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SEISMIC RETROFITTING GUIDELINES FOR HIGHWAY BRIDGES

Research, Development, and Technology
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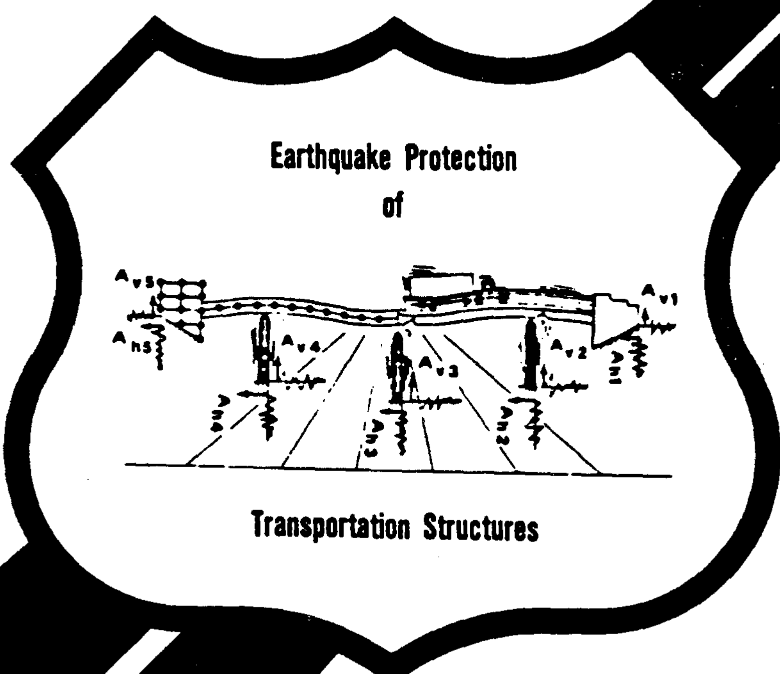

FOREWORD

Presented in this report are guidelines for use in the seismic retrofitting of typical highway bridges which will be of interest to highway bridge engineers, structural engineers, and researchers. This document was prepared by Applied Technology Council, Palo Alto, California, for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9295. Earthquake engineering research has been included in the FHWA Federally Coordinated Program of Highway Research and Development as Task 1 of Project 5A, "Improved Protection Against Natural Hazards of Earthquake and Wind."

The guidelines represent the collective knowledge of a distinguished group of academicians, designers and highway bridge engineers. They were formulated and based on both the observed performance of bridges during past earthquakes and on recent research conducted in the United States and abroad, and are applicable for use in all parts of the country.

Sufficient copies of the report are being distributed to provide a minimum of five copies to each FHWA Regional office, Division office, and State Highway Departments. Direct distribution is being made to the Division office.

Richard E. Hay
Director, Office of Highway
Operations and Engineering
Research and Development
Federal Highway Administration

NOTICE

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16. Abstract

This document contains guidelines for the seismic retrofitting of highway bridges. The guidelines are the recommendations of a team of nationally recognized experts, which includes consulting engineers, academicians, state highway engineers, and federal agency representatives from throughout the United States.

The guidelines are comprehensive in nature and embody several new concepts. An extensive commentary documenting the basis for the guidelines and a worked example problem illustrating their use are included.

The guidelines include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies. The preliminary screening procedures are used to identify bridges that are the most likely candidates for retrofitting. The methods for evaluating an existing bridge in detail involve the calculation of seismic capacity/demand ratios for each potentially vulnerable bridge component, overall assessment of the consequences of a design earthquake at the bridge site, and selection of the most appropriate retrofitting scheme for the bridge. Retrofitting schemes may be selected from among those presented and discussed in the guidelines.

These guidelines utilize many of the concepts presented in the "Seismic Design Guidelines for Highway Bridges" (Report No. FHWA/RD-81/081) and are intended to be used in conjunction with that document.

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PREFACE

This document, prepared by Applied Technology Council, contains guidelines for the seismic retrofitting of highway bridges. The guidelines are the recommendations of a team of nationally recognized experts, composed of consulting engineers, academicians, state highway engineers, and federal agency representatives from throughout the United States. The document represents a consensus of the project participants.

The guidelines are comprehensive in nature and embody several new concepts which are significant departures from existing retrofitting practice. An extensive commentary documenting the basis for the guidelines and a worked example problem illustrating their use are included.

The guidelines include a preliminary screening procedure, methods for evaluating an existing bridge in detail, and potential retrofitting measures for the most common seismic deficiencies. The preliminary screening procedures are used to identify bridges that are the most likely candidates for retrofitting. This procedure rates bridges based on the vulnerability of the structural system, the seismicity of the bridge site, and the importance of the bridge. The methods for evaluating an existing bridge in detail involve, as a first step, the calculation of seismic capacity/demand ratios for each potentially vulnerable bridge component. These capacity/demand ratios are used to assess the consequences of a design earthquake at the bridge site. The results of this assessment can then be used to select the most appropriate retrofitting scheme for the bridge. These retrofitting schemes may be selected from among those presented and discussed in the guidelines. Also included are special design requirements for various retrofitting measures.

These guidelines utilize many of the concepts presented in the "Seismic Design Guidelines for Highway Bridges", also prepared by Applied Technology Council, and are intended to be used in conjunction with that document.

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GLOSSARY

SYMBOLS AND DEFINITIONS

A = Acceleration coefficient as determined from Figures 1 and 2 (dimensionless)

A_a = Effective peak acceleration coefficient from ATC-3-06 report (dimensionless)

A_b = Cross-sectional area of a spliced reinforcing bar (sq. in. OR sq. cm.)

A_g = Gross cross-sectional area of a column (sq. in. OR sq. cm.)

$A_L(c)$ = Effective peak ground acceleration at which liquefaction failures are likely to occur (decimal fraction of acceleration of gravity)

$A_L(d)$ = Acceleration coefficiency, A, as determined from Figures 1 and 2 (dimensionless)

$A_{tr}(c)$ = Effective cross-sectional area of transverse reinforcement (sq. in. OR sq. cm.)

$A_{tr}(d)$ = Required cross-sectional area of transverse reinforcement (sq. in. OR sq. cm.)

A_v = Effective peak velocity-related acceleration coefficient from ATC-3-06 report (dimensionless)

b_c = Width of a column in the direction of loading (feet OR meters)

BCR = Benefit-to-cost ratio (dimensionless)

b_{max} = Maximum transverse column dimension (feet)

b_{min} = Minimum width of the column cross-section (inches OR centimeters)

BVR = Base Vulnerability Rating (dimensionless)

c = The lesser of the clear cover over the bar or bars, or half the clear spacing between bars (inches OR centimeters)

C/D = Capacity/Demand Ratio (dimensionless)

CVR = Column Vulnerability Rating (dimensionless)

d_b = Nominal diameter of reinforcing bar (inches OR centimeters)

$d(c)$ = Displacement capacity at abutments (inches OR centimeters)

$d(d)$ = Displacement demand at abutments (inches OR centimeters)

F = Framing Factor used to calculate column vulnerability (dimensionless)

f'_c = Compressive strength of concrete (psi)

f_y = Yield stress in longitudinal steel reinforcement (psi)

f_{yt} = Yield stress in transverse steel reinforcement (psi)

H = Height of a column or pier used to calculate minimum support length (feet OR meters)

IC = Importance Classification (dimensionless)

k_1 = A constant, which accounts for the volumetric ratio of transverse steel, used to calculate C/D ratios for transverse confinement (dimensionless)

k_2 = A constant, which accounts for the spacing of transverse steel, used to calculate C/D ratios for transverse confinement (dimensionless)

k_3 = A constant, which reflects the effectiveness of transverse bar anchorage (dimensionless)

k_m = A constant, which accounts for the effect of concrete side cover, used to calculate effective anchorage length for hooked anchorage (dimensionless)

k_s = A constant, which accounts for steel yield stress, used to calculate effective anchorage length for straight anchorage (dimensionless)

k_{tr} = A constant, which accounts for the effect of transverse reinforcement, used to calculate effective anchorage length for straight anchorage (dimensionless)

L = Length of bridge deck used to calculate minimum support length (feet OR meters)

$\ell_a(c)$ = Effective anchorage length of longitudinal reinforcement (inches OR centimeters)

$\ell_a(d)$ = Required effective anchorage length of longitudinal reinforcement (inches OR centimeters)

L_c = Effective column height (feet)

$Loss_B$ = Probable loss before retrofitting (dollars, lives, etc.)

$Loss_R$ = Probable loss after retrofitting (dollars, lives, etc.)

ℓ_s = Length of splices in column longitudinal reinforcement (inches OR centimeters)

SYMBOLS AND DEFINITIONS (CONT.)

M_u	=	Ultimate moment capacity of a column or footing (pound feet OR Newton meters)
N(c)	=	The support length provided at a bearing seat (inches or mm)
N(d)	=	The minimum support length as specified in Section 4.6 (inches or mm)
P_c	=	Axial compressive load on the column (pounds OR Newtons)
P_s	=	Percent main reinforcing steel (percent)
Q_{EQ}	=	Displacement or force demand for design earthquake
Q_i	=	Displacement or force demand due to non-seismic loading "i"
r	=	Seismic capacity/demand ratio (dimensionless)
R	=	Response modification factor (dimensionless)
r_{ad}	=	Capacity/demand ratio for abutment displacement (dimensionless)
r_{bd}	=	Capacity/demand ratio for bearing displacement (dimensionless)
r_{bf}	=	Capacity/demand ratio for bearing force (dimensionless)
R_c	=	Ultimate displacement or force capacity
r_{ca}	=	Capacity/demand ratio for anchorage of longitudinal reinforcement in columns (dimensionless)
r_{cc}	=	Capacity/demand ratio for transverse confinement in columns (dimensionless)
r_{cs}	=	Capacity/demand ratio for splices in column longitudinal reinforcement (dimensionless)
r_{cv}	=	Capacity/demand ratio for column shear (dimensionless)
r_{ec}	=	Ultimate moment capacity/elastic moment demand ratio for columns (dimensionless)
r_{ef}	=	Ultimate moment capacity/elastic moment demand ratio for footings (dimensionless)
r_{fr}	=	Capacity/demand ratio for footing rotation and/or yielding (dimensionless)
r_{sl}	=	Capacity/demand ratio for soil liquefaction (dimensionless)
s	=	Spacing of transverse reinforcement in a column (inches OR centimeters)

SYMBOLS AND DEFINITIONS (CONT.)

SPC	=	Seismic Performance Category (dimensionless)
T	=	Fundamental period of the bridge (seconds)
$V_b(c)$	=	Force capacity at the bearings (pounds OR Newtons)
$V_b(d)$	=	Force demand at the bearings (pounds OR Newtons)
$V_e(d)$	=	Elastic shear demand at the columns (pounds OR Newtons)
$V_f(c)$	=	The final shear capacity of a column damaged by spalling (pounds OR Newtons)
$V_i(c)$	=	The initial shear capacity of an undamaged column (pounds OR Newtons)
$V_u(d)$	=	The maximum shear force in a column due to plastic hinging (force units)
ϕ	=	Capacity reduction factor (dimensionless)
μ	=	Ductility indicator used to modify ultimate moment capacity/elastic moment demand ratios to account for post-elastic behavior (dimensionless)
$\Delta_{eq}(d)$	=	Movement at an expansion joint due to design earthquake (inches OR centimeters)
$\Delta_i(d)$	=	Movement at an expansion joint due to temperature, shrinkage, and creep shortening (inches OR centimeters)
$\Delta_s(c)$	=	Allowable movement at expansion joint (inches OR centimeters)
$\rho(c)$	=	Volumetric ratio of transverse confinement reinforcement in a column (dimensionless)
$\rho(d)$	=	Required volumetric ratio of transverse confinement reinforcement in a column (dimensionless)

CHAPTER 1

INTRODUCTION

1.1 PURPOSE

These guidelines offer procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Specifically they contain:

- A preliminary screening process to identify and rate bridges that need to be evaluated for seismic retrofitting.

- A methodology for quantitatively evaluating the seismic capacity of an existing bridge and determining the overall effectiveness, including cost and ease of installation of alternate seismic retrofitting measures.

- Retrofit schemes and design requirements for increasing the seismic resistance of existing bridges.

The guidelines do not prescribe rigid requirements dictating when and how bridges are to be retrofitted. The decision to retrofit a bridge depends on a number of factors, several of which are outside the realm of engineering. These would include, but not be limited to, the availability of funding as well as political, social, and economic considerations. The guidelines assist in evaluating the engineering factors.

Seismic retrofitting of bridges is a relatively new concept. Only a few retrofitting schemes have been used in practice. At the present stage of development, seismic retrofitting is an art requiring considerable engineering judgment. The guidelines present concepts in seismic retrofitting, but should not be interpreted as restricting innovative designs which are consistent with the principles of good structural engineering.

The primary goal of seismic retrofitting is to minimize the risk of unacceptable damage during a design earthquake. Damage is unacceptable if it results in:

- The collapse of all or part of the bridge, or

- The loss of use of a vital transportation route.

In most cases it is not feasible to strengthen existing bridges to the same standards used for new construction. However, the performance of a structure during an earthquake often can be greatly improved, and unacceptable damage averted, through relatively inexpensive and straightforward means. Although retrofitting is not intended to completely eliminate structural damage, retrofitting measures should be designed to limit damage to easily accessible areas. In this way, bridges can be readily repaired following an earthquake, if necessary, to restore them to their intended use.

When a decision is made to retrofit vulnerable structural components, these components should be strengthened to the standards for new construction if economically feasible. Usually this will not strengthen the entire structure to new design standards because some damage may occur in other components. The risk of damage in other components may be accepted either because the damage does not constitute an unacceptable failure, or because retrofitting of these other components is not practical or is too expensive.

1.2 BACKGROUND

These guidelines are intended to be used in conjunction with the report entitled, "Seismic Design Guidelines for Highway Bridges," herein referred to as the Seismic Design Guidelines. The Seismic Design Guidelines were developed for national use and contain provisions for considering the variable levels of expected seismic activity in the United States. The level of expected seismic activity is reflected in the Acceleration Coefficient, A, which is assigned to all locations covered by the Guidelines. Contour maps of Acceleration Coefficients are shown in Figures 1 and 2.

The Seismic Design Guidelines also consider the importance of the structure in social/survival and security/defense terms through the use of an Importance Classification (IC). Essential bridges are assigned to Importance Classification I, while all other bridges are placed in Importance Classification II. The Importance Classification is used along with the Acceleration Coefficient to assign bridges to one of four Seismic Performance Categories (SPC), A through D, as shown in Table 1. The complexity of analysis and design requirements vary depending on the Seismic Performance Category.

TABLE 1: SEISMIC PERFORMANCE CATEGORY

Acceleration Coefficient	Importance Classification I	Importance Classification II
$A \le 0.09$	A	A
$0.09 < A \le 0.19$	B	B
$0.19 < A \le 0.29$	C	C
$0.29 < A$	D	C

The Seismic Design Guidelines utilize a force design approach. Elastic response spectrum analysis procedures are used to determine seismic displacements and elastic member forces. Design forces are obtained by dividing elastic member forces by response modification factors which account for redundancy and ductility in structural members. Design forces may be reduced even further when column yielding will limit forces to certain maximum values.

The Seismic Design Guidelines consider design displacements to be as important as forces. To minimize the potential for a loss of support failure at bearings and expansion joints, minimum support lengths are required. These support lengths were selected to accommodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface-wave motions.

This approach to the determination of seismic forces and displacements and many other concepts used in the Seismic Design Guidelines have been adapted to these guidelines for retrofitting.

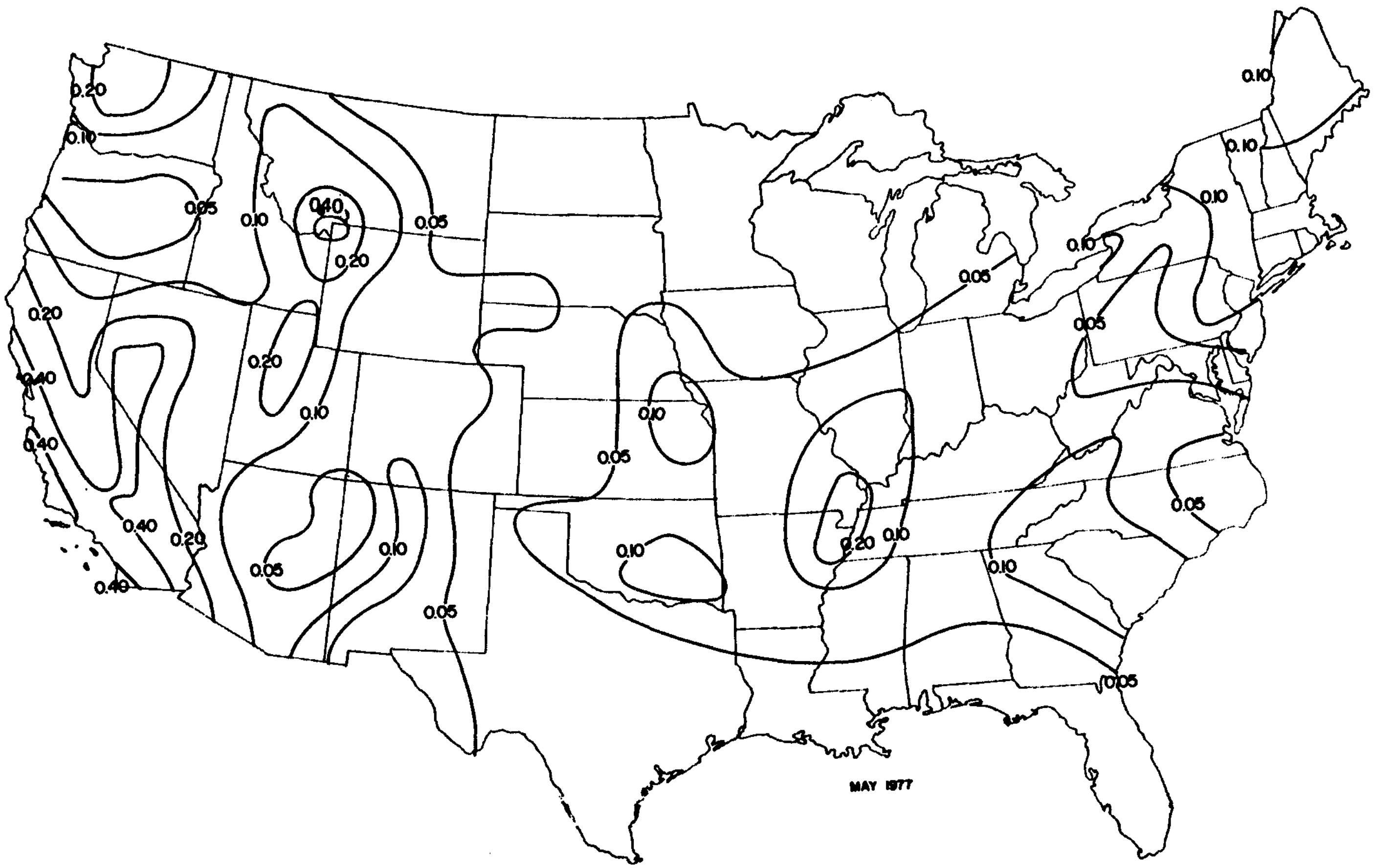


FIGURE 1. ACCELERATION COEFFICIENT — CONTINENTAL UNITED STATES

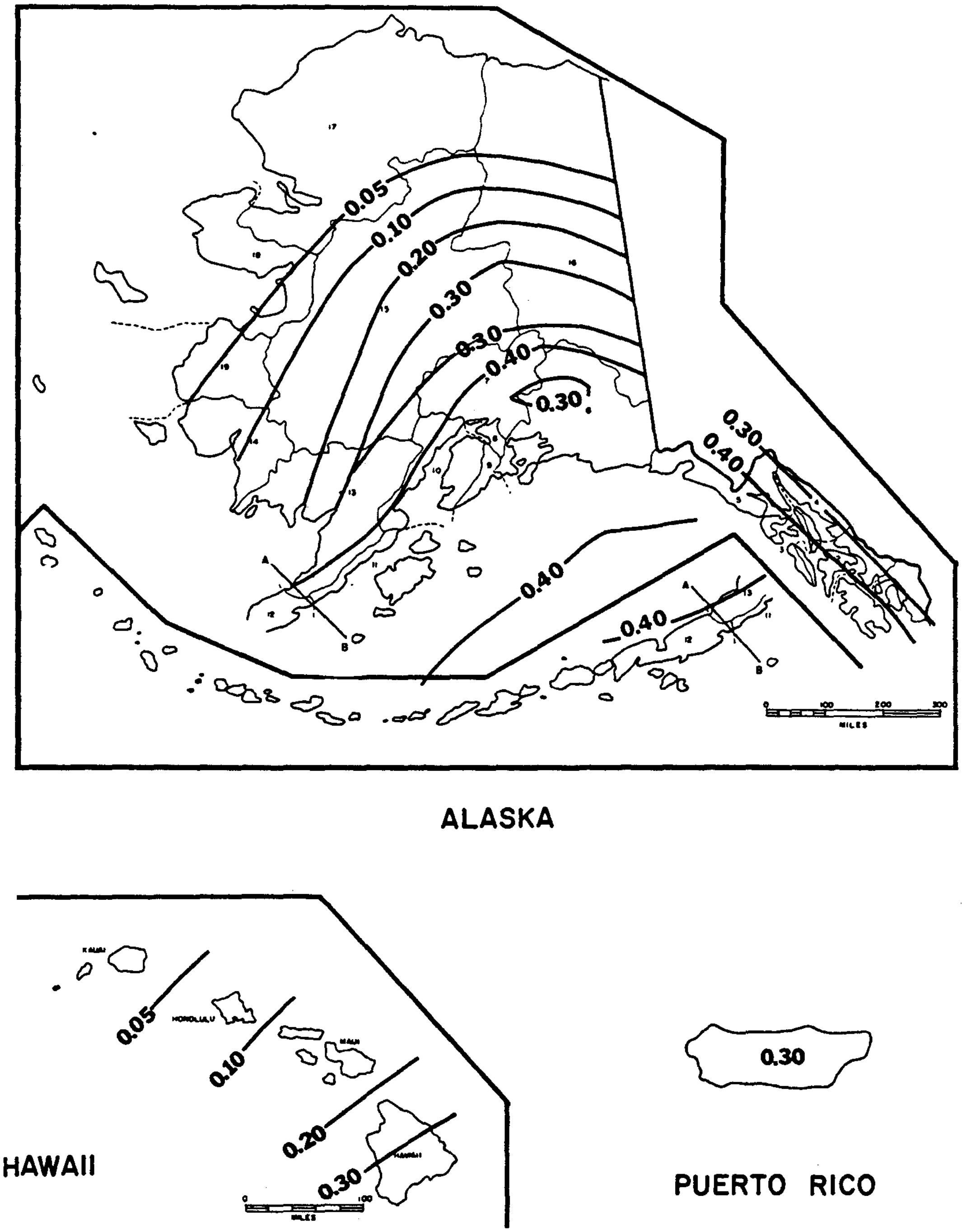


FIGURE 2. ACCELERATION COEFFICIENTS — ALASKA, HAWAII, and PUERTO RICO.

1.3 APPLICABILITY

These guidelines are intended for use on highway bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 ft. (152.4 m). Suspension bridges, cable-stayed bridges, arch-type, and movable bridges are not covered. However, many of the concepts presented here can be applied to these types of structures if appropriate judgment is used. Although specifically developed for highway bridges, these guidelines may also have some applicability to other types of bridges. These guidelines are recommended for all applicable bridge structures classified as Seismic Performance Category (SPC) B or greater. **Bridges in SPC A generally do not have to be considered for seismic retrofitting.** Minimum requirements for evaluation and upgrading will vary based on the Seismic Performance Category of the bridge.

Preliminary screening is optional for bridges classified in SPC B. However, seismic retrofitting of bridges in this Seismic Performance Category should definitely be considered for major structures and bridges undergoing non-seismic rehabilitation. These guidelines require that only the bearings, joint restrainers, and support width be included when screening, evaluating, or retrofitting bridges in Seismic Performance Category B.

A comprehensive program of retrofitting should be established for all bridges classified in Seismic Performance Categories C and D. This will require preliminary screening to identify the most critical bridges. Screening, evaluation, and retrofitting will include all major components subject to failure during a strong earthquake. The effects of soil failures such as liquefaction are also included.

1.4 THE RETROFITTING PROCESS

The seismic retrofitting process can be divided into three major steps. These are:

- Preliminary screening
- Detailed evaluation
- Design of retrofit measures

Preliminary screening of seismically deficient bridges is necessary to identify bridges which are potentially in the greatest need of retrofitting. This is partially important when a comprehensive retrofitting program is to be implemented. Certain elements of the screening procedure may also be used to quickly determine if seismic deficiencies exist in individual bridges.

The detailed seismic evaluation for retrofitting begins with a quantitative evaluation of individual bridge components and failure modes. The results from an elastic spectral analysis are used unless a simplified approach is warranted by the bridge location and configuration. The analysis is performed using the design earthquake loading. The subsequent force and displacement results, known as "demands," are compared with the "capacities" of each of the components to resist forces and displacements. In the case of reinforced concrete columns, ultimate capacities are modified to reflect the ability of the column to resist post-elastic deformations. Capacity/demand (C/D) ratios are calculated for each potential mode of failure in the critical components. These ratios are intended to represent the decimal fraction of the design earthquake at which a local failure of the components is likely to occur.

Therefore, a C/D ratio less than one indicates that component failure may occur during the design earthquake and retrofitting may be appropriate.

An overall assessment of the consequences of local component failure will be necessary to determine the need for retrofitting. Retrofitting should be considered when an assessment indicates that local component failure will result in unacceptable overall performance. The effect of potential retrofitting may be assessed by performing a detailed re-evaluation of the retrofitted bridge.

These guidelines provide retrofit measures for the types of bridge components which have performed poorly during past earthquakes. Retrofitting by these or other equivalent methods should be considered when components are identified by the detailed evaluation as being deficient. The decision to use a retrofitting scheme will be based on an assessment of its effectiveness in preventing unacceptable overall performance, the cost of retrofitting, and the remaining service life of the bridge.

Detailed design of retrofit measures should be performed using these guidelines in conjunction with the Seismic Design Guidelines. If possible, components which are selected for retrofitting should generally be strengthened to conform to the Seismic Design Guidelines for new construction, even though the structure may otherwise be seismically deficient.

A flow chart of the retrofit process as it applies to bridges in different seismic performance categories is shown in Figure 3. Chapters 2 through 5 contain detailed information about each of the major steps in this process. Chapter 2 covers the preliminary screening of bridges. Chapter 3 describes the detailed evaluation procedures. These procedures include a quantitative evaluation of the capacity/demand ratio for individual bridge components, which is described in Chapter 4. The procedure for evaluating bridges for retrofitting also includes the identification and assessment of retrofit measures. Several potential retrofitting concepts and retrofit design requirements are discussed in Chapter 5. The symbols used in this report are defined in the Glossary. The worked example problem included in Appendix A will help illustrate the use of the guidelines in planning the retrofitting of a typical highway bridge.

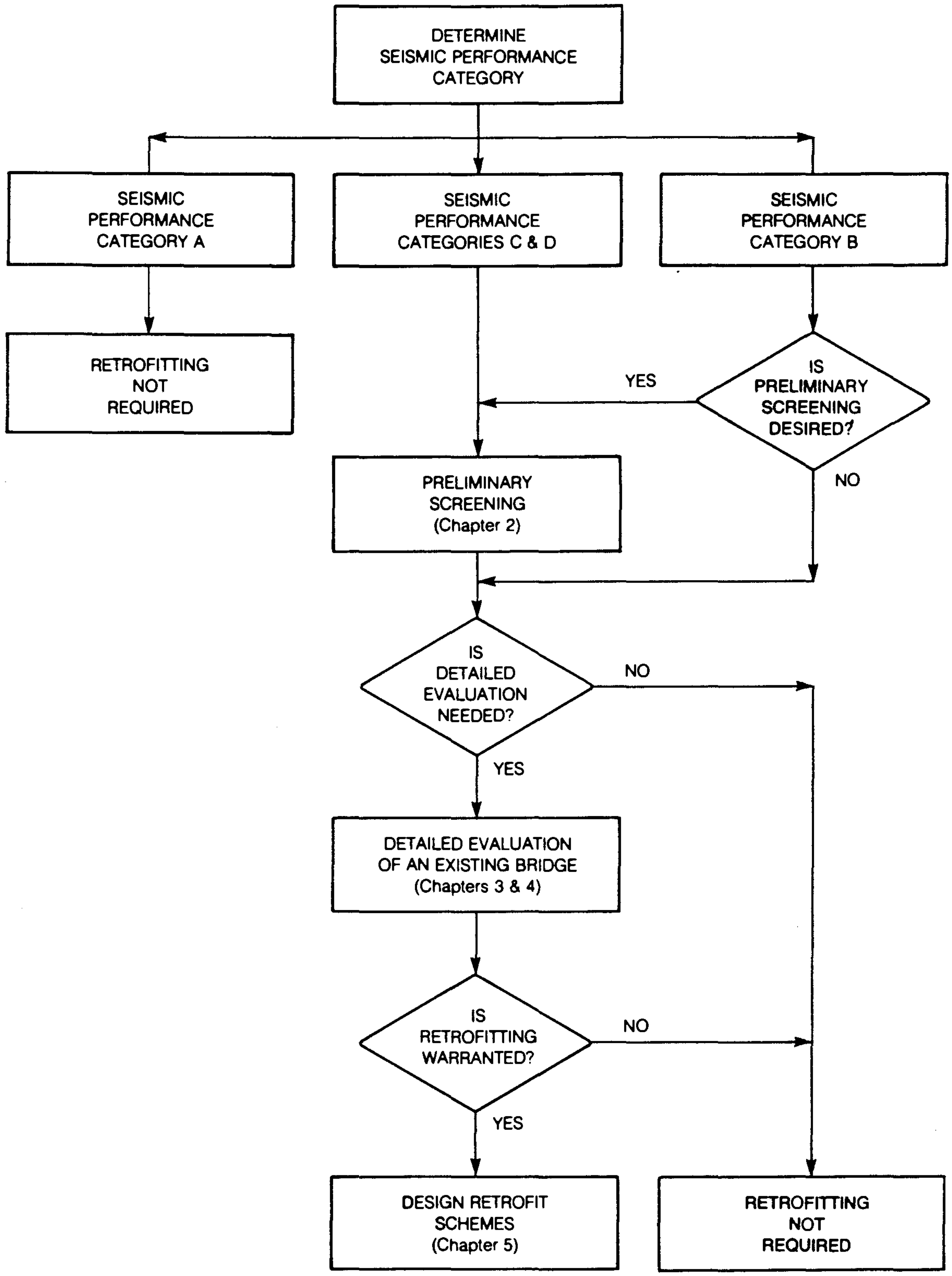


FIGURE 3. SEISMIC RETROFITTING PROCESS

CHAPTER 2

PRELIMINARY SCREENING OF BRIDGES FOR DETAILED EVALUATION

2.1 GENERAL

An efficient and comprehensive retrofitting program requires that structures be rated according to their need for seismic retrofitting by a preliminary screening process. It is recommended that this be done for all bridges classified as Seismic Performance Category C and D. Establishing priorities for retrofitting is optional and greatly simplified for bridges in Seismic Performance Category B. The flow chart shown in Figure 4 illustrates the preliminary screening proceedure as it applies to bridges in different Seismic Performance Categories.

In general, the Seismic Rating System described in this chapter shall be used as a basis for selecting bridges for the more detailed quantitative evaluation described in Chapter 3. The Seismic Rating System considers only the technical aspects of the problem and does not include administrative, economic, or political considerations. In cases where these other considerations are important, the Seismic Rating System will provide useful information but will not necessarily dictate the order in which bridges should be selected for evaluation and possible retrofitting.

2.2 SEISMIC INVENTORY OF BRIDGES

The first step in implementing the Seismic Rating System is to inventory all applicable bridges with the objective of establishing the following basic information:

- Structural characteristics needed to determine the vulnerability rating described in Section 2.3.1.
- Seismicity of the bridge site.
- Importance of the structure as a vital transportation link.

This information may be obtained from bridge records such as the Federal Highway Administration bridge inventory, "as-built" plans and maintenance records, the regional disaster plan, on-site bridge inspection records, and other sources.

Bridges will be classified according to Seismic Performance Category (SPC) as described in Section 1.2.

2.3 SEISMIC RATING SYSTEM

To calculate the seismic rating of a bridge, consideration is given to structural vulnerability, seismicity of the bridge site, and the bridge's importance as a vital transportation link. This is accomplished by making independent ratings of the bridges in each of these three areas as described in Sections 2.3.1 to 2.3.3. Each of these three areas are assigned a rating, weight, and score. The scores are added to arrive at an overall seismic rating according to the following procedure:

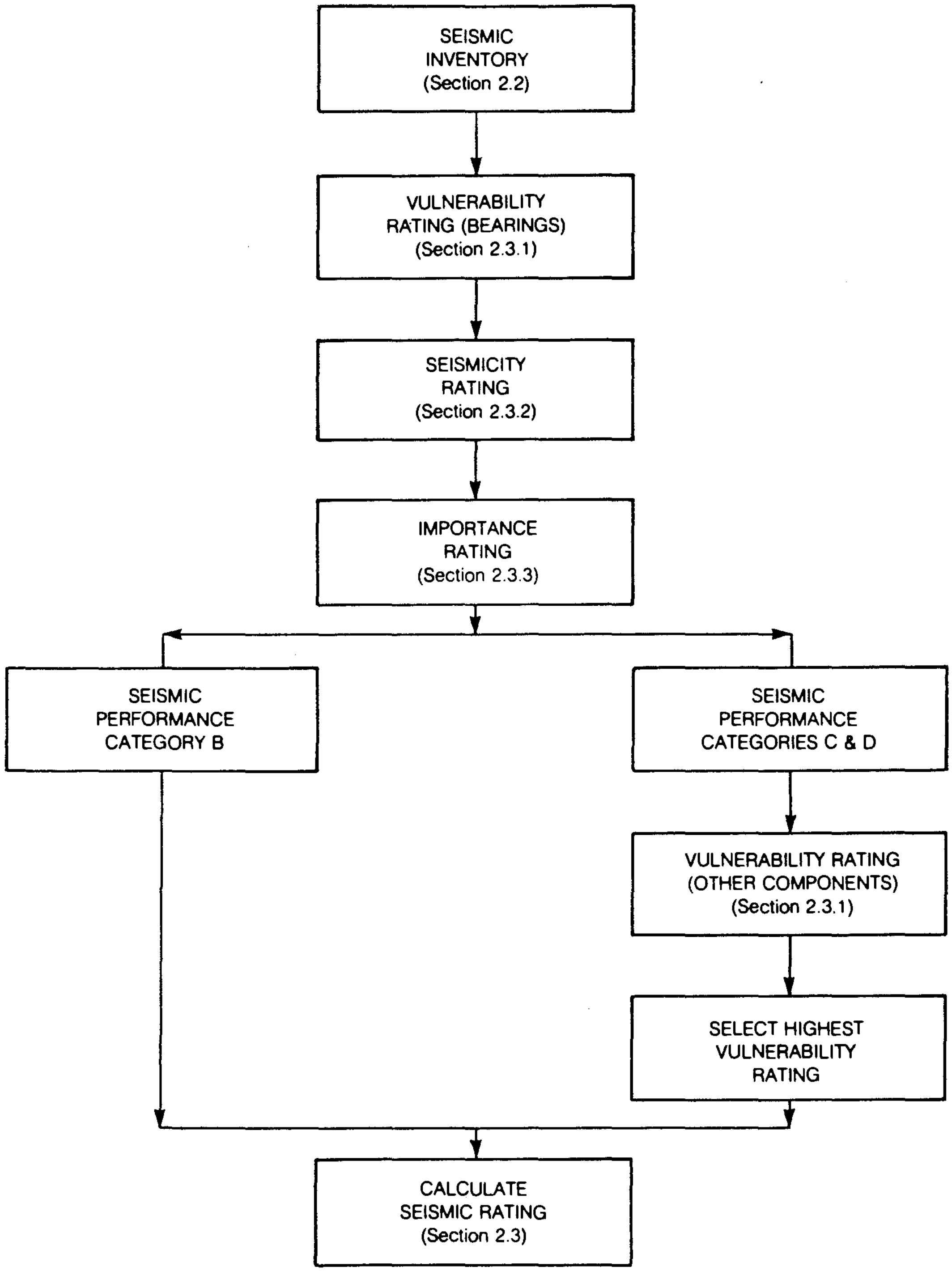


FIGURE 4. PRELIMINARY SCREENING PROCESS

Vulnerability Rating (rating 0 to 10) x weight	= score
Seismicity Rating (rating 0 to 10) x weight	= score
Importance Rating (rating 0 to 10) x weight	= score
Seismic Rating (100 maximum)	= Total Score

The higher the seismic rating score, the greater the need for the bridge to be evaluated for seismic retrofitting. It is recommended that each weight be taken as 3.33 unless different weights, which must total 10, are assigned by the engineer to reflect regional and jurisdictional needs. The rating, weight, and score in each of the individual areas, as well as the overall seismic rating score, should be recorded as part of the bridge seismic inventory.

It is obvious that the Seismic Rating System is very subjective. To enhance consistancy it is desirable to have the rating of all bridges in one geographical area performed by the same personnel. It is important that the current condition of the bridge be considered in determining these ratings. It is therefore recommended that maintenance personnel who are familiar with the current bridge condition participate in the rating process.

2.3.1 VULNERABILITY RATING

Vulnerability ratings may assume any value between 0 and 10. In general, a 0 rating means a very low vulnerability to unacceptable seismic damage, a 5 means a moderate vulnerability to collapse or a high vulnerability to loss of access, and a 10 means a high vulnerability to collapse. This should not be interpreted to mean that the vulnerability rating must assume one of these three values, however. It is useful to consider the seismic vulnerability of the bearings joint restrainers, and support lengths separate from the vulnerability of the remainder of the structure, which will include columns, piers, footings, abutments, and vulnerability resulting from ground liquefaction. Separate vulnerability ratings between 0 and 10 should be assigned to both of these areas. The overall vulnerability rating of the bridge shall be taken as the greater of the above two vulnerability ratings, although a record should be kept of both values.

For bridges classified as SPC B, only the vulnerability ratings for bearings, joint restrainers, and support lengths needs to be calculated. Determination of vulnerability ratings requires considerable judgment. A suggested methodology for determining vulnerability ratings is covered in the Commentary.

The vulnerability rating for bearings will reflect the susceptibility of the bridge to a bearing failure. Past experience and retrofitting methods are available to show that most bearing deficiencies can be economically corrected by seismic retrofitting. In general, retrofitting will be feasible for correcting a high vulnerability to bearing or expansion joint failure.

