Section 4: Commentary
DESIGN AND ANALYSIS PROCEDURES

C4.1 SINGLE SPAN BRIDGES

Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single span bridges need not be analyzed for seismic loads regardless of the SDR and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures.

C4.2 SDAP A1 AND A2

In areas of low seismicity only minimum seat widths (Article 6.3) and connection design forces for bearings and minimum shear reinforcement in concrete columns and piles in SDR 2 are deemed necessary for the life safety performance objective. These default values are used as minimum design forces in lieu of rigorous analysis. The division of SDAP A1 and A2 at a short period spectral response acceleration of 0.10 is an arbitrary expedience intended to provide some relief to parts of the country with very low seismicity.

This article describes the minimum connection force that must be transferred from the superstructure to its supporting substructures through the bearings. It does not apply if the connection is a monolithic structural joint. Similarly, it does not apply to unrestrained bearings or in the unrestrained directions of bearings that are free to move (slide) in one direction but fixed (restrained) in an orthogonal direction. The minimum force is simply 0.1 or 0.25 times the weight that is effective in the restrained direction. The calculation of the effective weight requires care and may be thought of as a tributary weight. It is calculated from the length of superstructure that is tributary to the bearing in the direction under consideration. For example, in the longitudinal direction at a fixed bearing, this length will be the length of the segment and may include more than one span if it is a continuous girder (i.e. it is the length from one expansion joint to the next). But in the transverse direction at the same bearing, this length may be as little as one-half of the span, particularly if it is supporting an expansion joint. This is because the expansion bearings at the adjacent piers will generally be transversely restrained and able to transfer lateral loads to the substructure.

It is important that not only the bearing but also the details that fasten the bearing to the sole and masonry plates (including the anchor bolts which engage the supporting members), have sufficient capacity to resist the above forces. At a fixed bearing, it is necessary to consider the simultaneous application of the longitudinal and transverse connection forces when checking these capacities.

Note that the primary purpose of this requirement is to ensure that the connections between the superstructure and its supporting substructures remain intact during the design earthquake and thus protect the girders from being unseated. The failure of these connections has been observed in many earthquakes and imposing minimum strength requirements is considered to be a simple but effective strategy to minimize the

risk of collapse. However, in low seismic zones it is not necessary to design the substructures or their foundations for these forces since it is expected that if a column does yield it will have sufficient inherent ductility to survive without collapse. Even though bridge columns in SDR 2 are not required to be designed for seismic loads, minimum shear reinforcement requirements will provide a minimum level of capacity for ductile deformations which is considered to be adequate for the magnitude and duration of the ground motion expected in SDR 2.

The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of γ_{eq} used in conjunction with Table 3.5-1.

C4.3 SDAP B — NO SEISMIC DEMAND ANALYSIS

The no analysis procedures are an important new addition to the provisions because they apply in the expanded areas now requiring more detailed seismic design. The purpose of these provisions is to provide the designers of regular bridges, that comply with certain restrictions, the ability to design their structure without the need to undertake a dynamic analysis. The bridge is designed for all non-seismic requirements and capacity design procedures are then used to determine shear reinforcement and confining reinforcement requirements. Capacity design principles are also used for the connection forces of the columns to the pile cap or spread footing and the superstructure or bent cap. There are no seismic design requirements for abutments except that integral abutments need to be designed for passive pressure. The superstructure displacements anticipated in these lower zones are expected to be relatively modest and significant abutment contribution to the response of the bridge is not anticipated but if it occurs it will reduce substructure displacements. The design forces for the soil and pile aspects of foundation design are the overstrength forces from the columns but using an overstrength ratio of 1.0 as specified in Article 4.8.1. The use of the lower overstrength ratio for SDR 3 implies that there will be some limited ductility demand on the piles in the event of the 3% PE in 75-year/1.5 mean deterministic earthquake. Since shear and confining reinforcement is also required in the top 3D of the piles this reduction in foundation design forces was believed to be prudent in the lower seismic risk areas. Current AASHTO Division 1-A requirements (SPC B) do not require capacity design of the foundation, rather the foundations are designed for twice the column design forces. Converting to a capacity design approach with an overstrength ratio of 1.0 will lead to a more uniform level of seismic resistance in these lower seismic areas.

C4.3.2 Restrictions

The restrictions on the application of this procedure were developed to ensure that the bridge had a reasonably regular configuration and that unusually high loads would not be present. Furthermore, it is important that most of the substructures contribute to resisting both the transverse and longitudinal lateral loads. A design in which only one of several piers resists longitudinal loads, needs to be analyzed prior to its detailed design. The restrictions on the method were based on engineering judgment and should be refined as research work on this design approach progresses.

