 100% of the absolute values of force and moment in transverse direction are added to 30% of the corresponding forces and moments from the longitudinal direction.

Load Case 2: 100% of the absolute values of force and moment from the analysis in the longitudinal direction are added to 30% of the absolute value of the corresponding forces and moments from the transverse direction.

Columns: Each column of the structure is designed to withstand the forces resulting from each load combination from the lower level event according to Caltrans specification.

Foundations

Foundations shall be designed for the forces resulting from plastic hinging of the top and bottom of the column. The column plastic moment capacity shall be 1.3 times the moment capacity obtained using a ϕ -factor of 1.

Transverse Abutment Shear Keys

The transverse shear keys, if used, shall be designed for the forces resulting from each load combination from the lower level event according to Caltrans specifications using a ϕ -factor of 1.

Abutment Piles

The combined pile and wingwall capacity at the abutments shall have sufficient lateral capacity to resist the design forces required. If fuses are provided for the lower level event, these will provide the lower limit of the design forces. If no fuses are provided, design forces should be obtained from the upper level event.

Connections

The connections of the columns to the foundations, superstructure or bent cap shall be designed for the forces resulting from column plastic hinging using a ϕ -factor of 1.3. If bent caps are used, the joints of the bent caps shall be designed to resist the plastic hinge moments developed by the column and the bent cap beam both determined using a ϕ -factor of 1.3.

Girder Support Length

The minimum support lengths (N) of all girders shall be the greater of either those obtained from an analysis using the maximum credible spectra or

$$N = 12 + 0.3L + 0.12H \text{ (inches)}$$

where

L = length (ft.) of bridge to adjacent joint or bridge end.
H = height (ft.) of columns.

Ductility Demand Design Check

The column forces and displacements resulting from the maximum credible analytical model shall be used to check that each column has a ductility demand less than 3 for single columns and less than 4 for multicolumn bents. The column forces shall be determined by incorporating the P- Δ effect if the column displacements are judged to be significant.

CONCLUSIONS

A two level design criteria has been developed for the bridges in California's first toll roads. The intent of the criteria is to have a higher level of safety and reliability than Caltrans current minimum requirements. The criteria when implemented should prevent significant damage for at least a 72 year return period event. By comparison Caltrans current requirements would prevent significant damage for a 25 to 50 Year return period event.

Several different design strategies were included in the performance and cost study that was performed to aid in the development of the criteria. It was shown that for relatively small increases in initial cost there were several options for enhancing the seismic performance of the bridges. These included designing the columns for higher force levels to reduce column damage; the use of energy dissipators at abutments for 2 to 5 span bridges which will provide a 30 to 50% reduction in displacements and column ductility demands; and the use of seismic isolation which can eliminate column damage.

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