The vulnerability rating for the remainder of the structure will consider weaknesses in components such as columns, abutments, and foundations. Past experience and retrofitting methods for these components are much more limited than for bearings. They are generally more difficult and less economical to retrofit than are bearings.

A comparison of the above two vulnerability ratings can be used to obtain an indication of the type of retrofitting needed. If the vulnerability rating for the bearings is equal to or less than the vulnerability rating of other components, simple retrofitting of only the bearings may be of little value. Conversely if the bearing rating is greater, then benefits may be obtained by retrofitting only the bearings. A comparison of these two ratings during the preliminary screening process may be helpful in planning the type of comprehensive retrofit program needed, but should not serve as a substitute for the detailed evaluation of individual bridges as described in Chapter 3.

2.3.2 SEISMICITY RATING

The seismicity rating shall be taken as 25 times A, where A is the acceleration coefficient taken from the maps in Figures 1 and 2. The maximum seismicity rating is 10.

2.3.3 IMPORTANCE RATING

The importance rating will be based on the Importance Classification, IC, of the bridge which is determined from Social/Survival and Security/Defense requirements as discussed in the Commentary and the Seismic Design Guidelines. The relative importance of bridges within each importance classification are assigned by considering utilities, available detours, and involvement of other lifelines. The importance rating may vary from 0 to 10, depending on the relative importance of the structure within each of the Importance Classifications as shown in Table 2.

TABLE 2: IMPORTANCE RATING

Importance Classification (IC)	Importance Rating
I	6-10 points
II	0-5 points

CHAPTER 3

PROCEDURE FOR DETAILED EVALUATION OF AN EXISTING BRIDGE

3.1 GENERAL

Bridges selected for detailed evaluation shall use the procedures of this chapter. Evaluation for retrofitting will include the determination of the seismic capacity/demand ratios for individual bridge components. A more detailed treatment of the methodology for determining these ratios is given in Chapter 4. An overall assessment of capacity/demand ratios and identification and assessment of retrofit measures are also part of the evaluation process. Some suggested retrofitting concepts are listed in Chapter 5. The requirements for evaluating a bridge for retrofitting will vary depending on the location, configuration, or type of the bridge. A flow chart detailing this procedure is shown in Figure 5.

3.2 REVIEW OF BRIDGE RECORDS

A detailed evaluation requires the determination of the in-situ condition of the bridge. Initially this involves a thorough review of the "as-built" plans and the construction and maintenance records if these are available. A review of the original design calculations and the specifications to which the bridge was designed should also be performed if possible. Information that will have an effect on the seismic response of the bridge and the capacity of the individual components should be obtained from these documents. Sufficient structural details can usually be obtained from the as-built plans. Information on material strengths and foundation conditions may in some cases be obtained from construction records. Maintenance records will often contain information about the actual condition of the structural materials or components. In addition, certain structural modifications may have been made which are not shown on the plans, but which may be noted in the maintenance records. When information about the in-situ properties of the materials is not available, the American Association of State Highway and Transportation Officials (AASHTO) Manual for Maintenance Inspection of Bridges may be used as a guide for determining material properties.

3.3 SITE INSPECTION

A field inspection of bridges selected for detailed evaluation should be made to verify the information obtained from a review of the bridge records and to talk to the bridge maintenance and inspection personnel. The items which should be noted in the field inspection are as follows:

- Unusual lateral movement under traffic loading.
- Unusual gap or offset at expansion joints.
- Damaged or malfunctioning bearings.
- Damage or deterioration to the main and secondary structural members.
- Extra dead load, such as wearing surface, utilities, sidewalks, etc., not shown on plans.

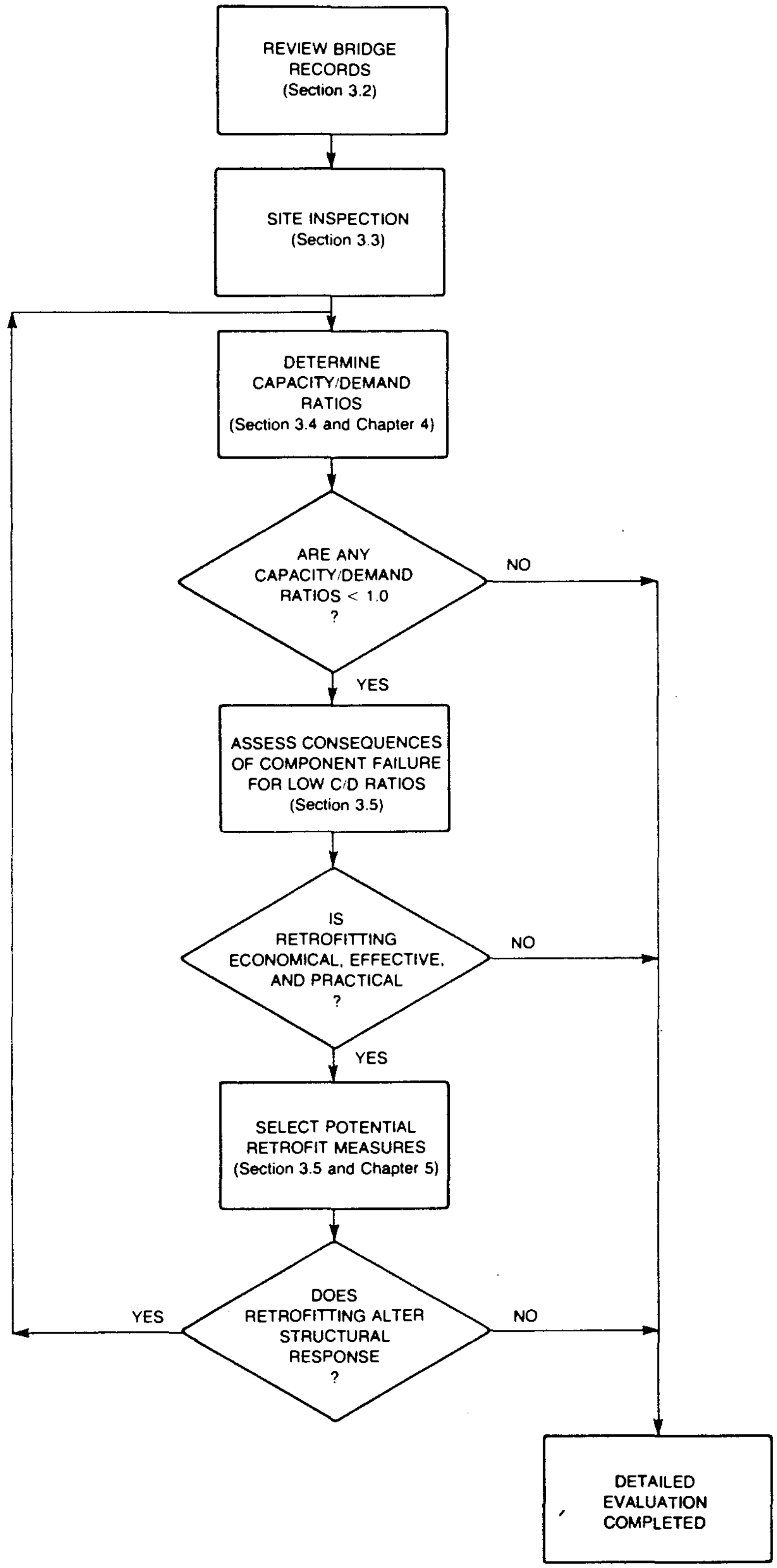


FIGURE 5. PROCEDURE FOR EVALUATING A BRIDGE FOR SEISMIC RETROFITTING

- Unusual erosion of soil at or near the foundation.

- Nonstructural items not shown on plans, such as continuous barrier rail, that could affect the lateral stiffness of the structure or its performance under seismic loading.

- Horizontal or vertical movement or tilting of the abutments, columns, or piers.

- Any deviations from the plans and specifications.

3.4 QUANTITATIVE EVALUATION OF BRIDGE COMPONENTS

Bridge components that have the potential of being damaged during a strong earthquake should be evaluated quantitatively to determine their ability to resist the design earthquake. This should be done by calculating the seismic capacity/demand ratio for each of the potential modes of failure for these components. The type of components subject to unacceptable failure during an earthquake will vary with the seismic performance category of the bridge. Table 3 indicates the components and failure modes that should be checked. For certain bridge configurations it is obvious that some component failures will not result in unacceptable damage. Capacity/demand ratios for these components need not be calculated. For certain other bridge configurations, components other than those listed should be calculated if their failure will result in unacceptable overall performance of the bridge.

TABLE 3: COMPONENTS FOR WHICH SEISMIC CAPACITY/DEMAND RATIOS MUST BE CALCULATED

Seismic Performance Category	B	C	C	D
Acceleration Coefficient	0.19>A>0.09	0.29>A>0.19	>0.29	>0.29
Component				
EXPANSION JOINTS AND BEARINGS				
Support Length	X	X	X	X
Forces	X	X	X	X
REINFORCED CONCRETE COLUMNS Piers, and Footings				
Anchorage		X	X	X
Splices		X	X	X
Shear		X	X	X
Confinement		X	X	X
Footing Rotation			X	X
ABUTMENTS				
Displacement			X	X
LIQUEFACTION		X	X	X

In general, seismic demands will be determined from an elastic spectral analysis performed using the design earthquake. Minimum bearing force and support length requirements are also specified. In certain cases the capacity/demand ratios will be calculated using these minimum requirements as demands.

Seismic capacities are calculated at their nominal ultimate values without capacity reduction (ϕ) factors. In cases such as reinforced concrete columns where post-elastic behavior is acceptable, capacity/demand ratios are modified by ductility indicators to accurately reflect the capacity of the column to withstand yielding. Chapter 4 describes the methodology for determining component capacity/demand ratios.

3.5 OVERALL ASSESSMENT OF THE EXISTING BRIDGE AND IDENTIFICATION AND ASSESSMENT OF POTENTIAL RETROFIT MEASURES

The capacity/demand ratios calculated according to the procedures outlined in Chapter 4 indicate the reduced load levels at which individual components may fail. The capacity/demand ratios for the as-built condition of a bridge should be tabulated as shown in Table 4. Values greater than one indicate that the corresponding component is not likely to fail during the design earthquake, whereas values less than one indicate a possible failure.

Beginning with the lowest capacity/demand ratio, each value less than one should be investigated to assess the consequences of local component failure on the overall performance of the bridge, to identify retrofit measures and to determine the effectiveness of retrofitting. Component failure is always considered unacceptable if it results in the collapse of the structure. If component failure results in a loss of access or loss of function, this may also be unacceptable if the bridge serves a vital transportation route. If component failure does not result in unacceptable consequences, then retrofitting is usually not justified for the component in question.

If the consequences of component failure are unacceptable, then the effectiveness of retrofitting the component should be evaluated. When retrofitting will affect the response of the remainder of the structure, new capacity/demand ratios should be calculated and tabulated as shown in Table 4. If an improvement in overall bridge performance will result from the component retrofit and this can be accomplished at a reasonable cost, then the bridge should be retrofitted. Each component with a capacity/demand ratio less than one should be investigated in this way.

TABLE 4: COMPARISON OF COMPONENT SEISMIC CAPACITY/DEMAND RATIOS

Component	As-Built Bridge	Retrofit Scheme 1	Retrofit Scheme 2
EXPANSION JOINTS AND BEARINGS			
Displacement - r_{bd}	____	____	____
Force - r_{bf}	____	____	____
REINFORCED CONCRETE, PIERS, AND FOOTINGS			
Anchorage of Longitudinal Reinforcement - r_{ca}	____	____	____
Splices in Longitudinal Reinforcement - r_{cs}	____	____	____
Confinement Reinforcement - r_{cc}	____	____	____
Column Shear - r_{cv}	____	____	____
Footings - r_{fr}	____	____	____
ABUTMENTS - r_{ad}	____	____	____
LIQUEFACTION - r_{sl}	____	____	____

The decisions regarding the need for retrofitting will require considerable engineering judgment. For more discussion about considerations in assessing each type of component failure, refer to Section C3.5 of the Commentary. Evaluation of the effectiveness and economics of retrofitting is also discussed in Section C3.5 of the Commentary.

CHAPTER 4

DETERMINATION OF SEISMIC CAPACITY/DEMAND RATIOS FOR BRIDGE COMPONENTS

4.1 GENERAL

Seismic capacity/demand (C/D) ratios give a reasonable indication of the decimal fraction of the design earthquake that is likely to cause serious damage to a particular bridge component. It should be pointed out, however, that these C/D ratios do not necessarily reflect the capacities or demands that would be used for design. The components for which C/D ratios need to be calculated vary with the Seismic Performance Category of the bridge as specified in Table 3, Section 3.4. Seismic demands will be taken from an elastic spectral analysis as described in Sections 4.2 through 4.4, or as minimum values as described in Sections 4.5 and 4.6. Capacities are taken as the nominal strength and/or displacement capacities of the components without modification by capacity reduction (ϕ) factors.

In concrete columns and certain types of footings where significant flexural yielding may occur before serious damage results, C/D ratios calculated using elastic moment demands are multiplied by ductility indicators (μ) to account for yielding. This can be done because it is assumed that inelastic and elastic displacements are of similar magnitude for a given earthquake loading. Therefore, the actual moment demands are the elastic moment demands divided by the ductility indicator. The effect of this is to increase the elastic C/D ratio by a factor equal to the ductility indicator.

Sections 4.7 through 4.10 describe procedures for calculating seismic C/D ratios for various bridge components.

4.2 ANALYSIS PROCEDURE

The recommended minimum spectral analysis procedure for a given type of bridge is presented in Table 5 and is dependent upon the number of spans, the geometric complexity, and the Seismic Performance Category. A more rigorous and generally accepted procedure may be used in lieu of the recommended minimum. A detailed seismic analysis is not required for regular bridges in Seismic Performance Category B. In this case only the bearings are considered critical, and the force and displacement demands may be taken as the minimums prescribed in Sections 4.5 and 4.6, respectively.

TABLE 5: ANALYSIS PROCEDURE

Seismic Performance Category	Regular* Bridges with Two or More Spans	Irregular** Bridges with Two or More Spans
B	Analysis not Required	Procedure 1
C	Procedure 1	Procedure 2
D	Procedure 1	Procedure 2

* A "regular" bridge has no abrupt or unusual changes in mass, stiffness, or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). For example, a bridge may

be considered regular if it is straight or describes a sector of an arc not exceeding 90° and has adjacent columns or piers that do not differ in stiffness by more than 25%. (Percentage difference is to be based on the lesser of two adjacent quantities as the reference.)

** An "irregular" bridge is any bridge that does not satisfy the definition of a regular bridge.

The analysis procedures designated in Table 5 are based on elastic analysis of the structure using the following methods:

Procedure 1: Single-Mode Spectral Method.
Procedure 2: Multimode Spectral Method.

Details of these procedures are given in Chapter 5 of the Seismic Design Guidelines. Some notes on the adaptation of these procedures to bridge evaluation are presented in the Commentary to these retrofit guidelines.

4.3 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS

The elastic forces and displacements should be determined independently due to loading along two perpendicular axes by use of the analysis procedure specified in Section 4.2. The foundation stiffnesses at the abutments and piers should also be considered in the analysis if they contribute significantly to the overall stiffness of the bridge. Elastic forces and displacements due to loading along each perpendicular axis should then be combined as specified in Section 4.4 to account for directional uncertainty of the earthquake motion. The perpendicular axes are typically the longitudinal and transverse axes of the bridge, but the choice is open to the engineer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments.

4.4 COMBINATION OF ORTHOGONAL ELASTIC SEISMIC FORCES

A combination of seismic forces and displacements resulting from orthogonal loading is used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. The elastic seismic forces, moments, and displacements resulting from the analyses of loading in the two perpendicular directions described in Section 4.3 should be combined to form two load cases as follows:

Load Case 1: Seismic demand forces and moments on each of the principal axes of a member and seismic demand displacements in each of the perpendicular directions should be obtained by adding 100% of the absolute value of the results from the analysis of loading in the first perpendicular (longitudinal) direction to 30% of the absolute value of the corresponding results from the analysis of loading in the second perpendicular (transverse) direction.

Load Case 2: Seismic demand forces and moments on each of the principal axes of a member and seismic demand displacements in each of the perpendicular directions should be obtained by adding 100% of the absolute value of the results from the analysis of loading in the second perpendicular direction (tranverse) to 30% of the absolute value of the corresponding results from the analysis of loading in the first perpendicular direction (longitudinal).

4.5 MINIMUM BEARING OR RESTRAINER FORCE DEMANDS

When determining the minimum bearing or restrainer force demands for the evaluation of an existing bridge, a minimum equivalent horizontal force of 0.20 times the deadload of the superstructure should be assumed. The minimum bearing or restrainer force demand will be the portion of the minimum equivalent horizontal force that must be resisted by the bearings or restrainers that are not specifically designed as force limiting devices.

4.6 MINIMUM SUPPORT LENGTHS

Minimum support lengths, N(d), for bearing seats supporting the unrestrained expansion ends of girders, as shown in Figure 6, are used to calculate bearing displacement C/D ratios, r_{bd}, by Method 1, as described in Section 4.7. These support lengths shall be measured normal to the face of abutment, pier, or mid-span joint. The values for minimum support length will vary with the Seismic Performance Category of the bridge as given by the following formulas:

Seismic Performance Category B:

$$N(d) = 8 + 0.02L + 0.08H \quad , \quad \text{(inches)} \qquad (4\text{-}1A)$$

or

$$N(d) = 203 + 1.67L + 6.66H \ . \quad \text{(mm)} \qquad (4\text{-}1B)$$

Seismic Performance Categories C and D:

$$N(d) = 12 + 0.03L + 0.12H \quad , \quad \text{(inches)} \qquad (4\text{-}2A)$$

or

$$N(d) = 305 + 2.5L + 10H \quad . \quad \text{(mm)} \qquad (4\text{-}2B)$$

where

L = Length of the bridge deck to the adjacent expansion joint or to the end of the bridge deck. For mid-span joints, L is 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. These lengths are shown in Figure 6 (feet: Eq. 4-1A and 4-2A; meters: Eq. 4-1B and 4-2B).

For abutments:

H = Average height of columns supporting the bridge deck to the next expansion joint. H = 0 for single-span bridges (feet: Eq. 4-1A and 4-2A; meters: Eq. 4-1B and 4-2B).

For columns and/or piers:

H = Average height of adjacent two columns or piers (feet: Eq. 4-1A and 4-2A; meters: Eq. 4-1B and 4-2B).

For mid-span joints:

H = Average height of adjacent two columns or piers (feet: Eq. 4-1A and 4-2A; meters: Eq. 4-1B and 4-2B).

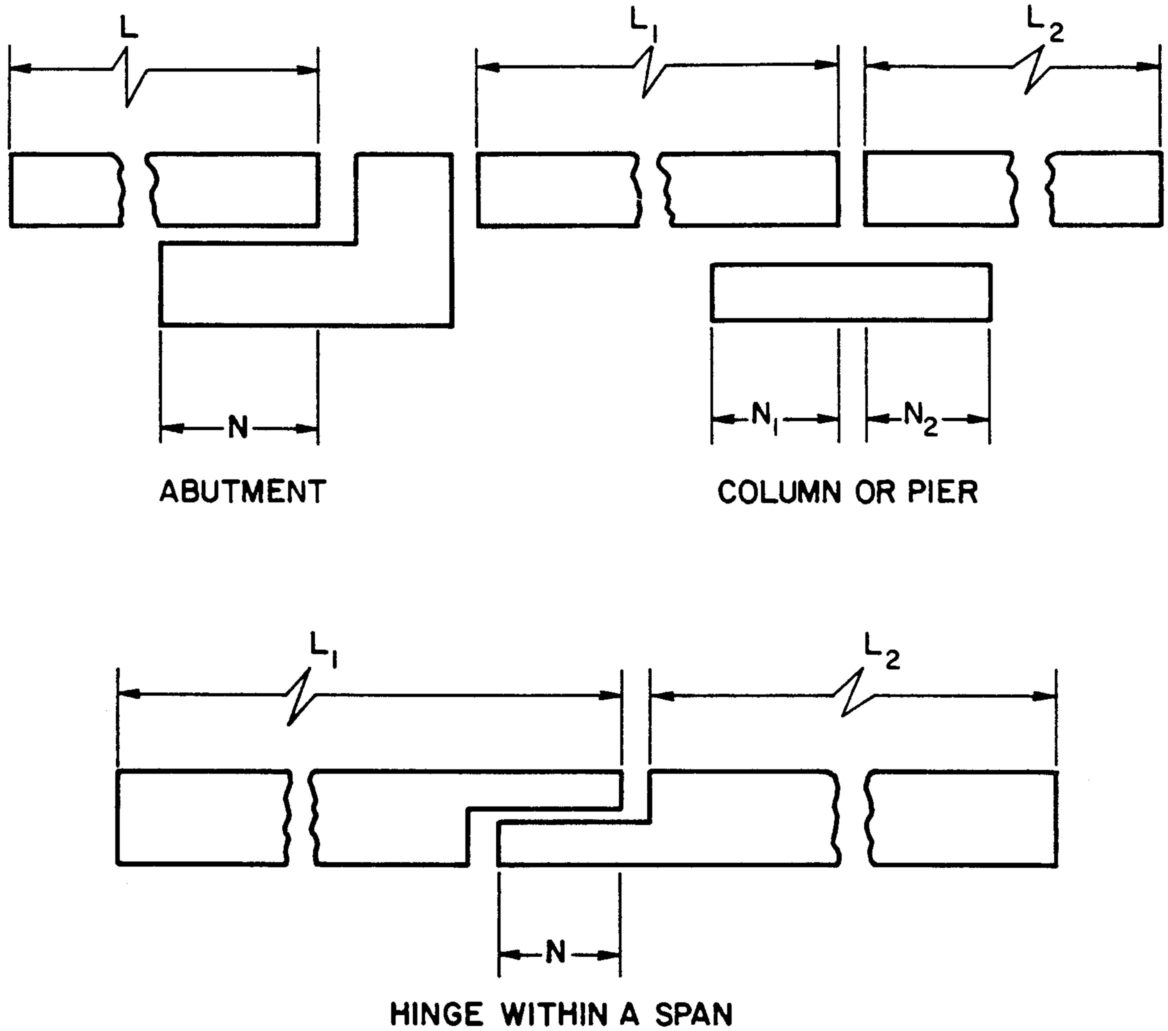


FIGURE 6. DIMENSIONS FOR MINIMUM SUPPORT LENGTH REQUIREMENTS

4.7 CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS AND BEARINGS

The displacement and force C/D ratios, r_{bd} and r_{bf}, for expansion joints and bearings are evaluated for the most critical horizontal direction(s) by the procedures outlined in this section.

4.7.1 DISPLACEMENT CAPACITY/DEMAND RATIO

The displacement C/D ratios should be calculated for restrained and unrestrained expansion joints and for bearings at which movement can occur due to the absence of fixity in a horizontal direction. The displacement C/D ratio is the lesser of the values calculated using the following two methods, except in the case where displacement-limiting devices such as restrainers are provided, in which case only Method 2 needs to be used.

Method 1:

$$r_{bd} = \frac{N(c)}{N(d)}, \tag{4-3}$$

where

N(c) = The support length provided. This length is measured normal to the expansion joint.

N(d) = The minimum support length defined in Section 4.6.

Method 2:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)}, \tag{4-4}$$

where

$\Delta_s(c)$ = The allowable movement of the expansion joint or bearing. For structures in SPC-D, unreinforced cover concrete should not be included in determining the allowable movement.

$\Delta_i(d)$ = The maximum possible movement resulting from temperature, shrinkage, and creep shortening. If field measurements have been made of a bridge in existence for some time, only the temperature effects need to be considered.

$\Delta_{eq}(d)$ = The maximum relative displacement due to earthquake loading for the load cases described in Section 4.4.

4.7.2 FORCE CAPACITY/DEMAND RATIO

The force C/D ratio for bearings and expansion joint restrainers are evaluated as follows:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} \qquad (4\text{-}5)$$

where

$V_b(c)$ = Nominal ultimate capacity of the component in the direction under consideration.

$V_b(d)$ = Seismic force acting on the component. This force is the elastic force determined from an analysis in accordance with Section 4.4 multiplied by 1.25. The minimum bearing force demand as specified in Section 4.5 is used when an analysis is not performed, or when it exceeds the force demand obtained from an analysis.

4.8 CAPACITY/DEMAND RATIOS FOR REINFORCED CONCRETE COLUMNS, PIERS, AND FOOTINGS

It is not uncommon for reinforced concrete columns, piers, and/or footings to yield and form plastic hinges during a strong earthquake. The interaction between these components will determine the probable mode of failure. To evaluate columns, piers, and footings, it is first necessary to determine the location of potential plastic hinges. Plastic hinges may form in the column end regions or within the footing. An effect similar to a plastic hinge may also develop due to yielding of the soil or pilings. Piers, which are defined as supports having a height-to-width ratio of 2.5 or less in the strong direction, may develop plastic hinges in the end regions about the strong as well as the weak axis.

Once potential plastic hinges have been located, it is necessary to investigate the potential modes of column and/or footing failure associated with the location and type of plastic hinging. A ductility indicator is used to account for the ability of the columns and/or footings to resist certain modes of failure controlled by the amount of yielding. The ultimate moment capacity/elastic moment demand ratios are multiplied by ductility indicators to enable elastic analysis results to be used for determining the seismic C/D ratios of components subject to yielding.

The following procedure should be used to determine the C/D ratio for columns, piers, and footings as illustrated in the flow chart in Figure 7. This procedure includes a systematic method for locating plastic hinges and evaluating the capacity of the columns and/or footings to withstand this plastic hinging. Sections 4.8.1 through 4.8.5 describe detailed procedures for investigating different column and/or footing failure modes sometimes associated with plastic hinging.

<u>Step 1:</u> Determine the elastic moment demands at both ends of the column or pier for the seismic load cases described in Section 4.4. Moment demands for both the columns and footings should be determined. The elastic moment demand shall be taken as the sum of the absolute values of the earthquake and deadload moments.

<u>Step 2:</u> Calculate nominal ultimate moment capacities for both the column and the footing at axial loads equal to the deadload plus, or minus, the seismic axial load resulting from plastic hinging in the columns, piers, or footings as discussed in Section 4.8.2 of the Seismic Design Guidelines.

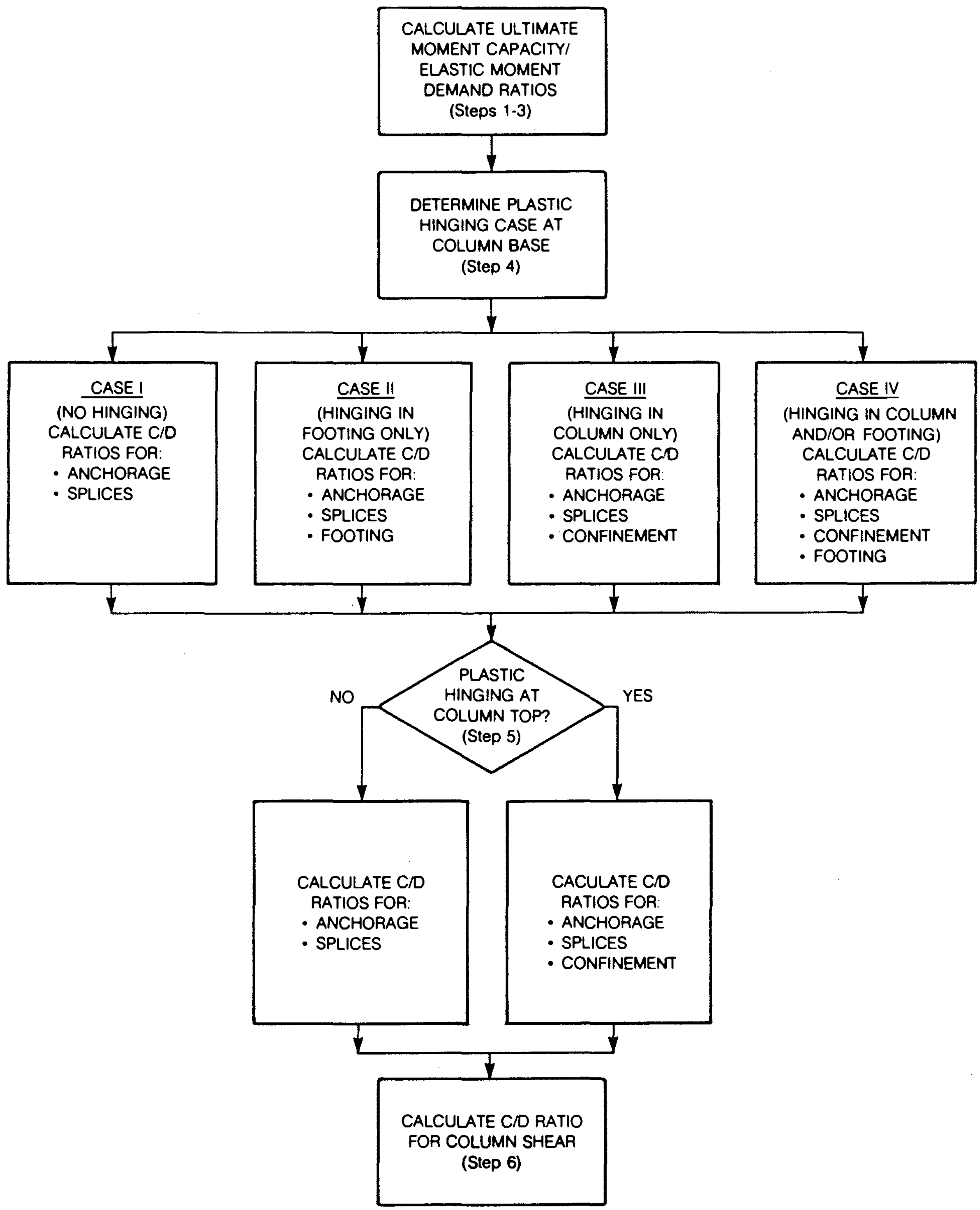


FIGURE 7. PROCEDURES FOR DETERMINING CAPACITY/DEMAND RATIOS FOR COLUMNS, PIERS, AND FOOTINGS

Step 3: Calculate the set of moment C/D ratios (nominal ultimate moment capacity and elastic moment demand), r_{ec} and r_{ef}, for each combination of capacity and demand, assuming first that the column will yield and the footing will remain elastic, and second that the footing will yield and the column will remain elastic.

Step 4: Calculate the C/D ratios for the anchorage of longitudinal reinforcement, splices in the longitudinal reinforcement, and/or transverse confinement reinforcement at the base of the column, and/or footing rotation or yielding for the most severe possible cases of plastic hinging as indicated by each set of r_{ec} and r_{ef}. The following cases describe the C/D ratios that should be investigated based on the location and extent of plastic hinging.

Case I: When both r_{ec} and r_{ef} exceed 0.8, it may be assumed that neither the footing nor the column will yield sufficiently to require an evaluation of their ability to withstand plastic hinging. In this case only the column C/D ratios for anchorage of longitudinal reinforcement (Section 4.8.1) and splices in longitudinal reinforcement (Section 4.8.2) should be calculated.

Case II: When r_{ef} is less than 0.8 and r_{ec} either exceeds 0.8 or exceeds r_{ef} by 25%, then the footing will require an evaluation of its ability to rotate and/or yield unless an anchorage or splice failure will occur and prevent footing rotation. Anchorage or splice failures may be assumed when either the C/D ratio for anchorage of longitudinal reinforcement (Section 4.8.1) or for splices in longitudinal reinforcement (Section 4.8.2) is less than 80% of r_{ef}. When this is not the case, only the C/D ratio for rotation and/or yielding of the footing should be calculated.

Case III: When r_{ec} is less than 0.8 and r_{ef} either exceeds 0.8 or exceeds r_{ec} by 25%, it may be assumed that only the column will yield sufficiently to require an evaluation of its ability to withstand plastic hinging. In this case the column C/D ratios should be calculated for anchorage of longitudinal reinforcement (Section 4.8.1), splices in longitudinal reinforcement (Section 4.8.2), and column transverse confinement (Section 4.8.4).

Case IV: When r_{ec} and r_{ef} are less than 0.8 and within 25% of one another, it may be assumed that both the column and footing have the potential to yield sufficiently to require further evaluation. Since yielding of the footing will be prevented by a column failure prior to column yield, column C/D ratios for anchorage of longitudinal reinforcement (Section 4.8.1), splices of longitudinal reinforcement (Section 4.8.2), and column transverse confinement (Section 4.8.4) should be calculated first. When all of these C/D ratios exceed 80% or r_{ef}, then the C/D ratio for rotation and/or yielding of the footing (Section 4.8.5) should also be calculated.

Step 5: Calculate the column C/D ratios for anchorage of longitudinal reinforcement (Section 4.8.1) and splices in longitudinal reinforcement (Section 4.8.2) at the top of the column. If the moment C/D ratio, r_{ec}, of the column is less than 0.8, the C/D ratio for column transverse confinement (Section 4.8.4) should also be calculated.

Step 6: Calculate the column C/D ratios for column shear (Section 4.8.3).

Seismic C/D ratios for anchorage of longitudinal reinforcement (r_{ca}), longitudinal reinforcement splice lengths (r_{cs}), column shear capacity (r_{cv}), column confinement reinforcement (r_{cc}), and rotation and/or yielding of the footing (r_{fr}) are dependent on the amount of flexural yielding in the column or footing. In columns with poorly detailed transverse reinforcement, one of the most critical consequences of flexural yielding is the spalling of cover concrete. Such spalling is followed by a rapid degradation in the effectiveness of the transverse steel which can lead to column failure. The procedure for calculating C/D ratios for column confinement reinforcement is based on the assumption that spalling will begin at a ductility indicator of 2. The effectiveness of poorly detailed transverse reinforcement is assumed to begin to degrade at the onset of spalling. This type of transverse reinforcement is considered totally ineffective beyond a ductility indicator of 5. Figure 8 shows the relationship between the ductility indicator and the effectiveness factor, k_3, for poorly detailed transverse reinforcement. The effectiveness factor gives the decimal fraction of the transverse steel reinforcing that can be considered effective.

4.8.1 ANCHORAGE OF LONGITUDINAL REINFORCEMENT

A sudden loss of flexural strength can occur if longitudinal reinforcement is not adequately anchored. The following terms are used to calculate the C/D ratio for anchorage of longitudinal reinforcement r_{ca}:

$\ell_a(c)$ = Effective anchorage length of longitudinal reinforcement as shown in Figure 9.

$\ell_a(d)$ = Required effective anchorage length of longitudinal reinforcement.

For straight anchorage the effective anchorage length in inches is given by

$$\ell_a(d) = \frac{k_s\, d_b}{\sqrt{f'_c}(1 + 2.5c/d_b + k_{tr})} \geq 30\, d_b \tag{4-6}$$

where

k_s = A constant for reinforcing steel with a yield stress of f_y (psi)
= $\dfrac{(f_y - 11000)}{4.8}$

d_b = Nominal bar diameter in inches.

f'_c = Concrete compression strength (psi).

c = The lesser of the clear cover over the bar or bars, or half the clear spacing between adjacent bars.

and

$$k_{tr} = A_{tr}(c)f_{yt}/600\, sd_b \leq 2.5\ , \tag{4-7}$$

where variables used to calculate k_{tr} are

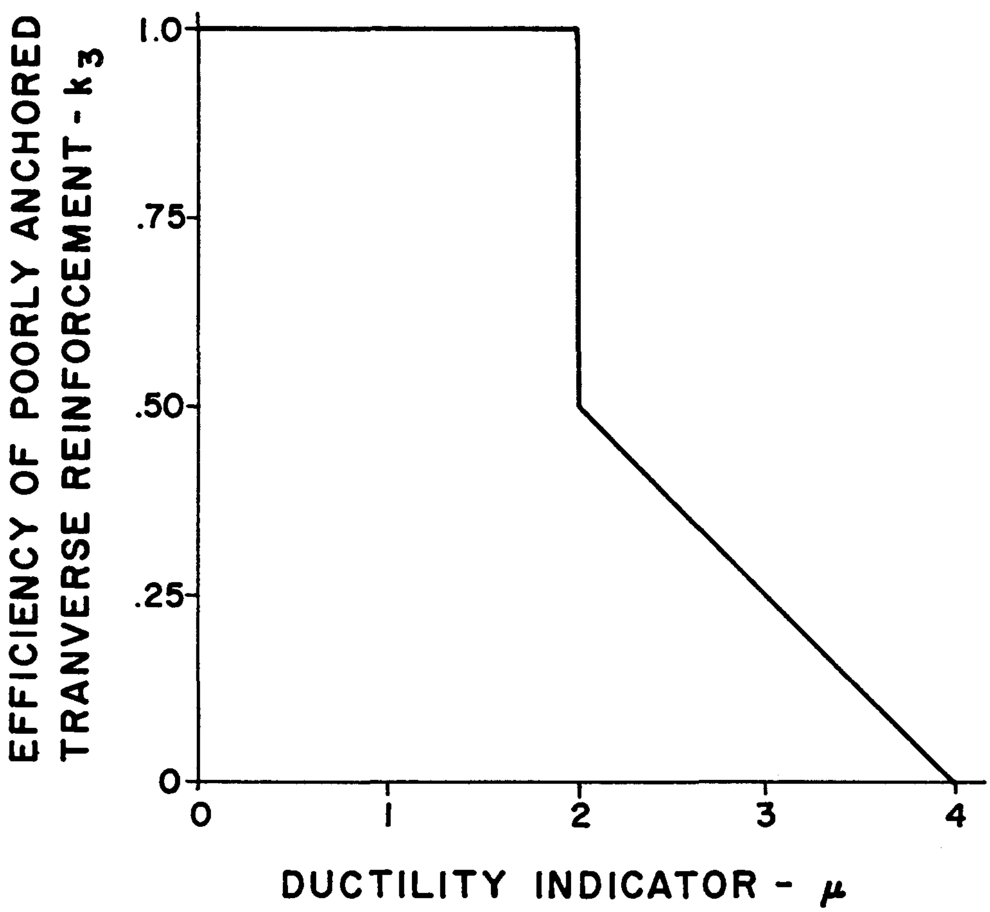


FIGURE 8. EFFECTIVENESS OF POORLY ANCHORED TRANSVERSE REINFORCEMENT AS A FUNCTION OF THE DUCTILITY INDICATOR

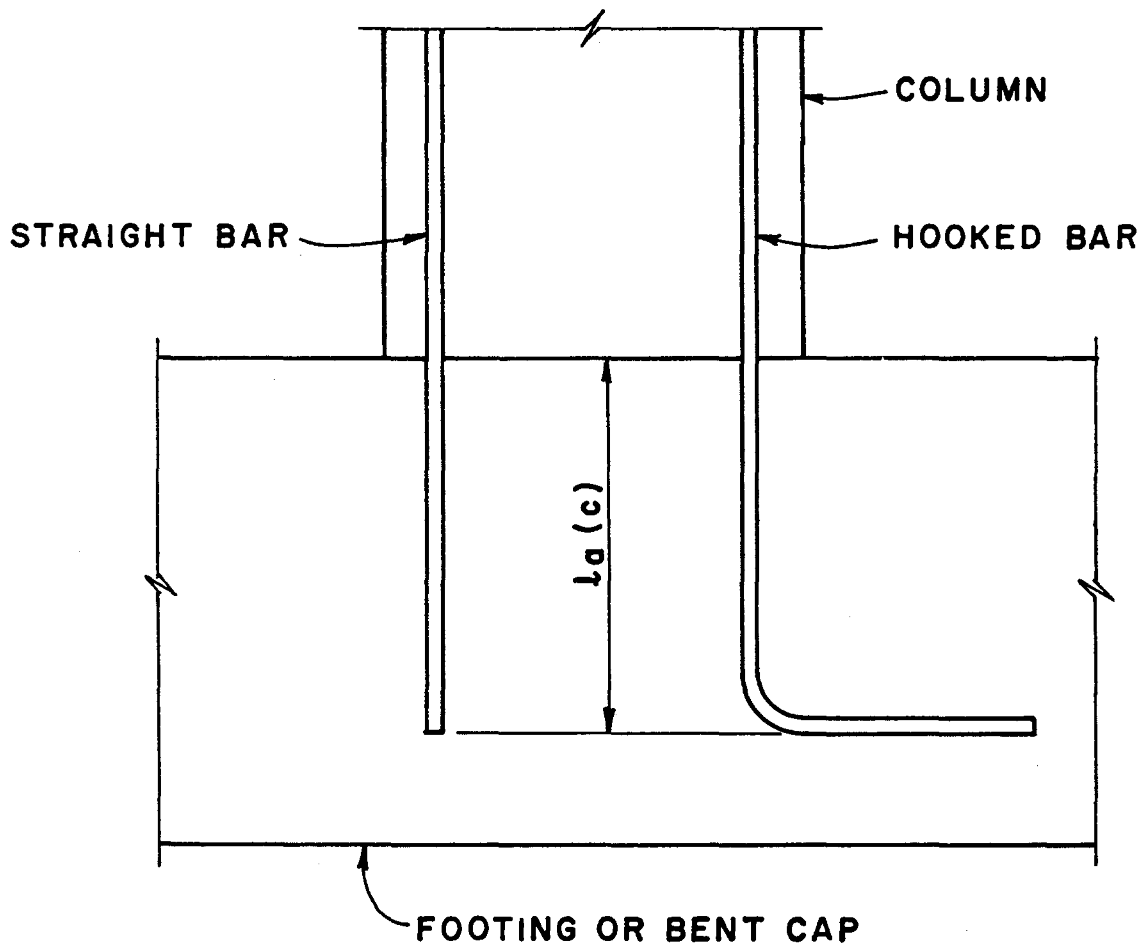


FIGURE 9. EFFECTIVE ANCHORAGE LENGTH OF LONGITUDINAL REINFORCEMENT

$A_{tr}(c)$ = Area of transverse reinforcing normal to potential splitting cracks (see Commentary). When splitting will occur between several bars in a row, $A_{tr}(c)$ is the total of the transverse steel crossing the potential crack divided by the number of longitudinal bars, in the row.

f_{yt} = Yield stress of transverse reinforcement (psi).

