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Seismic Analysis and Design of Concrete Bridge Systems

Reported by ACI Committee 341

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* Subcommittee members.

This document, intended for use by practicing engineers, provides a summary of the state-of-the-art analysis, modeling, and design of concrete bridges subjected to strong earthquakes. The material in this report is intended to supplement and complement existing documents from American Association of State Highway and Transportation Officials (AASHTO), California Department of Transportation (Caltrans) and Uniform Building Code (UBC). Procedures and philosophies of current and emerging codes and guidelines are summarized. Linear and nonlinear seismic analysis methods are also discussed, and important modeling considerations for different bridge elements including curved girders and skewed abutments are highlighted. The report also includes a summary of analysis and design considerations for bridges with seismic isolation as well as general seismic design considerations for concrete bridges.

Keywords: abutment; bridges; columns; concrete; connections; design; earthquakes; footings; hinges; restrainers; seismic analysis; seismic isolation.

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CHAPTER 1—INTRODUCTION

The primary objective of all current U.S. seismic codes is to prevent collapse of the structure under the design earthquake. The codes recognize that it is uneconomical to design a bridge to resist a large earthquake elastically, and therefore some degree of damage is permitted and expected (Figure 1). It is intended that this damage be limited primarily to ductile behavior (flexural yielding) of the columns or pier walls, to nominal abutment damage, and to shear key breakage. These bridge elements lend themselves to relatively easy inspection and repair should acceptable damage be sustained during a seismic event. Unacceptable damage includes loss of girder support, column failure, foundation failure, and connection failure.

These performance requirements indicate why proper modeling of the bridge system is important. The calculated internal distribution of forces, expected deformations, and prediction of collapse mechanisms are directly related to the adequacy of the overall system model. Yielding of a single element in a structure is acceptable in a particular mode providing it does not lead to collapse. The formation of a local failure mechanism must occur before overall collapse can take place. The distribution (or redistribution) of loads in the structure, their relation to the formation of plastic hinges, and the prediction of the eventual failure mechanism, are the central goals of bridge systems analysis.

Structural evaluation of an overall bridge system is a challenging undertaking. Evaluations are typically performed at ultimate conditions, and limit analysis is used where progressive yielding is permitted until the structure collapses. Traditional code-based analysis procedures generally do not lend themselves to accurate determination of overall bridge system behavior. The internal force distributions (or redistributions) are different for each structure and will require careful evaluation and engineering insight. In many instances a single model does not provide sufficient insight into the overall system behavior. A series of incremental models providing progressive yielding, a bracketing of likely behaviors, or a sophisticated nonlinear model may often be necessary to provide an appropriate indication of force distributions within the structure, and of overall bridge behavior.

The purpose of this document is to provide practicing engineers general guidelines for overall bridge modeling. Although the discussions presented here are in general applicable to all bridges, the intent was to address short- and medium-span bridges (those with span lengths less than 500 ft,

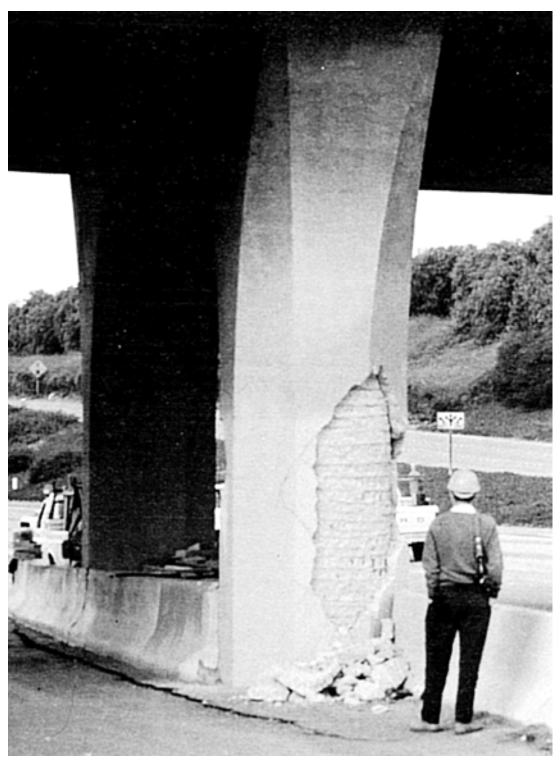


Fig. 1—Acceptable damage to a bridge column.

[150 m]). Most recommendations in this document are extracted from the cited codes and references. These recommendations should not be construed as absolute rules, and should in no way limit the creativity and responsibility of the Engineer in analyzing the structure with the best and most appropriate available tools. However, if followed, the recommendations should provide a good indication of the seismic behavior for a broad class of bridge types encountered in current practice.

CHAPTER 2—CODES

2.1—Historical perspective

The first United States code specifically addressing highway bridge design was published in 1931 by the American Association of Highway Officials (AASHO), which later changed its name to the American Association of State Highway and Transportation Officials (AASHTO). That code, and subsequent editions prior to 1941, did not address seismic design. The 1941, 1944, and 1949 editions of the AASHO code mentioned seismic loading, but simply stated that structures shall be proportioned for earthquake stresses. Those codes gave no guidance or criteria as to how the earthquake forces were to be determined or applied to the structure.

The California Department of Transportation (Caltrans) was the first organization within the United States to develop specific seismic criteria for bridges. Caltrans formulated its first general code requirements for bridge design in 1940, and in 1943 included recommendations for specific force levels based on foundation type. In 1965, the Structural Engineers Association of California (SEAOC) adopted provisions where building force levels varied according to the structure type. Following the 1971 San Fernando earthquake, which caused several freeway structures to collapse, a bridge-specific code was developed and more stringent seismic force levels were introduced. Most importantly, research was conducted that helped develop a more scientifically based seismic code, including ground motion attenuation, soil effects, and structure dynamic response. Those efforts led to development of the so-called “ARS Spectra,” where A, R, and S refer to the maximum expected bedrock acceleration (A), the normalized rock response (R), and the amplification ratio for the soil spectrum (S).

A major research effort, headed by the Applied Technology Council (ATC) and sponsored by the National Science Foundation, resulted in 1978 in the publication of ATC-3, Tentative Provisions for the Development of Seismic Regulations for Buildings. A similar study on bridges was funded by the Federal Highway Administration and resulted in publication in 1982 of ATC-6, Seismic Design Guidelines for Highway Bridges. Those guidelines were the recommendations of a group composed of Federal and State agency representatives, consulting engineers, and researchers. The ATC-6 recommendations represented the state-of-the-art specification in earthquake engineering for highway bridges.

The ATC-6 recommendations incorporate an Elastic Response Spectrum analysis, with adjustment factors (R factors) to consider redundancy, ductility, and over-strength provided by the various structural systems. In those guidelines, the primary departure from previous practice was (1) an emphasis on ductile details to enable safe accommodation of plastic deformations, (2) an emphasis on realistic survivability of structures with minor damage allowed, and (3) specification of bridge seating requirements that were substantially more severe than the practice at the time.

2.2—Current codes and manuals

2.2.1 *AASHTO*—The AASHTO bridge design specifications adopted the ATC-6 recommendations essentially without change, as a guide specification in 1983, as a standard specification in 1991, and finally as a part of the “Standard Specifications for Highway Bridges” in 1992. Those provisions focused on the following basic concepts:

- Hazard to life should be minimized;
- Bridges may suffer damage, but should have a low probability of collapse due to earthquake motions;
- Functioning of essential bridges should be maintained;
- The design ground motions should have a low probability of being exceeded during the normal lifetime of the bridge (10 percent probability of being exceeded in 50 years, or a 475-year-return period);
- The provisions should be applicable to all of the United States; and
- The ingenuity of design should not be restricted.

The AASHTO specification is based on analysis using elastic response spectra. The response moments at potential plastic hinge locations are subsequently divided by response modification factors (*R*-factors) to obtain design moments. The remainder of the structure is designed for the lesser of the elastic response forces or of the forces resulting from the plastic hinge moments and gravity loads, accounting for possible over-strength of the plastic hinges.

2.2.2 *Caltrans*—The 1990 Caltrans Code has provisions similar to the ATC-6 recommendations, but the ARS elastic response spectrum is based on a maximum credible event (10 percent probability of being exceeded in 250 years). Caltrans spectra are elastic, and elastic moments may be reduced by reduction factors (“Z” factors).

2.2.3 *NCHRP 12-33/AASHTO LRFD specification*—NCHRP (National Cooperative Highway Research Program) Project 12-33 has been adopted by AASHTO as a comprehensive load and resistance factor design (LRFD) Bridge Specification which will eventually replace the AASHTO specification (AASHTO, 1996). It was the intention of the committee developing the new AASHTO LRFD Code to move as much of the existing AASHTO seismic code as possible into the new code and at the same time update the technical portions to take advantage of new developments (Roberts and Gates, 1991). The primary areas where updates were included are as follows:

Soft soil sites

The dramatic amplification that can occur on soft ground was demonstrated by the Mexico City Earthquake of 1985 and the Loma Prieta Earthquake of October 1989. The proposed LRFD Bridge Specification introduces separate Soil Profile Site Coefficients and Seismic Response Coefficients (response spectra) for soft soil conditions.

Importance considerations

Three levels of importance are defined in the new code (as opposed to two levels in the current code): "Critical," "Essential," and "Other." The importance level is used to specify the degree of damage permitted by changing the force reduction factors (R). For "critical" facilities, the reduction factors are set at 1.5 to maintain nearly elastic response under the seismic event. For "essential" facilities the reduction factors vary from 1.5 to 3.5 for various bridge components, and for "other" facilities the reduction factor varies from 2.0 to 5.0 for various bridge components.(AASHTO, 1994).

2.2.4 *Seismic analysis and design manuals*—The FHWA has distributed five design manuals (listed below) that serve widely as authoritative references on seismic analysis and design. These manuals provide a practical source of information for designers and serve as a commentary on the design codes. They are:

- "Seismic Design and Retrofit Manual for Highway Bridges," FHWA-IP-87-6.
- "Seismic Retrofitting Guidelines for Highway Bridges," FHWA/RD-83/007.
- "Seismic Design of Highway Bridge Foundations," (3 volumes): FHWA/RD-86/101, FHWA/RD-86/102 and FHWA/RD-86/103, June 1986.
- "Seismic Design of Highway Bridges Training Course Participant Workbook," 1991, Imbsen & Associates.
- "Seismic Retrofitting Manual for Highway Bridges," FHWA/RD-94/052, May 1995.

2.2.5 *ATC-32*—The Applied Technology Council (ATC) has published improved seismic design criteria for California Bridges, including standards, performance criteria, specifications, and practices for seismic design of new bridge structures in California (Applied Technology Council, 1996). That project, termed ATC-32, uses results from current research plus observations in recent earthquakes to identify several significant improvements to the current Caltrans Bridge Design Specifications (BDS). The proposed changes are summarized as follows:

- Consideration of two design earthquakes, under certain circumstances: Safety Evaluation Earthquake, and Functional Evaluation Earthquake. The Safety Evaluation Earthquake is defined as the "maximum credible earthquake." This may alternately be defined probabilistically as an earthquake with a 1000-year return period. The Functional Evaluation Earthquake is a newly defined loading intended to represent an earthquake with a reasonable probability of occurring during the life of the bridge. Because no standard functional evaluation earthquakes have been defined at this time, the earthquake must be determined on a case-by-case basis through site specific studies. The intent of this distinction is to assign level-of-performance criteria to realistic earthquake levels. Level of performance is defined in terms of two criteria, the service level of the structure immediately following the earthquake, and the extent and repairability of damage.
- Caltrans currently uses design spectra (ARS curves) that are a product of maximum expected bedrock acceleration (A), normalized rock response (R), and soil amplification spectral ratio (S). New "ARS" design spectra developed as part of ATC-32 better represent high ground accelerations produced by different sources with different earthquake magnitudes.
- Current seismic procedures, including those of Caltrans, emphasize designing for assumed seismic forces that, when adjusted by response modification factors to account for ductility, lead to an acceptable design. In actuality, relative displacements are the principal seismic response parameter that determines the performance of the structure. Although the ATC-32 document retains a force design approach, it utilizes new response modification factors (factor Z) and modeling techniques that more accurately consider displacements.
- ATC-32 addresses several foundation issues that, although discussed in various documents and reports, have not been described in a comprehensive guideline. These include design considerations for lateral resistance of bridge abutments, damping effects of soil, large-diameter, cast-in-place shaft foundations, conventional pile foundations, and spread footings.
- Several aspects of concrete design are considered in the ATC-32 Report. These include design of ductile elements, design of non-ductile elements using a capacity design approach, and the detailing of reinforced concrete bridge elements for seismic resistance.

The ATC-32 project seeks to develop comprehensive seismic design criteria for bridges that provide the design community with seismic design criteria that can be applied uniformly to all bridges.

CHAPTER 3—ANALYSIS

3.1—Seismic input

3.1.1 *Response spectrum analysis*—The complete response history is seldom needed for design of bridges; the maximum values of response to the earthquake will usually suffice. The response in each mode of vibration can be calculated using a generalized single-degree-of-freedom (SDF) system. The maximum response in each individual mode can be computed directly from the earthquake response spectrum, and the modal maxima can be combined to obtain estimates of the maximum total response. However, it is emphasized that these are not the exact values of the total response, but are estimates.

