SEISMIC DESIGN CRITERIA FOR BRIDGES

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After the earthquake at San Fernando on February 9, 1971, significant changes were made in structural details of bridges designed by the Division of Structures, California Department of Transportation. New seismic design criteria were developed that consider both fault proximity and local soil conditions. Response coefficient curves, which are a key element of these new criteria, were developed by determining average elastic response spectra for rock motion and then modifying these spectra to reflect the influence of soil conditions. Considerable engineering judgment was used in assessing ductility and acceptable risk so that the elastic curves could be scaled down to a desirable level for design. Response coefficient curves can be used to find suitable equivalent static force coefficients or as design spectra for use in a dynamic analysis of a response spectrum. The simplified design method is adequate for small structures, but a dynamic analysis is required to accurately predict the response of long, curved structures with intermediate hinges and widely varying column lengths.

•PEOPLE have always been in awe of great earthquakes. The fear and confusion that they feel after witnessing sudden death and massive destruction have evoked profound contemplations. For example, after the great Lisbon earthquake of 1755, which killed more than 60,000 people, about half of them in the collapse of churches, John Wesley wrote a sermon on The Cause and Cure of Earthquakes, in which he blamed the Lisbon earthquake on the original transgressions of Adam and Eve. The Moslems might have thought it just retribution for the cruelties of the Portuguese Inquisition had not the Mosque of Al-Mansur in Rabat also lain in ruins (1).

The 1971 San Fernando earthquake was small compared to the one in Lisbon. Fortunately, the casualties were a thousand times smaller: 63 deaths instead of 60,000. Likewise, our contemplations were less profound. Rather than questioning why earthquakes occur, we concentrated on changes we should make in our design practice to improve the performance of future structures.

Before February 9, 1971, none of the approximately 11,000 bridges in the California highway system had failed or been extensively damaged by earthquakes. Even though many bridge structures in the region of extreme ground shaking at San Fernando survived with negligible to moderate damage, this event triggered a turning point in bridge design.

When the damage at San Fernando was investigated and the evidence examined, we reached two major conclusions:

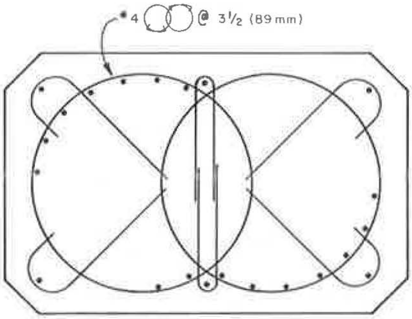
1. Deficiencies in details, especially at connections, played a major role in all of the spectacular failures; and
2. California earthquake design criteria, although more conservative and rational than current AASHTO specifications, needed a major revision.

IMPROVED DETAILS

The first conclusion led to immediate action to revise details that performed poorly. Two of the most important changes were the modification of column details and the addition of restraining devices at expansion hinges and bearings.

The column tie reinforcement for new bridges is predominantly of the spiral type.

Figure 1. Spiral reinforcing for rectangular column.



The spirals interlock to confine rectangular column concrete as shown in Figure 1. The superior performance of spiral columns over tied columns was abundantly evident in the review of both bridge and building damage. For columns 30 ft (9.1 m) high or less, lap splices are no longer permitted. For higher columns, 60 diameter lap splices are permitted for No. 11 and smaller bars except within 10 ft (3 m) of moment-carrying connections. Butt-welded and mechanical butt splices that conform to state specifications are permitted anywhere in main column reinforcement.

Narrow hinge seats fared badly under the violent shaking of the San Fernando earthquake. Figure 2 shows a typical hinge restrainer now used on new box girder structures. We also avoid the use of bearings that do not provide a positive tie down, such as the simple rocker bars that were once commonly used on simple span steel girder structures.

NEW DESIGN CRITERIA

Development of new criteria was necessarily preceded by a crash effort to become better educated in earthquake engineering. As an interim measure, we doubled the previously used static earthquake design factors for bridges with spread footings and increased by $2\frac{1}{2}$ times the factors for bridges founded on piles. This change brought our ceiling design force levels to about 0.20 and 0.25 *g*.