The value for c/d_b should not be taken as more than 2.5

For anchorage with 90^o standard hooks the effective anchorage length in inches is:

$$\ell_a(d) = k_m 1200\ d_b \frac{f_y}{60000 \sqrt{f'_c}} > 15\ d_b \qquad (4\text{-}8)$$

where

k_m = 0.7 For #11 bars or smaller, when side cover (normal to plane of the hook) is not less than 2 1/2 inches, and cover on bar extension beyond the hook is not less than 2 inches.

k_m = 1.0 For all other cases

The procedure for calculating the seismic C/D ratio for anchorage of the longitudinal reinforcement, r_{ca}, is shown in Figure 10. Methods for calculating r_{ca} will depend on the adequacy of the effective anchorage length provided and the reinforcing details at the anchorage. These methods are described in the two cases that follow.

<u>Case A</u>: If the effective development length provided is insufficient $(\ell_a(c) < \ell_a(d))$ then the C/D ratio for anchorage if longitudinal reinforcement, r_{ca}, is given by:

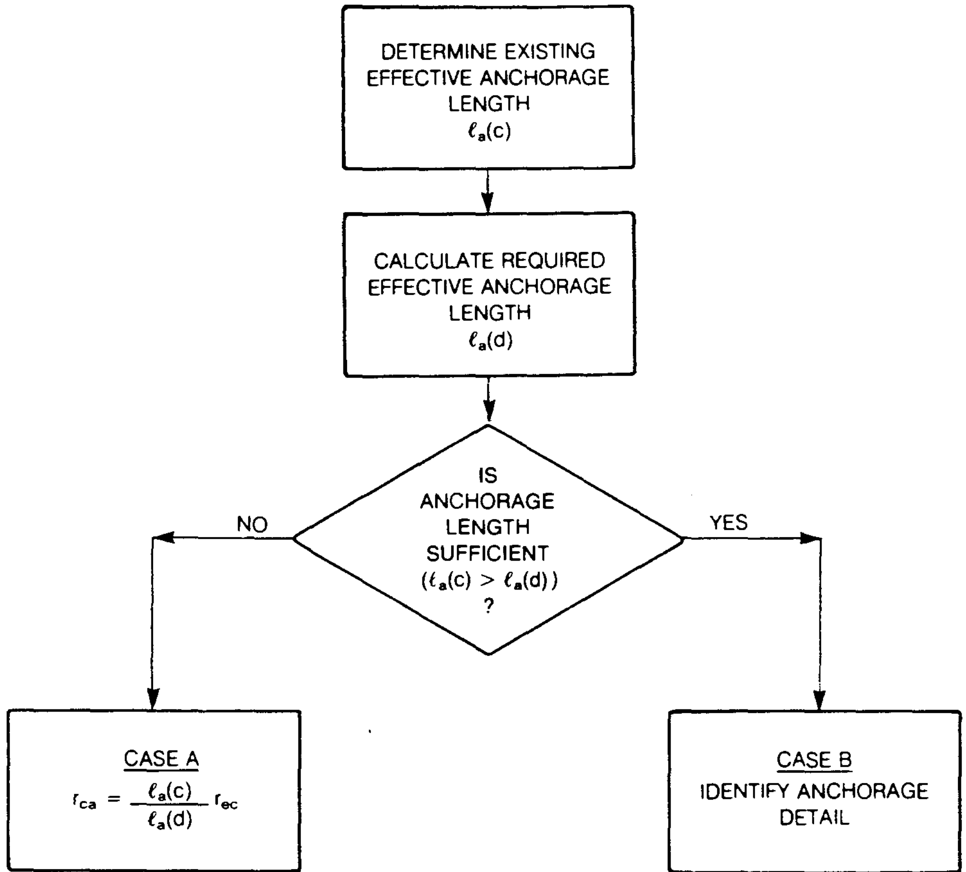
$$r_{ca} = \frac{\ell_a(c)}{\ell_a(d)} r_{ec} \ . \qquad (4\text{-}9)$$

<u>Case B</u>: If the effective development length is sufficient $(\ell_a(c) \geq \ell_a(d))$, the C/D ratio will depend on the reinforcing details at the anchorage. The six possible details and corresponding methods for calculating the C/D ratios are as follows:

<u>Detail 1</u>: <u>When no flexural tensile reinforcement</u> is present in the top of the <u>footing</u> and column bar development is by <u>straight anchorage</u>, i.e., no hooks are present at the bottom of the footings,

$$r_{ca} = r_{ef} \ , \qquad (4\text{-}10)$$

unless 1.25 times the soil overburden and/or pile anchorage is insufficient to overcome the negative moment capacity of the footing based on the modulus of rupture of the concrete, in which case r_{ca} = 1.0. This negative moment capacity will be used to calculate r_{ef} for this and the following two detail types.



DETAIL NO.	LOCATION	ANCHORAGE TYPE	TOP FOOTING REINFORCING	C/D RATIO
1	FOOTING	STRAIGHT	NO	$r_{ca} = r_{ef}$
2	FOOTING	90° HOOK AWAY FROM ℄	NO	$r_{ca} = 1.3r_{ef}$
3	FOOTING	90° HOOK TOWARD ℄	NO	$r_{ca} = 2.0r_{ef}$
4	FOOTING	STRAIGHT	YES	$r_{ca} = 1.5r_{ef}$
5	FOOTING	90° HOOK	YES	1.0
6	BENT CAP	N.A.	N.A.	1.0

FIGURE 10. PROCEDURES FOR DETERMINING CAPACITY/DEMAND RATIOS FOR ANCHORAGE OF LONGITUDINAL REINFORCEMENT

Detail 2: When no flexural tensile reinforcement is present in the top of the footing and the column bars are anchored with 90° or greater standard hooks, bent away from the column towards the edges of the footing,

$$r_{ca} = 1.3\ r_{ef} \leq 1.0\ . \qquad (4\text{-}11)$$

Detail 3: When no flexural tensile reinforcement is present in the top of the footing and the column bars are anchored with 90° or greater standard hooks turned toward the vertical centerline of the column,

$$r_{ca} = 2\ r_{ef} \leq 1.0\ . \qquad (4\text{-}12)$$

Detail 4: When the top of the footing contains adequately anchored flexural tensile reinforcement so that r_{ef} can be reliably computed from the flexural strength of the top reinforced footing section, and the column bar development is by straight anchorage only,

$$r_{ca} = 1.5\ r_{ef} \leq 1.0\ . \qquad (4\text{-}13)$$

unless soil overburden and/or pile anchorage is insufficient to overcome the negative moment capacity of the footing, in which case r_{ca} = 1.0.

Detail 5: When the top of the footing contains flexural reinforcement, as for the above detail, and the column bars have been provided with 90° standard hooks, the C/D ratio for anchorage should be taken as 1.0.

Detail 6: When the achorage is in a bent cap, the C/D ratios for anchorage should also be taken as 1.0.

4.8.2 SPLICES IN LONGITUDINAL REINFORCEMENT

Columns that have longitudinal reinforcement spliced near or within a zone of flexural yielding may be subject to a rapid loss of flexural strength at the splice unless sufficient closely spaced transverse reinforcement is provided. The minimum area of transverse reinforcement required to prevent a rapid splice failure due to reversed loading below the yeild strength of the spliced bars is given by:

$$A_{tr}(d) = \frac{s\ f_y}{\ell_s\ f_{yt}}\ A_b\ , \qquad (4\text{-}14)$$

where

s = spacing of transverse reinforcement
ℓ_s = splice length
f_y = yield stress of the longitudinal reinforcement
f_{yt} = yield stress of the transverse reinforcement
A_b = area of the spliced bar

If the clear spacing between spliced bars is greater than or equal to $4d_b$, where d_b is the diameter of the spliced reinforcement, $A_{tr}(c)$ will be the cross-sectional area of the confining hoop. If the clear spacing is less than $4d_b$, than $A_{tr}(c)$ will be the area of the transverse bars crossing the potential splitting crack along a row of spliced bars divided by the number of splices. Extra splice length by itself does not significantly improve the inelastic response of splices, but splice lengths should not be less than $1860 d_b \div \sqrt{f'_c}$,

The procedure for calculating the seismic C/D ratio for splices in longitudinal reinforcement, r_{cs}, is shown in Figure 11. This C/D ratio should be determined only when splices occur within locations potentially subject to column flexural yielding unless minimum splice lengths are not provided. This includes splices located outside the center half of columns with height-to-depth ratios greater than 3 and all splices located within columns with height-to-depth ratios less than or equal to 3. The following two cases will apply to these splices.

Case A: When splices length, transverse reinforcement amount, or transverse reinforcement spacing is inadequate $\left(\ell_s < 1860\ d_b \div \sqrt{f'_c}\ ;\ A_{tr}(c) < A_{tr}(d);\ \text{or } s < 6 \text{ inches (15.3 cm)}\right)$, then the C/D ratio for splices in longitudinal reinforcement, r_{cs}, is given by:

$$r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)} \left(\frac{\left(\frac{6}{s}\right) \ell s}{\left(\frac{1860}{\sqrt{f'_c}}\right) d_b} \right) r_{ec} < \frac{A_{tr}(c)}{A_{tr}(d)} r_{ec} \qquad (4\text{-}15)$$

where the ratio 6/s should not be taken larger than 1 and $1860/\sqrt{f'_c}$ should not be taken less than 30. The C/D ratio for splices, r_{cs}, need not be taken as less than .75 r_{ec} when the minimum splice length is provided.

Case B: When the splice is sufficient $\left(A_{tr}(c) \geq A_{tr}(d),\ \ell_s \geq (1860/\sqrt{f'_c})\ d_b;\ \text{and } s \leq 6 \text{ inches (15.3 cm)}\right)$ then the C/D ratio for splices in longitudinal reinforcement, r_{cs}, is given by:

$$r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)} r_{ec} \leq 2 r_{ec} \ . \qquad (4\text{-}16)$$

4.8.3 COLUMN SHEAR

Column shear failure will occur when shear demand exceeds shear capacity. This may occur prior to flexural yielding or during flexural yielding due to the degradation of shear capacity. The following terms are used to calculate the C/D ratio for column shear, r_{cv}:

$V_u(d)$ = The maximum column shear force resulting from plastic hinging at both the top and bottom of the column due to yielding in the column or footing ($V_u(d) = 1.3\ \Sigma M_u/L_c$) or due to an anchorage or splice failure in the column, whichever occurs first. (See Note 1 below.)

$V_e(d)$ = The maximum calculated elastic shear force.

$V_i(c)$ = The initial shear resistance of the undamaged column. This will include the resistance of the gross concrete section and the transverse steel. (See Note 2 below.)

$V_f(c)$ = The final shear resistance of the damaged column. This will include only the resistance of effectively anchored transverse steel. When the axial stress is greater than or equal to $0.10 f'_c$, an allowable shear stress of $2\sqrt{f'_c}$ may be assumed for the core of the concrete column.

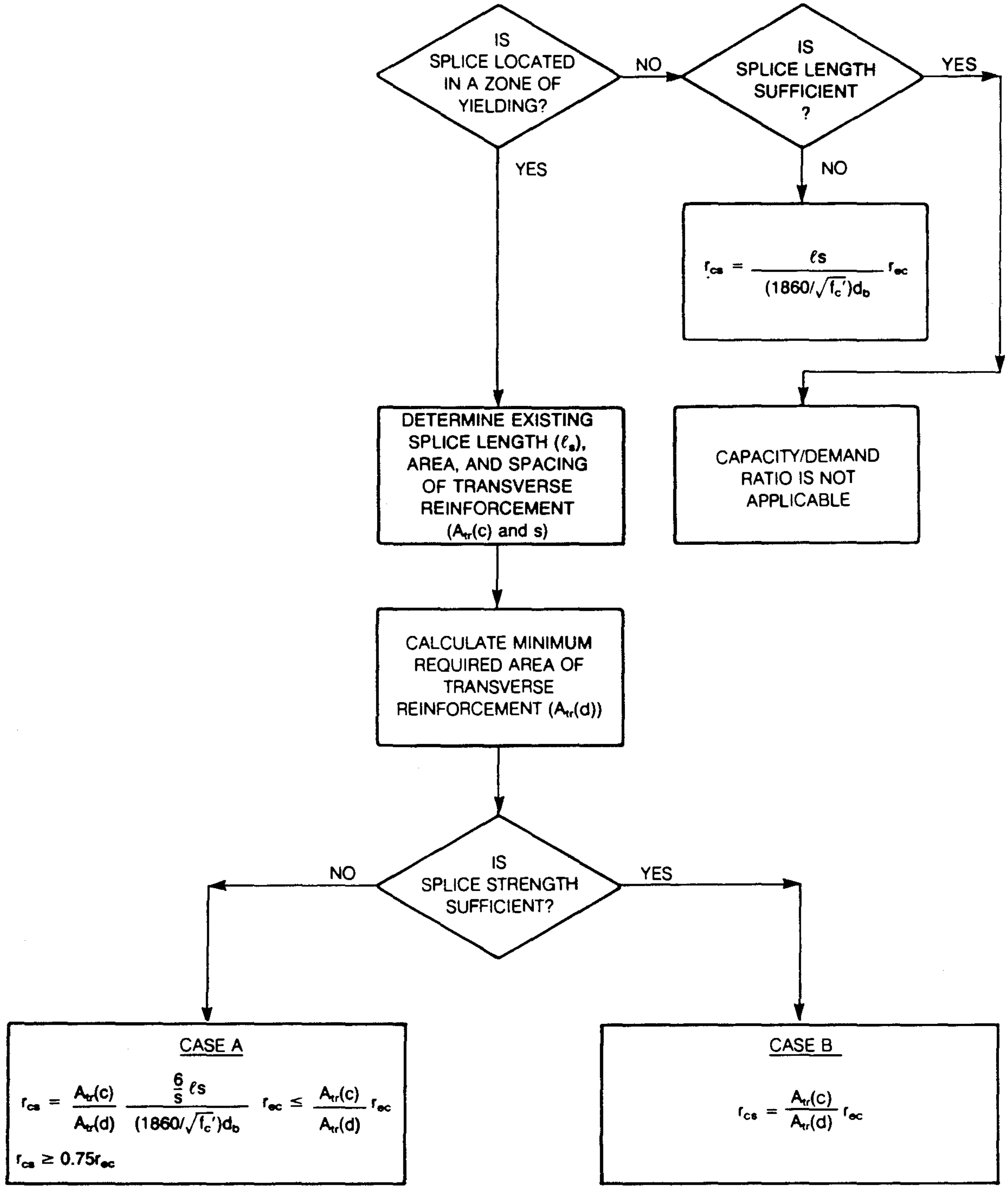


FIGURE 11. PROCEDURE FOR DETERMINING CAPACITY/DEMAND RATIOS FOR SPLICES IN LONGITUDINAL REINFORCEMENT

Note 1: Procedures for calculating shear forces resulting from column hinging are given in Section 4.8.2 of the Seismic Design Guidelines. These procedures may be extended to consider nominal moment capacities of the footings or a reduced column nominal moment capacity due to an anchorage or splice failure below the nominal ultimate column moment.

Note 2: Shear resistance of concrete columns is calculated using the provisions of Section 1.5.35 of the AASHTO Standard Specifications for Highway Bridges, except that capacity reduction (ϕ) factors are not used.

The procedure for calculating the C/D ratio for column shear is shown in Figure 12.

When columns do not experience flexural yielding ($r_{ec} \geq 1.0$), the C/D ratio for column shear should be calculated using the initial shear capacity, $V_i(c)$, and the elastic shear demand, $V_e(d)$. In columns subject to yielding ($r_{ec} < 1.0$), the C/D ratio for column shear, r_{cv}, is calculated according to the procedure outlined in Figure 12. Each of three possible cases are described below.

Case A: If the inital shear resistance of the undamaged column is insufficient to withstand the maximum shear force due to plastic hinging, $(V_{i(c)} < V_u(d))$ a brittle shear failure may occur prior to formation of a plastic hinge and the C/D ratio, r_{cv}, must be calculated using elastic shear demands,

$$r_{cv} = \frac{V_i(c)}{V_e(d)} . \tag{4-17}$$

In no case shall r_{cv} be greater than r_{ec}.

Case B: If the initial shear resistance of the column is sufficient to withstand the maximum shear force due to plastic hinging, but the final shear resistance of the column is not, $(V_i(c) \geq V_u(d) > V_f(c))$, then the C/D ratio for column shear will depend on the amount of flexural yielding which will cause a degradation in shear capacity from $V_i(c)$ to $V_u(d)$. The C/D ratio is given by:

$$r_{cv} = \mu r_{ec} , \tag{4-18}$$

where

$$\mu = 2 + \left(.75 \frac{L_c}{b_c}\right) \frac{V_i(c) - V_u(d)}{V_i(c) - V_f(c)} . \tag{4-19}$$

where

L_c = Height of the column.
b_c = Width of the column in the direction of shear.

The column height-to-width ratio should not be taken to be greater than 4 in Equation 4-19.

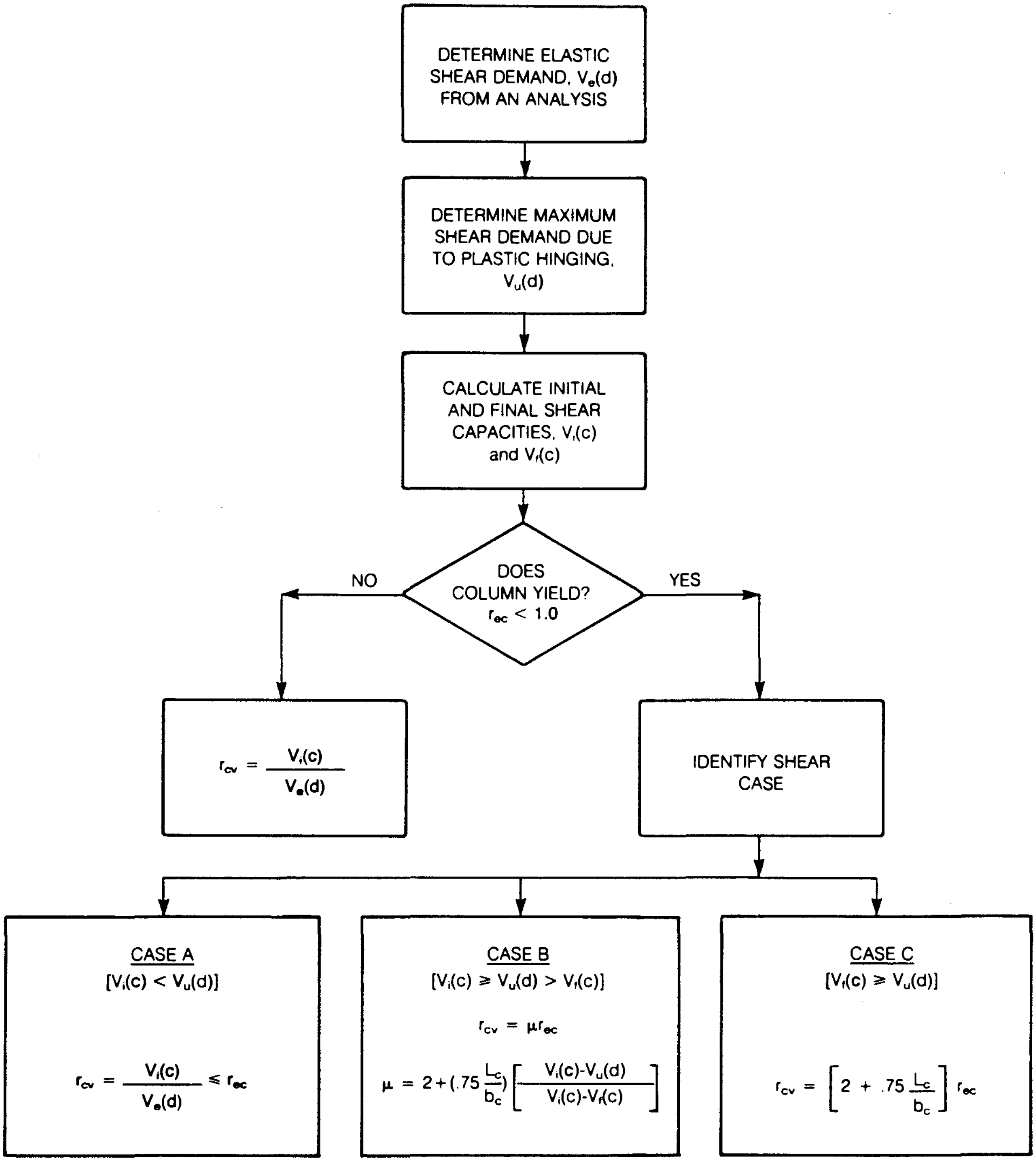


FIGURE 12. PROCEDURE FOR DETERMINING CAPACITY/DEMAND RATIOS FOR COLUMN SHEAR

Case C: If the final shear resistance of the column is sufficient to withstand the maximum shear force due to plastic hinging, $[V_f(c) > V_u(d)]$, then the C/D ratio for column shear is given by:

$$r_{cv} = \left(2+.75 \frac{L_c}{b_c}\right) r_{ec} \cdot \tag{4-20}$$

As with Case B, the column height to width ratio should not be taken to be greater than 4.

4.8.4 TRANSVERSE CONFINEMENT REINFORCEMENT

Inadequate transverse confinement reinforcement in the plastic hinge region of a column will cause a rapid loss of flexural capacity due to buckling of the main reinforcement and crushing of the concrete in compression. The following equation may be used to calculate the C/D ratio for transverse confinement, r_{cc}:

$$r_{cc} = \mu r_{ec} \; , \tag{4-21}$$

where

$$\mu = 2 + 4\left(\frac{k_1 + k_2}{2}\right) k_3, \tag{4-22}$$

where

$$k_1 = \frac{\rho\,(c)}{\rho(d)\ \left(0.5 + \dfrac{1.25 P_c}{f'_c\ A_g}\right)} \leq 1$$

$$k_2 = \frac{6}{s/d_b} \leq 1 \quad \text{or} \quad \frac{0.2}{s/b_{min}} \leq 1 \text{ , whichever is smaller}$$

k_3 = Effectiveness of transverse bar anchorage. This will be 1.0 unless transverse bars are poorly anchored in which case Figure 8 shall be used to determine k_3. Note that when this is the case an iteratative solution of Equaton 4-22 will be required.
ρ (c) = Volumetric ratio of existing transverse reinforcement.
ρ (d) = Required volumetric ratio of transverse reinforcement given by Section 8.4 of The Seismic Design Guidelines.
P_c = Axial compressive load on the column.
f'_c = Compressive strength of the concrete.
A_g = Gross area of column.
s = Spacing of transverse steel.
d_b = Diameter of longitudinal reinforcement.
b_{min} = Minimum width of the column cross section.

4.8.5 FOOTING ROTATION AND/OR YIELDING

Column footings may rotate and/or yield before columns can yield. This can occur due to any one of several failure modes. The amount of rotation and/or yielding allowed in the footing will depend on the mode of failure. The seismic C/D ratio for these types of footing failures, r_{fr}, are calculated as follows:

$$r_{fr} = \mu r_{ef} , \tag{4-23}$$

where μ, the ductility indicator, is taken from Table 6 depending on the type of footing and mode of failure. See the Commentary for discussion of method for calculating the nominal ultimate capacity of the footing.

TABLE 6: FOOTING DUCTILITY INDICATORS

Type of Footing	Factor Limiting the Capacity	μ
Spread Footing	Soil Bearing Failure	4
	Reinforcing Steel Yielding in the Footing	4
	Concrete Shear or Tension in the Footing	1
Pile Footing	Pile Overload (Compression 2 or Tension)	3
	Reinforcing Steel Yielding in the Footing	4
	Pile Pullout at Footing	2
	Concrete Shear or Tension in the Footing	1
	Flexural Failure of Piling	4
	Shear Failure of Piling	1

4.9 CAPACITY/DEMAND RATIOS FOR ABUTMENTS

Failure of abutments during earthquakes usually involves tilting or shifting of the abutment, either due to inertia forces transmitted from the bridge superstructure or seismic earth pressures. Usually these types of failures alone do not result in collapse or impairment of the ability of the structure to carry emergency traffic loadings. However, these failures often result in loss of access, which can be critical in certain important structures.

Large horizontal movement at the abutments is often the cause of large approach fill settlements that can prevent access to the bridge. Therefore when required, abutment C/D ratios are based on the horizontal abutment displacement. The displacement demand, d(d), will be the elastic displacements at the abutments obtained by properly modeling the abutment stiffness (see Commentary Section 4.2). The displacement capacity, d(c), is taken as three inches in the transverse direction and six inches in the longitudinal direction unless determined otherwise by a more detailed evaluation. Therefore:

$$r_{ad} = \frac{d(c)}{d(d)} . \tag{4-24}$$

4.10 CAPACITY/DEMAND RATIOS FOR LIQUEFACTION INDUCED FOUNDATION FAILURE

Many foundation failures during earthquakes are the result of loss of foundation support occurring as a result of liquefaction. A C/D ratio should be calculated when the preliminary screening indicates the potential exists for a major or severe liquefaction related foundation damage. To determine the C/D ratio for liquefaction failure, r_{sl}, a two-stage procedure is necessary. First, the depth and areal extent of soil liquefaction required for foundation failure and associated damage must be assessed. Secondly, the level of seismic shaking that will produce liquefaction of the above foundation soils must be evaluated. The C/D ratio is obtained by dividing the effective peak ground acceleration at which liquefaction failure is likely to occur by the design acceleration coefficient:

$$r_{sl} = \frac{A_L(c)}{A_L(d)}, \tag{4-25}$$

where

$A_L(c)$ = The effective peak ground acceleration at which liquefaction failures are likely to occur.

$A_L(d)$ = A = Design acceleration coefficient for the bridge site.

Although a great deal of work has been done with respect to determining earthquake induced liquefaction potential of soils, the parameter $A_L(c)$ is difficult to determine precisely. Selection of a realistic value for $A_L(c)$ will require considerable engineering judgment. For example, whereas a sand seam may liquefy, its influence on a pile foundation may be minimal. Significant lateral foundation displacement leading to damage may require a 10 feet depth of liquefaction near the pile head with subsequent continued ground shaking.

The amount of movement at a given site due to soil liquefaction is a function of the intensity and duration of shaking, the extent of liquefaction, and also the relative density of the soil, which controls post-liquefaction undrained or residual strength. In addition, different bridges will be able to sustain different amounts of movement. Therefore, when determining $A_L(c)$, both the site and the bridge characteristics must be taken into consideration.

The references to bridge related liquefaction failures noted in the commentary (Sections C2.3.1 and C 4.10) may be of assitance in evaluating this problem, as well as references related to assessing liquefaction potential of soils. Finally, it is recommended that geotechnical specialists participate in the determination of $A_{L(c)}$ at a specific bridge site and assist in the evaluation of the subsequent foundation displacement and damage potential.

CHAPTER 5

SEISMIC RETROFITTING CONCEPTS

5.1 GENERAL

Retrofit measures should be selected with the goal of minimizing the probability of total collapse and/or severe structural damage of the bridge. If practical, important bridges should be retrofitted so emergency vehicles can use the bridge following an earthquake. It is expected that retrofittng will not always increase the level of seismic resistance to that of a new bridge.

Bridges in Seismic Performance Category B will usually only require consideration of retrofitting at the bearings and expansion joints. In Seismic Performance Category C, columns, piers, and footings should also be considered. Only in Seismic Performance Category D should retrofitting of all components be considered.

When selecting appropriate measures for retrofitting, the overall capacity of the structure to resist earthquakes must be considered. An analysis of the existing structure is usually performed to identify weak links in the seismic resistance of the bridge. These weaknesses will be reflected in the capacity/demand ratios for various components and modes of failure. An assessment of the consequences of failure in each component or mode of failure will help identify components that need retrofitting. Retrofitting schemes which will increase the capacity of a component and/or reduce the demand on the component should be considered.

Several methods for retrofitting bridges have been proposed and some have been used in practice. The use of expansion joint restrainers is the most popular and has proved to be an economical method of retrofitting, whereas measures such as column and liquefaction retrofit are in general quite expensive. Sections 5.3 through 5.6 summarize many of these concepts. Economic and practical considerations will also be important in the final section of a retrofit scheme.

Bridge seismic retrofitting measures shall be designed and constructed in accordance with this chapter and the applicable requirements of the Seismic Design Guidelines. When a conflict exists the requirements of this chapter should govern.

5.2 SEISMIC PERFORMANCE REQUIREMENTS

Seismic retrofitting measures are designed to prevent collapse and/or severe structural damage of the bridge due to the following modes of failure:

1. Loss of support at the bearings which will result in a partial or total collapse of the bridge.
2. Excessive strength degradation of the supporting components.
3. Abutment and foundation failures resulting in loss of accessibility to the bridge.

When retrofitting bridges, care should be taken not to transfer excessive forces to other less-easily inspected and repaired components. The recommended minimum acceleration coefficient, A, to be used in designing seismic retrofitting measures should be as shown on the maps in Figures 1 and 2. Minimum analysis procedures, determination of elastic forces and displacements, and combination of orthogonal seismic forces should be accomplished as described in Sec. 4.2 through 4.4. Once it has been decided to retrofit a component, it is recommended, if possible, that the component retrofit be designed to the standards for new construction specified in the Seismic Design Guidelines. Reduced levels of seismic retrofitting may be used when the use of full design standards is not practical or economically feasible and partial strengthening significantly reduces the risk of unacceptable damage. The following sections also give special design requirements for each of the types of retrofit concepts discussed.

5.3 BEARINGS AND EXPANSION JOINTS

Several bridges have failured during past earthquakes due to a loss of support at the bearings. These failures are sometimes spectacular, but are also relatively simple and inexpensive to prevent by retrofitting. Because of this, most retrofitting efforts to date have been directed toward tying the bridge together at bearings and expansion joints. Several retrofitting methods have been used extensively, while other more exotic methods have been used only on a trial basis. Each of these methods are discussed in the following paragraphs.

5.3.1 LONGITUDINAL JOINT RESTRAINERS

Longitudinal joint restrainers are installed to limit the relative displacement at joints and thus decrease the chance of a loss of support at these locations. When bearing anchor bolts and similar details are deemed inadequate to prevent a loss of support at "fixed" bearings, longitudinal restrainers can be used as a method for improving these details.

The restrainer force capacity and stiffness will generally be determined from an analysis of the structure. The single mode spectral method (Analysis Procedure 1) must be adapted to determine restrainer forces. This adaptation is explained further in the Commentary. In no case should the restrainer force capacity be less than required to resist an equivalent horizontal static load of .35 times the deadload of the superstructure. When two superstructure segments are tied together, the minimum restrainer capacity should be the maximum of the two capacities obtained by considering each section independently. For "regular" bridges in Seismic Performance Category B, an analysis is not necessary, and the minimum restrainer force capacity may be used as the restrainer design force. Restrainers should be capable of developing the design force before bearings become unseated. In areas of low seismicity it may be desireable to restrain jonts having narrow bearing seats by using short, stiff restrainers designed to function below their yeild capacity. The stiffness of the restrainers will result in small joint movements while restrainer forces will be kept to reasonable levels because of the low seismicity.

Results from an analysis should always be carefully examined and interpreted with engineering judgement in light of the several assumptions usually made in dynamic analysis. When higher forces seem appropriate, they should be used for design.

Restrainers should be designed to resist the maximum forces in the elastic range. A minimum of two symmetric restrainers per joint will provide for redundancy and minimize eccentric movement of the joint. An adequate gap should be provided to allow for normal movement at expansion joints. For joints located at piers, restrainers should provide a direct and positive tie between the superstructure and the pier, unless pier caps are wide enough to prevent a loss of support at the end of the span and the anticipated maximum movement of the superstructure will not cause excessive damage to the bridge.

Connections of the restrainer to the superstructure or substructure should be capable of resisting 125% of the ultimate restrainer capacity. In addition, the existing structural elements subject to brittle failure should be capable of resisting 125% of the ultimate restrainer capacity. Both restrainer connections and existing structural elements should be capable of resisting the eccentricities caused by variations in the restrainer forces of at least 10% of the nominal ultimate restrainer capacity.

5.3.2 TRANSVERSE BEARING RESTRAINERS

Transverse restraint at bearings is intended to prevent unacceptable damage resulting from excessive transverse motion. A large number of bearings can be expected to move excessively during an earthquake. As discussed in the Commentary, however, this movement does not always result in unacceptable damage. When transverse bearing movement results in an instability of the structure that could lead to a loss of support, transverse restraint are be provided as a retrofit measure. Even when transverse restraint is not required to prevent structural collapse, it should not be ruled out, since it may be possible to prevent severe structure damage for very little cost.

The design forces used to design transverse restrainers are generally determined from an analysis. Transverse restrainer forces obtained from the elastic design spectra should be increased by a factor of 1.25 to account for transfer of load due to column yielding. The minimum transverse restrainer design capacity should be not less than required to resist an equivalent horizontal static load of .35 times the superstructure deadload. For single-span bridges or "regular" bridges in Seismic Performance Category B, an analysis is not necessary and the minimum transverse design force may be used.

5.3.3 VERTICAL MOTION RESTRAINERS

Vertical hold-down devices may be used at bearings to prevent uplift that could result in damage or loss of stability. Although uplift by itself is unlikely to result in structure collapse, vertical hold-down devices should be considered whenever the vertical seismic force due to Load Case 1 exceeds 50% of the deadload reaction. Vertical motion restrainers are usually not economically justified unless some additional bearing retrofit is being performed and the bridge is classified as Seismic Performance Category D.

5.3.4 BEARING SEAT EXTENSION

A bearing seat extension may be considered as a retrofit measure when it is impractical to restrain movement enough to prevent loss of support at the bearings. If possible, at abutments, these extensions should be supported directly on the foundation as shown in Figure 13.

All bearing seat extensions should provide a final minimum seat width equal to or greater than required by Section 4.6. Because bearing seat extensions will be subjected to large forces during an earthquake due to the superstructure dropping and sliding on the extension, they should be designed to resist vertical load of twice the deadload reaction plus the maximum live load reaction, or vertical load equal to the deadload reaction in conjunction with horizontal load equal to the deadload reaction times the acceleration coefficient.

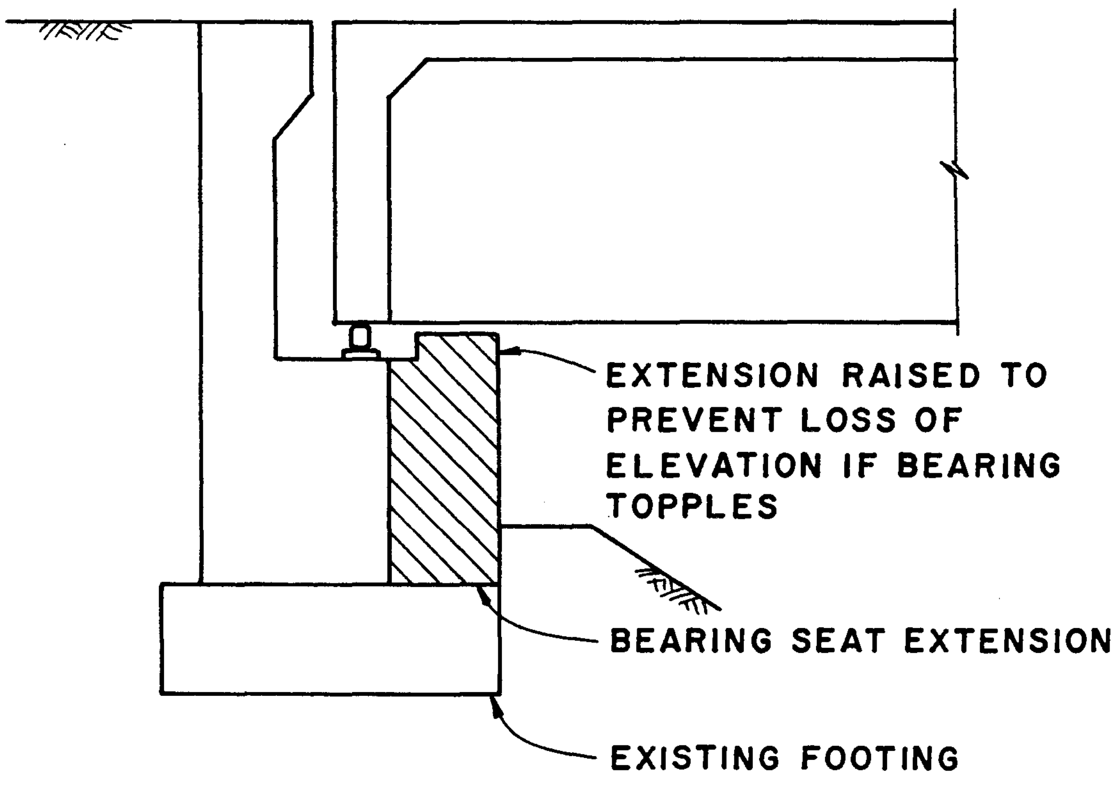


FIGURE 13. BEARING SEAT EXTENSION AT ABUTMENT

5.3.5 REPLACEMENT OF BEARINGS

Replacement of bearings should be considered if their failure will result in collapse or loss of function of the superstructure. Types of bearings that have performed poorly in past earthquakes are shown in Figure 14. When these bearings are present, consideration should be given to replacing them. The type of replacement bearing would be dependent upon the Seismic Performance Category and may include elastomeric bearing pads or more sophisticated energy-dissipating devices that have been recently developed.