A sufficient number of modes should be included in the analysis to ensure that the effective mass included in the model is at least 90 percent of the total mass of the structure. This can usually be verified by investigation of the participation factors in the analysis (with allowance that the reported mass may be scaled by a factor). Additionally, it should be confirmed that all important parts of the structure are represented in the response. For example, if a long structure with many piers has been modeled as a single unit, each pier base shear obtained from the response spectrum analysis should be compared with the product of the acceleration coefficient and the tributary mass of that pier. If the response spectrum analysis result is considerably lower than the result from hand calculation, the number of modes should be increased.

It would be unnecessarily conservative to directly add the contribution of each mode because the modal maxima do not occur at the same time. A widely accepted modal combination rule is the Square Root of the Sum of the Squares (SRSS) Method. This method is considered to provide an acceptable approximation of the structural response for structures with well-separated natural periods, where coupling is unlikely to occur. When closely spaced modes occur (a common occurrence for bridges), a preferred combination technique is the Complete Quadratic Combination (CQC) Method, which accounts for the statistical correlation among the various modal responses. The CQC method and other combination techniques are discussed by Wilson (Wilson et al., 1981).

3.1.2 *Time-step analysis*—Time-step analysis requires a detailed description of the time variation of the ground accelerations at all supports. It is obviously not possible to predict the precise nature of the future ground accelerations at a particular site. This uncertainty is accommodated by using at least five ground motions that represent the seismicity of the site.

3.1.3 *Vertical accelerations*—Measurements of earthquake ground motions indicate that during a seismic event, structures are subjected to simultaneous ground motions in three orthogonal directions. There has not been definite evidence of bridge failure due to vertical acceleration. As a result, current codes generally neglect the effect of vertical motions, and detailed analysis in the vertical direction is not required. Design provisions are available for hold-down devices and are discussed in Section 5.3.5.

3.2—Single-mode spectral methods

Single-mode, spectral-analysis methods may be used for final design of simple bridges and for preliminary design of complex bridges. This approach is reasonably accurate for response of straight bridges without a high degree of stiffness or mass irregularity.

Single mode spectral methods can generally be used with reasonable accuracy when the stiffness index $W1/W2 \leq 2$, where (see Fig. 2):

W1 = uniform transverse load to produce a maximum 1-in. (25-mm) lateral displacement at the level of superstructure considering the stiffness of both the superstructure and the substructure, and

W2 = uniform transverse load to produce a maximum 1-in. (25-mm) lateral displacement at the level of superstructure considering the superstructure stiffness only, spanning between abutments.

Where $W1/W2 > 2$, the single-mode spectral method is adequate only for those structures with balanced spans and equal column stiffness. For other cases, a multi-modal spectral analysis should be used.

Three general types of single-mode analysis techniques have been used in past codes; the "lollipop" method, the uniform load method, and the generalized coordinate method.

3.2.1 *"Lollipop" method*—The "lollipop" method models the entire structural mass and stiffness as a single lumped mass on an inverted pendulum. The main advantages of this method are that it is simple and it does not require a computer. The drawback is that it neglects the effects of continuity of the structure. Accordingly, it may not properly account for the distribution of seismic forces within the structure, and may introduce inaccuracies in the structural period that may give unrealistic values for the seismic forces. This method was widely used prior to the San Fernando earthquake of 1971, but is no longer in general use for final design. However, it may be adequate for preliminary analysis or as a check on complex dynamic response analysis.

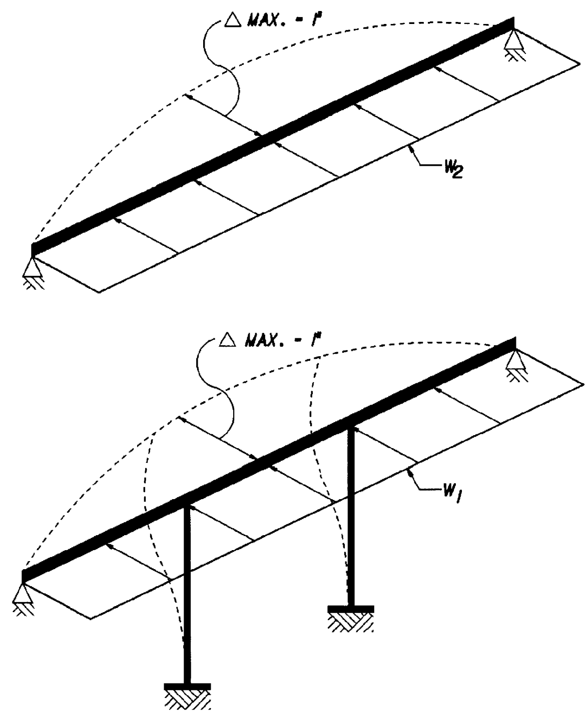


Fig. 2—Definition of stiffness index.

3.2.2 *Uniform load method*—The uniform load method is recommended by the pre-1991 AASHTO specifications and the current Caltrans Standard Specifications. The basic procedure is to determine the equivalent total structural stiffness by computing the uniform horizontal load that will produce a maximum 1-in. (25-mm) displacement in the structure. This stiffness is used in conjunction with the total mass to predict the fundamental period, which, in turn, is used in conjunction with a response spectrum to determine an equivalent seismic force. This force is converted to a uniform load and is reapplied to the structure to determine member seismic forces. The method provides a more representative distribution of seismic forces within the structure, as compared with the "lollipop" method, and accounts for continuity of the superstructure. However, it requires more effort than the "lollipop" method and may require a space frame computer analysis. The uniform load method may not give acceptable results for skewed bridges, curved bridges, and bridges with intermediate expansion joints.

3.2.3 *Generalized coordinate method*—The generalized coordinate method provides the best approximation of dynamic seismic responses using equivalent static methods, and is the method recommended by the current AASHTO specifications (AASHTO, 1996). The method is based on

Rayleigh energy principles. It differs from the uniform load method primarily in that the natural frequency is based on an assumed vibrational shape of the structure. This assumed shape can be approximated by determining the deflected shape associated with the dead load of the structure applied in the direction of interest. The loads should be applied in the same direction as the anticipated deflection. The maximum potential and kinetic energies associated with this deflected shape are equated to calculate the natural frequency, which is then used with a response spectrum (similar to the uniform load method) to determine an equivalent seismic force. This force is reapplied to the structure as a distributed load (with a shape and sense corresponding to the load used to calculate the assumed vibrational shape) to determine equivalent static member seismic forces.

The generalized coordinate method provides a more representative distribution of seismic forces, as compared with the uniform load method, and accounts for variations in mass distribution along the structure. However, the method is considerably more involved than either the "lollipop" or the uniform load methods because it requires an assumption of the vibrational shape and a computer analysis.

3.3—Multi-mode spectral method

The influence of higher modes can be significant in many regular and irregular structures. For structures with irregular geometry, mass, or stiffness, these irregularities can further introduce coupling of responses between vibrational modes. Higher mode responses and coupling between modes are not considered in the single-mode methods described above. Multi-modal spectral or time-step methods are required to evaluate these types of responses.

With the multi-modal spectral method the maximum response in each mode of vibration is calculated separately. Since these maximum responses do not occur at the same time, the responses are combined to approximate the total response (see Section 3.1.1).

A multi-mode spectral procedure should generally be considered where the stiffness index W1/W2 > 2, where significant structural irregularities exist, and where it is deemed appropriate by the Engineer due to unusual conditions, such as structures with unbalanced spans or unequal column stiffness.

Responses to higher vibrational modes may be calculated with Rayleigh energy methods by employing a procedure similar to that described previously for the generalized coordinate method and with assumed vibrational shapes corresponding to the anticipated higher modes. However computer programs are typically used for evaluation of the higher-mode responses.

3.4—Time-step analysis

Time-step analysis (response history analysis) should be used for structures that have unusual or novel configurations, that are particularly important, or that are suspected of having particular weaknesses. Time-step analysis may also be required for long structures where traveling wave effects can invalidate the response spectrum assumption that all supports have identical motions.

A key parameter in response history analysis is the length of the time step. This step is specified to ensure numerical stability and convergence in the time integration algorithm and to accurately capture the response of all significant modes. As a rule of thumb, the time step should be approximately one-hundredth of the fundamental period of the bridge. Unlike response spectrum analysis, the time variation of all response quantities is explicitly computed, and combination of modal maxima is not necessary.

3.5—Nonlinear analysis

3.5.1 *Nonlinear material behavior*—Although linear analysis is by far the more common method of analysis and design of bridges for earthquake loads, the true response of bridge elements to moderate and strong earthquake is nonlinear because element stiffnesses change during such earthquakes. Nonlinearity of the seismic response needs to be accounted for in order to obtain reasonably accurate estimates of internal forces, deformations, and ductility demands. The inclusion of nonlinear effects in analysis is particularly critical for bridges in areas with a history of moderate or high seismicity. Nonlinear analysis constitutes a significantly greater analysis effort, and requires careful interpretation of the results. In general, nonlinear analysis is not applied except under extraordinary circumstances, such as retrofit of complex structures.

Two types of nonlinearities generally exist in the response of structures, one due to material behavior and the other caused by large deformations that change the geometry of the system. Material behavior is discussed in this section. Geometric nonlinearity is addressed in a later section. Not addressed in this report are nonlinearities that may arise due to the failure of an element and the loss of support.

a) *Superstructures*

The analysis and design of bridge superstructures is usually controlled by non-seismic vertical loads. That is, the analysis and design are dominated by strength and serviceability requirements under dead loads and traffic live loads. The bridge width is controlled by the number of traffic lanes to be carried. As a consequence, bridge structures between hinges are very stiff and strong, particularly in the horizontal direction, where seismic inertial forces tend to be greatest. Past earthquakes have shown that concrete superstructures do not usually experience significant damage and their behavior usually remains in the linear range. The observed damage to some bridge superstructures during the Loma Prieta earthquake in 1989 was due to the failure of other bridge elements (Housner, 1990).

b) *Superstructure hinges*

Superstructure hinges are susceptible to damage from earthquake loads. There is considerable variation in the type of details used in superstructure hinges. Regardless of the detail, hinges typically consist of (1) a bearing to transfer the vertical loads to the supporting elements and (2) shear keys to limit horizontal movements in the transverse direction of the bridge. Since the 1971 earthquake in San Fernando, California, many highway bridges in the United States have been

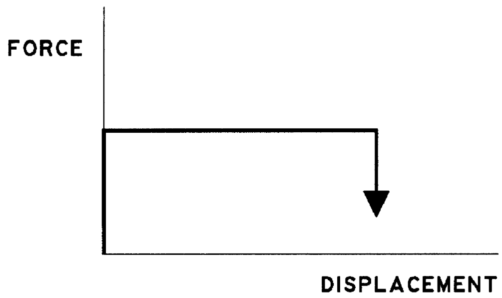


Fig. 3—Idealized nonlinear response of a roller bearing.

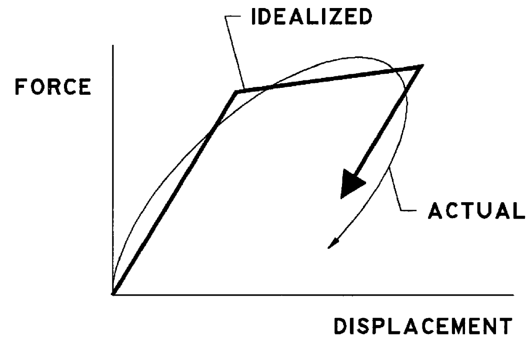


Fig. 4—Typical shear-displacement relationship for an elastomeric bearing.

equipped with restrainers to limit relative displacements at hinges (Yashinsky, 1990).

Two of the most common bridge bearings are steel and elastomeric bearings. Steel type bearings may be detailed to act as a roller or a pin. Assuming that the bearing is designed for the proper seismic loading, the pin should perform elastically. Corrosion may partially lock a pin, affecting the response for relatively small loads. A roller may, in practice, apply some friction forces on the superstructure before it allows for the movement of the bridge. This behavior is, of course, a nonlinear response that may be considered as shown in Figure 3.

The horizontal shear response of elastomeric bearings is nonlinear even under small loads. The bearing shear stiffness varies with shear displacement, dynamic frequency of the load, and the magnitude of the vertical load (Nachtrab and Davidson, 1965; Imbsen and Schamber, 1983a). Figure 4 shows a typical shear-displacement relationship for elastomeric bearings. This behavior may be simulated with reasonable accuracy using a piece-wise linearized relationship similar to the one shown in the figure (Saiidi, 1992). The dependence of stiffness on load dynamic frequency may be more complicated to simulate because the system is nonlinear and its frequency changes during the earthquake. Figure 5 shows elastomeric bearing shear-displacement relationship as a function of frequency (Imbsen and Schamber, 1983a). Because earthquake-induced, high-amplitude displacements are generally associated with lower frequencies, bearing stiffness may be based on an average frequency in the range of 0.5 to 5 Hz.

Bridge hinges usually have shear keys to avoid excessive movement of the superstructure. The shear keys typically are made of reinforced concrete blocks or steel angles. There is normally a nominal gap of approximately 1 in. (25 mm) between the contact surfaces of the shear key. The shear keys become active only when this gap closes. Stiffness changes occur when the shear keys are engaged, and when they reach their yield limit. The shear keys in many highway bridges subjected to the 1989 Loma Prieta earthquake suffered severe damage even under moderate superstructure displacements (Saiidi et al., 1993). Because there are no connecting elements between the contact surfaces of the shear keys, the shear keys are usually engaged only on one side when displacements exceed the gap. The resulting force-displacement relationship is generally similar to the one shown in Figure 6.

