Optimization of Seismic Design of Single-Column Circular Reinforced Concrete Bridge Piers

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A design algorithm is developed to incorporate design philosophy for seismic capacity in a computer program for the optimal design of single-column circular reinforced concrete bridge piers for seismic loading. The program designs the circular column as a single degree of freedom system under the combined effect of axial and lateral seismic loads over a broad range of axial load ratio, column height, and design displacement ductility capacity. For a given column height and axial load, results indicate the existence of an optimal column diameter and design displacement ductility level. As the column diameter is reduced, cost savings are effected by reduced volume of concrete but tend to be offset by $P - \Delta$ effects, increased longitudinal reinforcement for flexure, and increased transverse reinforcement for confinement and shear. On the basis of common trends, solutions are provided for the most economical range of axial load ratio and design displacement ductility capacity for a given column height.

Any structural design should include consideration of aspects of optimization. This means developing several technically feasible alternatives, evaluating their efficiency, and then making the best engineering choice. Comparative assessment of design efficiency requires predicting structural response and expected cost of design and construction, and estimating potential damage under design-level earthquake loading. Guidance on selection of structural alternatives and optimization procedure is not provided by design codes, the role of which is the specification of criteria to ensure that performance goals are met. The purpose of this study was to investigate cost-based optimal solutions for single-column circular reinforced concrete bridge bents subjected to transverse seismic loading. This is to be done by encoding a capacity design procedure in a computer program and performing comparative designs over a practical range of column height, column diameter, and design displacement ductility level.

The designer's prime variable will be the column diameter, which is directly related to the costs. A change in the column diameter would influence factors such as column stiffness and hence the natural period and lateral design force. For a given superstructure mass, a natural direction toward optimization thus would appear to be to decrease column diameter, which generally reduces the lateral seismic design forces because the reduction in stiffness normally will shift the natural period to a range of lesser dynamic response. Other effects, however, could become important: increased confinement steel requirements, greater longitudinal steel requirements, more prominent $P - \Delta$ effects, and increased shear requirements. Optimization of the design thus involves finding the right balance among these counteracting influences.

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SCOPE OF RESEARCH

Although the seismic capacity design approach for ensuring a ductile response of bridges has been prevalent for some time, not much has been done to investigate optimal choices in the seismic design of reinforced concrete columns. It is common practice in Japan to build bridge columns with low axial load ratios and low ductility demand and, hence, large diameter sections. On the other hand, in New Zealand and the United States, columns tend to be designed with higher ductilities and higher axial load ratios, and hence smaller diameter sections. The economics and structural desirability of the alternatives are unclear. To obtain insight into this area, a circular single-column bridge bent under transverse response was investigated. Various aspects of seismic design were considered by analysis for the following range of column axial load ratio, column height, and ductility capacity:

1. Axial load ratio, $A = P/f'_cA_g$, between 0.05 and 0.50, in steps of 0.05, where P is the axial load acting on the column and is equal to 1,200 kips (5338 kN) in this study; f'_c is the concrete compressive strength, which is assumed to be equal to 4 ksi (27.6 MPa) in this study; and A_g is the gross sectional area of the circular column.
2. Column height, L, ranging between 10 ft (3.05 m) and 50 ft (15.25 m), in increments of 10 ft (3.05 m).
3. Design displacement ductility demand, μ, varying between 2 and 10, in increments of 1.

Some of the more detailed aspects of capacity design approach, as applicable to the seismic design of circular bridge columns, have been discussed elsewhere (*1*); only issues relevant to the optimization computer program OPTCOL are addressed here.