Replacement bearings and their accompanying restraining features should be capable of resisting the longitudinal, transverse, and vertical design forces determined from an analysis. For single-span bridges or "regular" bridges in Seismic Performance Category B, an analysis is not necessary and the minimum bearing force demands as described in Section 4.5 may be used as the design forces.

5.3.6 SPECIAL EARTHQUAKE RESISTANT BEARINGS AND DEVICES

Certain types of recently developed bearings and devices have special performance characteristics which will alter the seismic response of the entire structure. Most of these are designed to act as force and/or displacement-limiting devices. Force-limiting devices minimize the force that can be transferred to supporting columns, piers, or abutments and thereby provide a retrofit measure for substructures as discussed in Section 5.4.1.

When the performance characteristics of special bearings include nonlinear characteristics as the basis for modifying seismic response of the structure, an analysis which will adequately consider this plus other pertinent nonlinear characteristics of the structure should be performed prior to using these special types of bearings as a retrofit measure. If nonlinear time-history analysis is performed, at least three ground motion time histories should be used. These ground motions should have different characteristics as represented by frequency content, duration of maximum shaking, and other characteristics that would reasonably reflect the variety of ground motions that could be expected at the bridge site. If design charts have been developed and are based on a series of nonlinear analyses consistant with the above-stated criteria, they may be used in lieu of a special nonlinear analysis.

Certain types of devices such as hydraulic dampers may leak and thus malfunction if not properly maintained. In these cases, a backup system of seismic resistance, such as restrainers, should be provided. When indicated by the analysis, retrofitting should also provide for additional displacements resulting from modified behavior, correction of permanent displacements following an earthquake, and increased component forces due to a redistribution of the seismic load.

5.4 REINFORCED CONCRETE COLUMNS, PIERS, AND FOOTINGS

Reinforced concrete columns, piers, and footings may fail in any of several ways during an earthquake, as discussed in the Commentary on Chapter 4. In general, it is more difficult and less cost effective to retrofit these components than it is bearings.

To date very few of these retrofit methods, with the exception of force-limiting devices used in New Zealand, have been used to retrofit for seismically deficient bridge columns. Column, pier, and footing retrofitting is discussed in the following paragraphs.

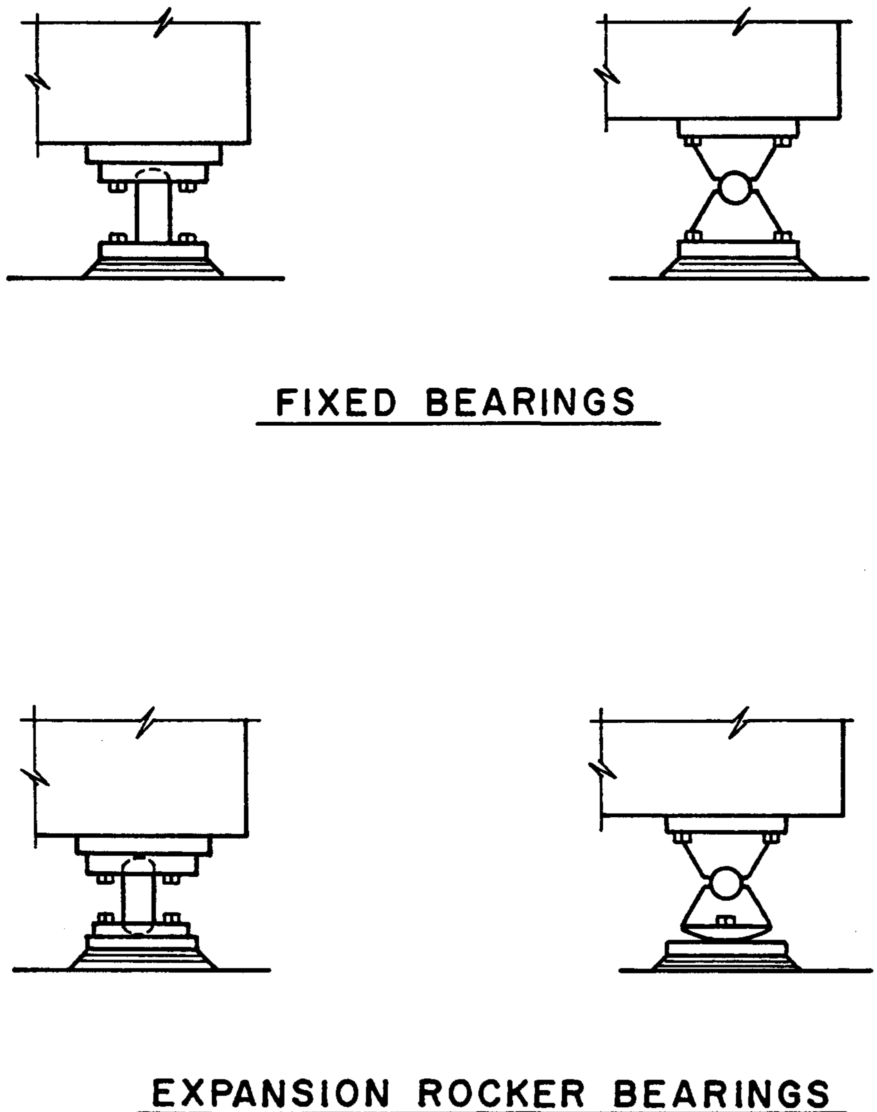


FIGURE 14. SEISMICALLY VULNERABLE BEARINGS

5.4.1 FORCE-LIMITING DEVICES

A force-limiting device provides a mechanism that limits the amount of force that can be transferred between the superstructure and supporting substructure. The most conceptually simple devices of this type are TFE ("Teflon") sliding bearings, which provide for a very small transfer of force. Other types of devices that transfer a greater but limited amount of force, as discussed in the Commentary, have also been developed.

The use of force-limiting devices should be restricted to devices whose dynamic performance has been demonstrated by physical testing. Design forces and displacements should be derived from an analysis of the structure which takes into consideraton the actual performance characteristics of the device.

To be effective in protecting the substructure from excessive forces, the yield level of the force-limiting device should not exceed 80 percent of the theoretical yield level of the substructure component to be protected. The device must also be capable of accommodating inelastic displacements equal to those expected during the design earthquake. To protect against unexpected differental displacements that could result in a loss of support and collapse of the bridge, it is recommended that restraining devices be provided as a fail-safe measure. Restrainers should be designed to allow for freedom of movement within a range that will maximize the effectiveness of the force-limiting devices. When engaged, restrainers used as a fail safe measure to prevent collapse should be capable of developing a force equal to 1.25 times the nominal ultimate capacity of the substructure elements prior to the bearings becoming unseated.

Care must be exercised in the use of force-limiting devices because they may cause increased relative displacements between the superstructure and substructure.

5.4.2 INCREASED TRANSVERSE CONFINEMENT

Improved confinement will increase the ability of a column to withstand repeated cycles of loading beyond the elastic limit and tend to prevent column failure due to shear, loss of anchorage or splice capacity of longitudinal reinforcement, and degradation of flexural capacity. The Seismic Design Guidelines have requirements for the spacing, amount, and anchorage of conventional transverse reinforcement. The use of conventional transverse reinforcement for retrofitting, however, would present construction difficulties and would be of questionable effectiveness.

Several different methods have been proposed which would increase confinement and allow the column to withstand repeated cycles of loading beyond the elastic limit. When designing retrofit measures that will increase transverse confinement, the following design requirements should be met.

Increased transverse confinement should be located within the column end regions. The end regions should be assumed to extend from the soffit of girders or cap beams at the top of columns, or the top of foundations at the bottom of columns, a distance not less than the greater of (a) the maximum cross-sectional dimension of the column, (b) one-sixth of the clear height of the column, and (c) 18 inches (457 mm).

The transverse confinement reinforcing should be capable of developing the confining force provided by the transverse confinement required for new construction as given in Section 8.4.1D of the Seisimic Design Guidelines. In addition, if the capacity/demand ratio for shear in the existing column is less than 1.0, the transverse reinforcement should be capable of resisting the maximum shear force due to hinging in the column as described in Section 4.8.3. Transverse confinement reinforcing should have a maximum spacing not to exceed the smaller of one-quarter of the minimum member dimension or 4 inches (102 mm). Anchorage schemes for transverse reinforcement should be capable of developing the ultimate capacity of the reinforcement, and should not be significantly affected by the spalling of cover concrete. The designer should be aware that retrofit schemes for increasing confinement may redistribute moments and shears, resulting in overstress in other members of the structure, i.e., footings and bent caps.

The effectiveness of transverse confinement in improving the seismic performance of the column should be evaluated by recalculating the seismic capacity/demand ratios for the column. Increased transverse confinement should result in capacity/demand ratios greater than one for each of the column failure modes. If this is not the case, then additional retrofit measures should be considered.

5.4.3 REDUCED FLEXURAL REINFORCEMENT

The ultimate shear force on a column can be reduced by decreasing the yield moment at one or both ends of the column. This retrofit method should only be considered when columns are over-reinforced for flexure resulting in little or no flexural yielding during an earthquake. The resulting high-yield moments could produce shear forces above the capacity of the column. By cutting longitudinal reinforcing bars, an increased amount of yielding is accepted in exchange for a reduced shear force. The net result could be an improvement in the overall earthquake resistance of the structure. This retrofit method has never been tried. Despite its conceptual appeal, it is controversial because it does reduce the flexural strength of the column. Because this retrofit method will increase the ductility demand at the points of flexural yielding, caution is advised. A loss of flexural capacity (i.e., the formation of a pinned joint) at the location of cut bars should not result in overall structural instability since the remaining uncut bars at this location may be expected to yield in the early stages of seismic shaking.

It is recommended that cutting of column longitudinal reinforcement as a retrofit measure be used only when it is infeasible to retrofit the column by adding additional transverse reinforcement and when the minimum capacity/demand ratio for the column can be improved by at least 50 percent to a minimum value of 0.75.

5.4.4 INCREASED FLEXURAL REINFORCEMENT

The use of increased flexural reinforcement has also been proposed. This retrofit technique will increase the flexural capacity of the column. Increased flexural capacity will increase the forces transferred to the foundation and the superstructure/column connections and will also result in an increased column shear force. In addition, the strengthened column will be stiffer and may be subjected to more seismic force. Since failure of the footings or failure of the columns in shear is usually more critical than excessive flexural yielding, this retrofit technique should be used with care.

The biaxial strength of columns retrofitted with increased flexural reinforcement should not be less than required to resist the bending moments determined according to Section 4.8.3 of the Seisimic Design Guidelines. The response modification factors shown in Table 3 of the Seismic Design Guidelines may be used if column transverse reinforcement conforms to Section 8.4.1C through 8.4.1E of the Seismic Design Guidelines. When these requirements are not met, a response modification factor of

2 should be used. Care should be taken that all other components are able to resist the forces developed by 1.25 times the nominal ultimate moments of the strengthened column. This retrofit technique should only be considered when loss of flexural strength would result in a collapse mechanism and when the ultimate moment capacity/elastic moment demand ratio, r_{ec}, is less than 0.125.

5.4.5 INFILL SHEAR WALL

The transverse resistance of multi-column bents can be increased by constructing an infill concrete shear wall between individual columns in the bent. This technique has been used to repair earthquake damage to bridges in Japan and California, and requires that individual column footings be extended to support the shear wall. The shear wall is tied into the existing structure with grouted bars or anchors.

5.4.6 STRENGTHENING OF FOOTING

In many cases column footings will fail before the column or pier yields. This is often due to the absence of a top layer of reinforcement capable of resisting uplift forces on the footing. During an earthquake this can result in the flexural cracking of footing concrete and the loss of anchorage for the column longitudinal reinforcement. This condition is usually most critical in single-column bents supported on pile footings.

Footings should be strengthened to prevent brittle failure that could lead to a collapse of the bridge. Design moments and forces should be equal to those generated by 1.25 times the nominal ultimate capacity of the soil and/or pilings. Design strengths should conform to the provisions for load factor design of the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

5.5 **ABUTMENTS**

Abutment failure very rarely results in the collapse of the structure unless associated with liquefaction failure. Lateral movement of an earth-retaining abutment or consolidation of the abutment fill may result in a loss of accessability to the bridge, which may be an unacceptable failure for a particularly important bridge. In addition, the use of restrainers to limit relative displacement at the abutment bearings may result in much larger abutment forces. Therefore, situations will exist in which abutment retrofitting should be considered. The following paragraphs discuss two possible retrofit measures that will mitigate the effects of abutment failure.

5.5.1 SETTLEMENT SLABS

Settlement (or approach) slabs are designed to provide continuity between the bridge deck and the abutment fill in the case of approach fill settlement. Settlement slabs should be positively tied to the abutment to prevent them from pulling away and becoming ineffective. It is recommended that they be considered only for bridges classified as SPC-D with approach fills subject to excessive settlement due to either soil failure or structural failure of the abutment. To minimize the discontinuity at the abutment following an earthquake, settlement slabs should be provided with a minimum length of 10 feet. Settlement slabs should be designed as simple span-reinforced concrete slabs spanning their full length. Positive ties to the abutment should be capable of resisting the slab deadload times the sum of the coefficient of friction between the slab and the abutment fill plus the Acceleration Coefficient.

5.5.2 SOIL ANCHORS

Horizontal displacement at the abutment may cause a loss of accessability to the bridge. Displacements of the abutment normal or parallel to the abutment face may be prevented or minimized by adding soil anchors.

The ultimate capacity of soil anchors should be greater than or equal to seismic forces transferred to the abutment from the superstructure and/or the seismic earth pressures generated behind the abutment backwall due to the design earthquake.

5.6 LIQUEFACTION AND SOIL MOVEMENT

Liquefaction and/or excessive movement have been the cause of the majority of bridge failures in some areas during past earthquakes. There are two suggested approaches to retrofitting that will mitigate these types of failure. The first approach is to eliminate or improve the soil conditions that tend to be responsible for seismic liquefaction. The second approach is to increase the ability of the structure to withstand large relative displacements similar to those caused by liquefaction or large soil movement. The first approach has been tried on dams, power plants, and other structures but to date has not been used as a retrofit measure for bridges. The second approach utilizes many of the retrofitting techniques discussed in the previous sections.

5.6.1 SITE STABILIZATION

Although site stabilization would only be used in exception cases, several methods are available for stabilizing the soil at the site of the bridge. Some possible methods include:

- Lowering of groundwater table.
- Consolidation of soil by vibrofloatation or sand compaction.
- Vertical network of drains.
- Placement of permeable overburden.
- Soil grouting or chemical injection.

Some of these methods may not be suitable or environmentally acceptable, and may even be detrimental in certain cases unless provisions are made to minimize the effects of soil settlement during construction. Therefore, careful planning and design are necessary before employing any of the above site-stabilization methods. Each method should be individually designed using established priniciples of soil mechanics to ensure that the design is effective and that construction procedures will not damage the existing bridge.

5.6.2 INCREASED SUPERSTRUCTURE CONTINUITY AND SUBSTRUCTURE DUCTILITY

Any method that will tend to prevent loss of support at the bearings will be useful in preventing structure collapse due to excessive soil movement. Therefore most of the methods for retrofitting bearings should be considered in a structure subjected to excessive soil movement. In addition, the ability of the substructure to absorb differental movements is important. If, for example, column shear is the critical failure mode, retrofitting methods such as cutting longitudinal reinforcing steel that will tend to make flexure the critical failure mode should be considered. This method is controversial and should be used only after a thorough investigation of all consequences. Usually retrofitting of the structures alone will not prevent the structure from being severely damaged in the event of large soil movements. Retrofitting is intended to prevent collapse and possibly provide for some restricted use of the structure immediately following an earthquake.

At a site subject to excessive liquefaction, methods to improve the structure may not be sufficient to prevent collapse unless coupled with methods to stabilize the site.

Design of retrofit methods to increase superstructure continuity and substructure ductility to mitigate liquefaction failures should conform to the requirements of this chapter for the type of retrofit measure used.

COMMENTARY

CHAPTER 1 - INTRODUCTION

C1.1 PURPOSE

It has become apparent in recent years that many bridges in the United States are inadequate to resist seismic loadings. Several bridge failures have occurred in Alaska and California as a result of seismic activity. Some of these failures occurred at relatively low levels of ground motion. Although the risk of bridge failure is lower in most other sections of the country, ground motions of sufficient magnitude to cause bridge damage have been estimated by seismologists to have a one-in-ten chance of occuring within the next 50 years at certain bridge sites in 37 of the 50 states and Puerto Rico.(1) Fifteen of these states plus Puerto Rico are subject to comparatively high levels of ground shaking. It is therefore necessary that an effort be made to identify seismically deficient bridges, evaluate the risk and consequences of seismic damage, and initiate a program for reducing the risk of seismic failure.

Seismic retrofitting of existing bridges is one method of mitigating the risk that currently exists. However, the goals and economics of retrofitting differ from those applied to new construction. The options of doing nothing and thus accepting the risk of failure, and of abandoning or replacing the bridge must also be considered. This requires that both the importance and degree of vulnerability of the structure be evaluated. The most important and/or more vulnerable bridges should be given highest priority for retrofitting.

Because of the difficulty and cost involved in strengthening an existing bridge to new design standards it is usually not economically justifiable to do so. For this reason, the goal of retrofitting is limited to preventing unacceptable failure. This allows for a considerable amount of structural damage during a major earthquake. The goal of preventing unacceptable failure requires that the collapse of the bridge be prevented. In important bridges, the ability of the bridge to carry light emergency traffic immediately following an earthquake is also important. The threshold of damage that will constitute unacceptable failure must be defined by the engineer by taking into consideration the overall configuration of the structure, the importance of the structure as a lifeline following a major earthquake, the ease with which certain types of damage can be quickly repaired, and the relationship of the bridge to other structures that may or may not be affected during the same earthquake. A decision to retrofit will be based in part on an evaluation of the likelihood of unacceptable damage due to earthquake loading. Because of the complexity of retrofitting decisions and the many nonengineering factors involved, a considerable amount of judgement will be required.

The recommendation that identified components be strengthened to new design standards may appear inconsistent with the overall goals of retrofitting and not economically justifiable if the structure as a whole will perform below the standards for new construction. There are two reasons for making this recommendation. One reason is that the cost to strengthen a component to new design standards is usually not that much greater than the cost of partial strengthening. The second reason is that it is possible that retrofitting will be a phased operation that takes place over the life of the structure. Changes in construction technologies and the economic situation may make it feasible to strengthen some components in the future even though it is not economical to do so now. If component retrofitting were performed to standards below those for new construction, it could become necessary to restrengthen these components during a second phase of retrofitting, resulting in a higher total cost.

There may be cases, however, where it is not feasible to strengthen components to new standards. In these cases, it would be preferable to strengthen components to lower standards rather than to reject retrofitting altogether. Selection of acceptable levels of strengthening requires the judgement of the engineer, taking into consideration the performance of the remainder of the structure.

There are some secondary factors that must also be considered when retrofitting. One of these is the repairability of the structure following an earthquake. If possible, component strengthening should not be done at the risk of forcing damage to other components that are more difficult to inspect and repair. For example, it would be undesirable to strengthen a ductile component if load would be transferred to a nonductile or brittle component. This would be the case even if calculations indicated an overall increase in seismic capacity.

Maintenance and inspection of retrofitted components should also be considered. Several years may pass before a structure is subjected to an earthquake, and the retrofit must be maintained to function as planned when and if the earthquake does occur.

C1.2 BACKGROUND

The report entitled "Seismic Design Guidelines for Highway Bridges" was published in 1981.[2] These guidelines, referred to in this document as the Seismic Design Guidelines, present seismic design and construction requirements applicable to the majority of new highway bridges to be constructed in the United States.

It was considered essential to the development of the Seismic Design Guidelines that representative segments of the bridge design and construction profession be involved. To ensure representative input and adequate consideration of the many factors involved, a Project Engineering Panel (PEP) comprised of members from a cross-section of the engineering community was assembled. The PEP was responsible for the development of the guidelines.

The methodology used in the Seismic Design Guidelines is based in part on the "force design" approaches employed by the California Department of Transportation[3] (CalTrans) and the New Zealand Ministry of Works.[4] Four additional concepts are included in the guidelines that are not included in either the CalTrans or New Zealand approach. These include minimum requirements for support lengths of girders, combination of elastic member forces for earthquake loading in two orthogonal directions to account for the directional uncertainty of earthquake motions, design requirements and forces for foundations intended to minimize foundation damage, and applicability to all parts of the United States.

In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) were defined. The four categories permit variation in the design requirements and analysis methods in accordance with the seismic risk associated with a particular bridge location. Bridges classified as SPC A are designed for the lowest level of seismic performance and bridges classified as SPC D for the highest level of seismic performance.

The Seismic Performance Category is determined from the Importance Classification (IC) and the Acceleration Coefficient (A).

Two Importance Classifications are specified. An IC of I is assigned for essential bridges and II for all others. Essential bridges are those that must continue to function after an earthquake. The determination of the Importance Classification of a bridge is necessarily subjective. Consideration should be given to Social/Survival and Security/Defense requirements. An additional consideration would be average annual daily traffic.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department, or public health agencies to respond to a disaster situation, a continuous route must be provided. Bridges on or crossing such routes should be classified as essential.

A basis for the evaluation of Security/Defense requirements is the 1973 Federal-Aid Highway Act, which required that a plan for defense highways be developed by each state. The defense highway network provides connecting routes to important military installations, industries, and resources not covered by the Federal-Aid primary routes. Bridges which serve as important links in the Security/Defense roadway network should be classified as essential.

The acceleration coefficients and associated elastic response spectra used in the Seismic Design Guidelines were originally developed as part of a similar and even more extensive study for buildings entitled "Tentative Provisions for the Development of Seismic Regulations for Buildings" (ATC-3-06 report).(1)

An Effective Peak Acceleration Coefficient, A_a, and an Effective Peak Velocity-Related Acceleration Coefficient, A_v, were developed in the ATC-3-06 provisions. A major policy decision by the Project Engineering Panel in the Seismic Design Guidelines was to use only the Effective Peak Velocity-Related Acceleration Coefficient, A_v, and to identify it as simply the Accleration Coefficient, A.

The recommended acceleration coefficient maps are based upon two major criteria. One criterion is that the probability of exceeding the design ground shaking should, as a goal, be 10 percent over a fifty year period regardless of location. The other criterion is that the regionalization maps should not attempt to microzone. In particular, there was no attempt to locate actual faults in the regionalization maps and variations of ground shaking over short distances (about 10 miles or less) were not considered.

An elastic analysis procedure is used to give the designer an indication of the force distribution to the structural members and to give some indication of the relative deformations. It also provides the basis for the design of the components.

Two analytical procedures are specified. Procedure 1, the single-mode spectral analysis, requires calculation of the fundamental period, T, of the bridge. A reasonable estimate of the elastic forces and displacements can be made for certain bridges by using this method. Procedure 2, the multimode spectral analysis, is the more sophisticated of the two procedures and generally requires the use of a digital computer. It is very effective for analyzing the response of any linearly elastic structure to any prescribed dynamic excitation, although it does not directly account for the phase relationships between the modes of vibration. Thus, a statistical approach (i.e., square root of the sum of the squares) is used to combine the contributions of different modes of vibration.

The actual forces and displacements in a bridge subjected to the design ground motions may be quite different from those obtained from an elastic analysis because at these high levels of excitation the bridge may respond inelastically. Response modification factors, R, are used to modify the component forces obtained from the elastic analysis. Inherent in this approach is the assumption that columns will yield when subjected to forces induced by the design ground motion and that connections and foundations are designed to accommodate the design ground motion forces with minor or no damage.

The Seismic Design Guidelines present methods for determining the forces resulting from plastic hinging (a column reaching its ultimate moment capacity) in the columns. The forces are based on the potential overstrength capacity of the materials and are valid only when the design detail requirements are satisfied. The overstrength capacity results from actual material strengths (steel yield strength, concrete compressive strength) that are greater than the minimum specified strengths. This fact must be accounted for when forces generated by yielding of the column are used as design forces.

In developing the Seismic Design Guidelines, the PEP considered design displacements to be as important as design forces because many of the loss-of-span type failures in past earthquakes have been attributed in part to relative displacement effects. The current state-of-the-art precludes an accurate determination of the differential column and abutment displacements to be expected when a bridge is subjected to an earthquake. The length of support provided at abutments, columns and mid-span joints must accommodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motions. The Seismic Design Guidelines specify minimum support lengths at abutments, piers, and hinge seats to provide for these effects. The displacements resulting from the elastic analysis should be used for design if these exceed the minimum specified values. The minimum support lengths specified are dependent on the deck length between expansion joints and the column height, since both dimensions influence one or more of the factors that cause differential displacements.

This report, "Seismic Retrofitting Guidelines for Highway Bridges," which will be referred to as the Retrofit Guidelines, is an extension of the Seismic Design Guidelines. Many of the principles just described were adapted to the analysis and design requirements for retrofitting.

C1.3 APPLICABILITY

The Retrofit Guidelines apply to the same types of bridges as do the Seismic Design Guidelines. It is estimated that this includes 85% to 95% of the existing highway bridges in the United States. Many of the concepts presented in the guidelines can be adapted to other types of bridges provided proper engineering judgement is used.

Bridges in SPC-A are exempt from these guidelines. The PEP decided that the relatively low level of seismic loading expected for these bridges was unlikely to result in unacceptable damage and therefore retrofitting was not warranted.

The requirements of these guidelines are greatly simplified for bridges in SPC B. Because the potential for unacceptable damage is less than it is for bridges in higher Seismic Performance Categories, a comprehensive seismic retrofitting program is optional. This means that preliminary screening as described in Chapter 2 is not required. When preliminary screening is performed, the screening process considers only the seismic vulnerability of the bearings and/or expansion joints, since the moderate levels of earthquake loading typical of bridges in this Seismic Performance Category are unlikely to cause unacceptable damage to other components. Regardless of whether bridges are screened, the PEP decided that the risk of unacceptable damage was great enough that seismic retrofitting should be considered whenever nonseismic rehabilitation of the bridge was planned or when obvious bearing deficiencies exist in a major structure. It should be pointed out that mobilization and traffic control costs represent a major part of the total seismic retrofitting cost, and therefore it is economical to include seismic retrofitting at the same time as nonseismic rehabilitation. Since the prevention of loss of support at the bearings is the primary concern for bridges in this Seismic Performance Category, only the bearings, expansion joints, and/or support width need to be considered when retrofitting.

Bridges in SPC-C and D are subject to the highest potential force levels during an earthquake. Because many bridges were constructed prior to modern seismic design standards, there is a great risk that these bridges will sustain unacceptable damage. Even though current practice is to concentrate on retrofitting bearings and/or expansion joints, these guidelines propose a methodology whereby all critical components can be evaluated in detail and considered for retrofitting. This will be increasingly important as more experience is gained and economical methods are developed for retrofitting other components.

C1.4 THE RETROFITTING PROCESS

Seismic retrofitting is one solution for minimizing the hazard of existing bridges that are vulnerable to serious damage during an earthquake. Because not all bridges in the highway system can be retrofitted simultaneously, the most critical bridges should be retrofitted first. The selection of bridges for retrofitting requires an appreciation for the economic, social, administrative, and practical aspects of the problem, as well as the engineering aspects. Seismic retrofitting is only one of several possible courses of action, such as bridge closure, bridge replacement, or acceptance of the risk of seismic damage. Bridge closure or replacement are usually not justified by seismic deficiency alone and will generally only be considered when other deficiencies exist. Therefore, for all practical purposes a choice must be made between retrofitting or accepting the seismic risk. This choice will depend on the importance of the bridge and on the cost and effectiveness of retrofitting.

The process of retrofitting bridges involves an assessment of a multitude of variables and requires the use of considerable judgement. The retrofitting process shown in Figure 3 of the Retrofit Guidelines involves several decisions. A discussion of these decisions is included in the following sections of this chapter.

SELECTING BRIDGES FOR RETROFIT

The Retrofit Guidelines describe a method for developing a seismic rating system. The purpose of this rating system is to prioritize all the applicable bridges on the highway system according to their need for seismic hazard mitigation. In other words, the most hazardous bridges are identified. Bridges high on the list should be investigated further to determine the benefits of retrofitting. Because the decision to retrofit depends on political, social, and economic as well as engineering factors, bridges high on the list will not necessarily be retrofitted. Similarly, there may be lower-priority bridges which will warrant immediate retrofitting.

One very important consideration that is not adequately reflected in the seismic rating system is the relationship of the bridge to other bridges on the system that may also be damaged during an earthquake. These types of considerations should be made prior to making a detailed evaluation of the seismic capacity of the bridge as described in Chapter 3 of the Retrofit Guidelines. A few examples will serve to illustrate the influence this consideration has on a decision to retrofit the bridge.

Assume that bridge A, a seismically vulnerable bridge, has a high seismic rating and is located on a major route in series with lower-priority bridges B and C, which are vulnerable to seismic loading but to a lesser degree than bridge A. This situation is shown in Figure 15. Assume that no convenient detour to this route exists. Assume also that each bridge can be economically retrofitted. Because retrofitting of the high priority bridge alone would only improve one point on the route and do nothing to prevent failure to bridges B or C, and because construction and administrative savings can be realized by retrofitting more than one bridge in a geographical area at a time, bridges B and C, although lower priority, should also be considered for retrofitting.

The opposite effect could occur if bridge B had a high priority rating but could not be economically retrofitted. Because bridge B is in series with bridges A and C, the route would be closed if bridge B were to collapse. Therefore, it may be advisable to give bridges A and C a lower retrofit priority because strengthening of these two bridges alone may not prevent closure of the route.

As another illustration, consider two bridges which have parallel functions, such as bridges D and E as shown in Figure 16. If bridge D is rated at a lower priority than bridge E, it is possible that bridge D could be more econimical to retrofit if less strenghening is required. If this is true, and the loss of the function served by the two bridges is more unacceptable than the collapse of only one of the bridges, then it might be more rational to retrofit bridge D before bridge E even though bridge E had the higher rating.

A further consideration when deciding if retrofitting is warranted is the age and condition of the bridge. It would not be rational to spend a large amount to retrofit a bridge with only five years of service life remaining. An unusually high seismic vulnerability may be a justification to accelerate closure or replacement of such a bridge.

A bridge in poor physical condition that is scheduled for nonseismic rehabilitation should be given a higher priority for seismic retrofitting, since construction savings can be realized by performing both the nonseismic and seismic work simultaneously.

The above examples do not represent all possible cases, but they do illustrate some of the principles involved in a retrofitting decision. In most cases the seismic rating system is used as a guide to making retrofitting decisions, but not as the final word. Common sense and engineering judgement will be necessary in weighing the actual costs and benefits of retrofitting against the risks of doing nothing. Also, the effect on the entire highway system must be kept in mind.

SELECTING RETROFITTING METHODS

The capacity/demand ratio for each potentially vulnerable bridge component should be calculated to identify the weakest points in the structure. The consequences of a local failure at each of these points should then be assessed in terms of the effect

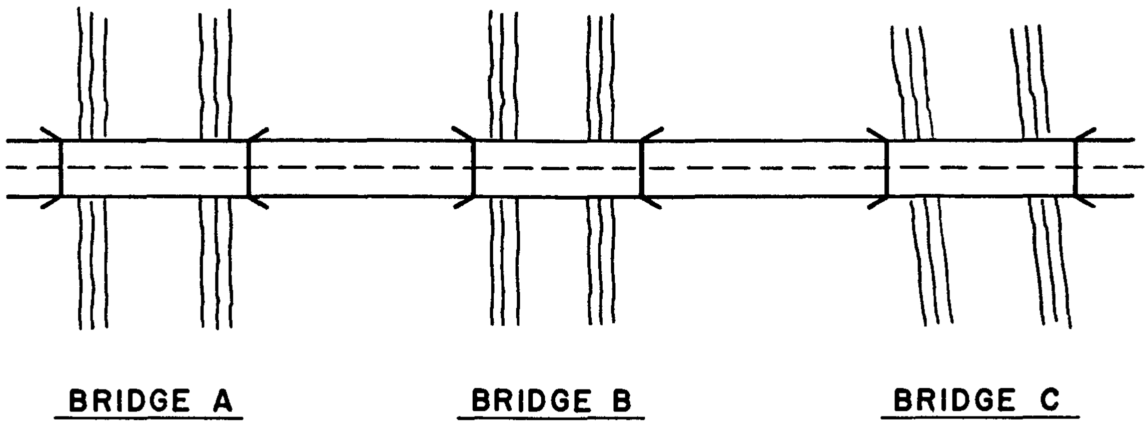


FIGURE 15. BRIDGES IN SERIES

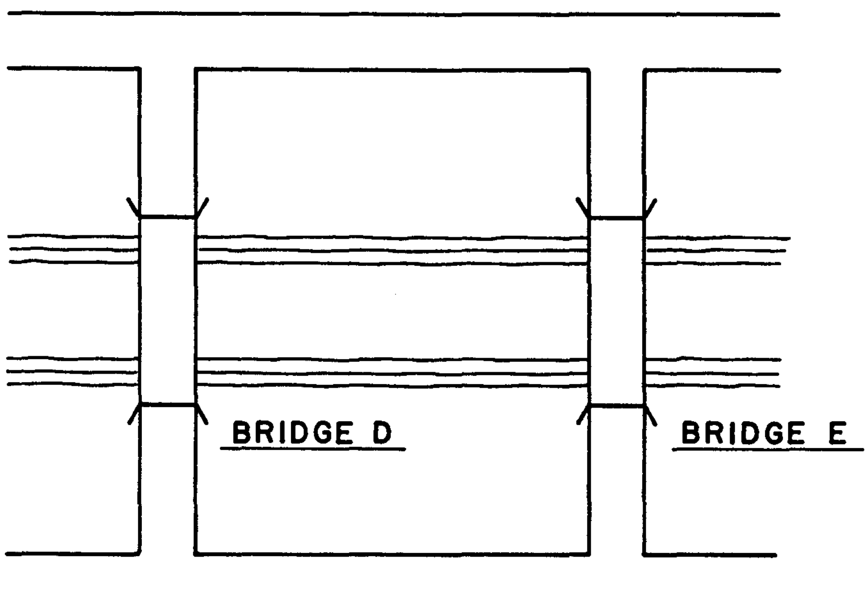


FIGURE 16. BRIDGES IN PARALLEL

on the global performance of the bridge. If local failure has a minor effect, then retrofitting methods directed toward preventing such a local failure will usually be rejected. If local failure has serious consequences, then retrofitting to prevent this failure should be considered. Several methods for retrofitting various bridge components are presented in Chapter 5 of the Retrofit Guidelines.

The determination of what constitutes a serious consequence of component failure will depend on the importance of the bridge. Collapse of the structure is serious in almost all cases since there is always a potental for loss of life in such an occurrence. In other cases, severe distortions or critical loss of strength will impair the ability of the bridge to carry light emergency traffic. This will be unacceptable for certain important bridges. Repairability of seismic damage is also a consideration. If repairs can be made quickly without serious delays to traffic, damage may be acceptable. This is an area in which engineering judgement is required.

Once it has been determined to consider retrofitting, an acceptable method may be selected from those suggested in Chapter 5 of the Retrofit Guidelines. If the seismic response of the structure is affected, then a reanalysis should be performed and new component capacity/demand ratios calculated. The new capacity/demand ratios will reflect a change in the size of the earthquake that will cause serious damage. A decision to use a retrofitting method will be based on a relative benefit-to-cost analysis. Hypothetically this benefit-to-cost analysis may be objective and rigorous, but it is more likely that it will be subjective and based on judgement. The following section discusses this in more detail.

EVALUATING THE EFFECTIVENESS AND ECONOMY OF RETROFITTING

Ideally, retrofitting should be performed to minimize the probability of unacceptable damage during an earthquake. Therefore a relative benefit-to-cost ratio equation could hypothetically be written as follows:

$$BCR = \frac{Loss_B - Loss_R}{\text{Retrofitting Cost}}, \qquad (C1\text{-}1)$$

where

BCR = Benefit-to-Cost Ratio, or the probable reduction in damage-losses per retrofitting dollar spent.

$Loss_B$ = Probable loss before retrofitting.

$Loss_R$ = Probable loss after retrofitting.

In a bridge structure, the unacceptable damage is a function of the importance of the bridge. Since bridge failure will result in losses, the probable losses are linearly related to the probability of an earthquake which will cause failure. By multiplying the probability of occurrence of this earthquake by the losses in the event of the earthquake, the probable losses may be determined. The methodology presented in the Retrofit Guidelines provides for the determination of capacity/demand ratios which can be used to determine the size of the damaging earthquake. By determining the probability of occurrence of this earthquake, the benefit-to-cost ratio of retrofitting could be evaluated. Although this highly theoretical approach is useful for conceptualizing retrofitting goals, it presents many practical problems because the variables are so difficult to define. Even if realistic benefit-to-cost ratios could be calculated, acceptable values must be determined by engineering judgement and experience with retrofitting.

Because assessment of benefit-to-cost ratios relies heavily on engineering judgement, it is presently impractical to calculate them. Currently acceptable practice uses a subjective assessment of benefits and costs based on past experience and engineering judgement. For example, the California Department of Transportation currently directs its retrofitting program toward to the installation of bearing and expansion joint restrainer devices, since these devices are perceived to provide the greatest benefit in preventing collapse for the least cost.