Many bridge hinges are equipped with restrainers of steel cables or high-strength steel rods (Figure 7). Restrainers are designed to remain elastic even during strong earthquakes (Caltrans, 1990). Nevertheless, they may introduce two types of nonlinearities in the seismic response of bridges. First, even though restrainers are intended to remain elastic, they may yield under strong earthquakes. Second, restrainers are active only when they are subjected to tension. An added complication is caused by restrainer gaps that are left in bridges to allow for thermal movements of the bridge without applying stresses to the restrainer. These restrainer gaps are a source of non-linearity, and introduce a significant nonlinear effect on the bridge, as illustrated in Figure 8.

The closure of gaps in superstructure hinges introduces a sudden increase in the stiffness during the earthquake. When the gap reopens, the bridge experiences another sudden change of stiffness. The impact associated with the closing of the gaps may also be thought of as another nonlinear effect because of the sudden dissipation of energy associated with impact (Maragakis et al., 1989). The closure of the gap and the impact effects may be modeled by very stiff springs that are inactive until the hinge gap closes. It is generally adequate to assume that these springs remain elastic.

c) *Columns*

Under moderate and strong earthquakes, the concrete in reinforced concrete columns may crack and the steel may yield. Therefore, the element flexural stiffnesses vary during the earthquake, and the response becomes nonlinear. The initial cracking of concrete is a stiffness consideration only, and does not affect calculated strength because the tensile strength of concrete is neglected in flexural design. Even if bridge columns are assumed to be cracked in flexural strength analysis, they may be uncracked under non-earthquake service loads because of the compressive stresses applied by the weight of the superstructure and the column. Cracking will affect the pre-yielding stiffness as shown in Figure 9. Normally, however, the difference between the actual and cracked stiffness is neglected and the column may be assumed to be cracked.

Nonlinear response of reinforced concrete columns may result from large moments, shears or axial loads. Nonlinear response (shear and axial deformations) under shear or axial loads, or both, should be avoided because shear and axial failures are normally brittle. In contrast, nonlinear response of a column in flexure is desirable because it is ductile and

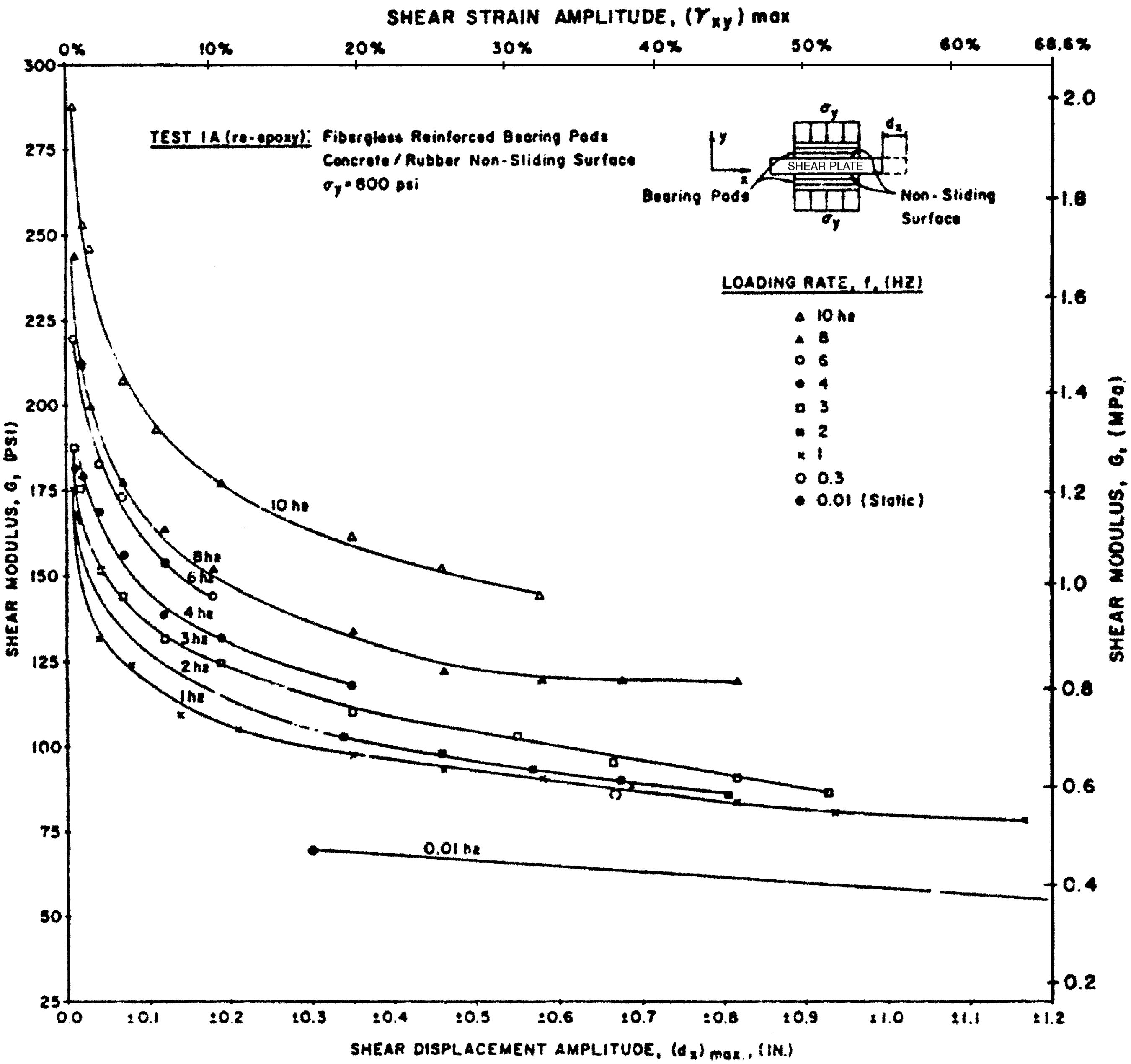


Fig. 5—Elastomeric bearing shear-displacement relationship as a function of frequency (Imbsen and Shamber, 1983a).

leads to energy dissipation through hysteretic action. During severe earthquake loading, the columns may experience several cycles of large deformations. A measure of deformation is the rotational ductility ratio, defined as the ratio of the maximum rotation to the yield rotation at the critical section. The rotational ductility ratio that a properly detailed bridge column may experience during strong earthquakes may be in the range of 6 to 10. Whether a column can withstand high ductility demands generally depends on the reinforcement details within and adjacent to a plastic hinge (Figure 10). Columns with confined cores and sufficiently-anchored reinforcement are known to have the necessary ductility capacity (Priestley and Park, 1979). The provisions of current codes are intended to satisfy these requirements (AASHTO, 1996; and AASHTO, 1994). Columns not adequately detailed should be expected to undergo nonlinear shear and axial deformations, and experience severe losses of strength as the core concrete crushes and longitudinal steel yields and buckles (Priestley and Seible, 1991). Figures 11a and 11b show the typical cyclic response of a column with inadequate confinement and a column with proper confinement, respectively. It is evident in the figures that the lack of confinement leads to a considerable degradation of strength and a reduction in ductility capacity. Furthermore, the energy dissipation capacity of the column, as indicated by the area within the hysteresis loops, is reduced substantially for inadequately confined columns. Even in columns that are properly detailed, a reduction in stiffness (stiffness degradation) is expected as the deformations increase (Figure 12). Upon unloading from Point B, the stiffness is lower than that for unloading from Point A.

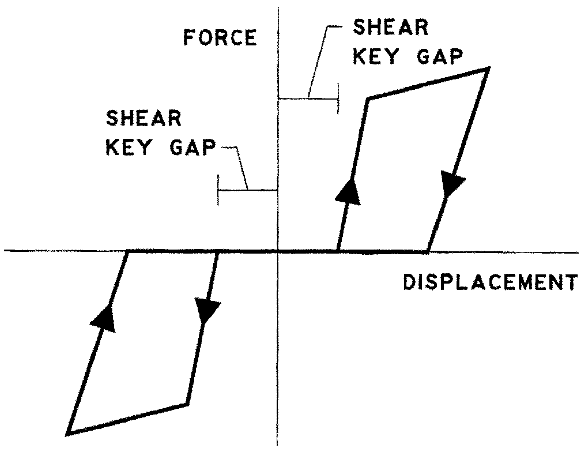


Fig. 6—Idealized force-displacement relationship for shear keys.

Fig. 7—Hinge restrainer detail.

The nonlinear cyclic response of bridge columns may be described by the available hysteresis models such as the Takeda and the Q-hyst models (Takeda et al., 1970; and Saiidi and Sozen, 1979). More complicated models are needed to simulate the hysteretic response of columns where shear nonlinearity or bond failure lead to strength degradation (Chang and Mander, 1994).

Most bridge columns carry relatively light axial loads, less than 10 percent of the concentric axial load capacity. This generally places the response of the columns below the

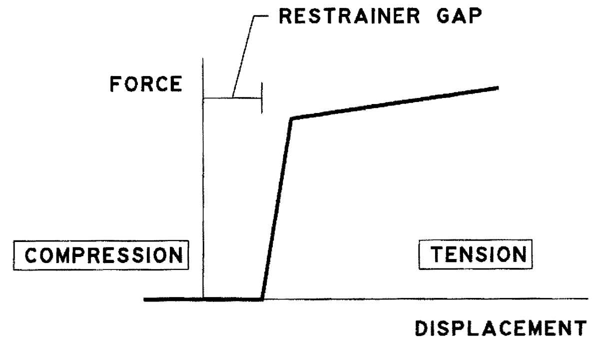


Fig. 8—Idealized force-displacement relationship for restrainers.

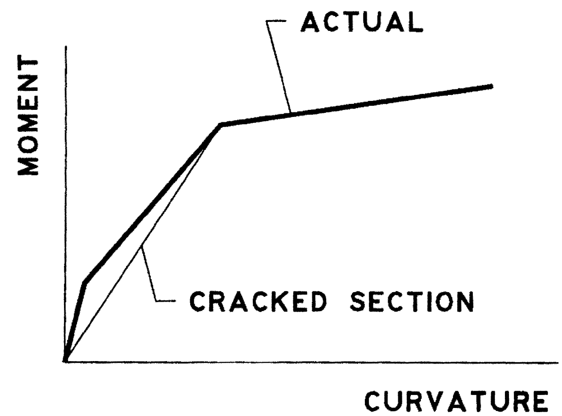


Fig. 9—Moment-curvature relationship for cracked columns.

balanced point, in the "tension failure" region of the axial load-moment column interaction diagram. Thus, properly reinforced bridge columns tend to exhibit ductile rather than brittle behavior when overloaded by an earthquake. However, regardless of the level of axial load, short or squat columns are dominated by shear forces and tend to exhibit brittle shear failures. As a ductile flexural column is cycled repeatedly in the inelastic range, degradation of stiffness and strength will result. This degradation needs to be accounted for in the inelastic analysis. Added complications arise when a column is subjected to biaxial loading. Additional stresses generally develop, which place higher demands on the column than when it is loaded uniaxially. To model biaxial bending, finite element models or spring models may be used (Jiang and Saiidi, 1990; Filippou, 1992).

One- or two-way hinged connections are used at the bases of many reinforced concrete bridge columns to reduce foundation forces. Studies have shown that, for columns with an aspect ratio (column height over depth) of two or more, only flexural nonlinearity needs to be accounted for in the analysis. Slipping shear deformations at the hinge need to be included in the analysis when the aspect ratio is less than two (Straw and Saiidi, 1992). Studies of well-confined two-way hinges have also indicated that flexure controls the cyclic response even when the column aspect ratio is as low as 1.25 (Lim and McLean, 1991).

d) *Wall piers*

Reinforced concrete wall piers loaded in-plane respond very differently from those loaded out-of-plane. In the out-of-