RELATIONSHIP AMONG DUCTILITY, FORCE REDUCTION FACTOR, AND TRANSVERSE REINFORCEMENT

Displacement ductility, related to lateral displacements measured at the center of mass, is the most convenient measure of gross deformation; it can be related directly to the force reduction factor reducing the design lateral force from the elastic response level. Thus, the displacement ductility ratio is given by

$$\mu_\Delta = \Delta_u/\Delta_y \tag{1}$$

where Δ_u is the ultimate, or maximum, displacement and Δ_y is the lateral displacement corresponding to the yield point of equivalent elastoplastic response. This concept of displacement ductility is shown in Figure 1 for a single-column cantilever bridge pier idealized as a single degree of freedom system subjected to an axial load and a transverse seismic force. Also shown are the assumed deflected shapes of the column at first yield and at maximum ductility demand.

The following relationship between displacement ductility factor, μ_Δ, and the force reduction factor, R, is used in this study to assess the required levels of inelastic seismic response forces (*1*):

$$R = 1 + \frac{(\mu_\Delta - 1)T}{1.5T_0} \geq \mu_\Delta \tag{2}$$

where T_0 is the period corresponding to peak spectral response and T is the fundamental period. Equation 2 provides a gradual reduction in R from the equal acceleration principle ($R = 1$, regardless of μ) at $T = 0$, through the equal energy approximation [$R = (2\mu_\Delta - 1)^{1/2}$] at about $T = 0.75T_0$, to the equal displacement approximation ($R = \mu$) for $T \geq 1.5T_0$. For this study, $1.5T_0$ was taken to be equal to 0.7 sec.

Concepts addressing issues related to column curvature ductility, its relationship with displacement ductility and plastic hinge length L_p, and required level of transverse reinforcement for confinement provision are discussed elsewhere (*1*). These concepts have been incorporated in the program OPTCOL.

FLEXURAL DESIGN PRINCIPLES

Based on tests on large-scale bridge columns, it has been noted that the prediction of flexural strength of confined columns on the basis of the American Concrete Institute (ACI) concrete compressive stress block—which assumes a mean stress of $0.85f'_c$, an ultimate compression strain of 0.003, and the measured material strengths—results in excessively conservative prediction of actual strength. This predicted moment is hereafter termed the "ACI moment." Based on a large number of tests, the average moment enhancement as related to the ACI moment may be expressed empirically (*1*) as

$$\text{for } P/(f'_c A_g) \leq 0.1\text{: } M_{\max}/M_{i,\mathrm{ACI}} = 1.13 \tag{3a}$$

$$\text{for } P/(f'_c A_g) > 0.1\text{: } M_{\max}/M_{i,\mathrm{ACI}} = 1.13 + 2.35\,\{(P/f'_c A_g) - 0.1\}^2 \tag{3b}$$

To avoid the unnecessary conservatism inherent in the ACI computations, flexural strength calculations should use a stress-strain relationship applicable to confined concrete (*2*) and a higher ultimate compression strain, say 0.005. Alternately, as suggested by Priestley and Park, the dependable flexural strength may be taken as

$$M_u = \phi K M_{i,\mathrm{ACI}} \tag{4}$$

where K is the moment enhancement ratio given by Equation 3 ($K = M_{\max}/M_i$), and ϕ is a flexural strength reduction factor, taken as $\phi = 0.9$ for all axial force levels of well-confined columns. This procedure has been included in the program OPTCOL for the enhanced level of column flexural strength.

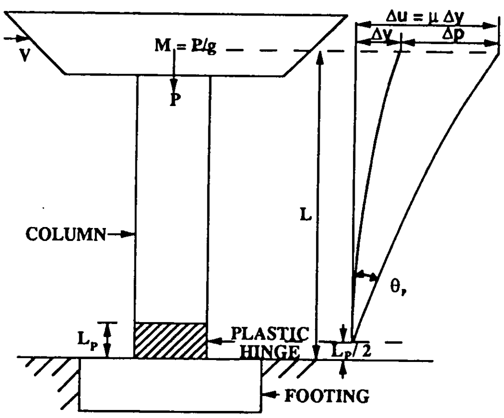

FIGURE 1 Lateral seismic response of a cantilever bridge column.