Structures with lower axial loads or stronger columns (i.e., more steel and large column and pile sizes) have a greater intrinsic strength and are able to resist the design ground motions with less damage. However, ductile detailing still needs to be provided in accordance with Articles 7.7 and 7.8.

The no analysis provisions are not applicable to steel braced frame substructures. In the case of a cantilever column, in a pile bent configuration, the length L in the L/b<10 criteria would be equal the length above ground to the top of the bent plus 3 pile bent diameters.

These provisions do not apply for bridges with variable height piers because one or more piers will attract significantly more lateral load. Designers are encouraged to design the portion of piers participating in a seismic mechanism to have similar column lengths.

Variable span lengths can also create uneven loading conditions on the piers resulting for unusual modal behavior and are therefore not permitted.

For highly skewed and curved bridges, biaxial loading of the piers can be problematic from a design point-of-view and hence this method is not applicable. Moreover, extra care needs to be taken in assessing the displacement demands at joints and bearings.

Designers are actively discouraged from using one pier to resist all longitudinal inertia loads when using this analysis method. The further limit on F_vS_1 due to a lesser number of bents participating is a first attempt to limit this type of configuration. Its use is most appropriate when all supporting bents participate in the ERS.

Careful and site specific analysis of the soil-structure interaction is needed at sites with liquefaction or lateral spreading potential and hence this is another limit on the method.

C4.3.3 Capacity Design and Strength Requirements of Members Framing into Columns

The principles of capacity design require that the strength of those members that are not part of the primary energy dissipating system be stronger than the overstrength capacity of the primary energy dissipating members—that is, the columns with hinges at their member ends.

The geotechnical features of foundations (i.e. soil bearing, and side friction and end bearing on piles) possess inherent ductility. At low to moderate levels of seismic input this manifests itself as minor rocking of the foundation and/or nominal permanent settlements which do not significantly affect the service level of the bridge.

Full capacity protection of the geotechnical features of the foundation in SDAP B is not required. Should the rare earthquake occur, some limited ductility demand may occur in the piles and some minor rocking and permanent settlement may occur. This trade-off, compared to current practice for SPC B in the existing AASHTO provisions, was believed to be prudent.

C4.4 SDAP C — CAPACITY SPECTRUM DESIGN METHOD

The capacity spectrum design method is conceptually the same as the Caltrans displacement based design method. The primary difference is that the capacity spectrum approach begins with the existing nonseismic capacity of the columns and then assesses the adequacy of the resulting displacements. The Caltrans procedure uses methods to estimate the maximum displacement that can be tolerated and then assesses the minimum strength requirements for the column.

The key equation used in the capacity spectrum method is the relationship between the seismic capacity coefficient, C_c, and displacement, Δ:

$$C_C\Delta = \left(\frac{F_V S_1}{2\pi B_L}\right)^2 g \qquad \text{C.4.4-1}$$

in which S_1 is the spectral acceleration coefficient at 1 second period, F_v is the site factor for the earthquake event, and g is the acceleration due to gravity (32.2 ft/sec^2 or 9.8 m/sec^2). The factor B_L reduces the demand to account for inelastic deformation capacity of the earthquake resisting elements; Table 5.4.1-1 gives B_L for the two earthquake events and two performance levels. This equation is valid in the velocity-sensitive region of the response spectrum and is applicable to most bridges of a regular configuration that respond essentially as a single-degree-of-freedom system. The complete design procedure includes steps for shorter period bridges, such as those with pier walls, but such cases are not discussed in this commentary.

Bridges that have elastomeric or sliding bearings at each pier shall be designed as an isolated structure using all of the provisions of Article 15 because it is essential that the columns remain essentially elastic (i.e., R = 1.5).

The following detailed summary of this method expands on the procedure outlined in the Specification. It focuses on conservative estimates of strength and displacement. More refined techniques may be used which still satisfy the capacity spectrum method, but for most cases the simple approach described herein provides efficient designs that will satisfy the performance requirements defined in the Specifications.

Step 1

With the design for all non-seismic requirements determine if the configuration and component requirements for a very regular bridge are

satisfied. If so, the capacity spectrum procedure may be used.

Step 2

Determine F_v and S_1 for the 50% PE in 75-year earthquake event. In the longitudinal and transverse direction, perform the following sub steps:

2-1 Compute the yield displacement, Δ_y, for each participating bent or pier; set Δ_y to 1.3 times the smallest value. Note that a participating pier or bent is one whose fixity conditions permits it to resist horizontal lateral loads. It is possible a pier may participate transversely but not longitudinally due to a bearing that has transverse fixity and longitudinal movement.