The criteria that we implemented in February 1974 are innovative in that they consider site peculiarities such as fault proximity and soil depths more accurately than older codes. They also consider the dynamic characteristics of the structure being designed, because they are essentially a design spectrum technique. Response coefficient curves (Figure 3) were developed by starting with average rock response spectra, then modifying them to reflect soil conditions, and scaling them down to force levels that we can afford to accommodate. To select the proper curve the designer must know the maximum anticipated rock motion and the approximate depth of alluvium.

To understand these criteria we will look first at the code format and then at the rationale used in developing its key element, the response coefficient curves. Finally, we will look briefly at application of the criteria in bridge design.

CODE FORMAT

$$EQ = C \cdot F \cdot W \tag{1}$$

where

EQ = equivalent static horizontal force applied at the center of gravity of the bridge,
F = framing factor = 1.0 for bridges where single columns or piers resist the horizontal forces and 0.8 for bridges where continuous frames resist horizontal forces applied along the frame, and
W = total dead load weight of the bridge.

$$C = \frac{A \cdot R \cdot S}{Z} \tag{2}$$

Figure 2. Typical hinge restrainer unit for a box girder bridge.

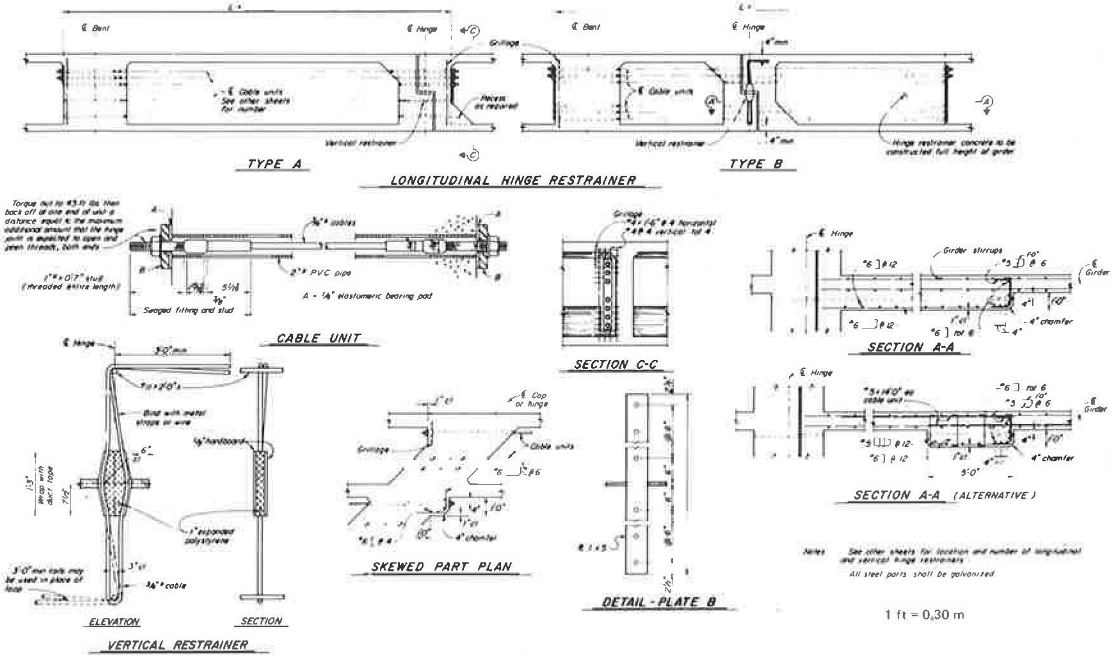


Figure 3. Response coefficient curves for (a) 0 to 10-ft-deep (0 to 3-m) alluvium, (b) 11 to 80-ft-deep (3.4 to 24-m) alluvium, (c) 81 to 150-ft-deep (25 to 46-m) alluvium, and (d) more than 150-ft-deep (46-m) alluvium.