DESIGN SHEAR FORCE

The current seismic design philosophy is to ensure against shear failure by setting the shear strength of a bridge pier higher than the maximum flexural strength that can be developed. These actual shear forces generated during earthquakes may be as high as three times the code-specified nominal values (*3*), if conventional flexural strength design is adopted. The ideal shear strength, V_i, should be matched to the overstrength shear force, V_D, because use of a reduction factor for shear strength is generally deemed inappropriate when the shear force is established on the basis of principles of capacity design. In this study, the shear strength of a column is based on the ACI approach of considering separate concrete shear-resisting mechanisms, V_c, and steel shear-resisting truss mechanisms, V_s. Thus the actual requirement is

$$V_i = V_c + V_s \geq V_D \tag{5}$$

Shear carried by concrete and steel can be assessed conservatively in accordance with the provisions of the New Zealand (NZ) Concrete Design Code (*4*), which is followed in the program OPTCOL for the design of column shear reinforcement, both within and outside the plastic hinge regions. It should be noted that recent research shows the shear strength of plastic hinge regions is a function of ductility demand and the NZ shear design equations for both hinging and nonhinging zones are somewhat conservative. Consequently, the amount of transverse reinforcement provided in this study for shear strength will be slightly conservative compared with more recently developed design criteria (*5*) currently under evaluation for design.

SEQUENCE OF OPERATIONS FOR OPTIMIZATION

The design approach outlined previously is appropriate for the design of any simple bridge column, given a knowledge of the required flexural strength and ductility. To draw conclusions about the economic consequences of adopting different design alternatives, the following sequence of operations was adopted for a specified column height:

1. Select the design response spectrum. Five elastic spectra were considered in this study.
2. Select an axial load ratio. This defines the circular column diameter.
3. Select the design displacement ductility. This enables the lateral design force to be selected by an iterative procedure requiring sequential estimation of member stiffness, natural period and, hence, the lateral seismic force.
4. Design longitudinal reinforcement for the provision of flexural strength.
5. Design transverse reinforcement for the provision of ductility and shear requirements.
6. Estimate the total costs involved in the design alternatives.

Some of the aspects of this optimization sequence are discussed in the following sections.

Response Spectra

The traditional method for describing ground shaking is a smoothed elastic response spectrum for single degree of freedom systems. Five response spectra, shown in Figure 2, were used to allow assessment of the influence of various spectral characteristics on the overall seismic design of bridge columns. They are the AASHTO Guide-based spectra specifications for an acceleration coefficient equal to 0.4 and for stiff clay or deep cohesionless conditions (*6*); the California Department of Transportation (Caltrans) A.R.S. Spectra for 150 ft or deeper soil alluvium, with peak ground acceleration equal to $0.5g$ and $0.7g$ (*7*); the Seed/Sun spectra proposed for deep soft soils after the Loma Prieta earthquake; the New Zealand Zone A inelastic design spectra for a peak ground acceleration equal to $0.5g$ (*8*); and a constant elastic design spectrum at $0.75g$.

Natural Period

Following the choice of a certain axial load ratio (and hence the column diameter, for a known axial load) and a design displacement ductility capacity for a circular column with a specified height, the next step involves calculating the equivalent natural period of vibration of the cantilever column idealized as a single degree of freedom system, as shown in

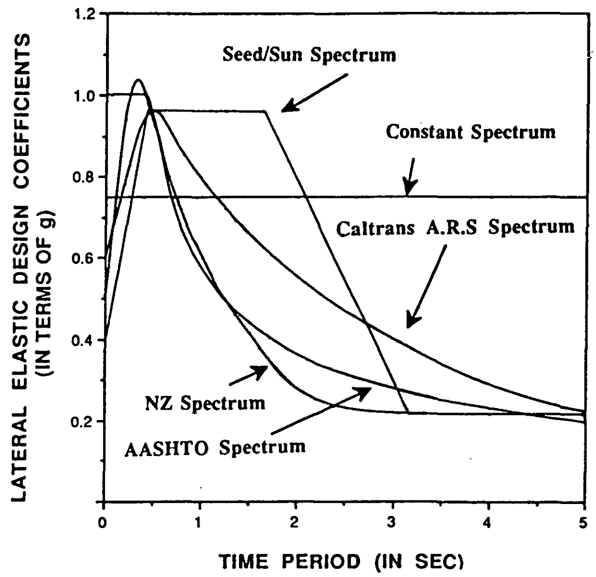

FIGURE 2 Comparison of five response spectra.