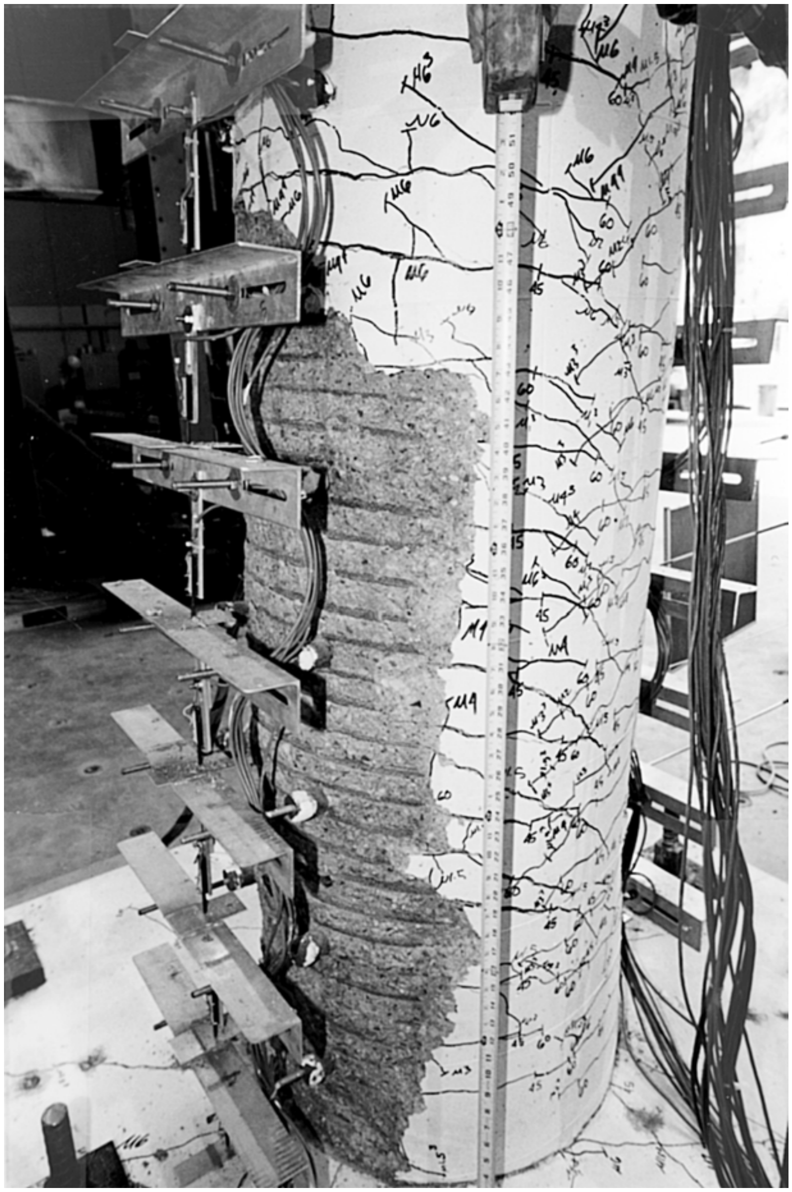


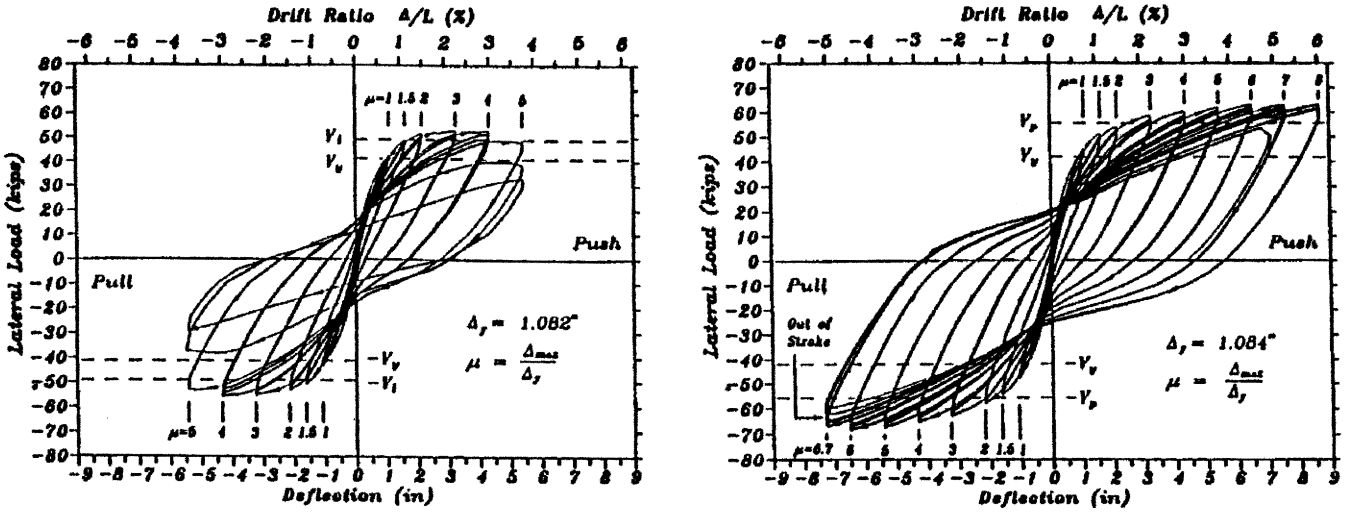
Fig. 10—Performance of a properly detailed column.

plane (weak) direction, walls behave essentially like uniaxially loaded reinforced concrete columns. Nonlinearities that may arise are primarily due to flexure and are caused by cracking of concrete and yielding of reinforcement. Confinement may be required, similar to a column, in order to assure flexural ductility. In contrast to reinforced-concrete columns, confinement of concrete in the weak direction of walls does not usually play a major role because shear and axial stresses are relatively small. In contrast, the wall pier response in the in-plane (strong) direction is dominated by shear, except in bridges with tall piers, in which case the combination of flexure and shear needs to be considered. A lack of confinement reinforcement leads to the buckling of the longitudinal reinforcement and a substantial reduction in stiffness and strength in the strong direction (Haroun et al., 1993).

e) *Foundations*

Soil stiffness is known to be a function of loading frequency and soil strain (Das, 1993; Dobry and Gazetas, 1986). During an earthquake, the loading frequency is highly variable. Therefore, it is not practical to consider the instantaneous changes of stiffness due to frequency changes. Past earthquakes have shown that larger earthquake acceleration amplitudes occur within a frequency range of 0.5 to 5 Hz. Accordingly, the soil stiffness may be based on an average frequency in this range. The variation of stiffness with soil deformation, however, should be accounted for by a nonlinear load-displacement relationship of the Ramberg-Osgood type (Saiidi et al., 1984), or other similar relationships.

Although the nonlinearity of the soil is the major source of nonlinearity in bridge foundation behavior, the geometry of the foundation affects how the nonlinearity is taken into account. For example, whether the bridge is supported by a shallow foundation or a deep foundation will influence the parameters that need to be considered (Norris, 1992). The lateral response of pile groups may include nonlinearity of the cap, the pile-cap connections, or any combination of these



(a) column with inadequate confinement (Priestley and Seible, 1991)

(b) column with sufficient confinement (Priestley and Seible, 1991)

Fig. 11—Typical cyclic response of columns.

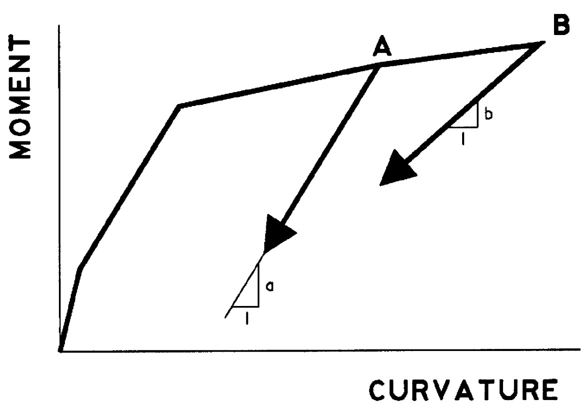


Fig. 12—Column stiffness degradation as a function of deformation.

(Figure 13). Piles may form plastic hinges due to moments imposed above ground or due to transitions in shear distortions from stiff to soft layers of soil as the soil itself responds. When the piles, pile caps, and the connections are properly proportioned, nonlinearity may be limited to the soil.

Although not addressed in detail in this document, unfavorable site conditions that may influence seismic behavior need to be considered. These conditions may include liquefiable soils, deep soft soils, fault crossings, and slopes with instability potential.

f) *Abutments*

Abutments affect the seismic response of bridges regardless of whether they are seat-type or integral with the superstructure. Similar to foundations, the nonlinearity of abutments generally stems from cracking and yielding of the abutment structure in addition to the changes in the stiffness of back-fill soil. Many abutment structures are sufficiently strong and are unlikely to yield, thus limiting the deformations to those due to soil displacements and sliding of the abutment system. The dependence of soil stiffness on loading frequency may be approximated by assuming an average frequency for the input earthquake.

3.5.2 *Geometric nonlinearity*—A potential cause of geometric nonlinearity in highway bridges is the lateral deflection of bridge columns and the closure of gaps in superstructure hinges, restrainers and seat-type abutments. Large lateral movement of bridge columns results in significant additional moments that are produced by the weight of the superstructure. This is the so-called "P-delta" effect. A simple method to account for this effect is to reduce the lateral stiffness of the column. This method has been successfully used in nonlinear seismic analysis of building structures (Saiidi and Sozen, 1979). While there are no general guidelines available as to when to neglect the P-delta effect, it is reasonable to ignore the effect if the product of the column axial load and its maximum estimated deflection is less than fifteen percent of the column flexural capacity.

3.5.3 *Methods of nonlinear analysis*—Analytical models that account for all the nonlinear effects discussed in the previous sections are inevitably complicated and typically not practical for design at this time. Several hysteresis models are needed to model the stiffness variation of different bridge components. Nonlinear analysis usually subdivides the earthquake record into small time steps. The structural stiffness is assumed to remain constant during each time step, and the instantaneous stiffness is based on the tangent stiffness of the nonlinear components. The bridge model may have a large number of elements that become suddenly active once a gap closes. The sudden increase in stiffness can make the microscopic response of bridge elements highly dependent upon the magnitude of the time interval used in the analysis. As a general rule, the analyst should use the shortest time interval that can be reasonably afforded in terms of the computational time. A time step equal to approximately one-hundredth of the fundamental

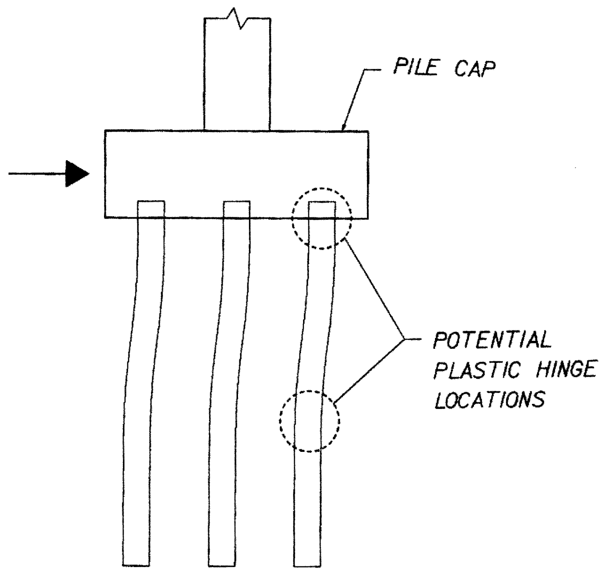


Fig. 13—Lateral response of pile groups.

period of the bridge should suffice in most cases. Different numerical methods are used to integrate the equations of motion. The integration parameters should be selected such that stable results are obtained.

The available analytical models do not adequately model all the aforementioned nonlinear effects. Examples of commonly used computer models for inelastic seismic analysis of highway bridges are discussed in the literature (Imbsen and Schamber, 1983b; Ghusn and Saiidi, 1986; Imbsen 1992; Saiidi et al., 1984).

CHAPTER 4—MODELING

4.1—General

The type and degree of refinement of modeling depends on the complexity of the bridge. The overall objective is to produce a model that captures the essential dynamic characteristics of the bridge so that the model produces realistic overall results. The essential dynamic characteristics of a bridge are not always easy to identify, and vary from bridge to bridge. This section will describe some of the significant modeling factors that influence dynamic behavior, and may therefore be important.

4.1.1 *Global modeling considerations*—It is important to keep in perspective a reasonable level of accuracy in the analysis for bridge seismic effects, particularly when performing iterative designs based on behavioral assumptions. Generally, results within 10 to 15 percent after one iteration are satisfactory. The additional refinement of computer models is not warranted, considering that final elastic forces are modified by response modification factors that are based on approximate assumptions.

a) *Modeling of skewed bridges*

Modeling of skewed bridges must consider the rotational tendencies caused by the orthogonal component of the loading. Longitudinal shaking produces transverse components of force, and vice versa. These bridges have a natural tendency to rotate in the horizontal plane, even under non-seismic loading. Transverse seismic forces can cause one end of the span to bear against the adjacent element while the opposite end swings free in response to the seismic loading, resulting in a ratcheting effect under cyclic loading. The modeling needs to recognize this possibility.

b) *Modeling of curved bridges*

Curved bridges must also consider rotational tendencies due to the orthogonal components of the loading. For curved bridges and some directions of loading, the abutments may provide only a small contribution to the overall stiffness, even for a “Compression Model” condition (a model representing the stiffness condition with closed expansion joints). For load cases resulting in transverse direction movement across the embankment, it is therefore not considered necessary to provide sophisticated modeling of the abutment stiffness for this condition.

4.1.2 *Stiffness modeling considerations*

a) *Section properties*

Uncracked element section properties are typically used when evaluating seismic performance. For a bridge with fundamental period greater than the period corresponding to the peak design response spectral ordinate, this approach is conservative for forces since shorter periods are obtained by modeling uncracked section properties, which results in higher force levels. However, this case is unconservative for estimating displacements. For the case of a bridge fundamental period less than the period corresponding to the peak design response spectral ordinate, the appropriateness of this assumption should be evaluated, and softening the elements or using cracked-section properties should be considered.

Other instances where modified section properties should be considered include:

- *Determination of deflections*—In this instance the above assumptions on use of uncracked section properties are unconservative.
- *Integral bent caps*—The modeled section properties need to be increased over the actual section properties of the bent cap properties to simulate the very stiff deck unit. When element weights are calculated by the computer program, the dead load should be checked, considering these increased section properties, to assure that the vertical loads are distributed properly to the columns with the modified cap stiffness.

For frames, multi-column bents, or both, an inelastic lateral load analysis (“pushover analysis”) should be used to determine the displacement demands (Priestley et al., 1992). These displacement demands are then compared with displacement capacities obtained using the moment-curvature analysis approach (Figure 14).