2-2 Compute the lateral strength of each participating pier or bent, and sum the strengths to give the lateral strength of the bridge, V_n. The seismic capacity coefficient for the bridge is $C_c=V_n/W$, in which W is the weight of the bridge responding to earthquake ground motion (generally the superstructure and a portion of the substructure).

2-3 If the following equation is satisfied for the 50% PE in 75-year values of F_v and S_1,

$$C_c\Delta_y \geq \left(\frac{F_v S_1}{2\pi}\right)^2 g \qquad \text{C4.4-2}$$

the bridge is expected to meet the performance requirement for the 50% PE in 75-year earthquake event.

Step 3

If the equation in step 2-3 is not satisfied, increase the strength of the participating piers or reconfigure the bridge so more piers participate such that V_n and C_c satisfy step 2-3.

Step 4

Determine F_v and S_1 for the 3% PE in 75-year/1.5 mean deterministic earthquake event. For the strength of the bridge in step 3, determine if the bridge has sufficient deformation capacity according to the following sub steps in the longitudinal and transverse directions:

4-1 Using the strength from step 3, determine the maximum displacement from:

$$\Delta = \frac{1}{C_c}\left(\frac{F_V S_1}{2\pi B_L}\right)^2 g \qquad \text{C4.4-3}$$

where B_L is obtained from Table 5.4.1-1.

4-2 Check that the maximum displacement is less than the deformation capacity for the shortest pier, with height H:

$$\Delta \leq \theta_p H \qquad \text{C4.4-4}$$

for reinforced concrete columns satisfying the requirements of Article 7.8 or 8.8, the plastic rotation capacity, θ_p, may be taken as 0.035 or as given in Article 7.8.6. A similar value is applicable for steel columns that satisfy the requirements of Article 7.7 or 8.7 or as given in Article 7.7.9.

4-3 Check that the P-delta requirement is met using the height of the shortest participating pier:

$$\Delta \leq 0.25 C_c H \qquad \text{C4.4-5}$$

If the displacement limits in steps 4-2 and 4-3 are met, the design is satisfactory for the 3% PE in 75-year/1.5 mean deterministic earthquake event.

Step 5

If the displacement limits in step 4 are not satisfied, the strength of the participating piers must be increased or additional piers must participate. For reinforced concrete columns it is necessary to increase the longitudinal reinforcement. If the reinforcement ratio exceeds 2.5%, the column size may need to be increased. The new strength can be determined as follows, in the longitudinal and transverse directions:

5-1 If step 4.2 is not satisfied, set the maximum displacement to $\Delta = \theta_p H$, where H is the height

of the shortest participating column. Determine the required seismic coefficient from,

$$C_c \quad \frac{1}{2}\frac{F_V S_1}{B_L}^{2} g \qquad \text{C4.4-6}$$

5-2 If step 4.3 is not satisfied, determine the required seismic coefficient from,

$$C_C = 4\frac{\Delta}{H} \qquad \text{C4.4-7}$$

where H is the height of the shortest column.

5-3 The required lateral strength is $V_n = C_c W$, where W is the total weight of the bridge, and C_c is a seismic capacity coefficient which is the ratio between the strength of the lateral load system and the inertial weight resisted by the lateral load system. Proportion V_n to the individual piers participating in resisting lateral loads in proportion to the tributary mass for the pier. Redesign the piers to provide the required strength.

Bridges that satisfy step 4 and 5 are expected to have satisfactory performance in the 3% PE in 75-year/1.5 mean deterministic earthquake event for each performance level.

Step 6

Capacity design procedures of Article 4.8 are used to determine the shear and confinement reinforcement requirements, the column connection forces and the foundation design forces. The bridge is designed so it can resist the 3% PE in 75-year/1.5 mean deterministic event without any contribution from the abutment and hence there are no seismic design requirements for the abutments.

C4.4.2 Restrictions

The configuration requirements for Capacity spectrum analysis restrict application to individual frames or units that can be reasonably assumed to respond as a single degree-of-freedom system in the transverse and longitudinal directions. When abutments do not resist significant seismic forces, the superstructure will respond as a rigid-body mass. The lateral load-resisting piers or bents must be uniform in strength and stiffness to justify the assumption of independent translational response in the longitudinal and transverse directions. Abutments are assumed to not participate in the lateral resisting system and therefore are not part of the stiffness or strength limitations. All the lateral load is resisted by the piers.

SDAP C may be appropriate for pier wall substructures in the longitudinal direction but will not work in the transverse direction if bearings are fixed. If bearings permit movement transversely, then the capacity spectrum method for isolation bearings (Article 15.4) shall be used.