Figure 1. This is given by

$$T = 2\pi\sqrt{\frac{m}{k_{eq}}} \tag{6}$$

where

m = mass of the superstructure = P/g,
P = specified axial load on column,
g = acceleration resulting from gravity, and
k_{eq} = equivalent lateral stiffness of the cantilever column including both flexure and shear flexibility terms.

The stiffness should correspond to conditions at first yield and thus is influenced by the axial force level and the reinforcement content (*9*). Since the latter is not known at the start of the analyses, an iterative approach was used in this study (*10*). The initial lateral stiffness of the section was calculated by assuming the sectional moment of inertia to be 50 percent of that for the gross circular section and ignoring the influence of longitudinal reinforcement. The effective shear area was assumed to be equal to $0.9Ag$, where Ag is the gross sectional area, and the value for the shear modulus G was assumed equal to $0.4E$. This was followed by the estimation of the lateral seismic design forces and assessment of the longitudinal steel requirements for the total flexural moment including the $P - \Delta$ moments. An iteration scheme was followed for achieving a desired level of accuracy in the amount of longitudinal steel. For the first iteration in every cycle in the column design and for all other subsequent cycles when the accuracy was not reached, the actual lateral stiffness of the cracked section was computed on the basis of a modified equation for the effective moment of inertia of the circular reinforced concrete column section.

Design Base Moment

Once the actual design lateral seismic force has been evaluated on the basis of a modified natural period of the section, an estimate of the total overturning moment acting on the column base is made, as shown in Figure 3. The base moment will thus equal

$$M_b = V \cdot L + P \cdot \Delta \tag{7}$$

where

V = lateral seismic design force,
P = axial load,
Δ = peak lateral displacement, and
L = column height.

Designing for the full $P - \Delta$ moments is known to be somewhat conservative, and approaches suggesting reductions of close to 50 percent have been made (*9*) on the basis of energy considerations. The trend toward more slender columns, mainly in the United States and New Zealand, has led to potentially more significant $P - \Delta$ effects, and thus the justification to include these effects in the calculation of the total bending moment acting on the column.

Figure 3 also compares the bending moment diagram and the deflected shape of the column at an initial ductility level of $\mu = 2$ and for an inelastic state at $\mu = 10$. As is evident from the diagrams, the influence of $P - \Delta$ effects is significant at higher ductility levels for the same column. Also, the deflected shape, and hence the bending moment diagram, are almost linear at high ductility levels, with the majority of rotation being concentrated at the plastic hinge forming at the column base.

To include the $P - \Delta$ effects in the program OPTCOL, an approximate approach was used to determine the column peak lateral displacements. In the elastic case, the equation for simple harmonic motion can be used to determine the peak yield displacement of the column. If Δ is the total maximum inelastic displacement occurring at the designed displacement ductility level μ, it can be shown (*10*) that

$$\Delta = |y_{\text{inelastic}}|_{\text{max}} = \mu \, |y|_{\text{max}} = \mu \, \frac{T^2}{4\pi^2} |\ddot{y}|_{\text{max}} \tag{8}$$

where $|\ddot{y}|_{max}$ is the magnitude of the peak ground acceleration and w ($= 2\pi/T$) is the angular frequency specified in radians per second, $|\ddot{y}|_{max}$ can be obtained, for a specified natural period of vibration and on the basis of any characteristic design response spectra, as the inelastic design coefficient times the acceleration due to gravity.