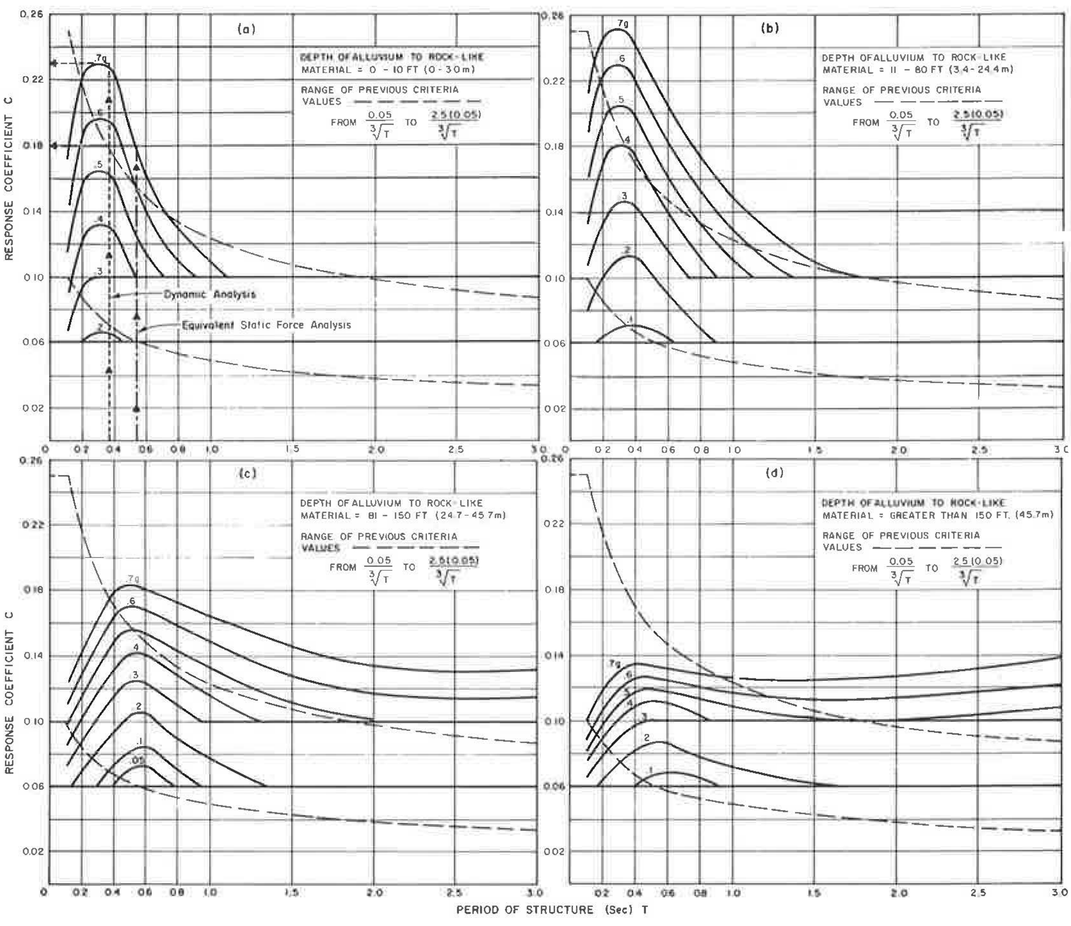