For composite concrete-steel members, the section properties should be adjusted to an equivalent concrete or steel section considering the modular ratio (ratio of moduli of elasticity). Where different concrete strengths are used in the same element (such as different concrete strengths in prestressed-concrete girders and cast-in-place concrete deck), the section properties should be transformed using a similar procedure. The composite densities should also be transformed to an equivalent concrete or steel density such that appropriate mass distributions will result when the mass is calculated by a computer program.

Torsional properties of superstructures can generally be approximated. For closed cross sections (box girders), an approximation that neglects interior webs typically may be used. For open-type cross sections, each component of the section behaves as a series of rectangular elements with the behavior of each element being similar to a solid plate.

b) *Boundary conditions*

Certain boundary conditions, such as abutment spring stiffness, have a significant influence on overall system behavior, and are nonlinear. These elements can be modeled as linear springs with an assumed equivalent stiffness in the initial analysis taken from the load-deflection ("P-Y") characteristics of the soil (Caltrans, 1990). After the first analysis these assumptions should be evaluated based on the calculated force level/displacement compared with the desired ultimate capacity of the element, and the spring stiffness appropriately adjusted in subsequent analysis.

c) *Element joint size*

The structural analysis should consider finite joint sizes. Structural models typically consider the member element length as the distance between centerlines of joints. It is not uncommon for ten percent or more of a bridge member length measured from centerline of joints to be in a nearly rigid zone, which if ignored will result in longer periods. If the fundamental period of a bridge is greater than the period corresponding to the peak response spectral ordinate, this omission will result in an unconservative estimate of the seismic forces.

d) *Anisotropic behavior*

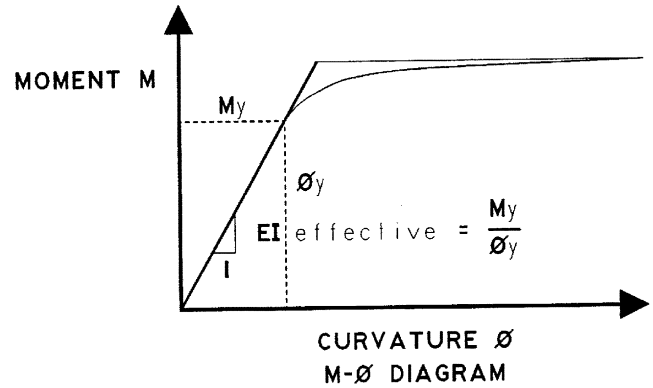
Bridges generally possess different characteristics in the longitudinal direction when they are subjected to tension or compression conditions. As a bridge opens up at the joints, it pulls at the restrainers (or offers little resistance if no restrainers are present). As a bridge closes at the joints the superstructure elements go into compression, offering considerable stiffness. This behavior is typically addressed by employing two models, a "tension model" representing opening of the joints and a "compression model" representing closing of the joints.

In the tension model the superstructure joint elements, including the abutments, are released longitudinally, but the stiffnesses of any hinge restrainer elements are included. In the compression model, restrainer elements are inactive, superstructure elements are locked together in compression, and the abutment and backfill stiffnesses are mobilized.

4.1.3 *Mass distribution*—The modeling of the mass distribution within a structure is fundamental to the accurate evaluation of the structural dynamic response to a seismic event. The total mass consists of three components:

a) *Self-mass*—mass of the girder/deck system, diaphragms and hinges.

b) *Superimposed dead loads*—All mass on the structure in addition to the structural mass. These masses include barriers and handrails, roadway surfacing, stay-in-place



where:

E = *concrete modulus of elasticity*
I = *flexural moment of inertia*
My = *yield moment capacity*
∅y = *curvature corresponding to first yielding of tensile longitudinal reinforcement*

Fig. 14—Moment curvature diagram

forms, and utilities. Where the roadway surfacing is to be placed as a future wearing surface, the more critical condition (with or without surfacing) needs to be considered.

c) *Live loads*—Current U.S. codes do not include the effects of live load concurrent with seismic loading, although inclusion of such a loading combination has been proposed (Caltrans, 1990). It has been suggested that it is inappropriate to combine seismic loads and live loads since the vehicle tires and suspensions will serve as damping devices and may reduce seismic response. For long-span bridges, live load is a very small portion of the total load. However, the added axial loads due to live load will increase the column moment capacity (if the column axial load is below the balance point on the interaction diagram, which is common), and this will increase the column overstrength, which is the most important consideration for live load.

Studies have been made by Caltrans (Caltrans, 1990) investigating the importance of including live load in addition to load combinations of dead loads and seismic loads. The analysis results from a combination of dead load and seismic load were 86 to 100 percent of the results that included dead loads, seismic loads and live loads for column axial load, and 89 to 92 percent of the results that included dead loads, seismic loads and live loads for shear forces and moments. However, those studies were based on limited live load combinations. A more realistic approach taking into account various vehicle types, loadings, and vehicle spacings was recommended by Caltrans. A study in Japan (Kameda et al., 1994) showed similar results, although that study was even more limited in scope, being based on a specific structure and specific live load conditions.

Until more conclusive results are available, it is recommended that live load be ignored in combination with dead load and seismic load, except in special circumstances. These may include special bridge configurations (cantilevers, outriggers, and C-bents) in regions of high seismic risk. If plastic hinges are allowed in a horizontal member, design for shear should be carried out by using dead load, seismic load, and an appropriate level of live load.

4.1.4 *Modeling of secondary effects*—Besides the primary structural forces resulting from inertial loading of a bridge during an earthquake, several types of secondary forces should be considered: P-delta moments, post-tensioning forces, thermal forces, and settlement forces.

Although P-delta effects are always present to some degree in columns, they are significant only for slender columns. Methods of determining the importance of P-delta effects are discussed in ATC-32 (Applied Technology Council, 1996).

Another class of secondary forces potentially affected by earthquakes are those arising from prestressing. Under static loading, the stresses in a prestressed member consist of those imposed by the prestress forces, together with those imposed by external static loads. When cyclic earthquake loading is also applied, the stress distribution within the member shifts. This can lead to undesirable deflections and stresses. For example, if downward gravity loads are removed from a prestressed highway bridge girder, the girder can deflect upwards under prestress forces alone. In addition to the shifting of internal stresses under cyclic earthquake loads, the intensity of stresses can be raised by a loss of member cross-sectional area. The compression-stress zone in some prestressed members, such as box column sections and T-girder sections, is concentrated in a relatively small flange area. If this compression zone is destroyed under severe cyclic loading, and redistribution of prestress forces is not possible, sudden failure of the member may result. The AASHTO specifications do not require prestress forces to be included when designing for earthquake loads, but Caltrans Specifications (Caltrans, 1990) do require prestress forces to be considered at the same time as earthquake loads.

Atmospheric temperature changes, spatial temperature variations, and support settlements can induce significant stresses in a bridge, particularly if there are redundancies in the structural configuration of the bridge and if there are multiple constraints on movements of the bridge at the supports. While thermal and settlement forces are important to the long-term performance of a bridge, they do not affect transient seismic performance at ultimate strength levels. Displacements induced by temperature variations and support settlements influence yielding displacement and influence the available displacement capacity, but do not affect the ultimate capacity. Therefore, these effects should not be combined; neither the AASHTO specifications (AASHTO, 1996) nor Caltrans requires consideration of thermal and settlement stresses simultaneously with earthquake loads.

4.2—Superstructure modeling

4.2.1 *Stiffness considerations*

a) *Overall modeling requirements*

Bridges generally need to be modeled as three-dimensional systems with 6 degrees of freedom at each node and a minimum of three nodes in each span (quarter points). Discontinuities (such as hinges) need to be modeled with double nodes with appropriate member releases or coupling elements representing the stiffness of the joints. Significant horizontal and vertical curves should be modeled with skewed supports included. Linear models cannot accurately model hinge restrainers because of the difference between the restrainer response in tension and in compression.

b) *Barrier/handrail contribution to stiffness*

Even though they are not considered to be primary structural elements in the conceptual development of a bridge design, barriers and handrails may contribute significantly to the mass and stiffness of the actual bridge. In a static analysis of a bridge, it is usually conservative to neglect the strength contribution of secondary materials and members. However, in a dynamic analysis the mass and stiffness contributions of secondary elements may substantially alter the dynamic response of the structure to an earthquake, in either a conservative or an un-conservative manner. Therefore, in the dynamic analysis of a bridge it is important to consider the contributions of such secondary components.

If concrete barriers are cast integrally with the superstructure, they should certainly be considered in computations of both stiffness and mass for the purposes of dynamic analysis. However, when considering the stiffness contribution of integrally cast barriers, careful attention should be paid to such conditions as the presence of cuts in the barriers (for crack control), block-outs for utilities or signage mounts, the existence of potential slip planes at horizontal or vertical construction joints, and the amount of reinforcement (if any) crossing any such potential slip planes. All of these conditions can affect the stiffness that barriers contribute to the overall superstructure stiffness.

Precast concrete barrier elements, positively anchored to the superstructure, can contribute to superstructure stiffness. However, since precast elements are attached to the superstructure at discrete points, it is difficult to determine their degree of interaction with the superstructure, and their stiffness contribution may have to be neglected. Precast concrete barrier elements that rest on the superstructure, but are not attached to it, contribute mass but not stiffness to the superstructure.

c) *Pavement continuity at abutments*

When the concrete pavement is continuous and adequately reinforced over the abutment gap, and where sufficient pavement length and mass are available beyond the abutment, rotational restraint about the vertical axis is introduced at the abutment. This restraint can affect the vibration period of the bridge and needs to be considered.

4.2.2 *Mass distribution*—A sufficient number of nodal points should be included along the superstructure to provide a representative mass distribution. Nodes should typically be provided at quarter points. Each node should have three translational dynamic degrees of freedom; masses associated with rotational degrees of freedom are generally ignored because they are relatively insignificant. However, for some cases such as single column bents, rotational degrees of freedom may be significant and should be considered.

4.2.3 *Damping*—Structural damping is a complex phenomenon that can be modeled in many ways. These include equivalent viscous damping (force is proportional to velocity); coulomb friction (constant force, such as slipping between dry surfaces or rubbing across cracks); and internal friction (proportional to the deformation amplitude, such as intermolecular friction).

It is not currently possible to analytically determine the overall damping for a structure. Evaluation of damping should be based on prior research of similar bridges. In lieu of such evaluations, use of 5 percent of critical viscous damping (as an approximation of the combined effect of all damping) is common practice for concrete bridges.

4.3—Substructure/foundations

There are many types and combinations of substructures, foundations and foundation/substructure connection systems. A further complication is that foundation behavior is typically nonlinear and depends on loading rate and time. A practical solution in foundation modeling can be based on bounding potential behavior, considering the sensitivities of various parameters and the implication of foundation effects on overall structural performance.

4.3.1 S*tiffness considerations*—The substructure typically provides the primary energy dissipation in bridge structures through yielding and formation of plastic hinges. Substructures should generally be proportioned to limit their inelastic action to the columns. Very stiff and strong columns, such as pier walls, should be avoided in areas of high seismic demand, since they may force the inelastic action into the foundation or superstructure, or both, which is undesirable.

AASHTO distinguishes between wall-type piers and columns in the evaluation of response modification factors. A column is defined as an element in which the ratio of clear height to the maximum plan dimensions is 2.5 or higher. Otherwise, the element is considered to be a pier wall.

a) *Overall modeling requirements*

A primary objective of foundation modeling is to obtain representations of foundation stiffness for overall bridge models, and representations of soil stiffness for determination of substructure stresses. Several approaches to foundation modeling are typically used, including:

- uncoupled translational and rotational springs,
- equivalent extended columns (cantilever model),
- fully coupled foundation spring models.

Uncoupled translational and rotational spring modeling has the advantages of simplicity and nearly universal implementation in computer codes. However, it has a significant disadvantage in that it cannot model cross-coupling of moment and shear in foundation behavior. This type of model is usually adequate for shallow foundations (spread footings and abutment walls).With proper care to account for the connection details between pile cap and piles, it can be used to model a pile group adequately. A foundation representation with uncoupled translational and rotational springs is inappropriate for single-column shaft foundations.

An equivalent extended-beam-column representation of foundation stiffness is likewise relatively simple and almost universally implementable in computer codes. It has the advantage of allowing some degree of cross-coupling of moment and shear in foundation behavior with a simple representation. It can be used for deep foundations, such as pile groups, pile bents, and single-column drilled shafts with proper bounding of likely equivalent lengths. The drawback to this type of modeling is that the simplified formulas for equivalent lengths are subjective, and a different length of equivalent cantilever is needed for proper representation of foundation stiffness and substructure moment. This method also does not provide an adequate basis for determining accurate moment distribution in piles.

Fully coupled foundation spring models provide the most accurate solution and can be rigorously applied to all types of foundation systems. The drawbacks of this type of modeling are that it requires a greater effort for implementation, and detailed soils information is required to develop all of the terms in the stiffness matrix. This type of stiffness representation also may not be possible in all computer codes. More detailed descriptions are given in the references (FHWA 1986 and FHWA 1987).

b) *Soft soil conditions*

The design engineer should be alert for warning signs of potential soft soil problems, such as saturated sands with blow-counts less than 20 (indicating a potential for liquefaction) or sites with 20 ft (6 m) or more of low-blow-count clays (potential case for acceleration amplification). Qualified geotechnical engineering advice should be obtained on a site by site basis concerning possible consequences of soft soil conditions (liquefaction, lateral spread, and amplification of bedrock acceleration).