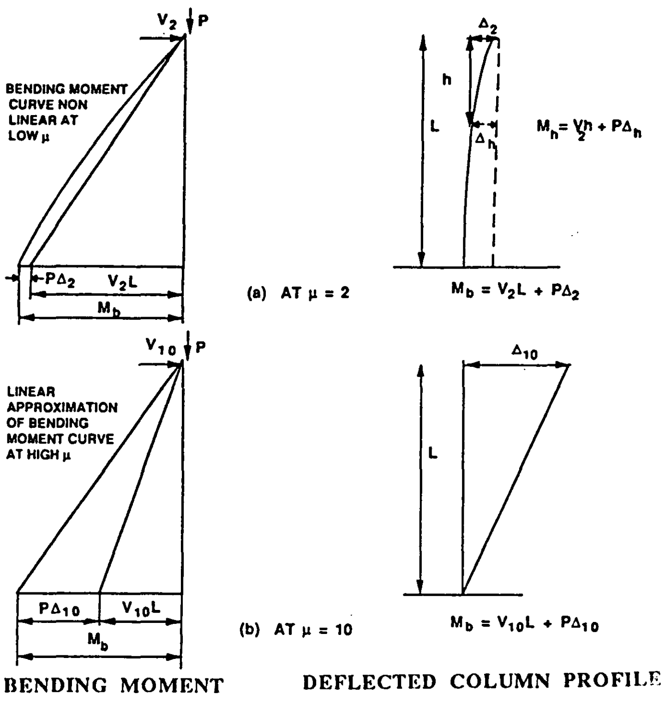

FIGURE 3 Design base moments and $P - \Delta$ effects for a cantilever column.

Concrete and Steel Cost Evaluation

The total amount of concrete and steel used in the various cases of column design was calculated. This calculation was followed by an estimation of the respective costs on the basis of current rates in California, including the cost of form work, pouring of concrete, and cost of cutting, bending, and placing the steel. The costs were assumed to be equal to \$350/yd^3 for concrete and \$0.60/lb for the steel. To assess the effect of varying the ratio of unit prices of concrete and steel, all cases also were analyzed for a concrete price of \$60/yd^3, while the steel price was held constant.

Figures 4 and 5 show typical total cost versus axial load ratio plots for a design displacement ductility level of μ equal to 6 for the AASHTO response spectra, the New Zealand Zone A spectra, and the Caltrans A.R.S spectra, respectively. Figure 6 shows total cost versus displacement ductility capacity plots for the AASHTO response spectra for column heights equal to 20 ft (*top*) and 40 ft (*bottom*), respectively.

RESULTS AND CONCLUSIONS

The following conclusions may be drawn from results for the five cases analyzed:

1. For a specified column height and required design displacement ductility level, an increase in the axial load ratio, and hence a decrease in the column diameter, results in cost savings for a limited range of the axial load ratio. Beyond the value of the optimal column diameter, other effects—such as dominant $P - \Delta$ effects, increased confinement requirements for shear and ductility, and greater longitudinal steel for flexure—result in either increased costs or "flattening" of the cost plots, as shown in Figures 4 and 5. The optimal axial load ratio decreases as the column height increases and is usually in the range of $0.10 \leq A \leq 0.30$ for column height in the range of 50 ft (3.05 m) $\geq L \geq$ 10 ft (15.24 m).

2. For a specified column height and a low value of the axial load ratio, the total cost decreases for increasing the design level of the displacement ductility capacity up to a certain optimal μ, beyond which the costs start increasing, as shown in Figure 6 (*top*). This subsequent increase is due to greater costs for provision of transverse confinement for the higher levels of ductility capacity. The optimal design ductility capacity is typically in the range of $6 \leq \mu \leq 10$. However, for slender piers with higher levels of axial load ratios, the