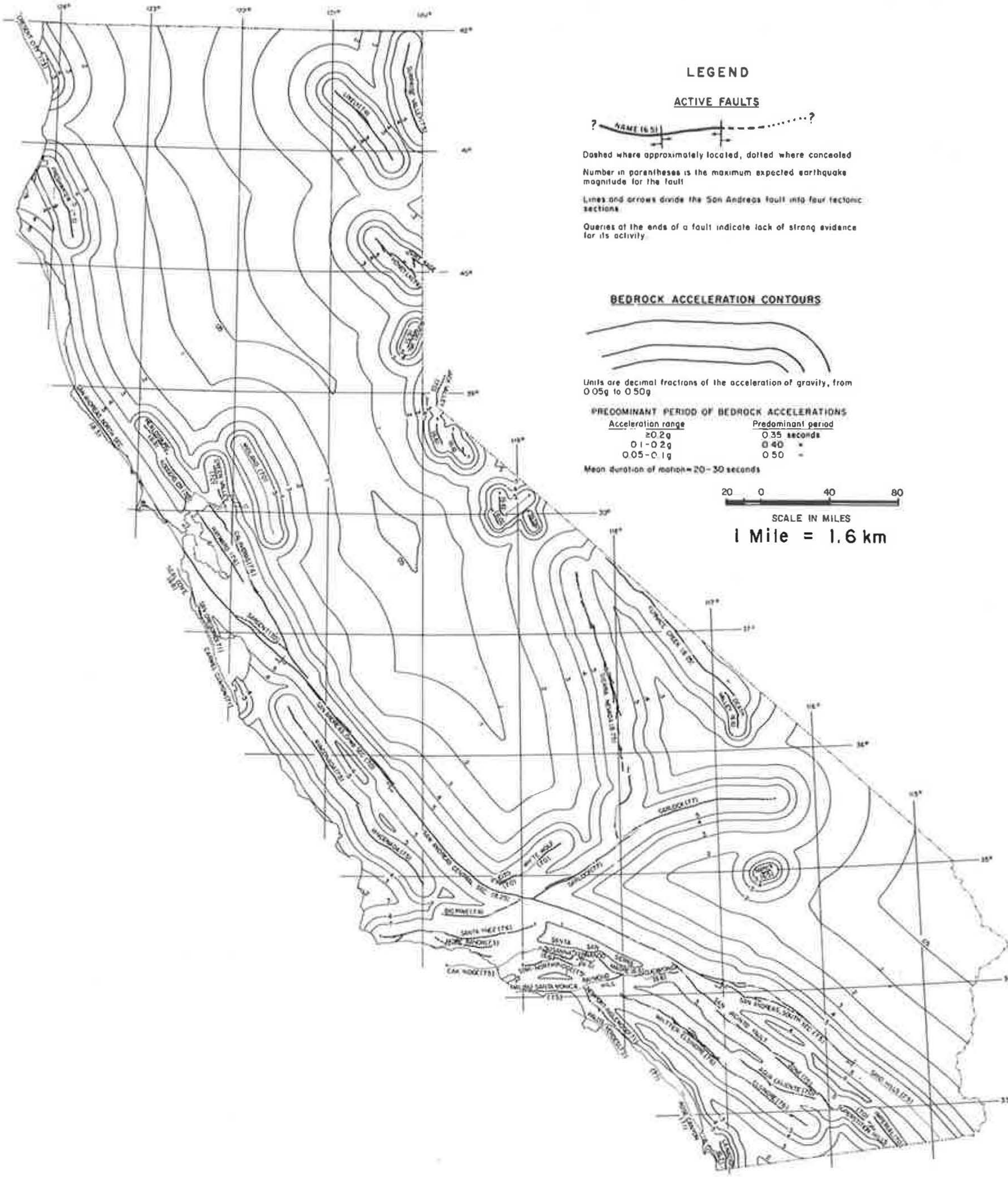