The restrictions are similar to the ones for no-analysis in Article 4.3.2.

C4.5 SDAP D — ELASTIC RESPONSE SPECTRUM METHOD

This is essentially a two level analysis procedure, however in many parts of the US, and in the eastern US in particular, the 50% PE in 75 year event will rarely govern. In most cases designers will be able to quickly assess which of the two events will produce the maximum column moments by dividing the ground response spectra by the respective R-Factors and comparing the relative values. Only when the two spectra are relatively close will two analyses be required.

C4.6 SDAP E — ELASTIC RESPONSE SPECTRUM METHOD WITH DISPLACEMENT CAPACITY VERIFICATION

This is a two step design procedure and is a key element in the philosophic development of these new provisions. The pushover method of analysis has seen increasing use throughout the 1990s, especially in the significant Caltrans retrofit program. This analysis method provides additional information on the expected deformation demands of columns and foundations and as such provides the designer with a greater understanding of the expected performance of the bridge. The use of the pushover method of analysis was used in two ways. First, it provided a mechanism to be as liberal as possible with the R-Factor for preliminary column design because there was an additional limit on the column plastic

rotations that must be obtained from a pushover analysis.

Second, it provided a mechanism to allow ERE that need owners approval (Article 3.3.1). The trade-off was the need for a more sophisticated analysis in order that the expected deformations in critical elements could be assessed. Provided the appropriate limits (i.e., plastic rotations for in-ground hinges) were met, the ERE requiring owners approval could be used. This method applies to all the ERE shown in Figure C3.3.1.1(b).

The early drafts of the provisions used stress and strain limits for the deformations resulting from the pushover analysis. The difficulty in translating plastic rotations that result from most nonlinear analysis programs to element stresses and strains led to the decision to provide limits on plastic rotations as given in Articles 7.7.9 and 7.8.6.

C4.7 RESPONSE MODIFICATION FACTORS

These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate R-Factor. Most other elements of the ERS are designed by capacity design procedures for the maximum forces that can be developed by plastic hinges in the columns or the elastic forces from the analysis.

The most important R-Factor is that of the supporting substructure. Since a bridge closely approximates a single-degre-of-freedom (SDOF) system, the design process is schematically shown Figure C3.3-2 and discussed in Article C3.3. There has been a considerable amount of research over the past ten years on the relationship between the ductility demand of a SDOF system and its design strength. For example, if we assume an element has a displacement ductility capacity μ at a given value, we would like to know the design force necessary to ensure that this ductility is not exceeded. A good overview of this issue can be found in ATC-18 (1997), which summarizes the work of Mirander and Bertero (1996), Nasser and Krawinkler (1991) and Chang and Mander (1994) Figure C4.7.1-1 shows a smoothed relationship (Mirander and Bertero, 1996) between the ductility factor μ and R for two sites. Note that R is less than μ for periods less than one second and hence the need for the short period modifier on R given by Equation 4.7-1.

The short period modifier should be applied separately in the transverse and longitudinal directions if there is a significant difference in the fundamental periods of the two directions (e.g. pier walls). If there is significant coupling in the fundamental mode shapes in the two directions then the lowest period should be used to determine the R-Factor in both directions.

If the abutments are dominating the period of the structure and causing a low R-Factor an engineer could evaluate the option of assuming all lateral loads are taken by the columns. This will result in a longer period structure and hence a higher R-Factor but will be offset by the columns taking more load.

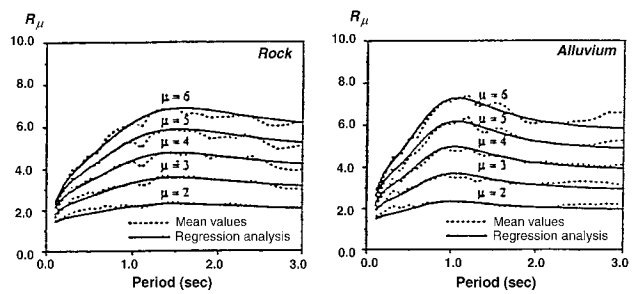

Figure C4.7.1-1 Comparison of Mean Strength-Reduction Factors of Rock and Alluvium Sites with Regression Analysis

The R-Factors of Table 4.7-1 were based on an evaluation of existing test data of structural components, parameter studies that were performed in conjunction with the development of these provisions and engineering judgment. The Project Team first reviewed the test data on reinforced concrete columns (Taylor and Stone, 1993; Hose, Silvan and Sieble, 1999) to establish the range of ductility capacity that could be relied upon. This was in the range of 6-10 for well-detailed columns, depending on the range of design parameters (e.g., axial load, longitudinal and confinement reinforcement, etc.). The parameter study associated with the development of this criteria showed that there were only a limited number of instances where use of an R-Factor greater than 6 would not be limited either by the minimum longitudinal steel requirement of

0.8% in concrete columns or the *P-Δ* requirements of Article 7.3.4 or 8.3.4. As a consequence the R-Factor for concrete and steel columns was set at 6 for SDAP E with a provision that the design forces could be further reduced (not lower than 70%) provided the displacement capacity of the element was satisfied in the pushover analysis.

Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.

C4.8 CAPACITY DESIGN

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top and/or bottom) where they can be readily inspected and/or repaired. To achieve this objective all members connected to the columns, the shear capacity of the column and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements is when all substructure elements are designed elastically (Article 4.10), seismic isolation design (Article 7.10 or 8.10) and in the transverse direction of columns when a ductile diaphragm (Article 7.7.8) is used.

C4.8.1 Inelastic Hinging Forces

The principles of capacity design require that the strength of those members that are not part of the primary energy dissipating system be stronger than the overstrength capacity of the primary energy dissipating members—that is, the columns with hinges at their member ends.

This clause permits three approaches of increasing sophistication (but also of increasing effort to conduct) for assessing the overstrength capacity of reinforced concrete columns. See Article 4.3.3 for foundation design in SDR 3.

Overstrength factors applied to nominal moment capacities are a simplified method for determining flexural overstrength. For reinforced concrete columns, detailed calculations of overstrength factors for a variety of column properties (Mander, Dutta and Goel (1997)) ranged from 1.25 to 1.50. A conservative default value of 1.5 is specified for the first approach but a designer can calculate a more precise project specific value using one of the remaining two approaches.

For the second approach, the flexural moment overstrength capacity (M_{po}) of reinforced concrete column/pier/pile members that form part of the primary mechanism resisting seismic loads may be assessed using the simplified plastic moment-axial load interaction formula method developed in Mander, Dutta and Goel (1997) – See Article C8.8.2.8. It is recommended that for this approach f'_{co} for concrete be assumed to be $1.7f'_c$ and f_{yo} of steel be $1.3f_y$

When assessing overstrength capacity of flexural members using the third approach, compatibility section analysis (i.e the moment-curvature method), it is important to differentiate between overstrength resulting from the response of the section to high curvature demands, and overstrength resulting from upper bound material properties.

For example, in the case of reinforced concrete columns, confined concrete will have enhanced capacity and reinforcing steel will strain harden at high plastic curvatures. This will result in increased flexural capacity of the column that will be captured by a moment curvature analysis that considers these factors. In addition, reinforcing steel can have a higher than nominal yield point, and concrete is likely to be stronger than specified and will gain strength with age beyond the 28 day specified strength. It has been recommended that for the purpose of a rigorous calculation that f'_{co} for concrete be assumed to be $1.7f'_c$ and f_{yo} of steel be $1.3f_y$. In this case the overstrength moment is taken at the design curvature from the moment curvature analysis (ATC, 1996).

For structural steel, f_{yo} may be taken as $1.2F_{ye}$ where F_{ye} is the expected yield strength considering the likelihood that higher than nominal strength steel will be used. The plastic section modulus should be used in overstrength moment calculations for steel members.

The conservative requirement to calculate the capacity design shear force of a column, pile bent or drilled shaft at the mud or ground line will provide adequate shear capacity if fixity of the column occurs at the ground line any time in the future. Such an event caused a column shear

failure in the 1994 Northridge earthquake on a major freeway bridge. If a concrete traffic barrier could reduce the fixity of the column then only the height above the barrier should be considered in the shear force calculation.

C4.10 ELASTIC DESIGN OF SUBSTRUCTURES

C4.10.1 All Substructure Supports are Designed Elastically

If all the supporting substructures elements (columns, piers, pile bents) are designed elastically for the 3% PE in 75 year/1.5 mean deterministic event, there will be no redistribution of lateral loads due to plastic hinges developing in one or more columns. As a consequence the elastic analysis results are appropriate for design. The recommended provisions attempt to prevent any brittle modes of failure from occurring.

C4.10.2 Selected Substructure Supports are Designed Elastically

If only one or a selected number of supporting substructure elements are designed elastically, there will be a significant redistribution of lateral loads when one or more of the columns develop plastic hinges. Generally, the elastically designed elements will attract more lateral load. Hence the need to either use capacity design principles for all elements connected to the elastically designed column. If this is not practical, the complete bridge needs to be reanalyzed using the secant stiffness of any columns in which plastic hinges will form in order to capture the redistribution of lateral loads that will occur.