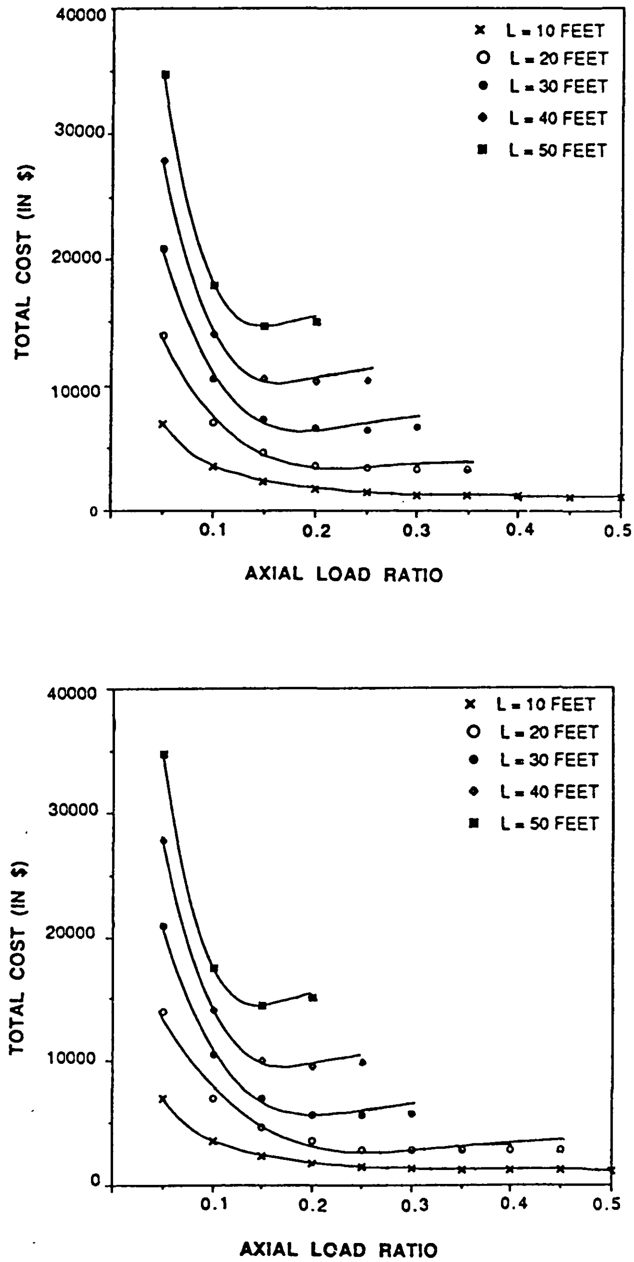

FIGURE 4 Total column cost versus axial load ratio (P/f'_cAg) for AASHTO spectra (*top*) and New Zealand spectra (*bottom*).