Figure 4. Maximum bedrock accelerations expected from earthquakes in California.



where

C = combined response coefficient (values of coefficients for various depths of alluvium to rock-like material are given in Figure 3),
A = maximum expected acceleration at bedrock at the site in terms of g,
R = normalized rock response,
S = soil amplification spectral ratio, and
Z = reduction for ductility and risk assessment.

ROCK ACCELERATION AND RESPONSE SPECTRA

The rock acceleration level A to be used at a particular site is taken from a map of California that delineates anticipated rock acceleration levels with a series of contour lines (Figure 4). This map was developed by the California Division of Mines and Geology (2). (The map is tentative and is intended as a tool for official use only. It is not intended for direct engineering use without consideration of foundation conditions and type of structure.) The technique used in its development involved plotting known active faults and assigning maximum credible earthquake magnitudes to them based on probable maximum length of rupture. These magnitudes were then related to acceleration levels, and the acceleration values were attenuated with distance from the fault.

The peak rock acceleration cannot be used by itself to get a design force. The dynamic characteristics of a structure greatly affect its response to a given ground motion. A structure with natural frequencies that are in resonance or "tuned in" with a ground motion will respond much more severely than one that has a natural frequency that is out of phase with the ground motion. Typically, a response spectrum is used to delineate this action.

Figure 5 shows normalized response spectra that we developed for 5 percent damped accelerations at rock outcrop locations. These curves, which define R in equation 2, were developed by using five actual recorded rock outcrop accelerograms. Multiplying the ordinates shown in the figure by the peak rock acceleration expected at a particular site will produce an average smoothed elastic rock acceleration response spectrum for that site.

INFLUENCE OF SOIL CONDITIONS

It is generally recognized that the type and depth of soil over bedrock will modify the rock motion dramatically. In some soils, the accelerations can be intensified by a factor of several hundred percent. Various methods for determining the influence of the soil have been proposed and are now being considered by various code-making bodies.

To develop the S-factor, we analyzed several hundred alluvial soil columns by using different earthquakes and varying the depth and density of the soil through use of a computer program called SHAKE that was recently developed at the University of California, Berkeley (3). This program analyzes a one-dimensional soil column for wave motions propagating from rock level to the top of the column and computes motions at the top of the soil column.

To separate the effect of soil from that of other factors such as unusual peaks and valleys in a particular response spectrum, we examined spectral ratios, i.e., the surface acceleration spectra divided by the rock acceleration spectra. The soil amplification curves thus computed were very smooth and regular and appeared to adequately represent soil effects only. The amplification curves varied only slightly with different earthquake motions. S-values from the soil amplification curves such as the one shown in Figure 6 are used as multipliers to find surface spectra from rock spectra.

We found that depth of the soil to rock-like material was the major variable affecting soil response. Figure 7 shows this phenomenon. For two sites where equal rock acceleration levels are anticipated, a structure with a natural period of 0.4 sec would experience a greater response when located on 50 to 80-ft-deep (15 to 24-m) alluvial deposits than when located on deposits of 150 to 250-ft-deep (46 to 76-m) alluvium. Conversely, a structure with a 2-sec period would experience a greater response when founded on the deeper soils.

We also found that the maximum rock acceleration has a significant influence on the amplification spectra. As the maximum rock acceleration increases, the amplification decreases and the predominant period of the soil column, indicated by the peak value on the amplification curve, lengthens.

To make use of these amplification curves requires that the site of the structure be investigated to determine the depth of alluvium. In California, this is no problem because we have been making borings at every bridge site for years to facilitate foundation design. We are now expanding this investigation to include an evaluation of soil response as well as an assessment of risks from liquefaction and landslides. For most sites, an experienced soils engineer or engineering geologist, using soil descriptions and standard penetration values, can select the proper amplification curve. For unusual sites, we have developed field procedures to measure shear wave velocities from which specific amplification values can be calculated.

REDUCTION FOR DUCTILITY AND RISK

Because our curves are based on elastic analysis, we were faced with the problem of scaling them down for use in design. Past experience has shown that, because of inelastic response, increased damping, ductility, and other factors, structures can resist considerably higher acceleration levels than are indicated by an elastic analysis. The state of the art has not progressed to a point where all these parameters can be adequately considered in the analysis. Therefore, considerable engineering judgment must be applied in reducing the elastic force levels to design levels. Forces determined elastically are commonly divided by ductility factors ranging from 2 to 6. We selected a general ductility factor of 4 for all bridges.

The theoretical elastic response values were adjusted further based on our assessment of reasonable risk. Our goal is to avoid total collapse under a severe earthquake. From a cost-effectiveness point of view, it is not a good investment to proportion bridges to avoid damage.