As of this writing, the retrofitting of bridge bearings and expansion joints with restraining devices has proved to be the most feasible. Most of these devices are relatively simple to install and cost only a very small percentage of the replacement value of the structure. Retrofitting of other components such as columns, footings and abutments has been proposed but has not been tried. Construction procedures for retrofitting these components are much more involved and would presumably be correspondingly more expensive. In addition, in many cases the sudden collapse of a bridge appears to be less likely due to deficiencies in these components alone. Therefore the retative benefit-to-cost ratio of retrofitting these components is not perceived to be as high as for bearing and expansion joint restrainers.

Because retrofitting may be new to construction personnel, it will be beneficial to standardize details as much as possible. This will eventually result in workmen becoming more familiar with construction techniques associated with retrofitting and will result in more efficient construction.

As with all construction associated with existing highway facilities, the disruption to traffic is an important consideration. Traffic control can add significantly to construction costs. It is important to select retrofit details and construction practices that will minimize these costs.

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COMMENTARY

CHAPTER 2 - PRELIMINARY SCREENING OF BRIDGES FOR DETAILED EVALUATION

C2.1 GENERAL

The Retrofit Guidelines recommend that only bridges classified as SPC-C and D be subjected to a preliminary screening. Screening of bridges in SPC-B is optional. This was done because there was a considerable difference of opinion regarding the need for screening of SPC-B bridges. Proponents for SPC-B screening argue that certain very vulnerable bridges may be subject to collapse even in areas of relatively low seismic activity. They point to the ease of screening when only the bearing vulnerability is considered as is the case for SPC-B bridges. Opponents to SPC-B screening point to the lack of an historical seismic bridge failure in areas of relatively low seismicity and argue that it would be an unwarranted hardship to require all SPC-B bridges to be screened.

Each bridge agency with bridges in SPC-B must weigh these arguments and decide for themselves whether preliminary screening should be carried out. Preliminary screening will usually be desirable if the agency in question has chosen to implement a comprehensive retrofitting program with the objective of upgrading the entire highway system to resist seismic action, or at the time of conducting a non-seismic bridge rehabilitation program.

C2.2 SEISMIC INVENTORY OF BRIDGES

Preliminary screening of seismically vulnerable bridges should be carried out efficiently and with a minimum of effort. The first step in this process is to accumulate critical information about each applicable bridge on the highway system. This information and the results of the seismic rating should be concisely organized and incorporated into the bridge records. The form shown in Figure 17 is suggested as one possible means of collecting and recording this information. This completed form should be incorporated into the existing bridge records.

C2.3 SEISMIC RATING SYSTEM

Although numerical ratings based on a few selected parameters are rarely a totally satisfactory means for determining the priority of needs, they provide a systematic way of considering the major variables involved in any decision. In the case of seismic retrofitting of bridges, there are three major variables that should be considered. These include the vulnerability of the structural system, the seismicity of the bridge site, and the importance of the bridge. The proposed Seismic Rating System addresses each of these variables separately by requiring that vulnerability, seismicity, and importance ratings be calculated for each bridge. These individual ratings are combined to arrive at an overall seismic rating.

In developing the Seismic Rating System, both addition and multiplication were considered as potential methods for combining the vulnerability, seismicity, and importance ratings. Both methods result in high ratings for bridges which are obvious

BRIDGE SEISMIC INVENTORY DATA

GENERAL:
Bridge Name: ____________________
Location: ____________________
ADT: ________ Detour Length: ______ Essential Bridge: Yes__ No__
Alignment: Straight___ Skewed___ Curved___ Remarks__________
Length: ______
Width: ______
Year Built: ______
Seismically Retrofitted: Yes___ No___ Description: __________
Classification: Regular___ Irregular___ Remarks: __________

SITE:
Peak Acceleration: ______
Soil Profile Type: I___ II___ III___
Liquefaction Potential: Yes___ No___

SUPERSTRUCTURE:
Material and Type: ____________________
Number of Spans: ___
Continuous: Yes___ No___ Number of Expansion Joints: ___

BEARINGS:
Type: ____________________
Condition: Functioning___ Not Functioning___
Type of Restraint (Trans): ____________________
Type of Restraint (Longit): ____________________
Actual Support Length: _____ Minimum Required Support Length : ______
Remarks: ____________________

COLUMNS AND PIERS:
Material and Type: ____________________
Minimum Transverse Cross-Section Dimension: __________
Minimum Longitudinal Cross-Section Dimension: __________
Height Range: _____ Fixity: Top _____ Bottom _____
Percentage of Longitudinal Reinforcement: __________
Splices in Longitudinal Reinforcement at End Zones: Yes___ No___
Transverse Confinement: ______ Conforms to Design Guideline: Yes___ No___
Foundation Type: ____________________

ABUTMENTS:
Type: ____________________
Height: ________
Foundation type: ________ Location: Cut______ Fill______
Wingwalls: Continous___ Discontinous___ Length___
Approach Slabs: Yes ___ No ___ Length: __________

SEISMIC RATINGS:
Vulnerability Rating:
Bearings: ______ Other: _____
Highest Rating: _____ Weight: _____ Score: _____
Importance Rating: _____ Weight: _____ Score: _____
Seismicity Rating: _____ Weight: _____ Score: _____
Total Seismic Rating _____

FIGURE 17. BRIDGE SEISMIC INVENTORY FORM

candidates for seismic retrofitting (i.e., important bridges with vulnerable structural systems located in regions of high seismicity). However, these two methods result in some minor differences in the relative priorities derived for bridges with less need for retrofitting.

An advantage of addition as a method for combining ratings is that each of the three ratings can be assigned relative weights. When relative weights are adjusted within reasonable limits, very little difference will result in the relative priorities of the bridges in the greatest need of retrofitting. This also applies to bridges with little or no need for seismic retrofitting. However, an adjustment in relative priorities will result for bridges in moderate need of retrofitting. An examination of the types of bridges affected by varying the relative weights of individual ratings reveals that these bridges lie in a "grey zone" where subjective judgement should play a much greater role in assigning priorities. Some of the types of bridges affected include the following:

- An important bridge with a highly vulnerable structural system located in a region of moderate seismicity.

- A bridge with a highly vulnerable structural system located on a route of moderate importance in a region of high seismicity.

- An important bridge with a moderately vulnerable structural system located in a region of high seismicity.

There may be considerable disagreement as to which of these three types of bridges should be retrofitted first. For most jurisdictions, these might be the types of bridges that would be retrofitted in the second or third phase of a comprehensive retrofitting program. Therefore, the ability to adjust the relative weights of the individual ratings allows the jurisdiction some flexibility in selecting the types of bridges they want to retrofit in the later phases of a retrofitting program in order to meet their particular needs or preferences.

The Retrofit Guidelines suggest that equal weight be assigned to each of the three individual ratings. This would result in equal priorities for each of the three bridges described above. If an adjustment in these priorities is desired, the weights should be adjusted accordingly. As a guide, weights should vary between 2 and 4. If one of the variables does not influence priorities, its weight may be set equal to zero and the remaining two weights raised to equal a total of 10. For example, such a case would occur when all applicable bridges are located in regions of equal seismicity. In this case the weight for seismicity would be set equal to zero and the weights for vulnerability and importance would be increased to equal a total of 10.

The Seismic Rating System will result in priority lists that should be used as a guide for selecting bridges for detailed evaluation. Although these priority lists are helpful, they will not reflect several important considerations in retrofitting bridges. It is not necessary that bridges be selected from the top of the list if there is sufficient cause to do otherwise.

C2.3.1 VULNERABILITY RATING

Although the performance of a bridge is based on the interaction of all its components, it has been noticed in past earthquakes that certain bridge components are most vulnerable to damage. These are the bearings; columns, piers, and footings; abutments; and foundations (liquefaction damage). Of these, bearings seem to be the most economincally retrofitted. For this reason the vulnerability rating to be used in the seismic rating system is determined by examining the bearings separately from the remainder of the structure. The vulnerability ratings of the remainder of the structure may be determined as the greatest of the vulnerability ratings for each of the other components which are vulnerable to failure. The remainder of this section is devoted to a discussion of the vulnerable features of each of these components and methods for calculating their vulnerability rating.

A. Bearings

Bearings are used at superstructure/substructure interfaces as well as at in-span joints. For the purpose of this discussion, bearings are considered to include restraints provided at these locations, including shear keys, restrainer bars, etc. Bearings may be "fixed" bearings, which do not provide for translational movement, or expansion bearings, which do. A bearing may provide for translation in one orthogonal direction but not in the other.

There are basically four types of bearings used in bridge construction. One type, the rocker bearing, is generally constructed of steel and rolls on a curved surface to provide for translation and/or rotational movement. It is the most seismically vulnerable of bridge bearings because it usually has a large vertical dimension, is difficult to restrain, and can become unstable after a limited movement. Another type of bearing, the roller bearing, is also usually constructed of steel. It is stable during an earthquake, except that it can become misaligned and horizontally displaced. The third type is the elastromeric bearing pad, which has become very popular in recent years. It is constructed of natural or synthetic elastomer and relies on the distortion of this material to provide for movement. It is very stable during an earthquake, although it has been known to "walk out" under severe shaking. The final bearing type is the sliding bearing which relies on the sliding of one surface over another and may consist of anything from asbestos sheet packing between two concrete surfaces to sophisticated TFE ("Teflon") and stainless-steel bearings.

Transverse restraint of some type is almost always provided at the bearings. Common types of restraint are concrete shear keys, keeper plates, or anchor bolts. They are usually not ductile, and are subjected to large seismically induced forces resulting from a redistribution of force from ductile components such as columns. In addition, when several individual bearings with keeper bars are present at a support, the keeper bars do not resist load equally because of slight variations in clearances. Therefore, individual keeper bars are subjected to very high forces. In vulnerable structures, serious failure usually is due to loss of support resulting from large relative transverse or longitudinal movement at the bearings. The expected movement at a bearing is dependent on many factors and cannot be easily analyzed. The Seismic Design Guidelines[1] require a minimum support length at all bearings in newly constructed bridges. Because it is very difficult to predict relative movement, the minimum support length formula, which represents an upper bound for movement at the bearing, may be used as the basis for checking the adequacy of longitudinal support lengths.

Support skew has a major effect on the performance of bridge bearings. Notice that for these guidelines, skew is defined as the angle between the support centerline and a line perpendicular to the bridge centerline. Rocker bearings have proved to be the most vulnerable in past earthquakes. At highly skewed supports these bearings may topple during moderate seismic shaking. When bearings may topple, it is necessary to consider the potential for collapse of the span. The potential for collapse will depend on the geometry of the bearing seat. Settlement of a span due to a toppled bearing may be assumed to be a minor problem resulting in a temporary loss of access which is easily solved by ramping with asphalt or other similar fill. The potential for total loss of support should be the primary criteria for rating the vulnerability of the bearings.

A suggested step-by-step method for determining the vulnerability rating of the bearings follows:

Step 1: Determine if the bridge has nonvulnerable bearing details. These bridges would include:

a. Continuous structures with integral abutments.

b. Continuous structures with seat-type abutments where all of the following conditions are met:

(1) The skew is less than 20° (.35 rad), or the skew is greater than 20° (.35 rad) but less than 40° (.70 rad) and the length-to-width ratio of the bridge deck is less 1.5.

(2) Rocker bearings are not used.

(3) The bearing seat on the abutment end diaphram is continuous in the transverse direction and the bridge has in excess of three girders.

(4) The support length is equal to or greater than one half the minimum required support length (Section 4.6)

If the bearing details are nonvulnerable, a vulnerability rating of 0 may be assigned and the remaining steps for bearings omitted.

Step 2: Determine the vulnerability to structure collapse or loss of bridge access due to transverse movement.

Before significant transverse movement can occur, the transverse restraint must fail. Nominal bearing keeper bars or anchor bolts should be considered subject to failure for bridges in SPC-C and D. Nominally reinforced, nonductile concrete shear keys should be considered subject to failure for bridges in SPC-D only.

When transverse restraint is subject to failure, girders are vulnerable to collapse if either of the following conditions exist:

a. Individual girders are supported on individual pedestals or columns.

b. The exterior girder in a 2- or 3-girder bridge is near the edge of a continuous-bearing support.

In either of these cases, the vulnerability rating should be 10.

Steel rocker bearings have been known to topple, resulting in a partial superstructure displacement. All bridges assigned to SPC-D are vulnerable to this type of failure. Bridges assigned to SPC-C are vulnerable only when the support skew is greater than 40^{o} (.70 rad). When bearings are vulnerable to a toppling failure but structure collapse is unlikely, the vulneribility rating should be 5.

Step 3: Determine the vulnerability of the structure to collapse or loss of accessability due to excessive longitudinal movement.

If the longitudinal support length measured in a direction perpendicular to the support is less than one, but greater than half the required longitudinal support length, the vulnerability rating shall be assigned a value of 5 unless in addition rocker bearings are vulnerable to toppling, in which case a value of 10 should be used. If the longitudinal support length is less than half the required longitudinal support length, then a vulnerability rating of 10 should be assigned.

B. Columns, Piers, and Footings

Columns have failed in past earthquakes due to lack of proper transverse reinforcement and poor structural details. Excessive ductility demands have resulted in degradation of column strength in shear and flexure. In several serious failures in past earthquakes, columns have failed in shear resulting in severe vertical settlements or total column disintegration. Another serious type of column failure resulted from longitudinal reinforcing steel pullout at the footings. Fortunately, serious bridge column failures only occur during earthquakes with fairly high ground accelerations of relatively long duration. The following step-by-step procedure may be used to determine the vulnerability of the columns, piers, and footings.

Step 1: Assign a column and footing vulnerability rating of 0 to bridges classified in SPC-B and to those bridges in SPC-C having an acceleration coefficient A less than 0.29.

Step 2: Assign a vulnerability rating of 0 if bearing keeper plates or anchor bolts are assumed to fail, eliminating the transfer of load to columns, piers, or footings.

Step 3: If columns and footings have adequate transverse steel as required by the Seismic Design Guidelines, assign a column vulnerability rating of 0.

Step 4: Calculate the Base Vulnerability Rating, BVR, which is an indicator of the vulnerability of a column to a sudden shear failure. The base rating shall be assigned as follows:

$$BVR = 13-6\left(\frac{L_c}{P_s Fb_{max}}\right), \qquad (C2-2)$$

where

L_c = Effective column length in feet.

P_s = Percent main reinforcing steel.

F = Framing factor:

= 2 (multi-column bents fixed top and bottom).

= 1 (multi-column bents fixed at one end).

= 1.5 (single-column bent fixed at top and bottom—box girder).

= 1.25 (single-column bent fixed at top and bottom—other than box girder).

b_{max} = Transverse column dimension (feet).

The column vulnerability rating, CVR, will be between 0 and 10 and will be taken as the BVR minus the points shown for each of the following conditions up to a maximum of 4 points unless larger CVRs are calculated in Steps 5 and 6.

a. $A < 0.4$ (3 points) (Notice that this will always be the case if the maps in Figures 1 and 2 are used. Only when microzoning is considered will a reduction of 3 points not be in order).

b. Right structure—skew $\leq 20^{o}$ (.35 rad) (2 points).

c. Continuous structures with diaphragm abutments of approximately equal stiffness in which the length-to-width ratio of the deck is less than 4 (1 point).

d. Grade 40 (or below) reinforcement (1 point).

Values of CVR less than zero or greater than 10 should be assigned values of 0 and 10, respectfully.

Step 5: To account for column flexural failure at a splice, the following CVR should be calculated for single-column bents supporting superstructures in excess of 300 feet in length or superstructure with expansion joints where the column longitudinal reinforcement is spliced at a potential plastic hinge location:

a. $A \leq 0.4$, CVR = 7.

b. $A > 0.4$, CVR = 10 (only when microzoning is considered).

Step 6: The following CVR should be calculated for single-column bents supported on pile footings unreinforced for uplift, or poorly confined foundation shafts. This step is only applicable if microzoning yields values of A greater than 0.40:

$0.40 \leq A < 0.50$ CVR = 5.

$0.50 < A$ CVR = 10.

C. Abutments

Abutment failures during earthquakes do not usually result in total collapse of the bridge. This is especially true for earthquakes of low-to-moderate intensity. Therefore, the abutment vulnerability rating should be based on damage that would temporarily prevent access to the bridge.

FIGURE 18. ABUTMENT AND APPROACH FILL SETTLEMENT—SAN FERNANDO EARTHQUAKE, 1971

One of the major problems observed in past earthquakes has been the settlement of fill at the abutment as shown in Figure 18. Elms[2] reports that in past earthquakes in New Zealand and New Guinea these settlements have been on the order of 10-15% of the fill height. However, observations of damage during the San Fernando and other California earthquakes suggest far less settlement. Bridges within the damage area of the San Fernando earthquake experienced average fill settlements on the order of 3-5%. This was assumed to be due to superior construction of fills, the absence of water, and the generally wider and better retained bridge approaches.

Additional abutment fill settlements are possible in the event of abutment failures due to excessive seismic earth pressures or seismic forces transferred from the superstructure. Certain abutment types are more vulnerable to this type of damage than others. Except in unusual cases, the maximum abutment vulnerability rating will be 5.

The following step-by-step procedure for determining the vulnerability rating for the abutments is based on engineering judgement and the performance of abutments in past earthquakes.

Step 1: If bridges are classified as SPC-B, assign a vulnerability rating of 0.

Step 2: Determine the vulnerability of the structure to abutment fill settlement. The fill settlement in normally compacted approach fills may be estimated as follows:

a. One percent of the fill height when $0.19 < A \leq 0.29$.

b. Two percent of the fill height when $0.29 < A \leq 0.39$.

c. Three percent of the fill height when $A > 0.39$.

The above settlements should be doubled if the bridge is a water crossing. When fill settlements are estimated to be greater than six inches, assign a vulnerability rating for the abutment of 5.

Step 3: For bridges classified as SPC-D, free-standing, earth-retaining abutments with skews greater than 40° (.70 rad) where the distance between the seat and the bottom of the foundation footing exceeds 10 feet (3.05 meters) should be assigned a vulnerability rating of 5.

D. Liquefaction

Although there are several possible types of ground instabilities that can result in bridge damage during an earthquake, ground instability resulting from liquefaction is the most significant. The vulnerability rating for foundation soil is therefore based on:

a. a quantative assessment of liquefaction susceptibility,

b. the magnitude of the acceleration coefficient and,

c. an assessment of the susceptibility of the bridge structure itself to damage resulting from liquefaction-induced ground movement.

The vulnerabilty of different types of bridge structures to liquefacton has been illustrated by failures during past earthquakes such as the 1964 Alaskan earthquake, as reported by Ross et al.[3] and various Japanese earthquakes, as reported by Iwasaki, et al.[4] The observed damage has demonstrated that bridges with continuous superstructures and supports can withstand large translational deformations and usually remain serviceable (with minor repairs). However, bridges with discontinuous superstructures and/or brittle supporting members are usually severly damaged as a result of liquefaction. These observations have been taken into account in developing the vulnerability rating procedure described below.

The procedure is based on the following steps:

Step 1: Determine the susceptibility of foundation soils to liquefaction.

High susceptibility is associated with conditions where:

a. foundation soil providing lateral support to piles or vertical support to footings comprise on average saturated loose sands, silty sands, non-plastic silts, and

b. where similiar soils underly abutment fills or are present as continuous seams which could lead to abutment slope failures.

Moderate susceptibility is associated with similar conditions where average soil conditions may be described as medium dense.

Low susceptibility is associated with dense soils.

Step 2: Determine the potential extent of liquefaction related damage where susceptible soil conditions exist:

a. Severe liquefaction related damage is likely to occur for conditions of high susceptibility when $A > 0.29$ and for conditions of moderate susceptibility when $A > 0.4$.

b. Major liquefaction related damage is likely to occur for conditions of high susceptibility when $0.19 < A \leq 0.29$. or for conditions of moderate susceptibility when $0.29 < A \leq 0.39$.

c. Moderate liquefaction related damage is likely to occur for conditions of high susceptibility for $0.09 < A \leq 0.19$, and for conditions of moderate susceptibility where $0.19 < A \leq 0.29$.

d. Low liquefaction related damage is likely to occur for conditions of low susceptibility.

For sites where ($A > 0.40$), engineering judgement should be applied to determine the possibility of greater damage.

Step 3: In general, bridges subjected to severe liquefaction related damage shall be assigned a vulnerability rating of 10. This rating may be reduced to 5 for single-span bridges with skews less than 20° (.35 rad) or rigid box culverts with floors.

Step 4: Bridges subjected to major liquefaction related damage shall be assigned a vulnerability rating of 10. This rating may be reduced to between 5 and 9 for single-span bridges with skew less than 40°, (.70 rad) rigid box culverts with floors, and continuous multi-span bridges with skew less than 20° (.35 rad) provided one of the following conditions exists:

a. Reinforced concrete columns are continuous with the superstructure and have a CVR less than 5 and a height in excess of 25 feet (7.63 meters).

b. Steel columns (except those constructed of brittle material) are in excess of 25 feet high (7.63 meters).

c. Columns are discontinuous with the superstructure and shifting of the superstructure will not result in instability.

Step 5: Bridges subjected to moderate liquefaction related damage should have a vulnerability rating of 5. This rating should be increased to between 6 and 10 if the vulnerability rating for the bearings is greater than or equal to 5.

C2.3.2 SEISMICITY RATING

The formula for the seismicity rating comes from the fact that 25 times the maximum acceleration coefficient from the maps in Figures 1 and 2 yields a maximum rating of 10. If microzoning has been carried out within a jurisdiction, that jurisdiction may wish to modify the seismicity rating to yield a value of 10 at the maximum acceleration coefficient obtained from microzoning.

C2.3.3 IMPORTANCE RATING

The Seismic Design Guidelines (1) provide for two importance classifications (IC). The first (IC = I) is for bridges defined as essential based on Social/Survial and Security/Defense requirements. The second classification (IC = II) is for all other bridges. For guidance in determining which of these two classifications should be assigned, refer to the Seismic Design Guidelines and Commentary.

In the case of seismic retrofitting, it is usually necessary to allocate limited funds and manpower resources according to need. Although two importance classifications are adequate for the purposes of determining design and construction requirements, there is clearly a difference in the importance of bridges that fall within each of these classifications. It is necessary to consider this fact when determining the priority of bridges for retrofitting. The seismic rating system allows for this by allowing importance ratings to vary from 0 to 10. Bridges classified as essential (IC = I) may be assigned ratings between 6 and 10, while bridges classified as nonessential (IC = II) may have ratings between 0 and 5. As with the determination of importance classifications, the determination of the importance rating is subjective. Since the goal of retrofitting is to minimize unacceptable damage, the relative importance of a bridge is determined by considering the consequences of bridge failure during an earthquake.

Immediate consequences will result from the collapse of the bridge. In this event, the loss of life among individuals on or under the bridge is likely to be high. One factor which will affect the loss of life is the amount of traffic on or under the bridge at the time of the earthquake. This is likely to increase with the amount of traffic that crosses a given point during a period of time (e.g., average daily traffic) and the physical size of the bridge (e.g., length, number of lanes, etc.). Another factor that should be considered is the presence of the other facilities (e.g., buildings on, under, or near the bridge) that could be damaged or destroyed by the collapsing bridge.

Other consequences of failure result from the loss of use of the bridge in the emergency situation that is likely to exist following a large earthquake. This is sometimes very difficult to assess, because there are so many possible situations that may develop in the aftermath of an earthquake. Some of the items that should be considered are discussed in the following paragraphs.

The seriousness of the emergency situation likely to result following an earthquake should be evaluated. This will depend on the size of the earthquake and the facilities and people in the area likely to be affected. Because earthquake size is reflected by the seismicity rating, it should not be duplicated in the importance rating. However, the type and number of surrounding facilities should be considered. A factor that may reflect this is population density near the bridge site. This should include temporary population such as would occur in a business district. High population densities imply a concentration of people in a large number of large buildings and thus indicate a much larger potential casualty rate in the event of an earthquake. The proximity of the bridge to special types of facilities such as dams or nuclear power plants, whose failures have far-reaching consequences necessitating rapid evacuation, should also be considered in determining the importance rating.

Another item to be considered is the type of function the bridge is likely to perform following a major earthquake. Some examples of important functions are:

- Primary route for special emergency traffic such as ambulances or firefighting equipment.
- Support for special utilities such as major water, gas, power, or communication lines.
- Major evacuation route.
- Access to other critical facilities.

A final item to be considered is the presence of alternate bridges or routes and the likelihood that these facilities may also be damaged. If, for example an overpass can be bypassed by using the off- and on-ramps, then a relatively convenient and reliable detour exists in the event of a bridge collapse. If, on the other hand, the structure in question is a critcal river crossing, the nearest detour may be several miles away, and in the event of a large earthquake it too may be seriously damaged. In this case a greater importance rating should be given to the river crossing.

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COMMENTARY

CHAPTER 3 - PROCEDURE FOR DETAILED EVALUATION OF AN EXISTING OF BRIDGE

C3.1 GENERAL

The selection of the best retrofitting technique for an existing bridge requires that the potential seismic weaknesses of the bridge be evaluated in detail. Retrofitting techniques that will eliminate these weaknesses must be identified and assessed for their feasibility and effectiveness. Chapter 3 of the Retrofit Guidelines outlines a procedure for evaluating existing bridges for seismic retrofitting.

C3.2 REVIEW OF BRIDGE RECORDS

Most agencies maintain a file of "as-built" bridge plans plus a bridge maintenance file with bridge inspection reports and information about major repairs or modifications to the bridge. This information is generally readily available and very useful when evaluating a bridge. Additional information may also be obtained from the original design calculations and construction records, although these documents are sometimes more difficult to obtain. Bridge rating calculations to determine live load capacity may also contain useful information about the condition and strength of the materials used to construct the bridge.

C3.3 SITE INSPECTION

Current federal legislation requires that most bridges be inspected biennially as part of the National Bridge Inspection Standards. In general, these inspections are designed to monitor deterioration of the structure as it may effect the live load rating and are not specifically directed toward seismic evaluation. It will usually be wise to make a separate inspection of the bridge site to detect seismically vulnerable conditions, or redirect maintenance personnel to monitor those conditions in their routine maintenance inspections.

C3.4 QUANTITATIVE EVALUATION OF BRIDGE COMPONENTS

The seismic capacity/demand ratio is defined as the ratio of the available capacity of the bridge for seismic loading to the demand (required capacity) for the seismic design loading at the site. The limiting available capacity is generally assumed to be one or more of the following:

- The displacement at expansion joints that will result in a total loss of support and collapse of the bridge.
- The ultimate force capacity of fixed bearings.
- The ductile capacity of columns, piers, and foundations beyond which unacceptable strength degradation can occur.
- Abutment displacements which could result in the bridge becoming inaccessible following an earthquake.

- Foundation movements which are excessive and will result in a collapse of the structure or loss of bridge accessibility.

The basic equation for determining the seismic capacity/demand ratio, r, is:

$$r = \frac{R_c - \Sigma Q_i}{Q_{EQ}}\ , \qquad \text{(C3-1)}$$

where

R_c = The nominal ultimate displacement or force capacity for the structural component being evaluated.

ΣQ_i = The sum of the displacement or force demands for loads other than earthquake which are included in the group loading defined by Equation 4-1 of the Seismic Design Guidelines.

Q_{EQ} = The displacement or force demand for design earthquake loading at the site.

Capacity/demand ratios should be calculated at the nominal ultimate capacity without the use of capacity reduction (ϕ) factors to account for possible understrength and/or undersize members. This is done because the objective of capacity/demand ratios is to determine the most likely level of failure.

Capacity/demand ratios are intended to be the ratio of the effective peak ground acceleration for a damaging earthquake to the design acceleration coefficient for the bridge site. Since these ratios reflect only component failures, they must be assessed in terms of the global effect of the failure or a cumulative number of failures. They can be used to evaluate the need for retrofitting or the effectiveness of various retrofitting techniques.

C3.5 IDENTIFICATION AND ASSESSMENT OF POTENTIAL RETROFIT MEASURES

Component capacity/demand ratios below 1.0 indicate that a localized failure is likely to occur which will cause damage resulting in a loss of structural strength or bridge accessibility. This damage, although undesirable, may not necessarily be unacceptable. The engineer must assess the consequences of local component failure on the global stability of the structure. A final decision about what to do with a deficient bridge will depend in part on this assessment and the level of seismic shaking at which unacceptable damage is likely.

If it is established that failure of a particular bridge component will have severe consequences and the component has a capacity/demand ratio less than one, then retrofitting should be considered. In certain cases it may not be economically feasible to retrofit all bridges or components that have substandard capacity/demand ratios. In these cases the values of the capacity/demand ratios for each component, and the global consequences of component failure, must be considered along with the importance of the structure and the cost of various retrofitting measures when making a decision of whether to retrofit or abandon the bridge, or to accept the risk of seismic failure. A discussion of the philosophy behind retrofitting decision-making is included in Section C1.4. The following paragraphs discuss some items that should be considered in assessing the consequences of failure in various components.

BEARINGS AND EXPANSION JOINTS

Displacement

The capacity/demand ratio for the relative displacement at unrestrained expansion joints is intended to reflect the reduced level of loading at which a loss of support failure may occur. Usually a loss of support failure results in a collapse of the span. In certain bridges with continuous superstructures, however, the bridge may still be capable of resisting the dead load moments and shears resulting from a loss of support at the expansion joint. This is often the case in reinforced concrete slab bridges. Although a structure which has failed in this manner is not capable of carrying traffic loadings, it is possible that following a major earthquake it will be inspected and the expansion joint failure discovered. Traffic can then be diverted or measures taken to shore up the unseated bearings. Although there is a risk that traffic will utilize the bridge before the discovery of the bearing failure, this risk can be accepted in some cases.

Conversely, certain structural configurations are exceptionally vulnerable to collapse in the event of a loss of support at the bearings. Such structures would be prime candidates for retrofitting. Simple or suspended spans in which no redundancy exists are particularly vulnerable. This is also true in the case of a structure with a small amount of redundancy, such as a continuous bridge in which only one support occurs between expansion joints.

Another factor that should be considered in assessing the consequences of loss of bearing support is the distance the spans will fall in case of collapse. If a structure simply comes off its bearings and drops a few inches, this is usually not critical. The slight vertical offset in the roadway can easily be bridged by emergency maintenance crews. The facility passing under a structure is also important. If a structure crosses a busily traveled roadway or railroad, its collapse is more unacceptable than a structure crossing a small stream.

Forces

A failure of bearing anchor bolts, keeper bars, or shear keys is usually not an unacceptable failure. If such a failure could result in relative displacements sufficient to cause a loss of support at the bearings, then the consequences of that failure must be assessed.

The loss of support of an edge girder due to transverse movement may render a portion of the superstructure unusable but will not result in a structure collapse, except possibly in a bridge with only two main girders or trusses. It will still be possible in most cases to utilize the remaining portion of the superstructure. In other cases, solid diaphrams between girders may prevent a total collapse of the span. If this results in some vertical displacement in the roadway, it can usually be bridged quickly by maintenance crews.

COLUMNS, PIERS, AND FOOTINGS

Bridge columns will almost always yield during strong seismic shaking. This is expected and provided for in the design of new structures. However, in existing structures the bridge may not be capable of withstanding as much yielding. Column failure may occur in any one of several modes. Column failures that have the potential for causing structure collapse are those that result in a sudden loss of flexural or shear strength. The force levels at which these local failures occur are reflected in the capacity/demand ratios for the various column failure modes. Each of these failures must be assessed in terms of its effect on the global stability of the structure. The cumulative effect of column failures elsewhere in the structure should also be considered in making this assessment.

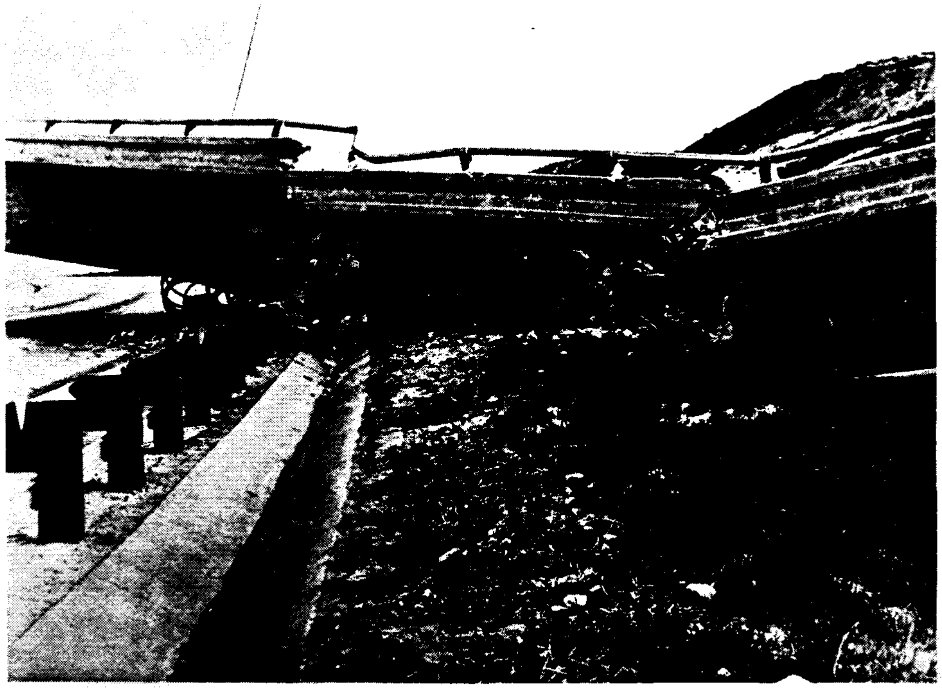
The loss of flexural strength in a column can result from an anchorage failure in the main reinforcing steel at the footing or the bent cap, a failure of splices in the main reinforcement, or a loss of transverse confinement followed by crushing of concrete and buckling of the main reinforcing steel. The capacity/demand ratios for each of these failure modes are calculated when evaluating a bridge.

Column shear failure occurs rather suddenly and can potentially result in a collapse of the bridge. The ability of the column to resist lateral loads will be lost. In addition, a column which has failed in shear will be rapidly pulverized by continued shaking. In columns which fail in shear at low force levels, the continuing degradation of the column in shear can result in an eventual structure collapse, as occurred in the case of the Route 5 (Truck Lane)/405 Separation during the San Fernando earthquake of 1971. This failure is shown in Figure 19. Because this type of failure will occur progressively over several seconds, there is less of a threat to safety than in a collapse resulting from loss of support at the bearings. Nevertheless, a potential for loss of life or serious injury exists. In addition, the function of the bridge will be lost immediately following the earthquake, which will result in delays to emergency traffic that could be unacceptable.

The loss of flexural or shear capacity in a column is unacceptable when it results in the formation of a collapse mechanism. In assessing the possibility for the formation of a collapse mechanism, both the configuration and geometry of the structure should be considered.

For example, a continuous structure with multi-column bents has a high degree of redundancy. Collapse is prevented in the longitudinal direction by the presence of the approach fills. In the transverse direction, collapse due to a loss of column flexural capacity would require as a minimum that columns loose capacity at both ends. With the exception of very long flexible structures, collapse would also require a total loss of shear capacity at the abutments since the superstructure acts as a deep beam in the lateral direction. When integral abutments are present, this is virtually impossible. Even in the event of a shear failure in the anchor bolts, keeper bars, or shear keys at abutment bearings, the friction between the superstructure and abutment will provide some continued shear resistance. Only in the event of a loss of support at the abutments due to transverse movement will total collapse be likely.

On the other hand, continuous structures with single-column bents are usually more vulnerable to collapse than structures with multi-column bents. Single columns tend to respond as cantilevers fixed at the footing. Loss of flexural capacity at the base of the column will result in a mechanism at the bent. When the superstructure is flexible, as it often is for structures with single-column bents, and other columns also fail, collapse can occur. The geometry of the structure is important and should also be considered. For example, a torsionally stiff (box girder) curved structure with three or more single-column bents will have stability even if flexural capacity is lost at the base of the columns. A collapse mechanism can only be formed if flexural capacity is lost at the top of the columns or in the superstructure. In a similar



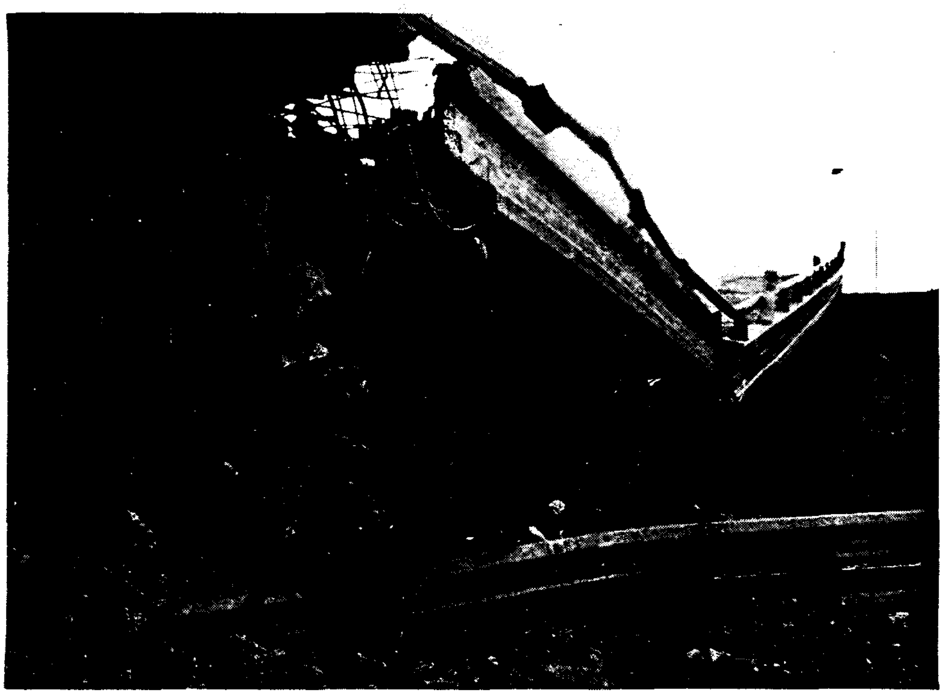


FIGURE 19. COLUMN FAILURE—SAN FERNANDO EARTHQUAKE, 1971

structure on a straight alignment the loss of flexural capacity at the base of the columns will transfer all lateral load through the superstructure to the abutments. If the superstructure is too flexible or discontinuous or if the abutments are incapable of resisting the shear and torsion forces that will be transferred to them, then collapse could occur. Structures with skewed abutments would be more likely to become unseated at the abutments and would be in greater danger of total collapse than would structures with nonskewed abutments.

Superstructure discontinuities, such as expansion joints, will affect the overall stability of the bridge. If expansion joints occur at the bents, superstructure forces will be transferred to the bents by way of anchor bolts, keeper bars, or shear keys. Although these components will usually fail at force levels below those capable of damaging the columns, enough force can be transmitted in some cases to damage the columns. If this occurs, the superstructure will provide little if any redundancy and the collapse of the bridge is a definite possibility.