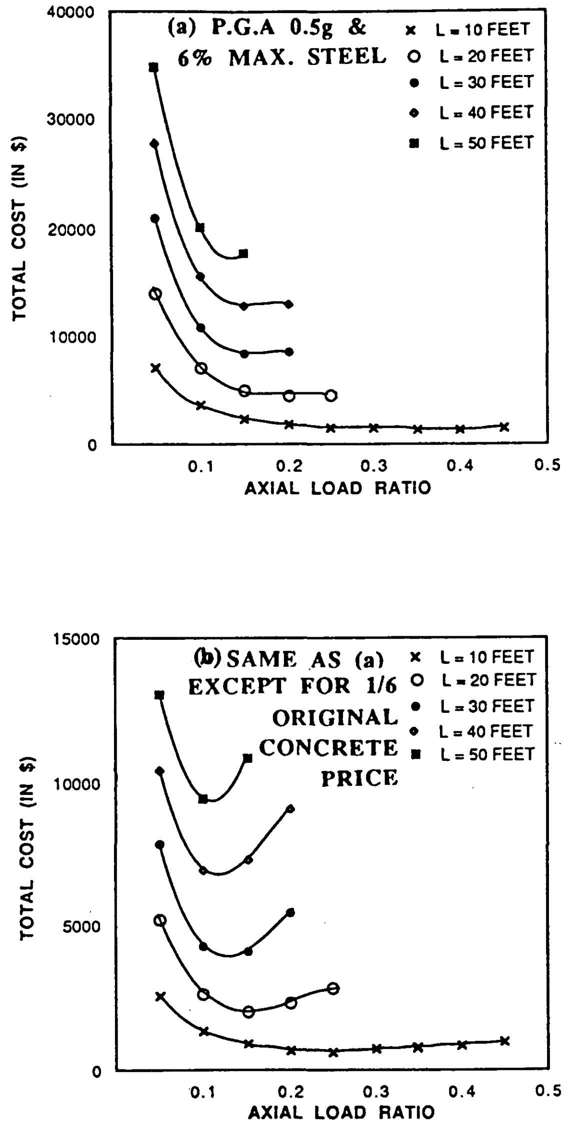

FIGURE 5 Total column cost versus axial load ratio (P/f'_cAg) for Caltrans A.R.S spectra.

costs simply go on decreasing for increased levels of design displacement ductility level, as shown in Figure 6 (*bottom*). This occurs because for slender piers, for a specified column height and a selected value of the column diameter, the increase in costs caused by increased confinement requirements for greater μ is offset by the decrease in the longitudinal steel requirements owing to the reduced design inelastic seismic forces for higher levels of ductile response.

3. An increase in the maximum allowable longitudinal steel ratio from 6 to 8 percent leads only to more design cases being possible, without any associated savings in the total design cost, as the steel cost is already dominant at that stage. The effect of increasing the design peak ground acceleration is to reduce the number of possible design cases as the maximum steel ratio limit of 6 percent is exceeded for higher seismic design moments. This effect also significantly increases the total costs for slender piers for the same reason.

4. To study the effect of altering the ratio of unit concrete to steel price, the concrete price was reduced from $350/yd^3 to almost one-sixth value at $60/yd^3 while the steel cost was held constant. The result: the existence of distinct optimal axial load ratios for given column heights and design displacement ductility levels is observed, in accompaniment with sharply rising cost curves beyond the optimal minima. This is shown in Figure 5(*b*) for the Caltrans spectra, which, when compared with cost plots at concrete price of $350/yd^3 in Figure 5(*a*), reflects the sensitivity of the shape of the cost plots to varying cost ratios. However, the influence of lower concrete cost on the optimal axial load ratio is not significant. The tendency, as expected, is to lower somewhat the values of the optimal axial load ratio.