In case of liquefiable material, the structural response should be investigated in its original state and also its liquefied state to ensure structural integrity. Because soil stiffness is nonlinear, soil-spring stiffness for soft-soil conditions should be consistent with expected footing displacements at the design seismic load level.

Soil/pile interaction may be modeled separately, and the results used with structural models that are adjusted by iteration to match the displacement/force levels predicted by the soil/pile interaction model. The group effects on soil/pile interaction should be included, either by reducing the stiffness of the single pile-response, or by using a pile-group analysis model.

4.3.2 *Mass distribution*—The mass considered for design is usually dominated by the superstructure structural mass contribution. The inertia of any soil mass that may act with the foundation is usually small, and may be neglected. In bridges with an approach slab, the contribution of the approach slab to the abutment mass and stiffness needs to be taken into account.

As with the superstructure, a sufficient number of nodal points needs to be included in the substructure in order to provide a representative mass distribution. Typically, third-point locations should be considered along the columns. Eccentricities between superstructure, substructure and foundation should be considered by the use of rigid links or rigid member end regions in the computer model. For the condition of submerged piers, the added mass of the water surrounding the substructure also may need to be considered in the design. This added mass, termed the "hydro-dynamic effect," decreases the natural frequency of the structure. Although this effect is usually small, under some circumstances (such as floating bridges and large piers) the effect is significant and should be included (Blevins, 1979).

4.3.3 *Damping*—Much more research is needed to properly account for damping of foundation systems. The problem is compounded by the fact that material damping is strain-dependent. Even for an identical foundation condition (load magnitude, configuration, and soil condition), the contribution of foundation damping to the overall system damping varies depending on the bridge structure.

For bridges shorter than 300 ft (90 m) with no internal hinges and less than 15 degrees of skew, the abutment dominates the dynamic response. In such cases the abutments are capable of mobilizing the soil, and damping in the range of 10 to 15 percent of critical is justified (Douglas et al., 1984).

4.4—Bearings

Bearings can broadly be classified into two categories. Forced-based bearings are designed to transmit the seismic force to adjacent components. These are the common bridge bearing types, such an neoprene pads, pot bearings, steel shoes and pin type bearings. Isolation-based bearings are designed to transmit a reduced force to adjacent components, with forces either reduced by energy dissipation ("isolation concept") or by redistribution ("fuse concept").

Ideally these isolation-based bearings resist the reduced seismic forces, restricts displacements, dissipates energy, and returns the structure to its original position after an earthquake.

CHAPTER 5—DESIGN

5.1—General

The following sections summarize current seismic design practices. Much of this information is explained in greater detail in the AASHTO specifications (and commentary) (AASHTO, 1996) and the Seismic Design and Retrofit Manual for Highway Bridges (Buckle et al., 1987) and their references.

5.1.1 *Single span bridges*—The design requirements for single-span bridges are greatly simplified in the codes, due to a history of acceptable performance of this type of structure provided there is sufficient support length to preclude unseating. A detailed seismic analysis is not required for single-span bridges. The basic design requirements are:

- Connections between the bridge span and the abutments should be designed both longitudinally and transversely to resist the gravity reaction force at the abutment multiplied by the acceleration coefficient at the site;
- Minimum support lengths should be provided as specified by the AASHTO specifications.

Design requirements for connections are necessary to minimize damage and control deflections. The design force level considers the structure to be very stiff, and acknowledges that the period of vibration is difficult to calculate for single-span bridges due to the significant contribution of the abutments.

5.1.2 *Design method*—Although the AASHTO specifications permit service load design, the load-factor method of design is preferred and recommended because it is consistent with the overall approach used to determine design forces.

5.2—Design forces

5.2.1 *Combination of orthogonal forces*—The directional uncertainty of seismic forces is addressed in the AASHTO specifications by combining member forces from separate independent seismic analyses in two perpendicular horizontal directions. AASHTO specifications recommend that two load cases be considered. Load Case I consists of 100 percent of the absolute value of each of the member elastic seismic forces resulting from the longitudinal seismic loading, combined with 30 percent of the absolute value of the member elastic seismic forces resulting from the transverse seismic loading. Load Case II consists of 100 percent of the absolute value of each of the member elastic seismic forces resulting from the transverse seismic loading, combined with 30 percent of the absolute value of the member elastic seismic forces resulting from the longitudinal seismic loading. These load combinations should be used for design of all substructure and foundation elements.

When considering plastic hinging of columns or pier walls as permitted by AASHTO Seismic Performance Categories C and D, the column, pier wall, and foundation elements should be proportioned in the direction of the hinging considering the forces resulting from plastic hinging in that direction only, and the above load combinations need not be considered. The specified combinations must be used for the orthogonal direction of the member.

5.2.2 *Load combinations*—AASHTO specifications require that the components of a bridge be designed to withstand the forces resulting from each of the specified load combinations. The load group that accounts for seismic loading is:

$$\text{Group load} = 1.0\,(D + B + SF + E + EQM)$$

where

D = dead load
B = buoyancy
SF = stream-flow pressure
E = earth pressure
EQM = elastic seismic force for either Load Case I or Load Case II described in Section 5.2.1 modified by the appropriate R-factor given in 5.2.3.

5.2.3 *Response modification factors*—The AASHTO specifications require that seismic design forces for individual components and connections of bridges be determined by dividing the elastic forces obtained from the analysis by the appropriate Response Modification Factor (R). The values of R for various components are given in Table 1.

R-factors are used to obtain the design forces for each component using the results of an analysis of the bridge when using the seismic load of the elastic design spectrum. The R-factors were developed assuming that the elements will yield when using the forces induced by the design ground motions and that connections and foundations are to be designed to

Table 1—Response modification factors

Substructure*	*R*	Connections	*R*
Wall-type pier	2	Wall-type pier	2
Reinforced concrete pile bents†		Expansion joints within a span of the superstructure	0.8
a. Vertical piles only	3	Columns, piers or pile bents to cap beam or superstructure‡	1.0
b. One or more batter piles	2	Columns or piers to foundation	1.0
Single column	3		
Steel or composite steel and concrete pile bents			
a. Vertical piles only	5		
b. One or more batter piles	3		
Multiple column bent	5		

* The *R*-factor is to be used for both orthogonal axes of the substructure.

† A wall-type pier may be designed as a column in the weak direction of the pier provided all the provisions for columns in Chapter 8 of the *AASHTO Guide Specifications* are followed. The *R*-factor for a single-column may then be used.

‡ For bridges classified in Seismic Performance Categories C and D, it is recommended that the connections be designed for the maximum forces capable of being developed by plastic hinging of the column or column bent. These forces are often significantly less than those obtained using an *R*-factor of 1 or 0.8 (AASHTO, 1996).

accommodate the design earthquake forces with little or no damage. It is emphasized that the use of *R*-factors implies the development of a ductile mechanism, and the designer must ensure that the detailing of the structure will allow this mechanism to form without brittle behavior.

The rationale used in the development of the *R*-factors for columns, piers and pile bents is based on considerations of redundancy and ductility provided by the various supports. The wall-type pier is judged to have minimal ductility capacity and redundancy in its strong direction, and is therefore assigned an *R*-factor of 2. A multiple-column bent with well-detailed, ductile columns is judged to have good ductility capacity and redundancy, and is therefore assigned the highest *R*-factor of 5. Although the behavior of single columns is similar to that of individual columns in a multiple-column bent, single columns have no redundancy and are therefore assigned a lower *R*-factor of 3.

The *R*-factors of 1.0 and 0.8 assigned to connections mean that these components are designed for the elastic forces and for greater than the elastic forces in the case of abutments and expansion joints within the superstructure. This approach is adopted to ensure that inelastic behavior is confined to the intended regions, and to maintain the overall integrity of the bridge structure at these important connections. Increased protection can be obtained with a minimum increase in construction costs by designing for these larger force levels. However, for bridges classified in Seismic Performance Categories C or D, the recommended design forces for column connections are the forces that can be developed by plastic hinging of the columns (AASHTO, 1996). Since these are the maximum forces that can be developed, and since they are generally smaller than the elastic values, the desired integrity can be obtained at lower cost. The connection design forces associated with plastic hinging are not specified for bridges in Seismic Performance Category B because the calculation of those forces requires a more detailed analysis. However, they may be used if desired.

A more rational method to determine the *R*-factor is presented in (FHWA, 1995). The *R*-factor is found based on the initial period of the structure and the ductility capacity of the structure.

5.2.4 *Forces resulting from plastic hinging*—A plastic hinge begins to develop when a column reaches its yield moment. For Seismic Performance Categories C and D, the forces resulting from plastic hinging at the top or bottom of the column is used as a second set of design forces. These forces, which tend to be less than the forces determined by elastic analysis, are recommended for designing most other components. The procedures for calculating these forces for single columns, and for multi-column piers and bents, are discussed in detail in the AASHTO specifications.

The forces are based on the potential over-strength capacity of the materials; to be valid, special design details must be used such that plastic hinging of the columns can occur. The increase in flexural strength due to confinement needs to be taken into account. The over-strength capacity results from (a) actual material strengths being greater than the minimum specified strengths, (b) confinement, and (c) strain hardening. The intent of a capacity design is to ensure that failure occurs in a flexural mode and not in shear, compression, torsion, or bond modes.

A shear mode of failure is usually brittle and may result in a partial or total collapse of the bridge. Because of the consequences of shear failure, it is recommended that conservatism be used in locating possible plastic hinges so that the smallest potential column length be used with the plastic hinging moments to calculate the largest potential shear force for design. For flared columns, plastic hinges may occur at the top or bottom of the flare. When a major portion of the column length is flared, the plastic hinge location may be between the ends of the flare. In such cases, the possibility of plastic hinging at different locations on the flare needs to be investigated. For multiple column bents, with a partial-height wall, the plastic hinges are likely to occur at the top of the wall unless the wall is structurally isolated from the column. For columns with deeply embedded foundations, the plastic hinge may occur above the foundation, mat or pile cap. For pile bents, the plastic hinge may occur above the calculated point of fixity.

If the column moments do not reach their plastic values, the shear forces from plastic hinging will not govern. The governing design forces will then be those from the elastic spectrum or from other load groups.

It is recommended that for shear stress calculations the section properties include only the “core” concrete for compression members, assuming the concrete cover has spalled away from the transverse reinforcement.

5.3—Design considerations

5.3.1 *Bearing seats*—Loss of support for the superstructure is the most severe form of bridge damage (Figure 15). One of the major causes of this damage is inadequate support length at the ends of the girders. The AASHTO specifications (AASHTO, 1996) require the following minimum support lengths for each Seismic Performance Category (SPC), as shown in Figure 16:

SPC A $N = 8 + 0.02L + 0.08H$ (in.)

$N = 203 + 1.67L + 6.66H$ (mm)

SPC B $N = 8 + 0.02L + 0.08H$ (in.)

$N = 203 + 1.67L + 6.66H$ (mm)

or the elastic displacement determined by the specified analysis, whichever is larger.

SPC C & D $N = 12 + 0.03L + 0.12H$ (in.)

$N = 305 + 2.5L + 10H$ (mm)

or the elastic displacement determined by the specified analysis, whichever is larger.

where:

L = length in ft (m) of the bridge deck from the support under consideration to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L_1 and L_2, the distances to either side of the hinge. For single-span bridges, L equals the length of the bridge deck.

N = minimum support length in inches (mm).

For abutments:

H = average height, in ft (m), of columns (from the top of footing to bottom of girder) supporting the bridge deck at the nearest expansion joint. $H = 0$ for single-span bridges.

For piers:

H = column or pier height in ft (m).

For hinges within a span:

H = average height of the adjacent two columns or piers in ft (m).

For skewed bridges, the above expressions are multiplied by $(1 + S^2/8000)$, where S = skew angle in degrees (Caltrans, 1990).

5.3.2 *Expansion joints and restrainers*—A positive linkage (i.e., a restraint to horizontal movement) is recommended between adjacent sections of the superstructure sections at expansion joints. Sufficient slack should be allowed in the linkage such that thermal movements are not restrained. Positive linkage can be provided by ties, cables, dampers or equivalent mechanisms. Friction should not be considered a positive linkage. Where adjacent frames have significant differences in stiffness, usually the lighter frame will govern the restrainer design.

Restrainer design forces should not be based on an elastic analysis because the extremely large column forces predicted by such analyses are not generally reached. According to the current AASHTO specifications, the linkage must be designed for the acceleration coefficient times the weight of the lighter of the two adjoining spans of the structure (AASHTO, 1996). An equivalent static method, (Caltrans, 1990) has also been used to design restrainers across superstructure hinges and simple-supports. Because the required restrainer area is sensitive to small changes in the estimated displacements, more refined methods may need to be used, particularly when the bridge is supported by tall, flexible columns (Saiidi et al., 1996).Restrainer design must consider in-plane rotational characteristics of skewed bridges, as described in Section 4.1.1a.

5.3.3 *Abutments*—For short to medium length bridges the abutments often attract most of the lateral seismic forces, due to their high lateral stiffness. These forces can be high, and may cause severe and often brittle failures. The interaction of the abutment with the backfill may also cause the wing walls to break loose from the abutments. Backfill settlement resulting from vibration is often observed. Wing walls are not generally designed for seismic forces.

The two most commonly used abutments are the integral abutment and the seat-type abutment. Integral abutments mobilize the backfill under both longitudinal and transverse load, and can dissipate significant energy during an earthquake. This type of abutment is preferred. However, severe damage may result in the end diaphragm, wing walls and piles because of the large forces to be resisted, unless these forces are properly accommodated.