When superstructure discontinuities exist within the spans, the stability of the structural section between adjacent discontinuities will determine the collapse potental of the bridge due to loss of column capacity. For example, an extremely vulnerable situation exists when only one single-column bent occurs between superstructure discontinuities. This was the case in the South Connector Overcrossing which suffered a partial collapse during the San Fernando earthquake of 1971.

Often the overall vulnerability of the structure to collapse resulting from column failure can be mitigated by the use of expansion joint restrainers at superstructure discontinuities. This will cause columns to work together during a severe earthquake and tend to stabilize the structure against collapse.

Footing flexural failures have much the same effect on overall structure stability as do column flexural failures at the column/footing interface. The capacity/demand ratios are dependent on the nature of the footing failure. A failure which is progressive and results in a gradual loss of flexural capacity will have a higher capacity/demand ratio than a footing which loses flexural strength rather suddenly. Footing failures are usually less desirable than column failures because they are more difficult to detect and repair.

Footing sliding failures will seldom result in a total loss of lateral strength. The passive resistances of the soil and friction between the footing and the soil will continue to resist load. The primary concern with this type of failure would be the consequences of the large displacements that could result. If these displacements could result in loss-of-support failures at the bearings, footing sliding failures should be considered important.

In assessing the consequences of column, pier, or footing failures, the effect of each failure on the stability of the structure should be considered in its order of occurrence. Because each failure will mean that additional forces will be transferred to other components, there will come a point when a component failure results in a high probability of total collapse. This will happen when the structure redundancy is severely reduced and formation of a collapse mechanism appears imminent. Since a rigorous assessment of the load at which a collapse mechanism would be formed requires sophisticated analytical procedures which are not typically available to the engineer, this assessment will usually be dependent on engineering judgement.

ABUTMENTS

Abutment failures usually result in excessive deformations and seldom result in structural collapse unless associated with liquefaction failures. Therefore, abutment failure is often considered acceptable when determining the need for seismic retrofitting. In the case of bridges that provide an indispensible function following an earthquake, the loss of accessibility resulting from an abutment failure may be sufficient cause to require retrofitting.

LIQUEFACTION

Liquefaction related failure are frequently dramatic because of the large relative displacements that often occur. Such failures can easily result in the collapse of the structure. Structures which are well tied together and have the ability to deform without undergoing a brittle failure usually will not collapse, although they may be severely damaged and rendered useless to emergency traffic. Discontinuous structures often collapse due to large relative movements. Capacity/demand ratios for liquefaction failure reflect the susceptibility of the structural configuration to serious failure. This will depend on the ability of the bridge to withstand the differential movements due to liquefaction. Since it is often difficult to accurately predict the magnitude and direction of foundation displacement resulting from liquefaction, it is necessary to rely on observations of past bridge liquefaction failures and the performance of various bridge systems when subjected to large displacements resulting from liquefaction.

Typically, the relative ground movements at the various supports of bridges is past earthquakes are as shown in Figure 20. Movements at the abutments of two feet are common, with movements as high as eight feet on record. Such large relative movements often result in severe structural damage.

The typical relative foundation movements shown in Figure 20 will subject the superstructure to compressive and bending forces. If the superstructure is continuous, the top of the abutment backwall will be restrained by the superstructure acting as a strut. The base of the abutment, which is without restraint, will move and the abutment will tilt away from the center of the bridge. Integral abutments displaced in this manner may continue to support vertical load. Bents which are monolithic with the superstructure will be subjected to similar tilting with decreasing magnitude as the distance from the abutment increases. This will subject the bents to high forces. If ductile plastic hinging can occur at the top and bottom of the bents, then the bents may continue to support vertical load. If the angle of column tilting is limited by the ratio of liquefaction movement to bent height, then it is likely that the structure will also retain enough integrity to carry light emergency traffic. In the event that the columns are unable to deform in a ductile manner, then it is likely that vertical support will be lost or greatly reduced and partial collapse could occur.

In bridges with discontinuous superstructures, expansion joints will close before compressive forces will be developed in the superstructure. The discontinuities at expansion joints will make the superstructure more susceptible to buckling under these compressive loads. The susceptibility to buckling will be aggravated by skewed supports or a curved horizontal or vertical alignment.

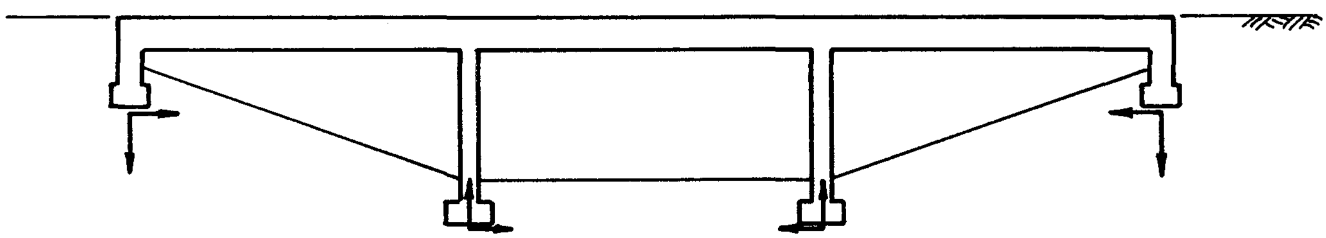


FIGURE 20. RELATIVE FOUNDATION MOVEMENTS DUE TO LIQUEFACTION

If expansion joints occur at the bents, the superstructure will be supported on bearings at these locations. Since the bents will usually move relative to one another, large forces will be induced in the anchor bolts and keeper bars of bearings fixed against movement along the axis of the superstructure. Very often the bearings will be incapable of resisting these forces and failure of the bearings will result. When this happens, the chances of the spans falling from the bents are high. If, as sometimes occurs, two expansion bearings exist at the same bent, span collapse is almost assured in the event of liquefaction because of the movement of the bent relative to the expansion joint. As might be expected, simply supported multi-span structures have historically been the most susceptible to collapse due to liquefaction.

The final issue with regard to retrofitting bridges to mitigate liquefaction failures is the cost of such retrofitting. In the case of severe liquefaction, the prevention of collapse due to relative movements may be the only benefit of retrofitting. In such cases, prevention of the loss of bridge function is often impractical or economically infeasible. The retrofitting of such bridges to mitigate liquefaction failure is almost surely less economical than other types of retrofitting. Therefore, the importance of the bridge will be a large factor in determining the need for retrofitting.

COMMENTARY

CHAPTER 4 - DETERMINATION OF SEISMIC CAPACITY/DEMAND RATIOS FOR BRIDGE COMPONENTS

C4.1 GENERAL

Seismic capacity/demand (C/D) ratios are indicators of the way a given structure will perform under earthquake loading. Although the C/D ratios are intended to give a reasonable estimate of the percentage of the design earthquake that is likely to cause a component to be seriously damaged, the consequences of this damage must be assessed in terms of its effect on the stability and usability of the structure following an earthquake.

It is very difficult to estimate the level of earthquake loading that is likely to cause serious damage to an existing bridge component that may have been designed to meet much different criteria than a corresponding component in a new bridge. Very little research has been performed relating to the behavior of many types of existing bridge components. For this reason, many of the equations developed to determine C/D ratios rely heavily on engineering judgement. This will be discussed in more detail in the sections that follow.

C4.2 ANALYSIS

The analysis procedures recommended for use in evaluating bridges for retrofitting are the same as those described in the Seismic Design Guidelines.[1] The objective of performing an analysis for retrofit is to approximate the seismic demands on various structural components. These demands are compared with the capacities of the components to determine the percentage of the design earthquake load at which failure of the various components can be expected.

The elastic response of the bridge to an elastic response spectrum is first determined, for two earthquake loadings applied in orthogonal horizontal directions. These directions will usually be parallel and perpendicular to a straight line between the bridge abutments. To account for the directional uncertainty of the earthquake, two load cases will be considered, each of which consists of 100% of the results from loading along one of the horizontal directions and 30% of the results from loading in the other direction. The effect of vertical ground motion is not explicitly considered.

C4.2.1 BRIDGE SEISMIC RESPONSE

The actual response of a bridge during a major earthquake is usually not elastic. Inelastic or nonlinear response occurs because of yielding of components such as columns and footings and the nonlinear response of abutment backfill, expansion joints, and piles if these are present. Clearly an inelastic analysis to determine the capacity/demand ratios of components cannot be used routinely at this time because of the difficulties and uncertainties that are involved. Therefore an elastic analysis is specified to approximately determine both the displacement and force demands on the bridge components during an earthquake.

The use of an elastic analysis to simulate actual dynamic response is based on the assumption that elastic and inelastic displacements of a bridge structure are of similar magnitude. Although Gulkan and Sozen (2) have shown this assumption to be significantly in error for short-period single degree of freedom oscillators, there is some evidence to indicate that this assumption may be reasonably accurate for the overall response of an actual bridge structure.[3] This seems to be true because column yielding is localized and affects only a portion of the total structural stiffness. However, the difference between elastic and inelastic results for relative displacements of individual components (e.g., restrainers) is considerable. Given the other uncertainties involved in predicting seismic behavior, an elastic analysis is usually accurate enough for the purpose of design and evaluation. To ensure realistic elastic displacement results, care should be taken to correctly model the structural components when performing an analysis.

Force results from an elastic analysis will only be realistic when the component does not yield or exhibit nonlinear behavior. For columns, the demands resulting from the elastic analysis are modified considerably in some cases to account for the anticipated mode of failure and ductility expected from the column. The method of accounting for this depends on the anticipated mode of failure and the ductility of the column as discussed in later sections of this chapter.

It is important that foundation flexibility at the abutments be considered in the elastic model. Although replacing the highly nonlinear and complex relationship of forces and displacements at the abutment with a linear spring or system of springs will never be totally satisfactory, this approach can yield reasonable results. Selection of appropriate spring stiffnesses for the abutment may be obtained by following the procedure outlined in Figure 21. Methods for calculating initial elastic spring stiffnesses are discussed in a workshop manual entitled "Seismic Design of Highway Bridges" (Report No. FHWA-IP-81-2).[4] If, when an analysis is performed, the ultimate force capacity of the abutments is exceeded by more than 10%, then abutment yielding may be assumed to occur. This yielding will be equivalent to a softening of the assumed elastic springs at the abutments. Therefore, the abutment spring coefficients should be reduced until the elastic forces at the abutment approximate the ultimate force capacity of the abutment.

The modeling of expansion joints is also important. For unrestrained joints, movement can occur relatively freely within a certain range. As expansion joints close, however, further movement is restricted by the impacting of adjacent structural sections. This is often ignored in a multi-modal response spectrum analysis when unrestrained expansion joints are modeled to have total freedom of relative longitudinal movement.

In expansion joints fitted with longitudinal motion restrainers, motion is restricted in both directions. Most restrainers are unidirectional, being only effective in preventing joint separation. Restrainers are usually designed to engage after movement of a small distance which is provided to allow expansion joints to function properly. Joint closure prevents movement in the same manner as in unrestrained expansion joints. The behavior of expansion joints is very complex and nonlinear computer programs have been written to account for this behavior.[5] In the case of an elastic analysis, however, it has been common practice to model the joint and restrainers as springs with a stiffness equal to the tensile stiffness of the restrainers. Analytical case studies have shown that this approach often yields results considerably different from a more sophisticated nonlinear analysis.[3] Usually, the elastic force results are greater than nonlinear results but inituitively appear to be reasonable in magnitude. Therefore, elastic analysis is often considered to be conservative for design and evaluation. Fortunately, the method of modeling restrained expansion joints for an elastic analysis appears to have a small effect on the elastic response of the remainder of the structure.

The difficulties involved in accurately analyzing the response of expansion joints is one of the main reasons that the Seismic Design Guidelines specify minimum support lengths and motion restrainer forces.

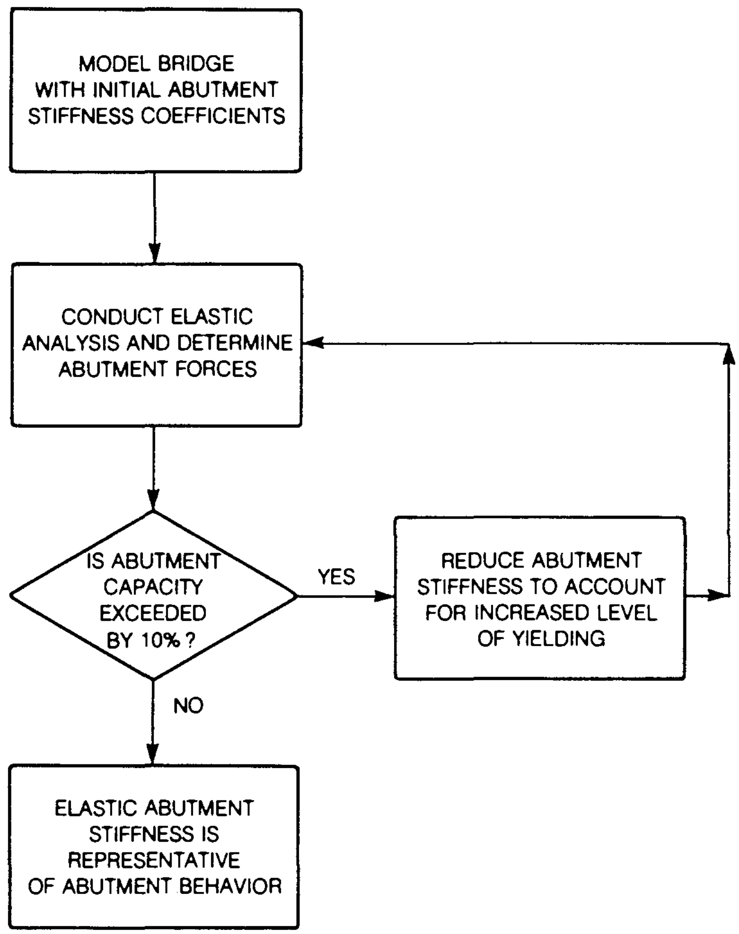


FIGURE 21. ITERATIVE PROCEDURE FOR DETERMINING ABUTMENT FLEXIBILITY EFFECTS

C.4.2.2 SINGLE-MODE SPECTRAL METHOD (SIMPLIFIED APPROACH)

The single-mode spectral method of analysis is allowed in different situations as shown in Table 5 of the Retrofitting Guidelines. This method is essentially an equivalent static force approach and assumes that the dynamic response of the structure can be accurately represented by a single, easily determined mode of vibration. A description of this method is given in the Seismic Design Guidelines.

Bridges with intermediate expansion joints will have at least two significant modes of vibration and therefore an adaptation must be made if the single-mode spectral approach is to be used. This can be done by performing separate analyses for various sections of the structure, as illustrated in Figure 22(a). The elastic force in the expansion joint restrainer should be assumed as the lesser force derived from the analyses of the two models shown. The column and foundation forces due to a longitudinal earthquake loading would be the greater of the forces obtained from an analysis assuming the free relative movement of the expansion joint or an analysis assuming no relative movement at the joint connecting the two structures, as shown in Figure 22(b).

C4.2.3 MULTI-MODE SPECTRAL METHOD

More complex structures in higher seismic performance categories should be analyzed using the multi-mode spectral approach. This will require the use of a linear dynamic analysis computer program.

The use of computer programs of this type is not difficult but requires a basic conceptual understanding of the theory of structural dynamics. Proper modeling of a bridge system for a multi-mode spectral analysis is different from the modeling required to perform a static analysis. This is because inertia effects must be included in a dynamic analysis. This is usually done by lumping the mass of the structure at various locations on an otherwise weightless structural frame. The number of mass lumps to be included is critical to the analysis. Too few mass lumps will result in unsatisfactory answers, and too many will increase the computer costs unnecessarily. As a rule of thumb for the type of structures covered by these guidelines, masses lumped at the ends and quarter points of spans and the ends and midpoints of columns will yield satisfactory results at reasonable costs.

Other modeling considerations related to the abutments and expansion joint hinges were discussed in Section C4.2.1.

Several general-purpose computer programs are available which can be used to perform a multi-mode spectral analysis of a bridge. However, a computer program specifically designed for the seismic analysis of bridge structures has recently been developed with funding from the National Science Foundation.[6] This program, known as SEISAB-I, is written in standard FORTRAN and is designed to be user-oriented and machine-independent. Many of the features of this program are developed specifically to assist in the implementation of the Seismic Design Guidelines. The worked example problem presented in Appendix A illustrates the use of this program for bridge seismic evaluation.

C4.3 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS

The first step in calculating seismic force and displacement demands is an elastic analysis of the structure response due to earthquake loading in two orthogonal horizontal directions. This analysis should consider all those factors which contribute significantly to

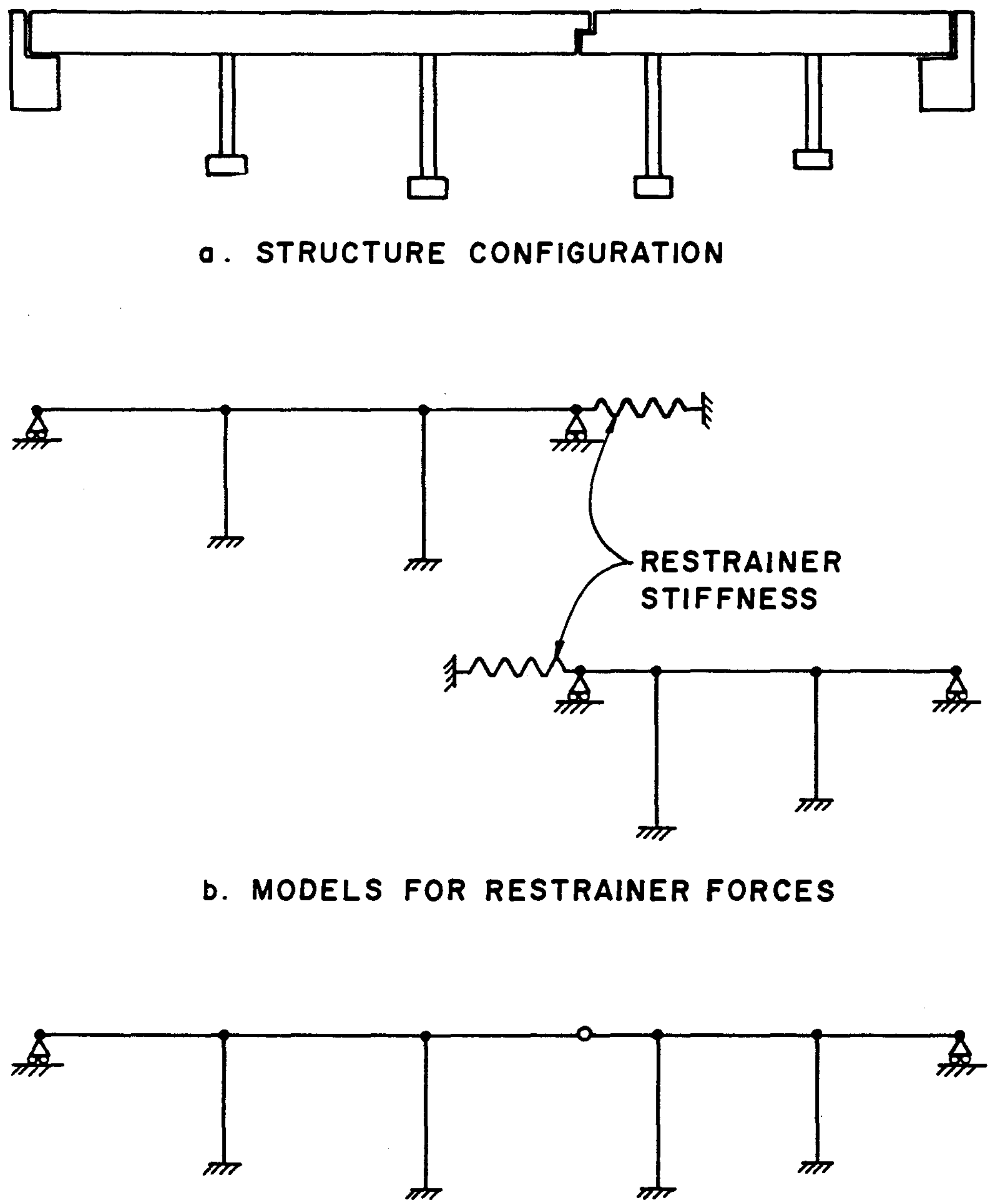


FIGURE 22. TYPICAL STRUCTURE IDEALIZATION, SINGLE MODE SPECTRAL METHOD

the overall flexibility of the bridge. In many cases the abutment foundation flexibility will have a large influence on the analyzed structure response and therefore should be included in the structure idealization.

C4.4 COMBINATION OF ORTHOGONAL ELASTIC SEISMIC FORCES

In an actual earthquake it is unlikely that the motion will be directed along either of the two horizontal axes used in the analysis. To account for the possibility of motion being directed in another direction that could result in higher forces, the force and displacement responses from each of the two elastic analyses are combined. Each of the two combinations are defined as load cases and consist of a combination of 100 percent of the absolute value of the response from the analysis for loading in one of the orthogonal horizontal directions plus 30 percent of the absolute value of the response to loading in the other direction. From a design point of view it is convenient to use the force responses along and about the principal axes of a bridge component.

C4.5 MINIMUM BEARING OR RESTRAINER FORCE DEMANDS

Bearing or restrainer force demands are generally obtained from an analysis of the structure. However, bearing or restrainers forces derived from an elastic analysis do not include the effects of nonlinear response of the structure or variations in motions at the supports due to traveling surface waves. Because a linear analysis of a bridge often results in relatively low bearing or restrainer forces, minimum force demands are specified to account for uncertainties in the analysis and to keep from overlooking bearings with unreasonably low force capacities. These minimum forces (.20 times the deadload) are intended for evaluation and should not be confused with minimum design forces (.35 times the deadload) for bearing and restrainers. Different minimum forces for evaluation and design are consistant with the rest of the Retrofit Guidelines in which evaluation and design are treated differently. Minimum force demands are not applicable to devices specifically designed to limit the transfer of forces.

The engneer performing the evaluation may use simplified methods to determine the portion of the minimum equivalent horizontal force carried by the bearings and restrainers. As an example, the minimum equivalent horizontal force may be distributed to each horizontal force resisiting element based on the portion of the total dead load included within the plan area of the bridge bounded by imaginary lines midway between adjacent horizontal force resisting elements. When the ultimate force capacity of a ductile horizontal force resisting element is insufficient to resist its share of the minimum equivalent horizontal force, then the excess of that force should be distributed to non-ductile bearings or restrainers.

C4.6 MINIMUM SUPPORT LENGTHS

The supports at the abutments, columns, and expansion joints must be of sufficient length to accommodate anticipated relative displacements. Because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves, minimum support lengths are specified. The minimum support lengths were originally developed as part of the Seismic Design Guidelines and represent the collective engineering judgement of the project engineering panel responsible for developing those guidelines.

C4.7 CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS AND BEARINGS

Bridge superstructures are often constructed with intermediate expansion joints to accommodate anticipated superstructure movements such as those caused by temperature variation or to allow for the use of incompatible materials. Discontinuities necessitate the use of bearings which provide for rotational and/or translational movement. During earthquakes, bridge bearings have proven to be one of the most vunerable of all bridge components.

In major earthquakes, the loss of support at bearings has been responsible for several bridge failures. Although many of these failures resulted from permanent ground displacements, several were caused by vibration effects alone. The San Fernando, California earthquake of 1971,[7] the Guatamala earthquake of 1976,[8] and the Eureka, California earthquake of 1980[9] are some recent examples of earthquakes in which bridge collapse resulted from bearing failure. Even relatively minor earthquakes have caused failure of anchor bolts, keeper bar bolts or welds, and nonductile concrete shear keys. In many of these cases the collapse of the superstructure would have been imminent had the ground motion been slightly more intense or longer in duration.

The dynamic behavior of bridge bearings is often very nonlinear and difficult to analyze using conventional linear-elastic analysis techniques. Elastic bearing forces obtained from a conventional analysis are likely to be lower than those actually experienced by bearings during an earthquake. This is because bearings, which are nonductile components, often do not resist loads simultaneously. This has been demonstrated in past earthquakes by the failure of anchor bolts or keeper bars on some, but not all, of the bearings at a support. In addition, the yielding of ductile members, such as columns, can transfer load to the bearings. This phenomenon was observed in the results from nonlinear analytical case studies of three bridge structures.[3] For these reasons it is necessary to increase elastic analysis force results when evaluating the force demand on nonductile motion-restraining components.

In the case of differential horizontal displacements at expansion joints during earthquakes, elastic response spectrum analysis results yield displacements that are often below those intuitively expected based on observed bridge behavior during past earthquakes. In addition to the nonlinear behavior of expansion joints, possible independent movement of different parts of the substructure and out-of-phase movement of abutments and columns resulting from travelling surface-wave motions also tend to result in larger displacements.

The Seismic Design Guidelines[1] recognize the difficulty in accurately determining the seismic displacements at supports and provide for a minimum support length in the design of new structures. The minimum support lengths specified are dependent on the structure length between expansion joints and the average column height because both dimensions influence one or more of the factors that cause differential displacements. The minimum support lengths are useful in evaluating the support lengths of existing bridges at unrestrained expansion joints.

When retrofitting expansion joints, however, it is often difficult or impossible to increase the existing support length. In these cases, longitudinal restrainers or other displacement-limiting devices may be the only feasible means of preventing a loss of support at the bearings. To evaluate the effectiveness of these devices in reducing displacements, it is necessary to more accurately analyze the movement at bearings. To obtain a reasonable estimate of the actual displacements, a multi-mode spectral method of analysis including the effect of foundation flexibility should be performed.

When evaluating the effect of seismic displacements, it is necessary to remember that the entire seat width will not be available during an earthquake. Shortening of the bridge superstructure due to shrinkage, temperature, or creep may reduce the effective support width. In addition, the impacting of adjacent superstructure sections during strong seismic shaking is likely to cause localized damage of the expansion joints. This damage will involve crushing of concrete and a probable loss of concrete cover which will further reduce the available seat width. This is shown schematically in Figure 23.

A bridge with a sloping vertical alignment may have a tendency to shift downhill during an earthquake, leaving some expansion joints closed and others open. This same tendency to move downhill may also result from non-earthquake loadings such as temperature movement, traffic vibrations, and vehicle breaking forces. This latter phenomenon should also be considered in determining the available support length.

In determining the force capacity of bearings, careful consideration should be given to the following shortcomings of bridge bearings:

- Grout pads under bearing masonry plates have traditionally given trouble during and after construction and have been one of the main sources of trouble in minor earthquakes. Failure of a grout pad will allow the bearing assembly to move, subjecting the anchor bolts to combined bending and shear.

- Anchor bolts which pass through an elastomeric bearing pad will be subjected to combined bending and shear.

- Anchor bolts are frequently threaded below the top surface of the pier or abutment seat. This gives a reduced area for shear and may reduce flexural capacity of the bolt due to notch sensitivity at the root of the threads.

- In some cases anchor bolts are too close to the edge of the bearing seat and will spall the concrete when subjected to horizontal loads.

- All of the bearings at the end of a span do not resist horizontal forces simultaneously. Because keepers or other devices are not set with exactly the same clearances, the bearings will not be equally effective in resisting load. It is not uncommon for bearings at one end of a span to be damaged to varying degrees after an earthquake.

- Bridge bearings may not be what they are represented to be on "as-built" plans or maintenance records. Adjustments to keepers and other details are occasionally made after construction is completed. The details and workmanship in such cases are often inferior to the original construction.

C4.8 CAPACITY/DEMAND RATIOS FOR REINFORCED CONCRETE COLUMNS, PIERS, AND FOOTINGS

Reinforced concrete columns or piers and the footings to which they are attached form a group of interacting components that are among the most vulnerable to earthquake damage. During high levels of ground shaking it is likely that one of these components will be subjected to yielding. Because of the interaction between yielding in one component and the response of the remaining components, the columns, piers, and footings should be considered as a group. The weakest of these components will determine the type of failure that is likely to occur.

In quantitatively evaluating the strength of the columns and piers, four failure modes should be considered. These are: pullout of main reinforcement, splice failures in the main reinforcement, sudden shear failure, and loss of flexural capacity due to insufficient

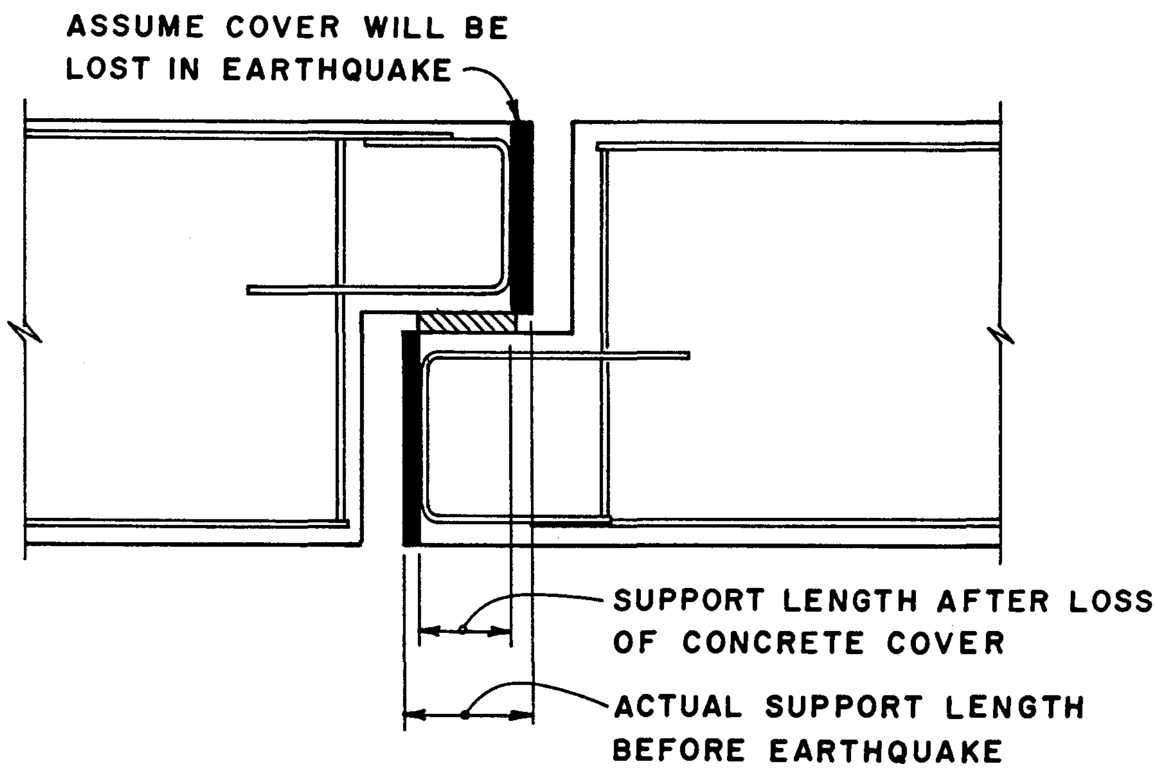


FIGURE 23. EFFECTIVE SEAT WIDTH

confinement. Each of these failure modes is a function of the level of column yielding that takes place in the column and depends on the amount of transverse confinement of the main longitudinal reinforcing steel. Although some useful research has been performed with respect to the behavior of bridge columns under cyclic loading,[10,11] the state-of-the-art is such that column evaluation must rely heavily on engineering judgement, especially in the case of existing columns which may have vulnerable details. The methods proposed for evaluting the C/D ratios are based on the latest research related to the behavior of reinforced concrete columns, but still reflect considerable judgement on the part of the researchers.

Most existing bridge columns not only have an insufficient quantity of transverse reinforcing steel, but the details with regard to the placement of this steel make it less effective than new construction in resisting cyclic column loading. Evaluation of the effectiveness of this reinforcement is necessary if a reasonably accurate analysis of seismic capacity is to be made.

The effectiveness of this steel will be greatly reduced when the concrete cover in the vicinity of the plastic hinge spalls. Transverse steel in the region of spalling will then be partly exposed, which will greatly reduce anchorage. To some extent, the reduction of efficiency of lap splices in transverse reinforcement depends on the degree of spalling. It is assumed that spalling of cover concrete will commence at a ductility indicator of approximately 2 and at this ductility indicator the efficiency of the lap splice drops to approximately 50%. At higher ductility indicators a greater amount of spalling of the cover concrete is assumed, and the efficiency of lap splices is assumed to be reduced linearly, eventually reaching zero at a ductility indicator of approximately 4. These estimates of the efficiency of lap splices in transverse steel are based mostly on engineering judgement, although observed column behavior during past earthquakes lends support to the conclusions drawn.

When a column failure in any of the four potential failure modes occurs due to insufficient transverse reinforcement, it is likely that poorly anchored transverse reinforcement will unravel and become totally ineffective. Therefore, this reinforcement should not be considered in calculating C/D ratios for the remaining indicators above the level where the initial column failure occurred.

C4.8.1 ANCHORAGE OF LONGITUDINAL REINFORCEMENT

The pullout of longitudinal reinforcement can occur at the footings or at the bent cap. This may result either due to an inadequate anchorage length or as a result of bond degradation due to flexural or shear cracking of the concrete in the footing or cap. In either case, a sudden loss of flexural capacity may result.

If inadequate anchorage length is provided for the reinforcing steel, the ultimate capacity of the steel cannot be developed and failure will occur below the ultimate moment capacity of the column.

If anchorage failure results from bond degradation that accompanies the flexural cracking of footing concrete, the load level at which failure occurs will depend on the amount of yielding in the footing. This is accounted for by multiplying the moment capacity/demand ratio for the footing, r_{ef}, by a ductility indicator. Since most existing footings are not reinforced to resist flexural cracking resulting from footing uplift, failure may occur as a result of the negative moments developed in the footing due to overturning. Usually this will not be a problem in spread footings, since they are not sufficiently restrained by the soil overburden to develop high tensile stresses in the concrete. On the other hand, pile footings are usually anchored, although only nominally, to the piles. This will allow high tensile stresses to be developed due to overturning of the footing and the resulting flexural cracking of the concrete could cause anchorage failures.

The ductility indicator that is applied to the footing moment capacity/demand ratio to evaluate anchorage failure due to flexural cracking in a footing depends on the details of the anchorage and the extent to which flexural cracking will occur. Straight anchorage in a footing without a top layer of reinforcement may fail rather suddenly when flexural cracking occurs, and therefore a ductility indicator of 1.0 is used. Failure will be delayed somewhat when anchored bars are hooked. When the hooks are bent away from the centerline of the column, the concrete in the vicinity of the hook may eventually be subjected to flexural cracking, and therefore a ductility indicator of 1.3 is specified. A greater ductility indicator is allowed when hooks project toward the centerline of the column because concrete in the vicinity of the hook will be in compression, which will tend to mitigate an anchorage failure. When nonstandard hooks are present, the required anchorage length will be determined by interpolating between equation 4-6 and 4-8 based on the rato of the actual length of the hook extension to the length of a standard hook extension.

When a top layer of footing flexural reinforcement is provided, flexural cracking may occur if the reinforcement is inadequate, but will progess more slowly and allow a larger ductility demand indicator to be used. When straight anchorage is provided, anchorage failure may still occur, although the ductility indicator related to this detail is specified as 1.5. If hooks are provided, the performance of the splice is assumed to be dependent on the nominal adequacy of the anchorage.

It must be stressed again that the procedures for evaluating loss of anchorage in a footing are based largely on engineering judgement.

This type of failure can also occur in pier shafts if bars are not extended below the level of fixity a sufficient distance to develop the ultimate stress in the reinforcement. Similarly, if splices occur in a pier shaft, sufficient confinement of the shaft must exist within the area of potential yielding to provide for a transfer of stresses in the reinforcing steel.

Development lengths used to evaluate columns for retrofitting were determined by research carried out at the University of Texas.[12] The failure hypothesis presented as a result of this research assumes that the radial component of reactions on the lugs of an anchored bar will produce stresses analogous to bursting stresses on a thick-walled hollow-concrete cylinder as shown in Figure 24. The resistance to bursting is a function of the wall thickness of the hypothetical concrete cylinder taken as the lesser of the clear bar cover or half the clear bar spacing. In addition, bursting will be prevented by transverse reinforcement crossing a potential splitting crack in the hypothetical cylinder wall. In some cases, the proposed equation for development lengths will result in lengths significantly below those specified by previous design codes. In the case where the clear bar cover is much larger than half the clear bar spacing, such as in footings, the confining effect of this large cover may be considered by assuming the cover to be equivalent to transverse steel of equal tensile strength.

In circular columns potential spitting cracks may occur between adjacent bars resulting in all bars failing in anchorage as a group and pulling out of the footings as a plug. In this case the amount of transverse steel, $A_{tr}(c)$, can be assumed as twice the cross-sectional are of a single hoop divided by half the number of anchored bars. A similar group anchorage failure can occur with columns of different shaped cross-sections.