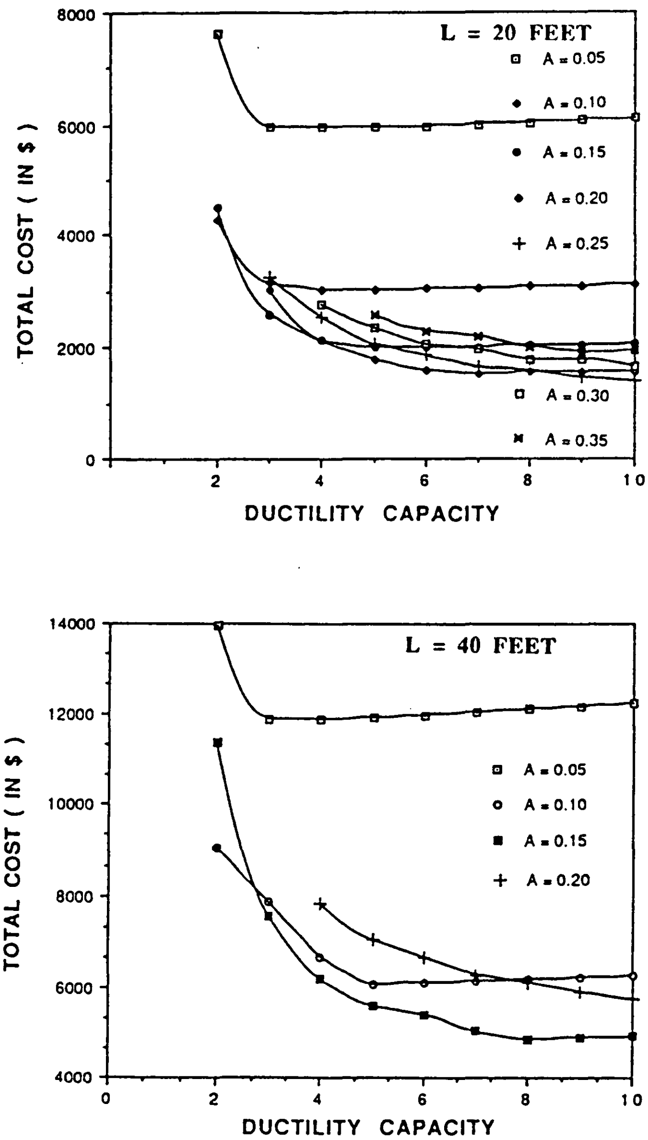

FIGURE 6 Total column cost versus design displacement ductility capacity, μ, for AASHTO spectra.

5. From the cost plots for this study, the general trends in the seismic design solutions for single-column reinforced concrete bridge piers are clear. Table 1, based on the results of the optimal seismic design of bridge bents of single-column circular reinforced concrete and using five design response spectra, presents the range of axial load ratio A and the corresponding range of the possible design displacement capacity, μ, for the most economical solutions at a specified column height L, in the range of L = 10 ft (3.05 m) to 50 ft (15.24 m).

6. The solutions recommended by this optimization approach are based solely on ultimate seismic displacement consideration. In many cases, gravity load considerations may dominate and make the recommended high ductility factors impractical. The use of high ductility factors also may result in excessive incidence of minor damage, such as spalling of cover concrete, under a moderate earthquake that may be expected to occur several times in the design life of a bridge.

TABLE 1 Optimal Range of Design Axial Load Ratio and Displacement Ductility Capacity

COLUMN HEIGHT (FT.)	OPTIMAL AXIAL LOAD RATIO RANGE	OPTIMAL DUCTILITY CAPACITY RANGE
10	$0.25 \le A \le 0.35$	$6 \le \mu \le 10$
20	$0.20 \le A \le 0.25$	$6 \le \mu \le 10$
30	$0.15 \le A \le 0.20$	$7 \le \mu \le 10$
40	$0.10 \le A \le 0.15$	$7 \le \mu \le 10$
50	$0.10 \le A \le 0.125$	$7 \le \mu \le 10$

This will be particularly pronounced in the case of columns with high axial load ratios and high longitudinal reinforcement ratios.

Nevertheless, the results appear to justify, both technically and economically, current practice for seismic design of columns in axial load ratio and ductility level. However, current Caltrans practice (*7*) of limiting displacement ductility factors for single-column piers to values as low as μ = 3 (dependent on the period) does not appear to be justified based on the results of this study.

RECOMMENDATIONS FOR FUTURE RESEARCH

This study focused on single-column piers in their transverse response to seismic excitation. Extending the study for multiple-column bridge bents, especially with taller piers and under bidirectional seismic attack, should then follow as a logical step. Rectangular column piers also may be included in the study. Comparative study should be made as to the most efficient number of columns at any bent (e.g., comparing the cases for two- and three-column bents with the results for the single-column bent).

Influence of foundation flexibility on the optimization of the seismic design of the bridge bent also should be investigated further and included in the optimization procedure. Its effects will particularly pronounce the $P - \Delta$ moments for slender piers and could alter the existing trends. Also, a change in the column diameter, and hence the stiffness, would affect both the footing and the superstructure design. These aspects also need to be examined.

A more consistent approach should be used to assess the shear strength of the bridge columns during ductile response. Further research needs to be done on aspects related to dynamic shear strength of single- and multiple-column bridge piers, behavior of columns in double curvature, influence of practical levels of reinforcement on shear strength, and effects of axial tension and multidirectional loading on the concrete shear carrying capacity.

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