Seat-type abutments typically sustain less damage than integral abutments because they allow movements, permitting better control over the seismic forces transferred to the abutment. The size of the seat can be adjusted to accommodate thermal movements and small-to-moderate earthquakes. During a major earthquake, impact against the abutment wall should be expected, and some damage tolerated. Schemes used to mitigate damage include a break-away

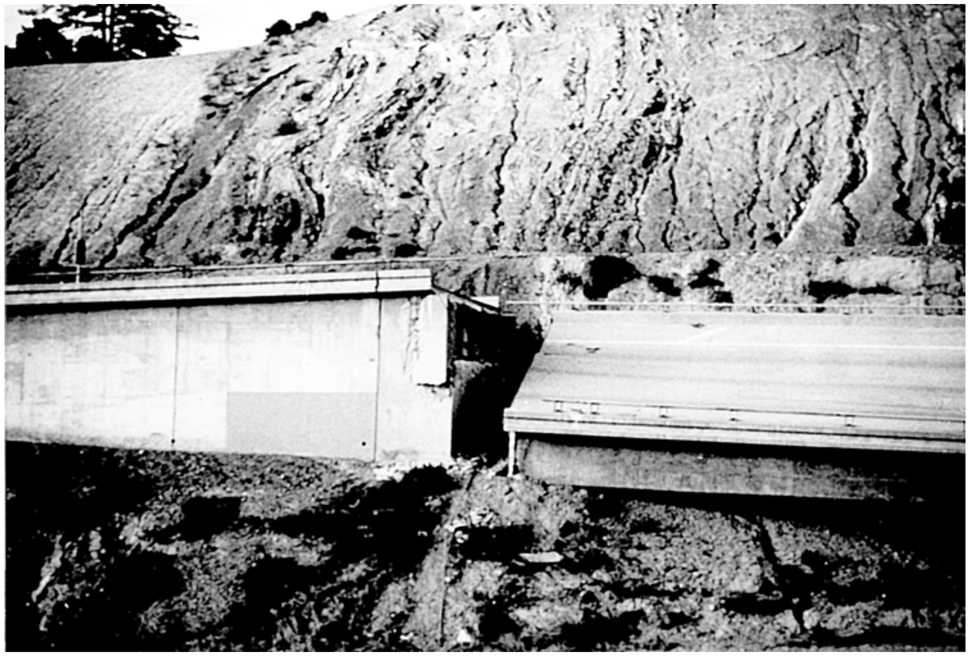


Fig. 15—Bridge unseating due to inadequate support width.

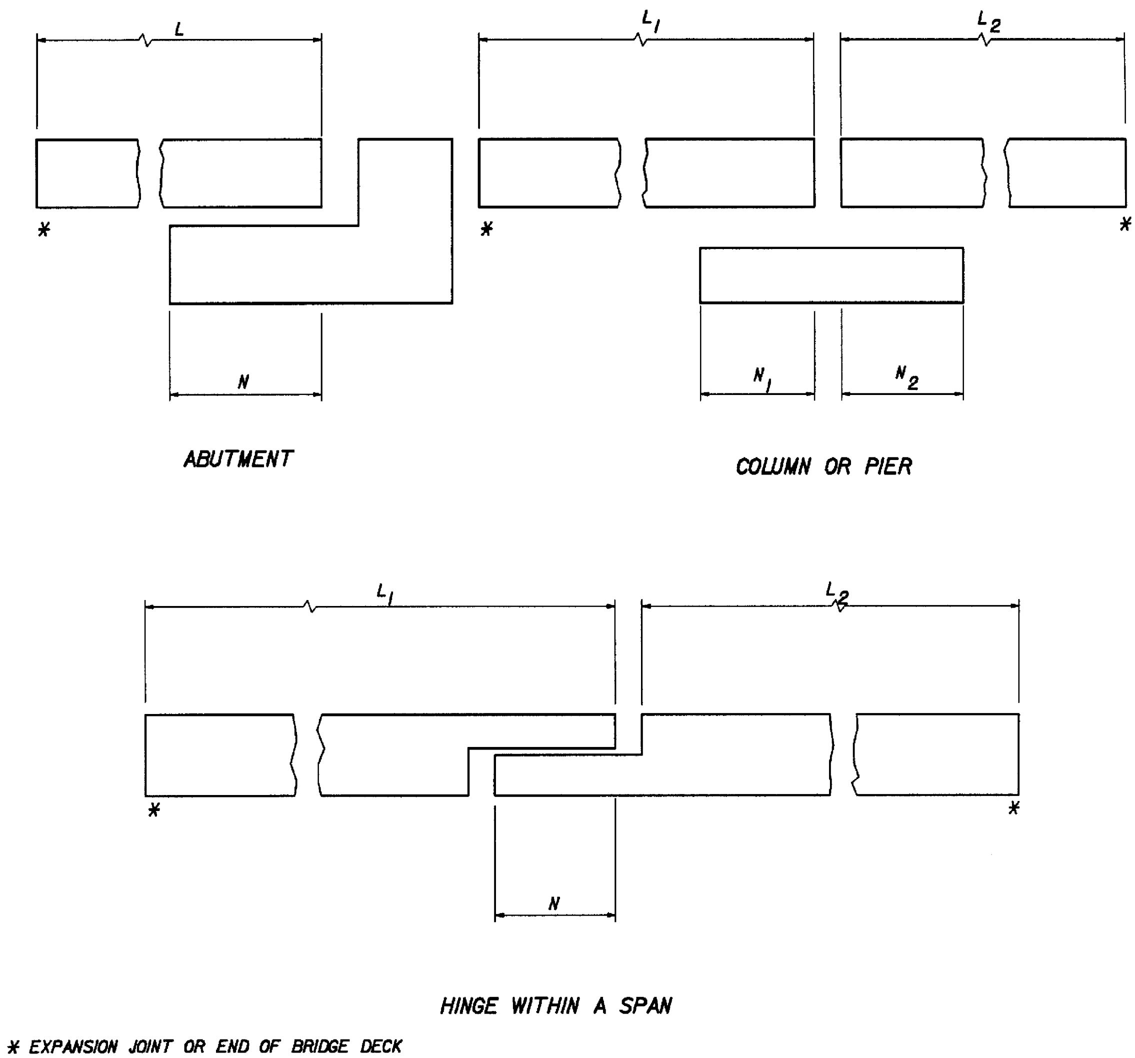


Fig. 16—Minimum support lengths (AASHTO, 1992).

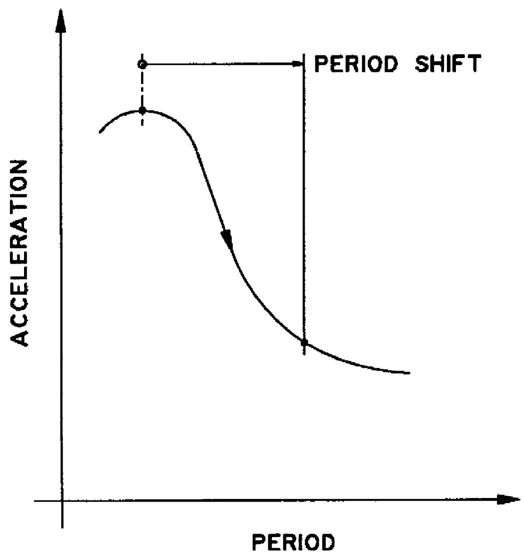


Fig. 17—Idealized acceleration response spectrum.

section near the top of the back wall. After failure of the break-away section, the superstructure would be free to move. Temporary repairs of the back wall can be completed quickly, and permanent reconstruction of the damaged section is inexpensive.

Seat-type abutments must have generous seat lengths. The seat lengths required by AASHTO are given in Section 5.3.1. The larger deflections that occur with a seat-type abutment can be accommodated by elastomeric or sliding bearings. If the deflections become excessive, the required seat lengths noted above should provide adequate protection against loss of support.

5.3.4 *Shear keys*—Shear keys, bearings with side stops or keeper-bars, and other restraining devices should be designed for the full elastic seismic forces (R-factor = 1.0). However, when these devices are located at abutments, the elastic forces should be divided by an R-factor of 0.8. This means that the attachments at abutments are designed for 125 percent of the calculated seismic force, reflecting the uncertainty in calculating abutment forces.

Alternatively, restraining devices may be designed to serve as fuses. At a predetermined force level, the restraint fails; then either the forces are transferred to adjacent restraint devices or deflection is allowed to occur.

5.3.5 *Hold-down devices*—For bridges in AASHTO Seismic Performance Categories C and D, it is recommended (Caltrans, 1990) that hold-down devices be provided where vertical seismic force is greater than 50 percent of the dead load reaction. The hold-down device should be designed for a force of 10 percent of the upward reaction force that would be exerted if the span were simply supported. If the vertical seismic force (Q) exceeds 100 percent of the dead-load reaction (DR), the design upward force for the hold-down device should be 1.2 (Q-DR), but not less than 10 percent of the upward reaction force.

For special structural configurations such as outriggers (straddle bents), C-bents, and cantilevered sections in regions of high seismic activity, criteria for accommodating vertical accelerations should be established on a project-by-project basis.

5.4—Seismically isolated bridges

One design approach to reduce seismic forces is to decouple the superstructure from the substructure by isolator bearings. This approach is commonly termed seismic isolation.

Any practical seismic isolation system, therefore, has three basic elements:

1) A flexible support, such that the period of vibration is lengthened sufficiently to reduce the force response;

2) A damper or energy dissipater such that the relative deflections across the flexible support can be limited to a practical design level; and

3) High stiffness under service loads from wind and braking.

Flexibility

The most common means of introducing a flexible support is by using an elastomeric bearing, although other isolator types such as sliding bearings are also available (FHWA, 1995). The idealized force response with increasing period (flexibility) is shown schematically in the acceleration response curve of Figure 17. Reductions in base shear occur as the period of vibration is lengthened. The extent to which these forces are reduced depends primarily on the nature of the earthquake ground motion, the soil type, and the period of the fixed-base structure. However, as noted, the additional flexibility needed to lengthen the period can give rise to relative displacements across the flexible support (Figure 18).

Energy dissipation

Relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. The additional damping also reduces the peak acceleration (Figure 19).

One of the most effective means of providing a substantial level of damping (greater than 20 percent equivalent-viscous damping) is hysteretic energy dissipation. Figure 20 shows an idealized force-displacement loop. The enclosed area is a measure of the energy dissipated during one cycle of motion. Mechanical devices that use the plastic deformation of either mild steel or lead to achieve this behavior have been developed (Billings and Kirkcaldie, 1985; Buckle and Mayes, 1990a, 1990b; Dynamic Isolation Systems, 1993; Kelly and Jones, 1991).

Rigidity under low lateral load

While lateral flexibility is highly desirable for high seismic loads, it is undesirable to have a bridge that will exhibit significant lateral deflection under frequently occurring loads, for example, wind or braking load. Mechanical energy dissipaters may be used to provide rigidity at these service load by virtue of their high initial elastic stiffness. As an alternative, a separate restraint device may be used for this purpose, typically a rigid component designed to fail at a predetermined level of lateral load (Applied Technology Council, 1986, 1993; Earthquake Engineering Research Institute, 1990).

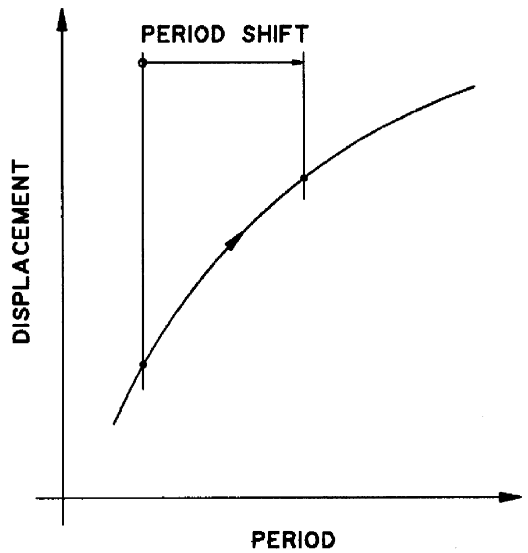


Fig. 18—Response spectra for increased damping.

5.4.1 *Design principles of seismic isolation*—The design principles for seismic isolation are shown in Figure 21. Curve 1 in this figure shows the elastic forces imposed on a non-isolated structure founded on rock, assuming the structure has sufficient strength to resist this level of load. Curve 4 shows the elastic forces divided by the response modification factor, *R*, for which the AASHTO specifications (AASHTO, 1996) require a multi-column, bent bridge to be designed. Curve 3 shows the probable strength, assuming the structure is designed for the AASHTO forces. The probable strength is 1.5 to 2.0 times the design strength because of the design load factors, actual material strengths (which commonly are greater than those specified), conservatism in structural design, and other factors. The difference between the maximum elastic force and the probable strength is an approximate indication of the energy that must be dissipated by ductility in the structural elements. However, when the bridge is isolated, the maximum forces are reduced (Curve 2 in Figure 21). If a seismically isolated bridge is designed for the AASHTO forces in the period range of 1.5 to 3.0 sec as shown in Figure 21, then the probable strength of the isolated bridge is approximately the same as the maximum forces to which it will be subjected. Therefore, there will be little or no ductility demand on the isolated structural system.