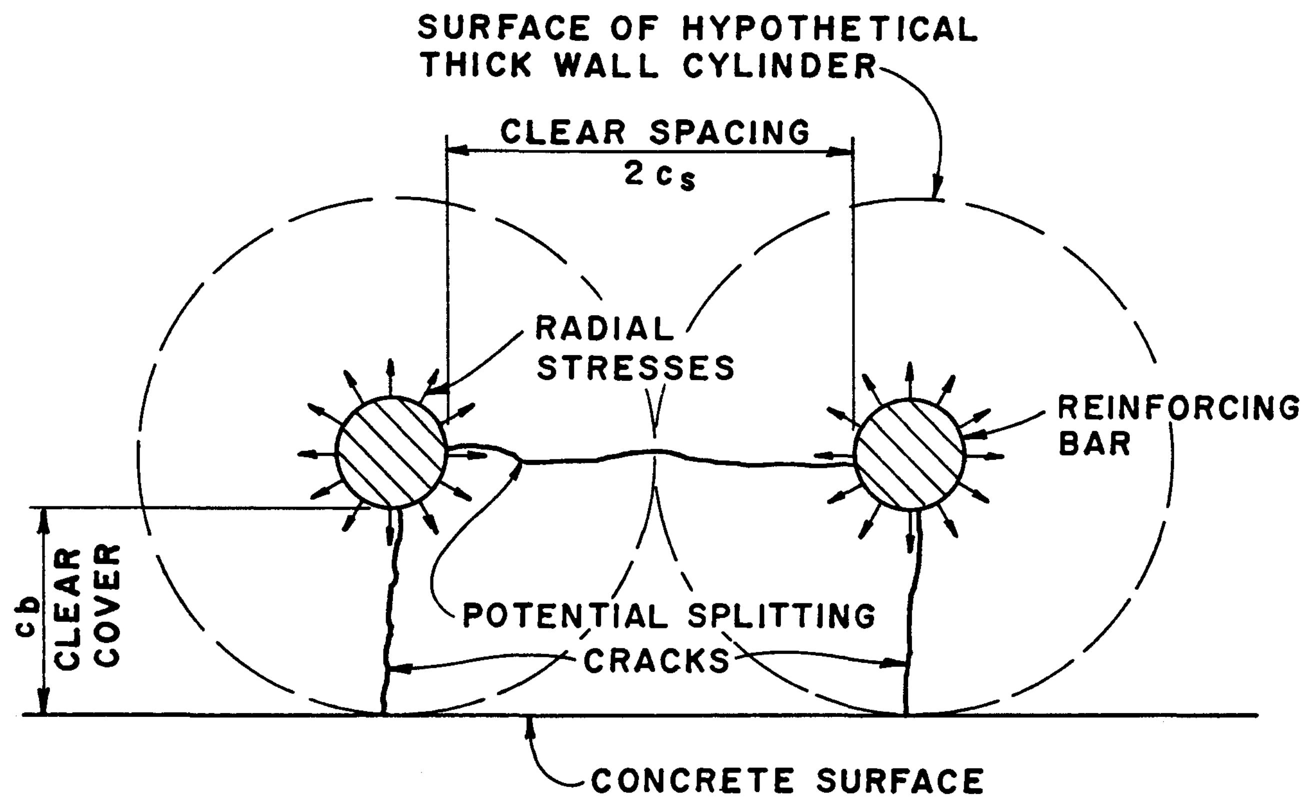


FIGURE 24. RADIAL STRESSES DEVELOPED DUE TO BAR ANCHORAGE

C4.8.2 SPLICES IN LONGITUDINAL REINFORCEMENT

Stress is transferred between spliced bars by the longitudinal component of diagonal compressive stresses that are developed in the concrete between the bars. The transverse component of this concrete stress acts against the spliced bars and may cause a longitudinal split to form in the concrete between the bars unless sufficient reinforcement is provided across the potential splitting surface. Splitting cracks may also develop between adjacent sets of spliced bars if sufficient spacing is not provided between bars. In the absence of sufficient reinforcement, failure will be initiated by splitting at the ends of the splice. This splitting will propogate along the splice under progressive cyclic reversed loading, eventually causing the splice to "unzip." Therefore, additional splice length will not necessarily prevent failure. The key to preventing a splice failure in a bridge column is the presence of sufficient, closely spaced transverse reinforcement that will prevent the initiation of splitting. It is necessary, however, to provide a minimum splice length.

The provisions for evaluating the potential for a splice failure are based on the results of experimental research conducted at Cornell University [13,14] and the University of Canterbury in New Zealand [15]. The research has been directed primarily at splices in building columns, which are typically subjected to stress reversals slightly below yield stress. This research was successful in indentifying the amount and maximum spacing of transverse reinforcement required to prevent a splice failure under these loading conditions. In addition, minimum splice lengths were determined based on concrete strength.

For the case where splices could be expected to yield, as might be the case in the zones of maximum moment in a bridge column, testing showed that rapid degradation in the stiffness and strength of the splice would occur when transverse reinforcement equal to or less than that required for non-yielding splices was provided. Further testing indicated, however, that an improvement in splice performance resulted when additional transverse reinforcement was used. If approximately twice the transverse reinforcement required for the non-yielding case was used, splices were shown to be capable of withstanding reversed loading with displacement ductilities as high as six in some cases, although at these extreme ductilities tensile fracture of the spliced bars occurred. Other tests did not result in bar fracture, but indicated strength losses at somewhat lower ductility demands. In evaluating splice performance for the yielding case, a conservative estimate of the maximum allowable ductility is proposed in the guidelines. This is because of the small amount of testing done for the yielding case, and the poor transverse reinforcing anchorage details typical of most existing bridge columns. Based on judgement, the allowable ductility was assumed to be linearly related to the amount of transverse reinforcement in excess of that required for the non-yielding case.

When spliced bars are not stressed beyond 75 percent of the yield capacity, the splice will not degrade when subjected to reversed loading. Therefore, the capacity/demand ratio for splices should not be less than .75 r_{ec} when sufficient splice length is provided.

C4.8.3 COLUMN SHEAR

Column shear failure is critical because it results in a comparatively sudden loss of shear strength. When this occurs, the resulting excessive deformations may cause disintegration of the column and the loss of vertical support. This happened to the Route 5 (Truck Lane)/405 Separation (California Bridge No. 53-1548) during the San Fernando Earthquake. Several other bridges in the San Fernando Earthquake were in various stages of this type of failure and probably would have collapsed had the intensity of the ground motion been higher or longer in duration.

The method proposed for evaluating a column for shear failure is based on engineering judgement and assumes an idealized model of column behavior. This method may be visualized by examining the assumed relationship between shear capacity and shear demand as shown in Figure 25. Three possible cases are considered in evaluating C/D ratios for column shear.

Case A occurs when the column cannot achieve flexural yielding because of a low initial shear capacity. In this case, column C/D ratios for shear are calculated by dividing the inital shear capacity of the column by the elastic shear demand. This is possible because the initial shear strength of the column is not expected to degrade in the absence of plastic hinging, although a brittle shear failure can be expected when the initial shear capacity is exceeded. Case B will result when a shear failure is expected to occur due to shear capacity degradation resulting from plastic hinging of the column. In this case, column C/D ratios for shear are calculated by multiplying the column moment C/D ratio, r_{ec}, by the ductility indicator corresponding to the amount of yielding at which the column shear demand is assumed to exceed the column shear capacity. Case C is assumed when the degradation in column shear capacity is not expected to result in a shear failure. In this case, the column C/D ratio for shear will be calculated by multiplying the column moment C/D ratio by the ductility indicator corresponding to an assumed maximum allowable level of flexural yielding.

The assumed relationship between shear demand and shear capacity in reinforced concrete columns is used to identify which of the three cases applies and to determine the ductility indicator for Case B. Both demand and capacity are assumed to be dependent on the level of flexural yielding as measured by the ductility indicator.

The relationship between column shear demand and the ductility indicator is based on the observation that column behavior will be linear-elastic at a ductility indicator of one or less. At a ductility indicator above one the shear demand is assumed to be constant and may be determined from statics assuming that, where possible, plastic hinges have formed in the column end regions. The moments developed in the plastic hinges are assumed to be the maximum ultimate column moments adjusted for the possibility of overstrength. Actual shear demands at a ductility indicator above one will vary due to variation in the column axial load, strain hardening of the column flexural reinforcement, degradation of column ultimate moment capacity, failure of the column to form plastic hinges at both ends simultaneously, and other factors. The proposed model of shear demand was selected because it provides a simple yet conservative method for relating shear demand to flexural yielding.

The assumed relationship between shear capacity and flexural yielding, as measured by a ductility indicator, is based on observations of column shear behavior during experimental investigations and past earthquakes. These observations have established a qualitative relationship between shear capacity and flexural yielding, but the quantification of this relationship, as proposed in the guidelines, is based largely on the judgement of specialists in reinforced concrete column behavior.

Writers of design codes have found it convenient to subdivide the shear capacity of reinforced concrete columns into two parts. The first part is the resistance provided by shear reinforcement such as hoops or spirals. The assumed resistance provided by the reinforcement has been derived from a logical model of shear behavior that is based on a truss analogy.

The shear resistance of the concrete portion of the column is assumed to provide the second part of the total column shear capacity. However, attributing all shear resistance

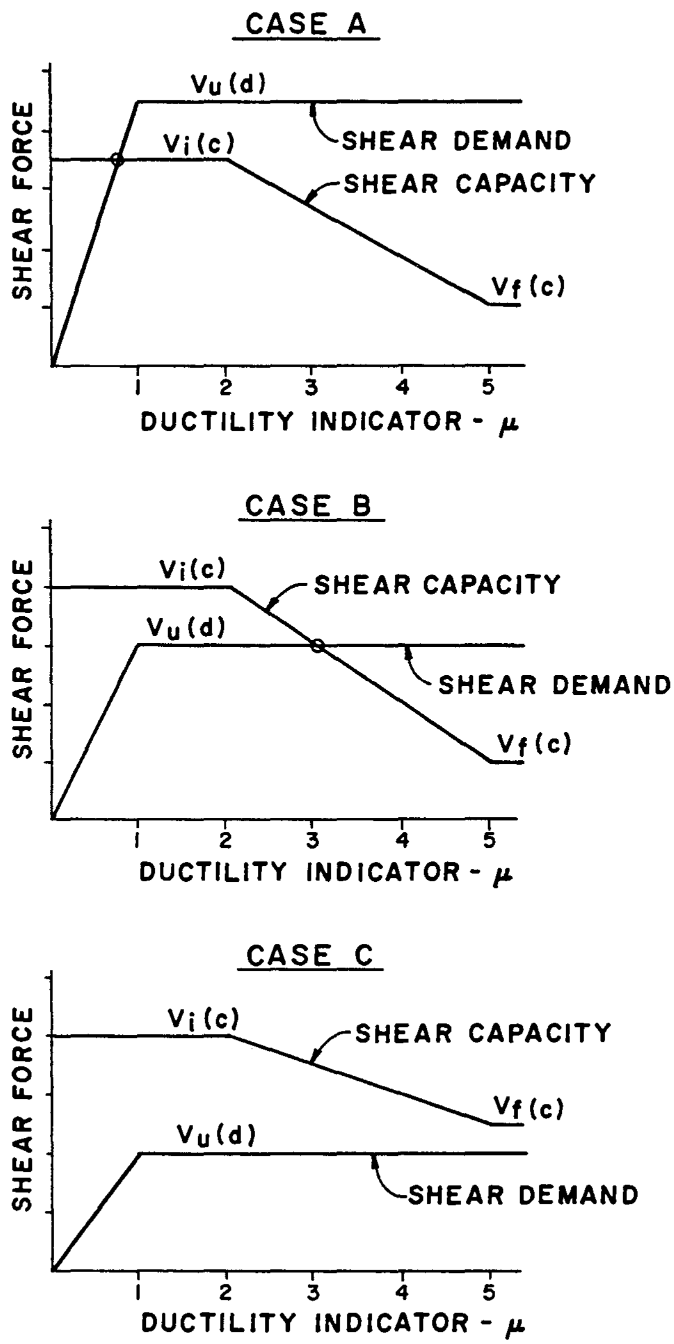


FIGURE 25. RESOLUTION OF SHEAR DEMAND AND SHEAR CAPACITY

other than that provided by shear reinforcement to the concrete portion of the column is an oversimplification of what actually occurs. This shear resistance in reality is composed of shear resistance of concrete within the zone of flexural compression stress, dowel action of the flexural reinforcement, and aggregate interlock along diagonal cracks. In present design codes these mechanisms of shear resistance are usually lumped into a single empirically derived effective shear stress that conservatively approximates the actual shear capacity not provided by shear reinforcement. This simplified approach is usually adequate when designing for static loads, since the mechanisms of shear resistance will remain intact at these load levels. Such an approach is probably also adequate for low levels of seismic loading that will not result in excessive flexural yielding of the columns.

The sum of the shear resistance provided by shear reinforcement and the concrete portion of the column is termed the initial shear capacity in the Retrofit Guidelines. It is assumed that the initial shear capacity will not be significantly affected prior to the commencement of spalling of the cover concrete, and therefore column shear capacity is assumed constant at a ductility indicator of two or less.

With reversed cyclic loading beyond the elastic limit, many of the mechanisms of shear resistance will begin to break down. This breakdown in shear resistance, which is assumed to commence at a ductility indicator of two, is more rapid in columns with a low height-to-width ratio because shear demands are typically higher than in more slender columns. For this reason the Retrofit Guidelines consider height-to-width ratios when evaluating columns for a Case B or C shear failure.

Shear reinforcement will not be seriously affected by moderate flexural yielding provided it is adequately anchored into the core of the column. However, many exisiting columns were built prior to 1973, when transverse reinforcement anchorage requirements were first included in the AASHTO bridge design specifications. Reinforcement that is not adequately anchored into the core of the column will be subject to a rapid loss of effectiveness when cover concrete spalls.

Reversed cyclic flexural yielding will usually have a detrimental effect on the shear resistance of the column not provided by shear reinforcement. Flexural and diagonal cracks may open under loading in one direction and never close on subsequent stress reversals. The mechanism of aggregate interlock will be affected as small transverse movements occur along these crack interfaces. The shear resistance due to the doweling action of flexural reinforcement will also be reduced as the concrete cover spalls and transverse confinement either fails or yields.

Some experimental research has shown that this degradation of the concrete shear resistance seems to be mitigated by increased column axial loads. Although the relationship between axial load and concrete shear resistance under reversed cyclic loading has not been precisely quantified, many researches have suggested that the concrete shear resistance be ignored in columns with an average axial stress below $0.10f'_c$, and considered totally effective for columns with greater average stresses. While this is a rather crude treatment of axial load effects, it has been used in other seismic design provisions and is therefore adopted in these guidelines until a more precise relationship can be derived.

For purposes of simplicity, it is assumed that a linear degradation of shear capacity will occur between a ductility indicator of 2 and a ductility indicator based on the maximum allowed level of flexural yielding. At the maximum level of yielding, the column shear capacity, termed the final shear capacity in the Retrofit Guidelines, will be assumed to consist of the shear resistance provided by adequately anchored shear reinforcement and the effective concrete shear resistance based on the magnitude of the axial loads. This method seems appropriate until research establishes a more precise relationship between shear capacity degradation and flexural yielding.

C4.8.4 TRANSVERSE CONFINEMENT

Transverse confinement reinforcement is required to prevent strength degradation in a column subjected to reversed cycles of flexural yielding. Degradation is prevented because confinement increases the capability of the concrete core to develop significant stress at high compressive strains and prevents buckling of longitudinal compressive reinforcement by providing lateral restraint for the reinforcing bars. The degree to which degradation will be prevented is dependent on the amount and spacing of transverse reinforcing and the adequacy of the anchorage of this reinforcing.

Current requirements for transverse confinement used in the Seismic Design Guidelines were developed by calculating the amount of reinforcement required to prevent a loss of axial strength in a reinforced concrete column due to the loss of cover concrete. Although this approach is simple and will result in column designs that can withstand high ductility demands, it is based on an inappropriate criteria for column performance and is of limited use for evaluating existing columns.

A more rational approach to calculating the effect of confinement was initially suggested by Priestley and Park.(10) This approach uses the calculated moment curvature relationships of a concrete column based on the assumed stress strain behavior of reinforcing steel and concrete at various levels of confinement. The available curvature ductility of a column would be assumed at a curvature that corresponds to a predetermined reduction (e.g., 80%) in the column moment capacity. This approach was subsequently used to develop the transverse confinement requirements for the New Zealand Concrete Design Code (NZS 3101).(16) The confinement provisions of NZS 3101 are based on the confinement requirements used in the Seismic Design Guidelines modified to account for the effect of axial load level. For low axial loads, NZS 3101 results in as much as a 50 percent savings over the amount of confinement reinforcing required by the Seismic Design Guidelines. The New Zealand Code requires that maximum spacing of transverse steel for adequate concrete confinement be 0.2 of the minimum cross-section dimension or 6 times the longitudinal bar diameter, whichever is less. Testing of near full-scale columns demonstrated the validity of the New Zealand transverse confinement requirements.(17)

Despite the work mentioned in the previous paragraphs, the evaluation of transverse confinement in existing columns must be tempered with judgement based on experience gained from past earthquakes. It is assumed that spalling of cover concrete commences at a ductility indicator of 2 and that even poorly confined columns can withstand yielding up to this level because the cover concrete provides some confinement. Columns with transverse reinforcement complying with the New Zealand Code are assumed to be capable of withstanding cyclic yielding corresponding to a ductility indicator of 6. Most existing columns have deficiencies in transverse reinforcement and are assumed to be able to withstand a limited level of yielding corresponding to a ductility indicator between 2 and 6. The equation developed to determine the appropriate ductility indicator utilizes three factors for assessing the relative effectiveness of transverse reinforcement. These factors are intended to account for reduction in the efficiency of confinement due to deficiencies in the amount, spacing, and anchorage of reinforcement. Factors for amount and spacing are averaged because they affect the efficiency of confinement in parallel but separate ways. A product of these factors would yield results that are too conservative. Deficiencies in anchorage, however, will effect the overall efficiency of transverse reinforcement and therefore the factor for anchorage is multiplied by the average of the first two factors to obtain the overall confinement efficiency. Although this approach is based largely on engineering judgement, it will allow for a reasonably accurate evaluation of the C/D ratio for transverse confinement.

C4.8.5 FOOTING ROTATION AND/OR YIELDING

Footing failures may be classified in one of two ways. The first type of failure involves large displacements of the foundation material resulting from instabilities generated within the soil by the earthquake ground motion. Liquefaction or slope instability would fall into this category. More discussion of these types of failures is included in Section C4.10.

The second type of failure, which will be discussed in this section, involves the yielding or rupture of foundation elements due to excessive seismic forces transmitted from the structure itself. This would include steel and/or concrete failure, bearing failure of the soil, footing failure due to sliding or overturning, and pile failure. These failures may result in ductile behavior or in sudden brittle failure.

Ductile yielding in the footing is avoided in the design of new bridges because of the difficulties involved in inspecting and repairing foundations. Such yielding results in structural damage, but will not usually result in structure collapse unless yielding is particularly extensive. Therefore, in the case of existing structures, the prospects of yielding in the footings is generally not sufficient grounds to justify seismic retrofitting. In fact, from the standpoint of preventing collapse, footing yielding may have a beneficial effect, since it can limit shear and flexure in the columns and thus decrease the chances of a brittle column failure.

A sudden brittle failure of the footing, on the other hand, could have serious consequences in terms of the ability of the structure to remain standing. The chances of total collapse will depend on the configuration of the structure and the nature of the footing failure. For example, the sudden loss of flexural capacity in the footings supporting a multi-column bent would probably not result in a structure collapse, since the bent would remain stable. However, similar failure in a structure with single-column bents would be much more serious. Structure collapse due to a sliding failure of the footing is difficult to imagine unless the movement is extensive and the structure is discontinuous and supported on narrow bearing seats. In summary, therefore, structures with single-column bents are most threatened by a footing failure.

In evaluating a structure, it is important to determine the capacity of the footings even if footing failure will not result in the collapse of the bridge. Footing failure modes will depend to a certain extent on the type of footing that is being examined. The following sections contain recommended procedures for determining the capacities of the two major types of footings used in bridge construction: spread and pile footings.

A. Spread Footings The capacity of the footing to resist the loads transmitted from the column or pier should be determined. There is an interaction between vertical load and moment capacity which may be governed by the following types of footing failures which are shown in Figure 26:

- Tilting of the footing due to a soil-bearing failure.
- Flexural yielding of footing reinforcing.
- Concrete shear failure of the footing.
- Bond failure of the main column steel.

The last two failure modes could have serious consequences which in some cases

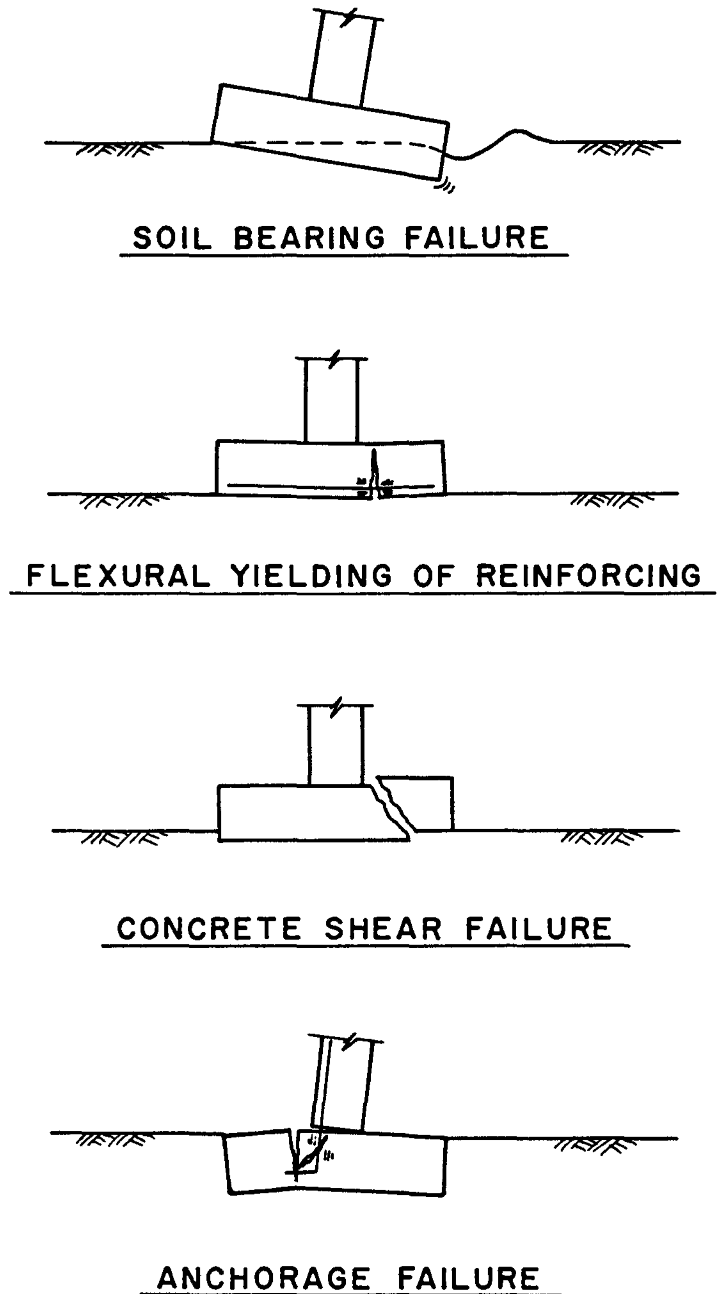


FIGURE 26. MODES OF FAILURE FOR SPREAD FOOTINGS

could potentially result in a structure collapse. Bond failure will be the most critical and should be evaluated based on the strength of the anchorage of the column main reinforcement in the footing as discussed in Section C4.8.1. Insufficient anchorage indicates that the yield capacity of the reinforcing cannot be developed and that failure will occur before the column reaches its ultimate capacity. A reduction in the effectiveness of the anchorage due to a flexural cracking of the footing is usually not a problem for unanchored spread footings because the tensile strength of the concrete is usually sufficient to prevent cracking.

A concrete shear failure in the footing could be serious because it could result in a fairly sudden loss of overturning resistance. In determining the possibility of a shear failure, the shear capacity at the critical section determined according to the AASHTO specifications should be sufficient to resist a uniform pressure equal to 1.3 times the ultimate soil-bearing capacity.

Flexural yielding of the footing is also possible but will not result in a rapid loss of overturning resistance as is the case with shear failure. Flexural capacity should be checked at the critical section according to AASHTO. This capacity should be sufficient to resist uniform footing pressure of 1.3 times the ultimate soil-bearing capacity. Flexural yielding of the footing will cause the column shear force to be limited because of statics.

If neither shear nor flexural failure will occur in the footing, then the footing capacity will be governed by a soil-bearing failure. The interaction between axial force and moments at the yield capacity of the footing may be calculated by assuming various areas of the footing to be loaded with a uniform pressure equal to the ultimate soil pressure. This will produce an interaction surface which will indicate the possibility of bearing failure only at the locations where this surface falls within the column interaction surface factored for overstrength.

B. Pile Footings Possible failure modes for pile footings are shown in Figure 27 and may be classified as follows:

- Tilting of the footing due to uplift or compression failures in the piling.
- Pullout of a pile from the footing.
- Flexural yielding of footing reinforcing.
- Concrete shear failure of the footing.
- Bond failure of the main column steel.
- Flexural or shear failure of the piling.

Unlike spread footings, the tensile stress in the concrete of footings unreinforced for uplift may be insufficient to prevent flexural cracking of the footing and a subsequent loss of column steel anchorage. This type of failure is accounted for in Section 4.8.1 of the Retrofit Guidelines.

A concrete shear failure in the footing could be serious because it could result in a fairly sudden loss of overturning resistance. In determining the possibility of a shear failure, the shear capacity at the critical section determined according to the AASHTO specifications should be sufficient to resist the shear produced by 1.3 times the ultimate capacity of the piles.

A flexural failure of the footing is also possible but will not result in a rapid loss of overturning resistance as is the case with shear failure. Flexural capacity should be checked at the critical section according to AASHTO. This capacity should be sufficient to resist the moment produced by the piles acting at 1.3 times their ultimate capacity.

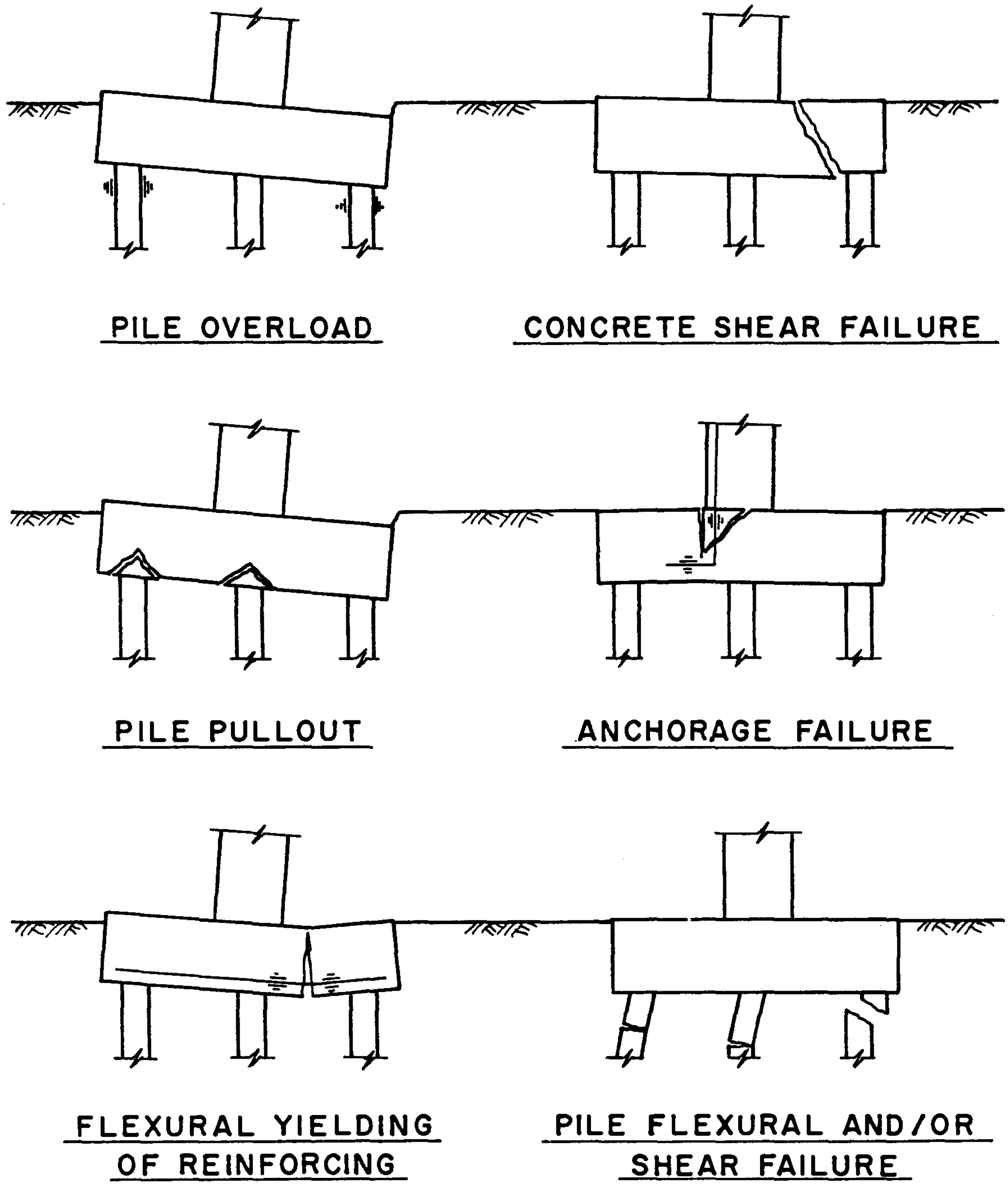


FIGURE 27. MODES OF FAILURE FOR PILE FOOTINGS

Flexural yielding of the footing will cause the column shear force to be limited because of statics.

If neither shear nor flexural failure will occur in the footing, then the footing capacity will be governed by a pile failure. The interaction surface for axial force and moments at the yield capacity of the footing may be produced by assuming the ultimate compression or uplift in various combinations of piles. Pile uplift may be limited by pullout of the pile from the footing or by the pile withdrawal force. In the case where piles will pull out of the footing, a lower ductility indicator is proposed because of the more brittle nature of this type of failure.

C4.9 CAPACITY/DEMAND RATIOS FOR ABUTMENTS

Abutment displacement capacities are limited to those which are likely to cause problems with accessibility to the bridge. Based on experience from past earthquakes, displacement capacities of 3 inches in the transverse direction and 6 inches in the longitudinal direction were chosen. These values are based largely on engineering judgement and are likely to be modified as more experience is gained.

C4.10 CAPACITY/DEMAND RATIOS FOR LIQUEFACTION

Bridge failures resulting from seismic activity have often been classified as failures resulting from permanent displacement of the foundations, or from structural failures arising from dynamic loading. The majority of severe seismic bridge failures have resulted from liquefaction induced permanent displacement of the foundation systems. Despite this fact, the emphasis in both research and design has been on preventing structural failures. This perhaps reflects the problem that foundation failures are difficult to treat quantitatively, whereas structural response is more amenable to analysis and generally represents a preventable type of failure.

Designers have generally approached the problem of liquefaction by attempting to select bridge sites at which such failures are unlikely. In many cases, however, the use of such sites is unavoidable. In the case of existing bridges, vulnerable sites may have been used without a full understanding of the consequences. Designers faced with improving the earthquake resistance of such bridges should take advantage of knowledge gained from the performance of bridges in past earthquakes to identify collapse mechanisms and evaluation procedures.

A qualitative description of mechanisms of foundation failure or displacement arising from liquefacton is provided in Chapter C3. Bridge failures in recent Alaskan and Japanese earthquakes are probably the best documented examples [19]. In many other earthquakes where bridge damage has been reported as a result of liquefaction, modes of foundation failure have been similar to the Japanese and Alaska earthquake case histories. Multispan bridges with unrestrained simply supported spans have usually suffered the most damage.

Foundation conditions which are susceptible to liquefaction are common to bridges that cross waterways where foundation soils have been deposited over the years by flowing water. These soils are often loose, saturated cohesionless deposits, and are most susceptible to liquefaction. It is noteworthy that it is a combination of earthquake intensity and duration that can cause liquefaction. In the 1964 Alaskan earthquake, it is estimated that maximum ground accelerations as low as 0.1 to 0.2 g were responsible for the extensive and widespread bridge foundation failures [18]. The duration of strong shaking was rather long, however, lasting more than 90 seconds. Therefore, bridge sites located some distance from a major fault could still be subjected to liquefaction failure if the necessary soil conditions are present.

Methods for assessing the liquefaction potential of site soils are provided in the Seismic Design Guidelines [1]. Two basic approaches are typically used, namely empirical methods based on blow count correlations for sites which have not liquefied, and analytical techniques based on the laboratory determination of liquefaction strengths and dynamic site response analyses. A rough indication of the potential for liquefaction may be obtained by making use of empirical correlations between earthquake magnitude and epicenter distance as described in the Seismic Design Guidelines.

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COMMENTARY

CHAPTER 5 - SEISMIC RETROFITTING CONCEPTS

C5.1 GENERAL

Seismic retrofitting of bridges has only become customary in the United States since the 1971 San Fernando earthquake. The California Department of Transportation has initiated a retrofitting program designed to mitigate the chances for the more spectacular types of bridge failures experienced during that earthquake. Retrofitting efforts have primarily been directed toward tying bridge superstructures together and preventing a loss of support at the bearings. Other retrofitting techniques, such as column strengthening, have not been developed to the same extent as joint restrainers because they are generally more expensive and do not result in the maximum amount of protection per dollar spent. This does not mean, however, that other retrofitting techniques should be ruled out on future retrofitting projects.

A comprehensive study of bridge retrofitting was conducted by the Illinois Institute of Technology Research Institute with funding from the Federal Highway Administration.(11) This study was a pioneering effort in this country to develop procedures for determining when and to what degree bridges should be retrofitted. Several retrofit methods were also proposed.

New Zealand has also retrofitted several of their bridge structures in recent years. They have used elastrometric bearings with a lead core as a replacement for conventional bearings. This approach is designed to alter the dynamic response of the bridge, thus increasing its seismic resistance.

C5.2 SEISMIC PERFORMANCE REQUIREMENTS

The seismic performance requirements for retrofitting should be selected with the goal of preventing a catastrophic failure during a major earthquake. Performance requirements will vary depending on the importance of the bridge. A potential loss of accessibility, for example, may not warrant retrofitting except for the most important bridges. Because of the many variables involved in retrofitting existing bridges, the designer is expected to use his best judgement in selecting the most appropriate seismic performance requirements for a particular bridge.

C5.3 BEARINGS AND EXPANSION JOINTS

Many retrofit techniques for bearing and expansion joints will require the coring of existing concrete. When coring is to be used, there are at least two items that should be considered.

One item is the clearance required for coring equipment. The minimum distance between the center of a cored hole and an adjacent surface should be 3 inches. For holes larger than 6 inches, the edge of the hole may be flush against the adjacent surface. In addition, cored holes should be located so that a minimum of 4 feet clearance exists on at least one side along the centerline of the hole. These clearances are shown in Figure 28.

The other item that needs to be considered is the potential for interference with major reinforcing steel, expansion joint hardware, and prestressing tendons. Special

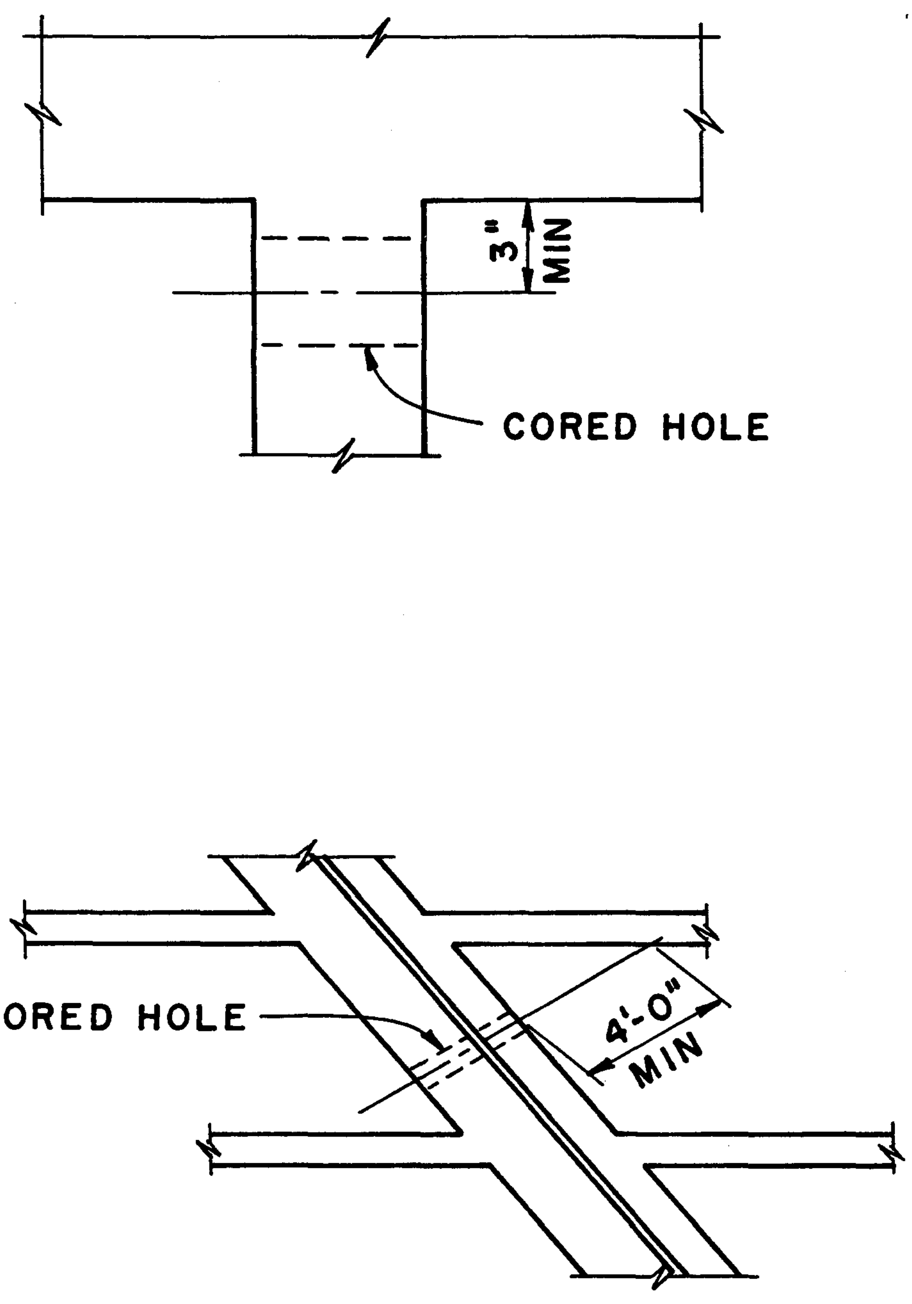


FIGURE 28. REQUIRED CLEARANCES FOR CONCRETE CORING

care should be taken to avoid structurally critical reinforcement and prestressing rods or large multi-wire or multi-strand tendons in post-tensioned bridge members. If the type of prestressing system used cannot be determined from the "as built" plans or construction records, rods and large tendons should be assumed. Construction personnel should be alerted to the presence of these elements so that appropriate precautions can be taken in the field.

Restrainers must be physically attached to the existing structure and care should be taken that critical components are not weakened or overloaded. Brackets and connections should be designed for a 25% overstress in all the restrainers. In addition, they should be designed to resist the eccentricity resulting from the possible failure of some of the restrainer elements with the remainder of the elements working at ultimate strength.

Bearing plates on concrete surfaces should be designed to prevent concrete failures when the restrainer elements are working at 25% overstress. Concrete walls subject to a punching failure should be strengthened. This type of strengthening is often required at expansion joint diaphrams.

Restrainer devices attached to easily accessible areas should have bolt threads peened after installation is complete to prevent loss of components to vandalism.

C5.3.1 LONGITUDINAL JOINT RESTRAINERS

Longitudinal joint restrainers may be used as a retrofit measure to tie a structure together and prevent a loss of support at the bearings. An ideal restrainer should be capable of resisting appropriate forces, resisting movements of bridge segments, dissipating energy, and returning the structure segments to their relative pre-earthquake positons. These restrainers should have redundancy to allow for defects in single restrainer units.

Restrainers should be placed symmetrically to minimize the introduction of eccentricities. The consequences of a premature restrainer failure should also be carefully considered. For example, the restrainer detail shown in Figure 29 may be undesirable. In the event of a premature failure of one of the cables, the resulting eccentric load could tear the web out of the girder and cause a serious loss of structural capacity unless the web has been adequately reinforced to prevent such a failure.