Short, low-period bridges such as the common two-span overcrossings are quite stable and less vulnerable to collapse than high structures. Even if columns are severely damaged, the stability provided at the abutments prevents the bridge from overturning. With increased column ties and greater use of spirals, collapse due to column disintegration is not likely to occur. These lower bridges can be readily shored up and restored to traffic use or removed under controlled conditions. Because the probability of collapse and loss of life is minimal for this type of structure, these bridges can be designed for lower force values than high, single-column bent, long-period bridges, which are more vulnerable to collapse.

An additional reduction factor for risk of 2.0 was assigned to the short stiff bridges with periods of 0.6 sec and less. This factor was then decreased linearly to 1.0 for bridges with a period of 3 sec. The ductility and risk factors were combined to produce a reduction curve, Z (Figure 8). The resulting total reduction values ranged from 8 at a 0.6-sec period to 4 at a 3-sec period. This curve was used to reduce the elastic response spectra to the final design coefficient curves for C.

In addition, a frame factor F was applied to the final design coefficient. The framing factor for single-column bents and piers is 1.0 and for continuous frames is 0.8. This factor is simply a numerical coefficient reflecting the increased stability and energy-absorbing characteristics of continuous frames compared with single-column bents.

MINIMUM FORCE LEVELS

No doubt faults exist in California that have not yet been identified and thus are not reflected on the seismic map. The fundamental assumption of the criteria, i.e., that ground motion can be predicted by attenuating rock acceleration with distance and by correcting it to a surface motion by modeling the overlying soil, is controversial. Some engineers think this procedure is a gross oversimplification. These critics point out that factors such as variations in source mechanism, the influence of terrain, and the chance combination of surface and body waves make it impossible to predict surface accelerations with reasonable accuracy by using our simplified procedure. We readily admit that this method is an idealization that does not account for all possible variables. However, we and many other professionals think it is the only practical method currently available for quantifying major variables. Because of the many uncertainties involved, as part of our design criteria we have established threshold values to be used regardless of an apparently favorable combination of factors. A minimum C-value of 0.10 is used where we expect a peak rock acceleration of 0.3 *g* or greater. For areas where peak rock acceleration is less than 0.3 *g*, a minimum C-value of 0.06 is used.

APPLICATION OF THE CODE

A designer currently has two approaches that he may use to apply these criteria. He may use the C-value to determine an equivalent static force, or he may perform a dynamic analysis. Using the equivalent static force method, the designer must obtain the period of the first mode of vibration in the direction under consideration. In addition, he must distribute the earthquake force to the substructure elements. This approach can give reasonable results for simple bridges that respond in one predominant mode of vibration. Dynamic analysis techniques are being used for geometrically complex structures that respond in many significant modes of vibration. The curves developed as part of these criteria can be used for the response spectrum technique that accounts for the effects of several modes. This approach results in a more accurate prediction of the response of structures subjected to earthquake loadings.

The equivalent static force method does, however, require an accurate determination of the fundamental period of vibration. As an illustration, compare the fundamental transverse period obtained for a two-span bridge by using three methods:

1. The formula for a single degree-of-freedom, lumped mass system (Figure 9),
2. A dynamic analysis using a multiple degree-of-freedom, lumped mass idealization (Figure 10), and
3. Field testing the actual structure.

The results are given in Table 1 along with the resulting force coefficients and column shears.

The formula method ignores the bending and torsional stiffness of the superstructure. This introduces an error in the determination of the period, which results in an incorrect C-value. In addition, it results in an inaccurate distribution of forces to the substructure elements.

SUMMARY

At the present time in California we are designing most simple structures by using the equivalent static force method. We are using dynamic analyses for all major structures. We think our new criteria used in a static analysis are a great improvement over older codes because they account for the major variables that influence earthquake force levels. However, because of the inaccuracies inherent in any static approach, the trend is toward using the response coefficient curves to define a design earthquake for a response spectrum analysis for most bridges with a fundamental period of less than 3.0 sec. For major structures with longer periods or those that have unusual configurations or foundation conditions, we will use a more rigorous individual dynamic analysis.