Additional information may be found in (Applied Technology Council, 1986, 1993; Billings and Kircaldie, 1985; Buckle et al., 1987; Buckle and Mayes, 1990a, 1990b; Earthquake Engineering Research Institute, 1990; Mayes et al., 1992).

5.4.2 *Objective of AASHTO seismic isolation guidelines*—The AASHTO seismic isolation design requirements for bridges were developed with three basic objectives:

1) To be as consistent as possible with the AASHTO specifications for conventional seismic design;

2) To be as consistent as possible with the Uniform Building Code (UBC) provisions for seismically isolated buildings; and

3) To be applicable to a wide range of possible seismic isolation systems.

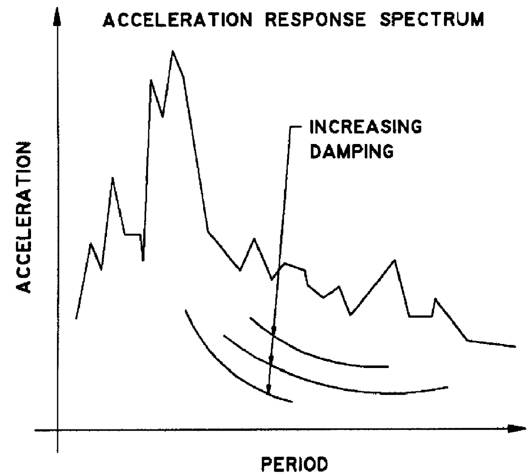


Fig. 19—Response spectra for increased damping.

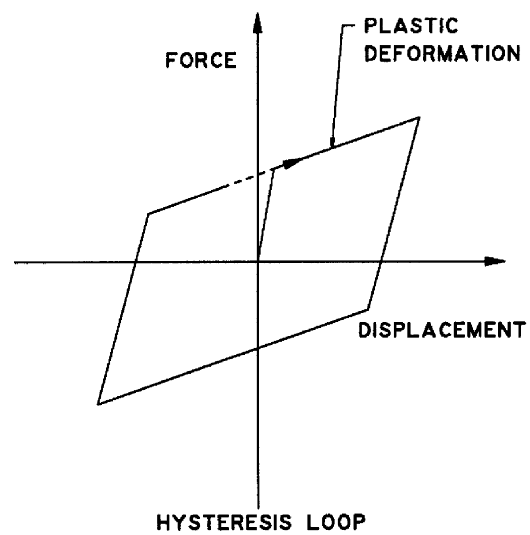


Fig. 20—Hysteretic force-deflection curve.

The first objective necessitated that the requirements fit within the seismic performance category (SPC) concept of the new seismic design provisions. The second objective formed the primary basis for the isolation design requirements. The third objective necessitated that the requirements remain general and, as such, rely on mandatory testing of isolation system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Both static- and dynamic-analysis procedures are included (depending on the SPC). They are based on the same level of seismic input and require the same level of performance from the bridge. The design basis earthquake load corresponds to a level of ground motion that has a 10 percent probability of being exceeded in a 50-year period.

5.4.3 *Philosophy of AASHTO seismic isolation guidelines*—Two design philosophies are included in the AASHTO isolation guide specifications (AASHTO, 1991). The first is to take advantage of the reduced forces and provide a more economical bridge design than conventional construction. This option uses the same response modification factors (*R*-

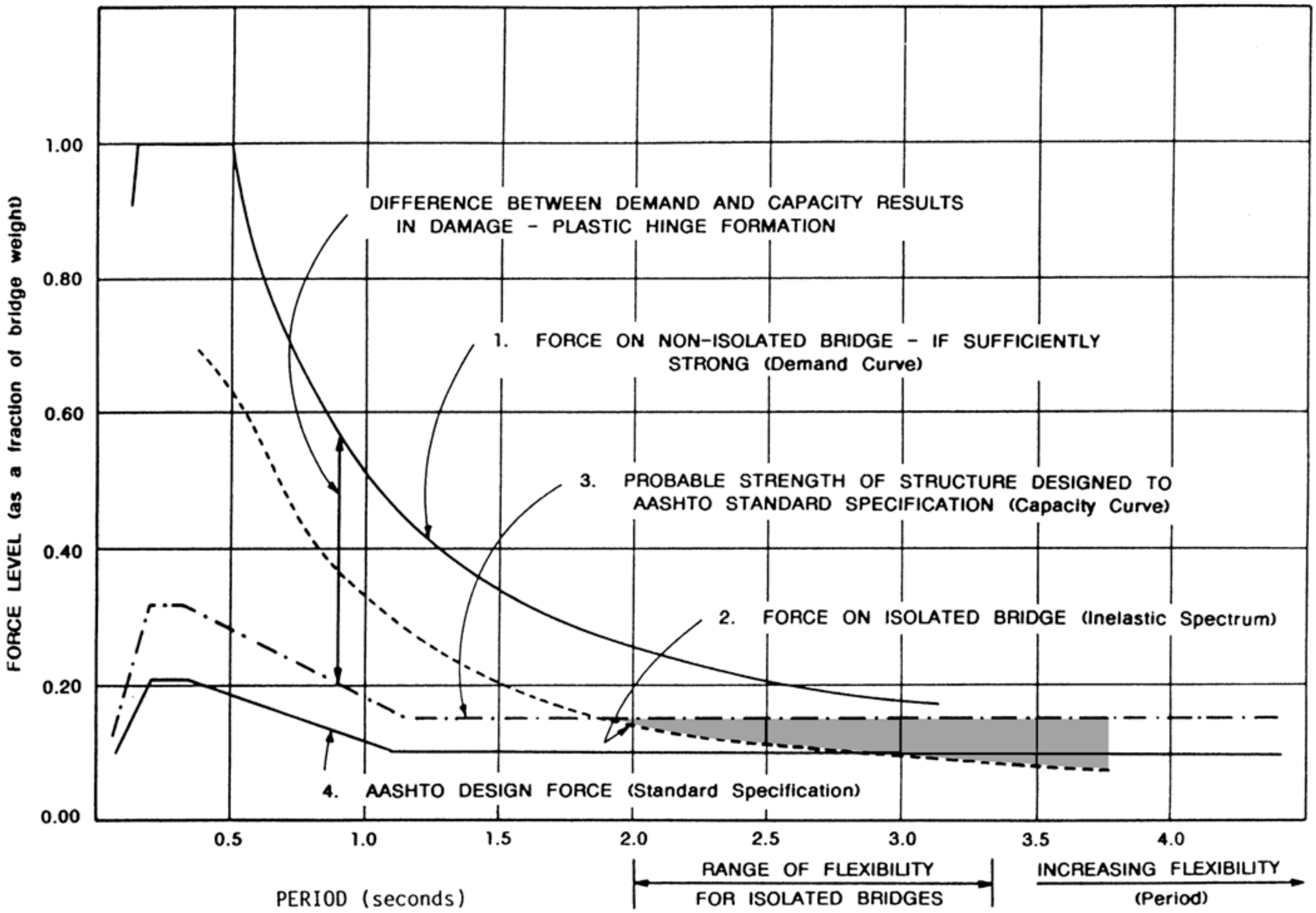


Fig. 21—Behavior of seismically isolated bridges.

factors) as the AASHTO specifications and thus provides the same level of seismic safety.

The intent of the second design option is to eliminate or significantly reduce damage (inelastic deformation) to the substructure and abutments. In this case, an R-factor of 1.0 to 1.5 will ensure an essentially elastic response by eliminating the ductility demand on the substructure.

5.4.4 *Methods of analysis for seismic isolation design*—The basic premise of the seismic isolation design provisions is two-fold:

1) The energy dissipation of the isolation system can be expressed in terms of equivalent viscous damping; and

2) The stiffness of the isolation system can be expressed as an effective linear stiffness.

These two basic assumptions permit both single- and multi-modal methods of analysis to be used for seismic isolation design.

For sliding systems without a self-centering mechanism, or for pure elasto-plastic isolation systems, the equivalent-viscous-damping concept is no longer valid. Consequently, it may be necessary to perform a nonlinear response history analysis for these systems.

Single-mode spectral analysis

The single-mode method of analysis (AASHTO, 1991) is also appropriate for seismic isolation design. The method is applicable to bridges equipped with seismic isolation because the superstructure essentially will have only a rigid-body movement.

Multi-mode spectral analysis

The AASHTO guidelines (AASHTO, 1991) are also appropriate for the response spectrum analysis of an isolated structure, with the following modifications:

- The isolation bearings are modeled by use of their effective stiffness properties determined at the design displacement (Figure 22).
- The ground response spectrum is modified to incorporate the damping of the isolation system (Figure 23).

The response spectrum required for the analysis must be modified to incorporate the higher damping value of the isolation system. This modified portion of the response spectrum should only be used for the isolated modes of the bridge.

5.5—Construction

5.5.1 *Constructability*—Construction plans and details must in general enable the Contractor to estimate quantities and construction procedures involved, at the bidding stage. It is particularly important that seismic details, which may be unique or unfamiliar to the contractor, be shown clearly enough such that the contractor can determine an appropriate erection procedure.

One such detail is the elimination of splices in the vertical reinforcement at the footing for column piers. This detail requires that the column cage be safely supported in position while the footing concrete is placed. The designer should be certain that all special details shown in the plans can be constructed, and that reinforcement is not overly congested.

5.5.2 *Seismic events during construction*—Care must be taken by the designer to provide a design that allows the Contractor to economically stabilize the superstructure elements during construction. Also, special provisions can be written to alert the contractor of the need to stage construction in a way that provides a high level of stability throughout the construction process.

Although there are no code provisions recommending seismic force values during construction, a rational approach may be to provide an "equivalent risk" considering the construction period. For especially vulnerable structures or structures where the public is exposed to construction activities (such as superstructure erection over traffic), consideration should be given to design for seismic effects during erection. Guidance is provided (Caltrans, 1990) for stage constructed projects where partially constructed bridges carry traffic or go over traffic.

In any case, the force levels to be considered during construction should be approved by the owner or governing agency.

5.5.3 *As-built analysis*—For cases where the bridge will be built with significant deviations from the approved design, an as-built seismic evaluation should be performed. Substantial redistribution of loads within a structure may result from as-built changes, such as alteration of the foundation size or characteristics, changes in column or cap sizes, changes in structural-dead-load mass, or changes in connection details. If there is substantial redistribution of the load, the structure should be reanalyzed to determine if it will perform adequately or if retrofitting is required.

CHAPTER 6—REFERENCES

The documents of the various standards-producing organizations referred to in this report are listed below with their serial designation, including year of adoption or revision. The documents listed were the latest effort at the time this report was written. Since some of these documents are revised frequently, the user of this report should check directly with the sponsoring group to refer to the latest revision.

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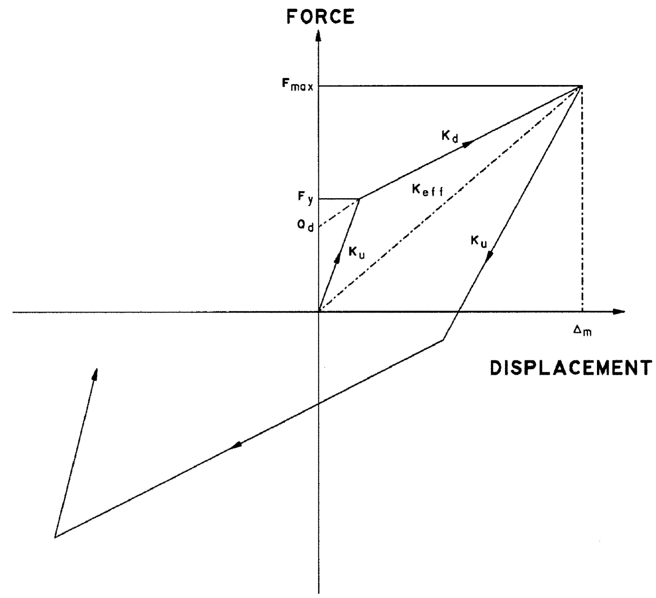

where:
Q_d = *characteristics strength (kips)*
F_y = *yield force (kips)*
F_{max} = *maximum force (kips)*
K_d = *post-elastic stiffness (kip/in.)*
K_u = *elastic (unloading) stiffness (kip/in.)*
K_{eff} = *effective stiffness*
Δ_m = *maximum bearing displacement*

Fig. 22—Bilinear behavior of isolation bearings.

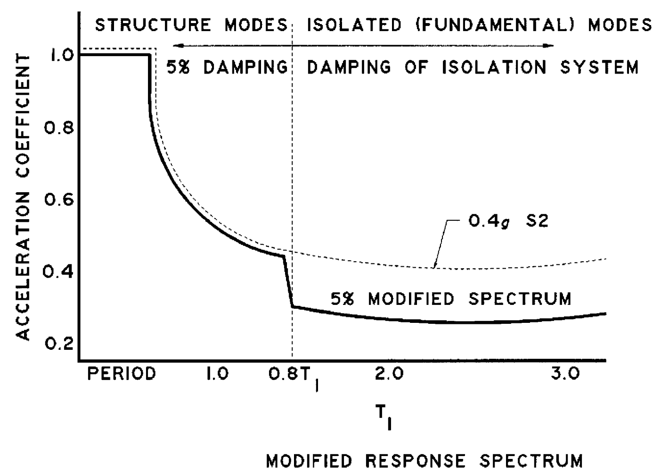

Fig. 23—Modified ground response spectrum for an isolated system.

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This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting procedures.