Figure 5. Normalized rock acceleration spectrum.

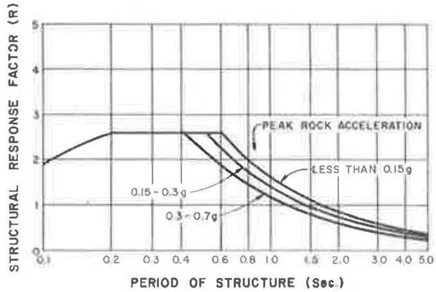


Figure 6. Soil amplification curves for 80 to 150-ft-deep (24 to 46-m) alluvium.

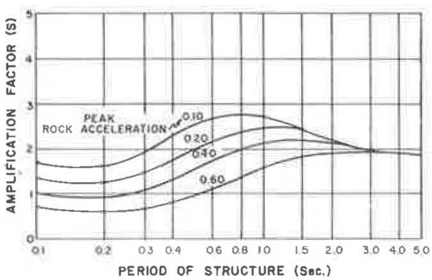


Figure 7. Soil amplification for different depths.

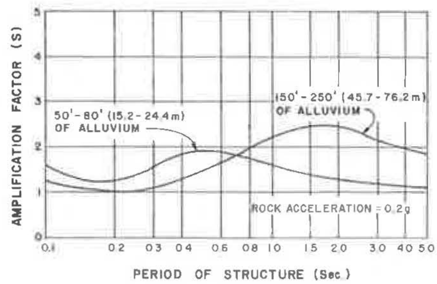


Figure 8. Reduction curve for ductility and risk assessment.

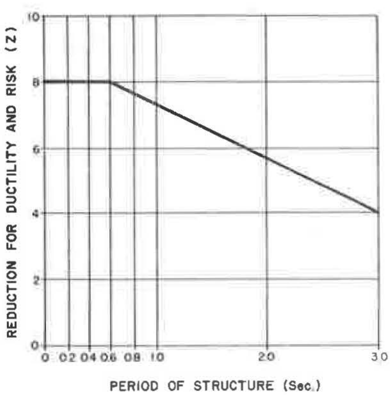


Figure 9. Simplified method for period determination.

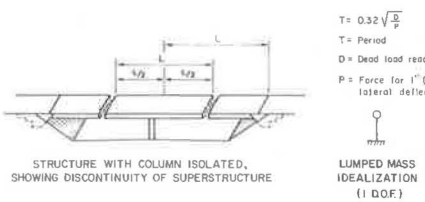


Figure 10. Dynamic analysis for period determination.

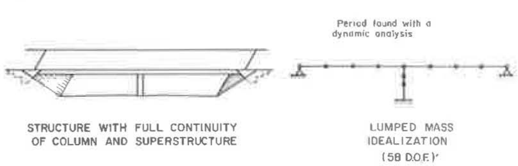


Table 1. Comparison of results from different analyses.

Method of Analysis	Period (sec)	Force Coefficient[a]	Column Shear (kips)
Simplified	0.54	0.18	212
Dynamic	0.37	0.23	270
Field test	0.33		

Note: 1 kip = 4.45 kN.
[a]From Figure 3.

ACKNOWLEDGMENTS

Many engineers and several engineering geologists in the Division of Structures, California Department of Transportation, have contributed greatly to improvements in earthquake engineering of bridges. The following employees have been particularly effective in these developments: James H. Gates developed normalized rock spectrum and soil amplification curves. Roy A. Imbsen developed guidelines for applying these criteria so that reasonable accuracy can be maintained with simplified methods, and he is responsible for sophisticated dynamic analysis of major structures. Adlai F. Goldschmidt developed techniques for measuring shear wave velocities in soils and expanded the geological report to include an assessment of potential soil problems from seismic forces. Oris H. Degenkolb developed hinge restrainer details. John B. Poppe improved column reinforcing details. I. Nagai coordinated the overall development of these criteria.

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