Consideration should also be given to minimizing access problems during construction and maintenance. For example, in box girders, the number of bays in which restrainers are placed should be kept to a minimum.

Longitudinal restrainers should be oriented along the principal direction of expected movement. If piers are rigid in the transverse direction as shown in Figure 30, the movement of the superstructure will be along the longitudinal axis of the bridge and the restrainers should be placed accordingly. However, in a skewed bridge with transversely flexible supports, superstructure rotation can occur. In this case restrainers will be more effective if placed normal to the expansion joint as shown in Figure 31.

When an expansion joint exists at a pier, restrainers at the expansion joint should provide a positive tie to the pier as shown in Figure 32. This detail will tend to prevent the bearings from becoming unseated. Since each of the restrainers can only resist movement in one direction, and because closure of the expansion joint will transfer the inertia forces of one span to the adjacent span, each restrainer must resist the

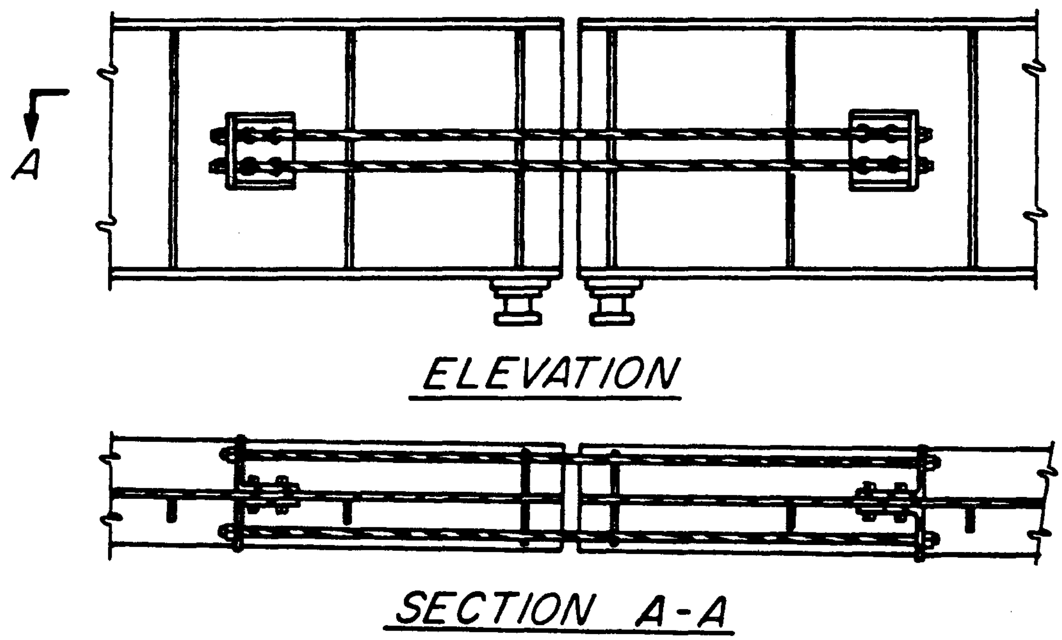


FIGURE 29. UNDESIRABLE RESTRAINER DETAIL

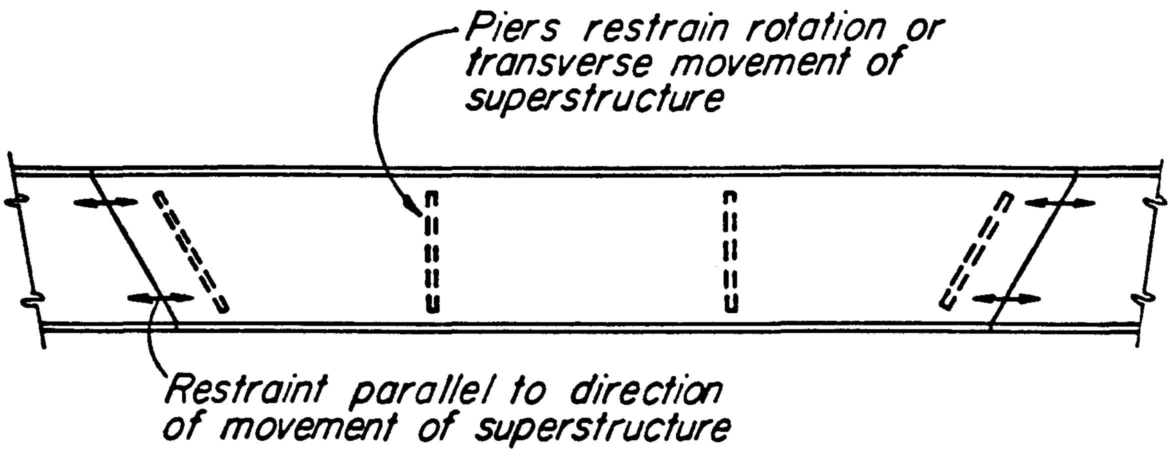


FIGURE 30. RESTRAINER ORIENTATION—TRANSVERSELY RIGID SUPPORTS

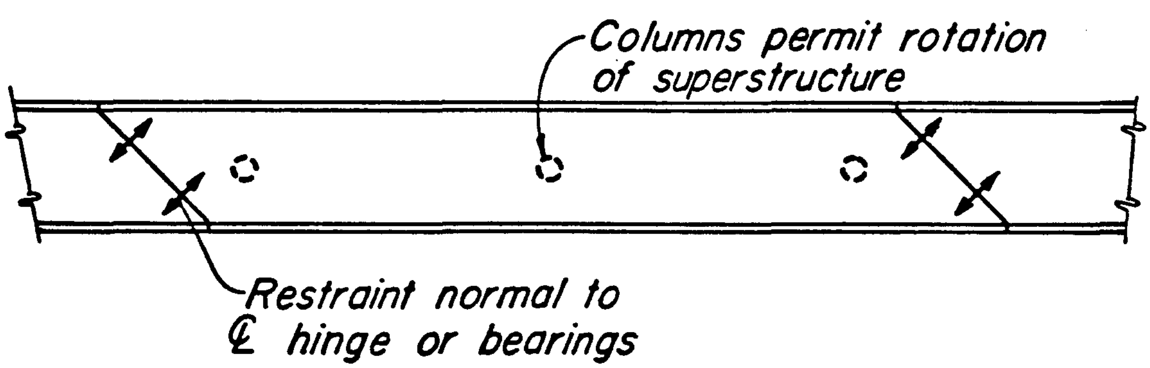


FIGURE 31. RESTRAINER ORIENTATION—TRANSVERSELY FLEXIBLE SUPPORTS

inertia forces of both spans. Depending on the configuration of the restrainers at adjacent expansion joints it is possible that the inertia forces of other spans should also be included. Notice in Figure 32 that the restrainers are connected to the bottom flange. This will prevent the possibility of tearing the web as mentioned earlier, but will reduce vertical clearance under the bridge.

In some cases it may be appropriate to forego the positive tie to the pier. In this case adjacent spans may be tied as shown in Figure 33. This should be considered only when the cummulative openings of expansion joints is small enough to prevent the spans from becoming unseated, when positive ties could excessively overload the pier and/or when one of the spans has an adequate existing connection to the pier. Although this retrofit technique is unlikely to prevent rocker bearings from toppling, collapse of the span will be prevented by the pier cap. Minor emergency repairs could quickly restore the usefulness of the bridge.

Steel cables and bars acting in direct tension have been the most frequently used method for restraining expansion joints against excessive movements. These devices do not dissipate any significant amount of energy because they are generally designed to remain elastic. Cable and bar restrainers may permit the ends of girders to be damaged, but the damage will usually be repairable and not extensive enough to allow the spans to lose support. Although cables and bars do not meet all the criteria of an ideal restrainer, they are relatively simple to install and are an economical means for preventing a catastrophic failure during an earthquake.

The California Department of Transportation (Caltrans) has been retrofitting bridges with longitudinal expansion joint restrainers since the San Fernando earthquake of 1971.(1) They have used two types of restrainer materials. The first type is 3/4 inch galvanized steel wire rope (6 strands with 19 wires per strand) identical to the material commonly used to anchor the ends of barrier railings. The second type of material is 1-1/4 inch high-strength steel bars. These bars are also galvanized and conform to ASTM A-722 standards. In addition, these bars are required to provide elongation of at least 7% in 10 bar diameters before fracture.

Caltrans performed several tests to study the performance of wire rope and bars under repeated cycles of loading near or beyond the yield stress. The graph shown in Figure 34 was developed by loading specimens to the specified yield stress (assumed to be 0.85 f_y for the wire rope) for 14 cycles and then to failure. Notice that both materials are capable of elongating beyond the elastic limit. The 1-1/4 inch bars are stiffer, yielding at approximately 0.7 inches of elongation over a 114-inch length. These bars are also more ductile and will continue to stretch to about 7.4 inches before fracture. On the first cycle of loading, wire rope undergoes a conditioning in which slack in the strands is taken up. On subsequent loadings a 114 inch specimen will elongate approximately 1-1/2 inches before yield. Total elongation after the initial conditioning is approximately 4.5 inches prior to failure.

In a second series of tests, specimens were loaded by applying one-inch increments of displacement up to failure. Between each displacement increment the specimen was unloaded to zero tension. Typical results from these tests are shown in Figure 35. It is interesting to note that 1-1/4 inch bars will withstand displacements up to about 11 inches, which is greater than experienced in the first series of tests where loading conditions were different. The wire rope, on the other hand, fails at 5 inches of elongation, which is slightly less than demonstrated by the first series of tests.

The California Department of Transportation has no established rule as to when

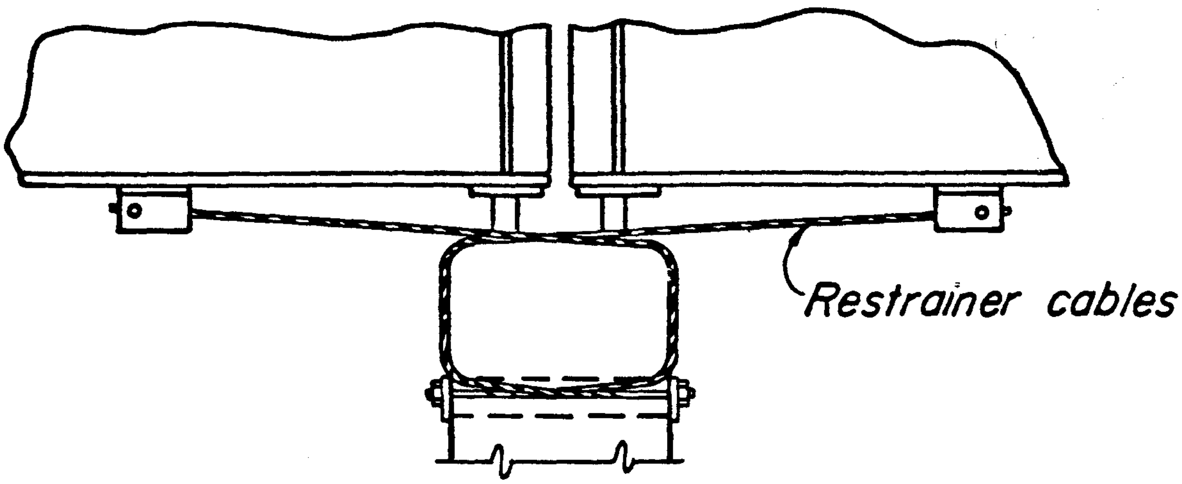


FIGURE 32. RESTRAINER AT PIER—POSITIVE TIE TO PIER

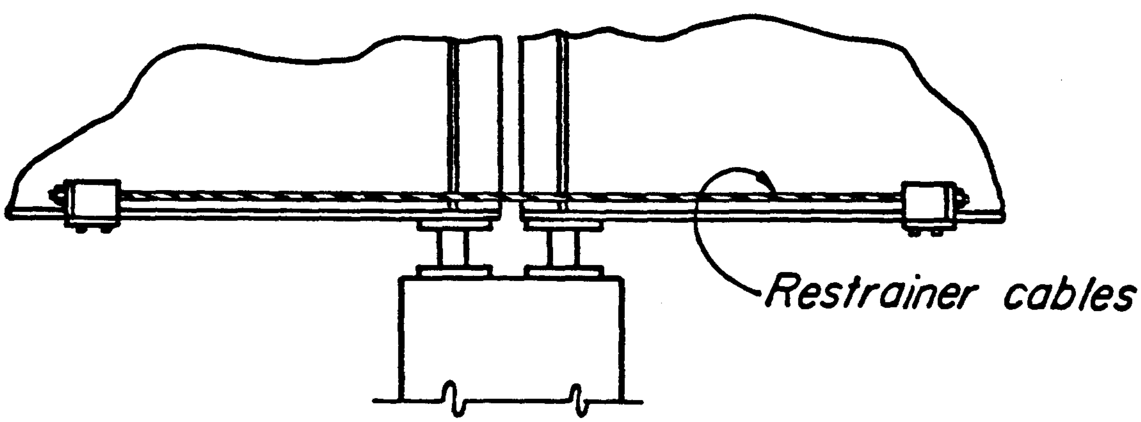


FIGURE 33. RESTRAINER AT PIER—NO POSITIVE TIE TO PIER

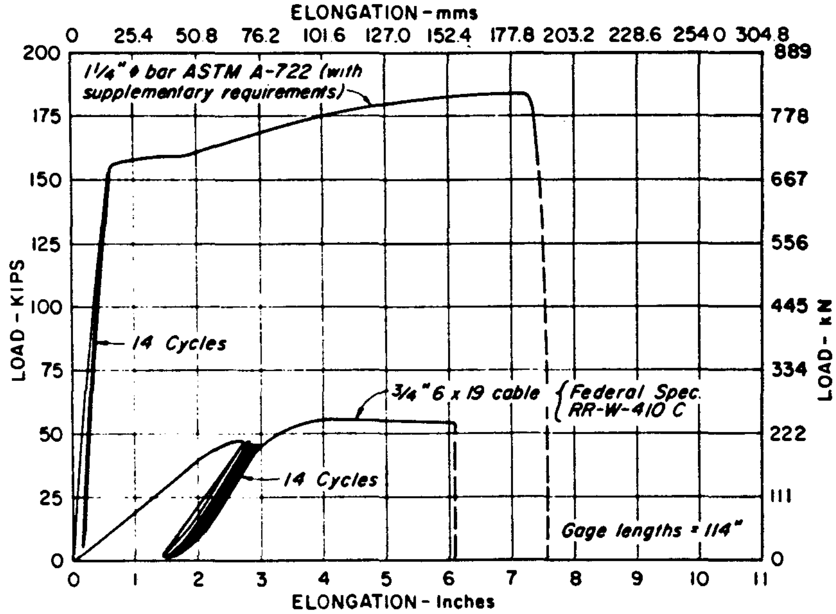


FIGURE 34. CYCLIC TESTS OF RESTRAINER MATERIAL TO YIELD

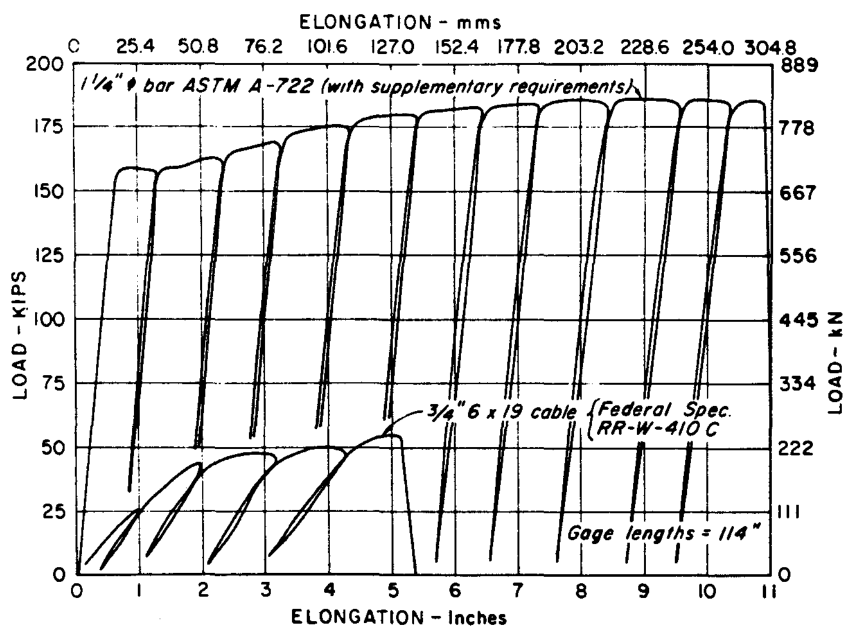


FIGURE 35. CYCLIC TESTS OF RESTRAINER MATERIAL BEYOND YIELD

wire rope or bars are preferred. Since restrainers are designed to perform elastically, the extra ductility of the 1-1/4 inch bars is not considered to be a particular advantage. An important consideration is the amount of movement allowed at the expansion joint. Elastic stretching should be limited because excessive movement can result in a loss of support at narrow bearing seats. On the other hand, an overly stiff restrainer, although more effective in limiting movement, will be subjected to more force. In California, the results of multi-modal spectral analyses are used to select the right combination of restrainer stiffness and strength. The number and length of wire ropes or bars are selected on this basis. A tentative simplified method for designing longitudinal restrainers has recently been developed by the California Department of Transportation and is described in Appendix C.

Wire ropes often has an economic advantage, since shorter lengths are required to allow for a given amount of movement. In addition wire rope is flexible and more able to accommodate transverse and vertical movements. If bars are used, transverse and vertical restrainers may be required to prevent a shear failure in the bars.

Figure 36 shows a method for retrofitting a mid-span expansion joint in a concrete box girder. Either cables or rigid steel bars may be used to prevent separation of the joint. Concrete bolsters are sometimes necessary to strengthen the concrete diaphrams to accommodate the force transmitted from the restrainers.

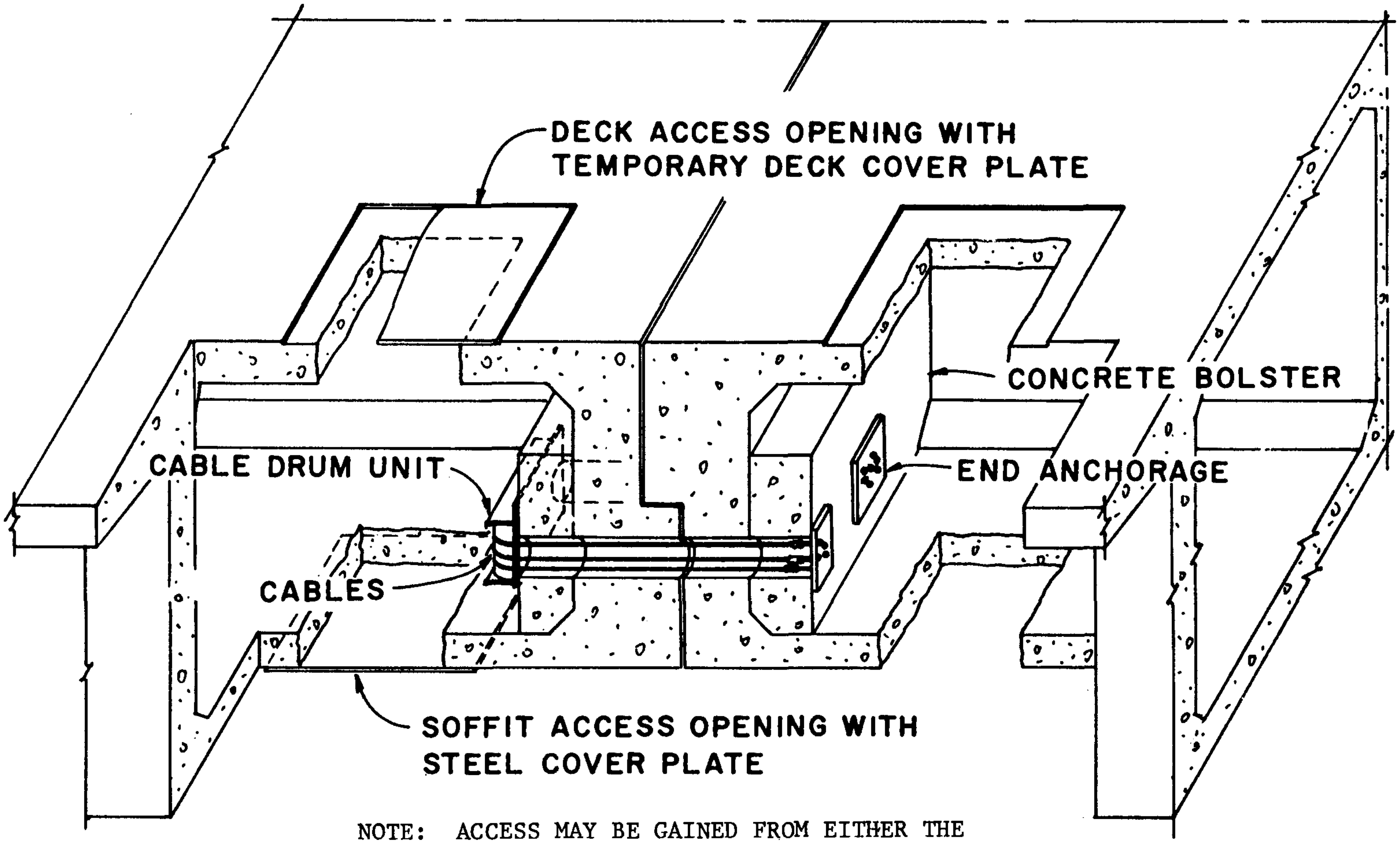
In open-web concrete bridges such as "T" beams, the lack of support at the bottom edge of the diaphram may make it necessary to locate restrainers as shown in Figure 37. This detail is usually restricted to situations where the restrainer force requirements are relatively low. When the joint is located at a bent, a positive tie between the substructure and the superstructure is preferred to this detail, unless the bridge is relatively short with a small number of spans and bent caps wide enough to prevent loss of end support.

An alternate method for restraining joints when the diaphram is weak is to attach restrainers to the sides of the girders or to the underside of the deck. In this case, it is necessary to locate restrainer anchors a sufficient distance from the joint to prevent damage to the ends of the span. A detail in which restrainers are anchored to the deck is shown in Figure 38. A direct tie to the bent is difficult when anchoring restrainers in this way, but is easier when restrainers are anchored to the girders.

Certain special situations permit some variation in the use of restrainer details. For example, Figure 39 shows continuous cables used to restrain a suspended span. Large restrainer lengths often make it necessary to increase the number of restrainers to limit the relative movement at the joints. Therefore, although anchorage costs are reduced with this detail, it is uneconomical to use excessively long restrainers.

C5.3.2 TRANSVERSE BEARING RESTRAINERS

Transverse restrainers are necessary in many cases to keep the superstructure from sliding off the bearings. As discussed in Section C3.5, the potential for the failure of existing transverse restraint may not always warrant retrofitting. Conditions that are particularly vulnerable exist when high concrete pedestals serve as bearing seats at individual girders, when bearing seats are narrow and highly skewed, or on a two-girder bridge in which the transverse distance between the bearing and the edge of the seat is small.



NOTE: ACCESS MAY BE GAINED FROM EITHER THE DECK OR SOFFIT BUT NEED NOT BE BOTH

FIGURE 36. LONGITUDINAL JOINT RESTRAINER FOR CONCRETE BOX GIRDER

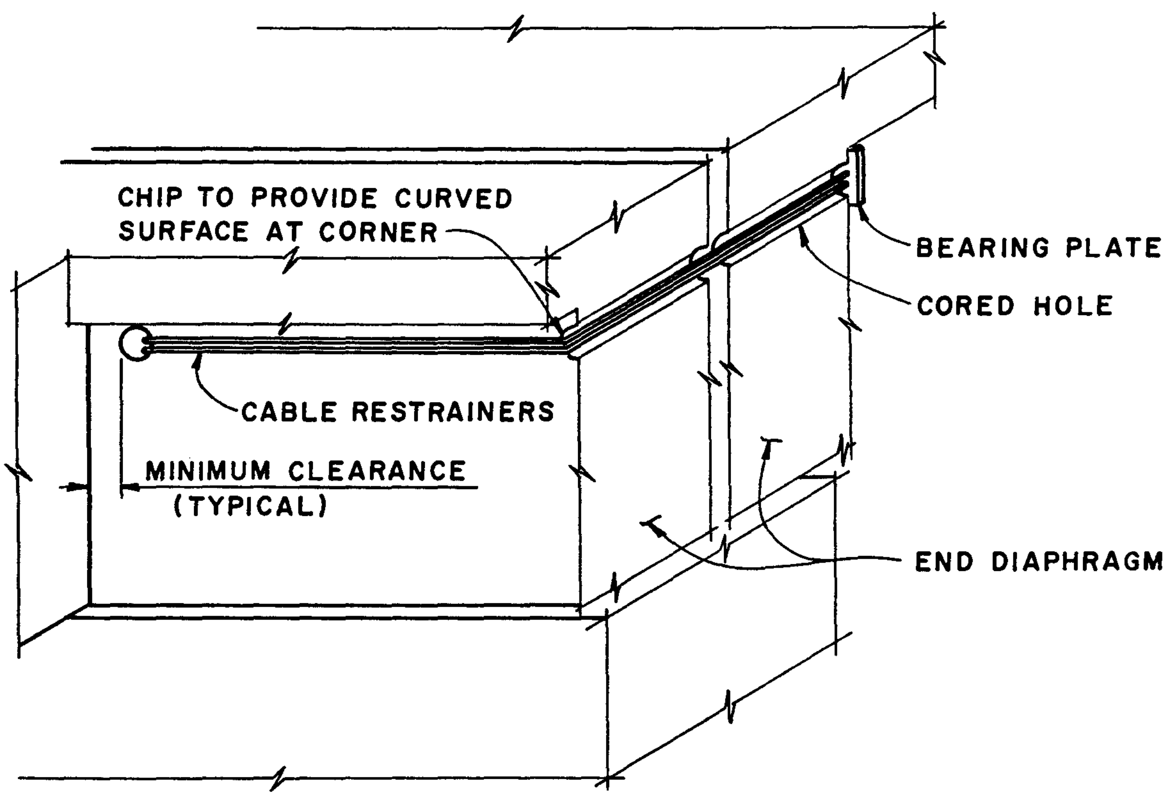


FIGURE 37. EXPANSION JOINT RETROFIT DETAIL FOR CONCRETE T-BEAM

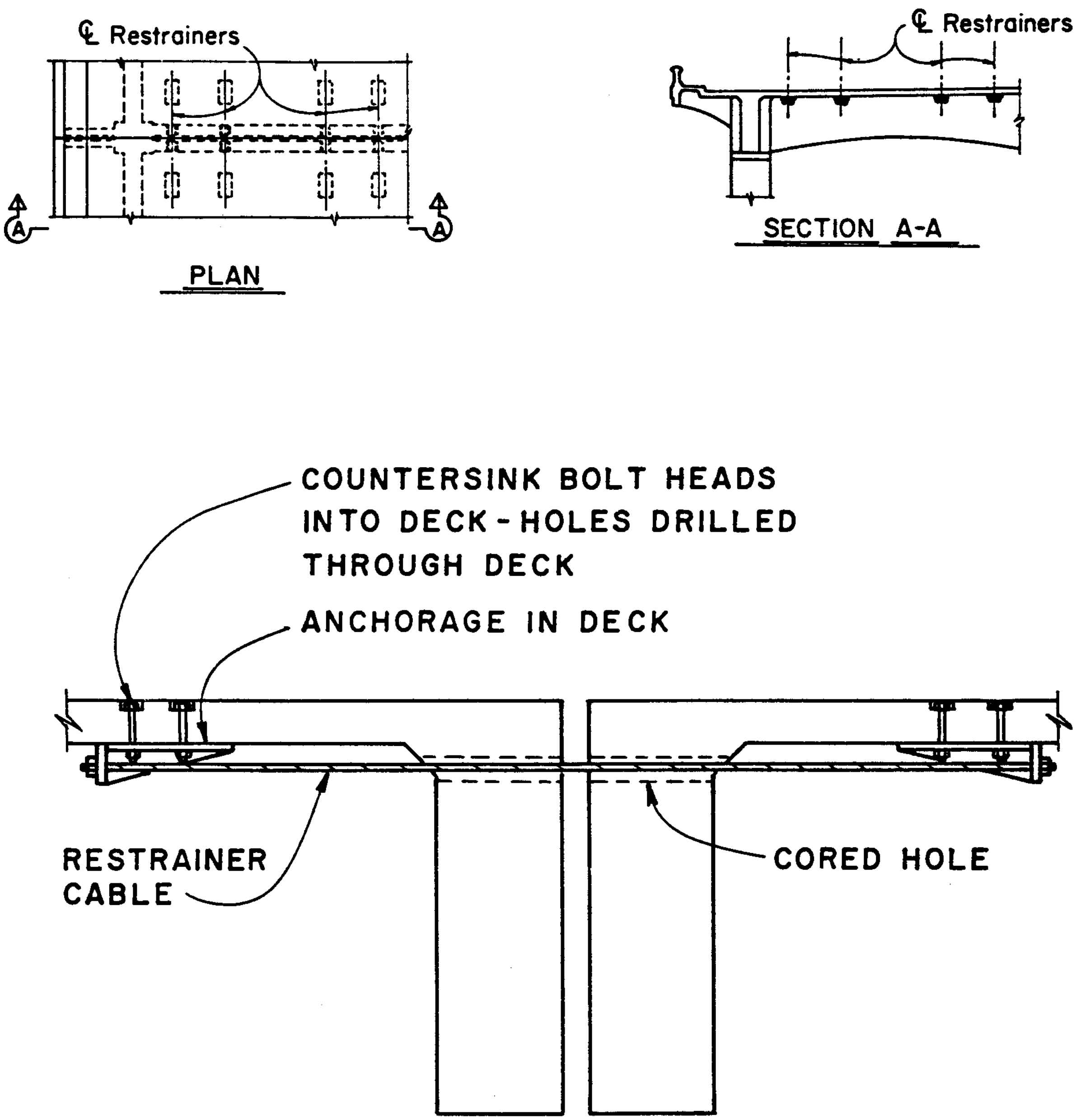


FIGURE 38. EXPANSION JOINT RESTRAINERS TIED TO THE CONCRETE DECK

Transverse bearing restrainers are usually designed to resist load elastically. Analytical studies have shown that when columns yield, additional force will be transferred to elements that are designed not to yield.[2] In addition, transverse restrainers may be built with slightly different construction tolerances which will cause them to resist load unevenly. To account for the possible increased load carried by these restrainers, the elastic forces from an analysis are increased by 25% for design.

One method that has been used to provide transverse restraint in concrete structures employs a double extra strong steel pipe filled with concrete that passes through the joint. This concept is shown in Figure 40. Design is based on bearing of the pipe against the walls of the cored hole. The full concrete compressive strength may be relied on in well reinforced expansion joint diaphragms. Care should be taken not to rely on the full strength of accute corners at highly skewed joints because they can easily break off.

C5.3.3 VERTICAL MOTION RESTRAINERS

Although vertical accelerations are not considered explicitly in the seismic analysis, vertical motion restrainers may be desirable in some situations because these can prevent damage or loss of stability to the bearings. Use of vertical motion restrainers should be considered only when longitudinal restrainers are contemplated and the bridge is in SPC-D. It is felt they can be installed at a very low unit cost at the time longitudinal restrainers are installed compared to the cost of installing them separately at some other date. A possible hold-down detail is shown in Figure 41.

C5.3.4 BEARING SEAT EXTENSION

It is preferable that bearing seat extensions be constructed so that load transfer will be directly to the foundation. Bearing seat extensions anchored to an existing vertical face of concrete with dowels or anchor bolts are not considered as reliable because of the large vertical and horizontal forces to which a bearing seat will be subjected in the event the superstructure were to fall off its bearings onto the extension. Consideration should be given to post-tensioning bearing seat extensions when direct load transfer to the foundation is not feasible.

If bearing seats are extended, their width should be increased to the minimum seat width recommended in the Seismic Design Guidelines.[3] These recommended seat widths are the best judgement of the team of engineers that developed the guidelines and are intended to reflect the possibility of large relative movements at the bearings resulting from the overall inelastic response of the bridge, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motion.

The design forces for bearing seat extensions are intended to encourage designers to consider the large forces to which a bearing seat may be subjected during an earthquake large enough to cause bearings to become unseated. Two loading conditions are specified. The first load considers vertical forces only and is intended to account for the large impact forces that can result when the superstructure drops from the bearings onto the bearing seat. The second load considers both the horizontal and vertical loads that can develop when the superstructure is resting on the bearing seat extension and is still being subjected to earthquake ground motions.

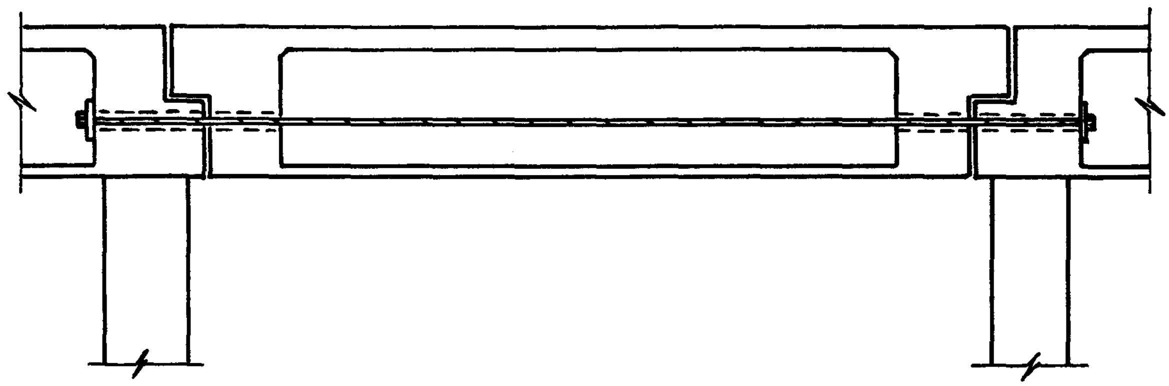


FIGURE 39. RESTRAINER RETROFIT OF A SUSPENDED SPAN

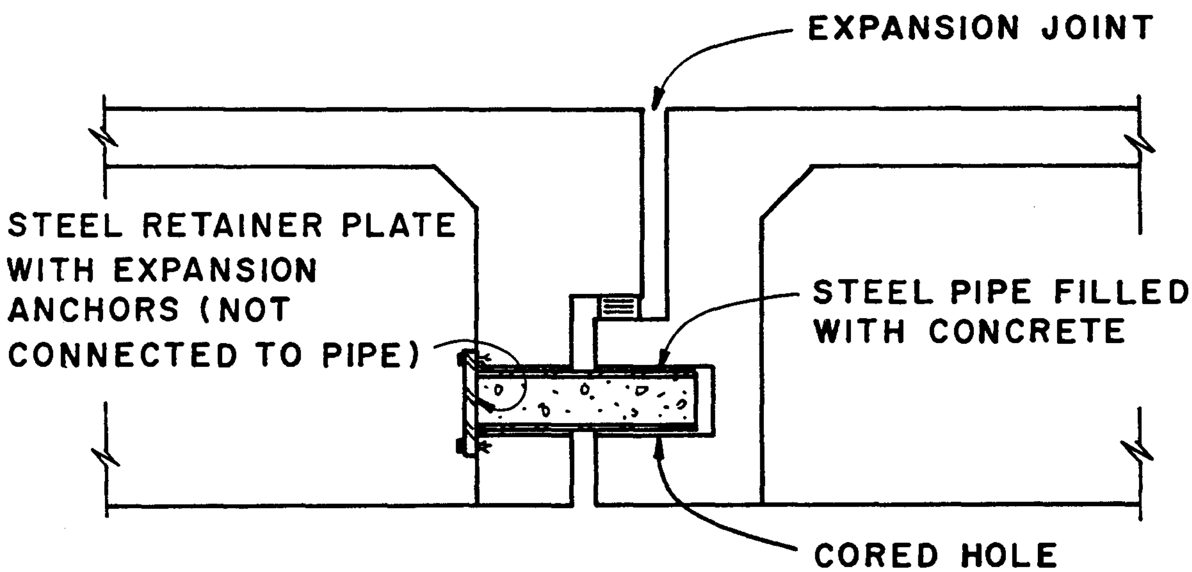


FIGURE 40. TRANSVERSE RESTRAINER RETROFIT FOR A CONCRETE BRIDGE

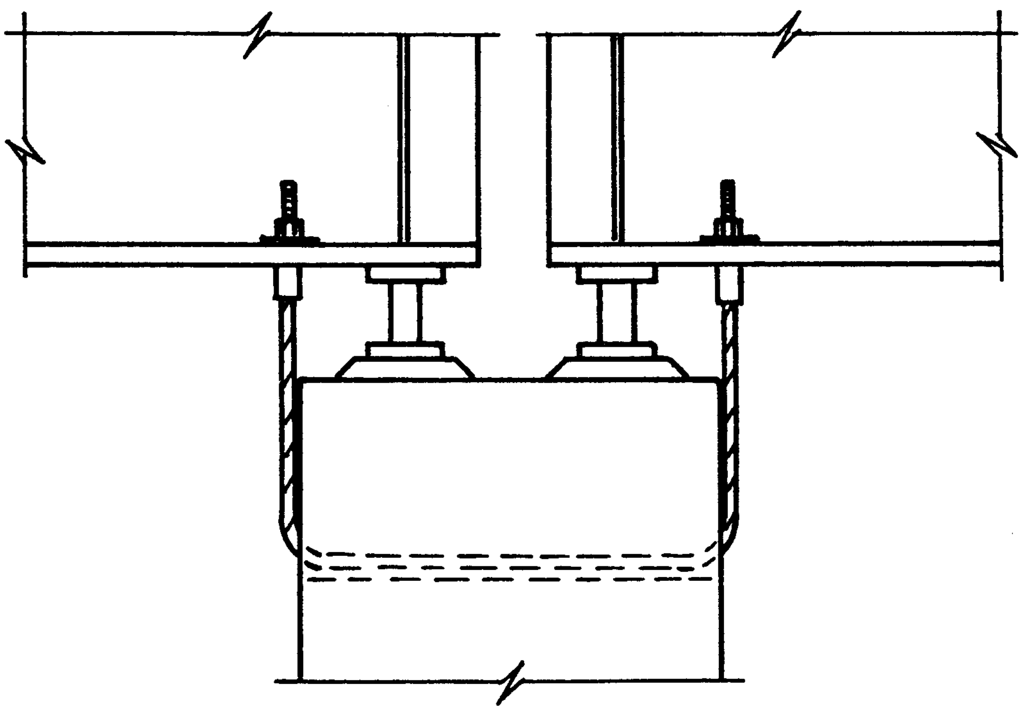


FIGURE 41. VERTICAL MOTION RESTRAINER RETROFIT

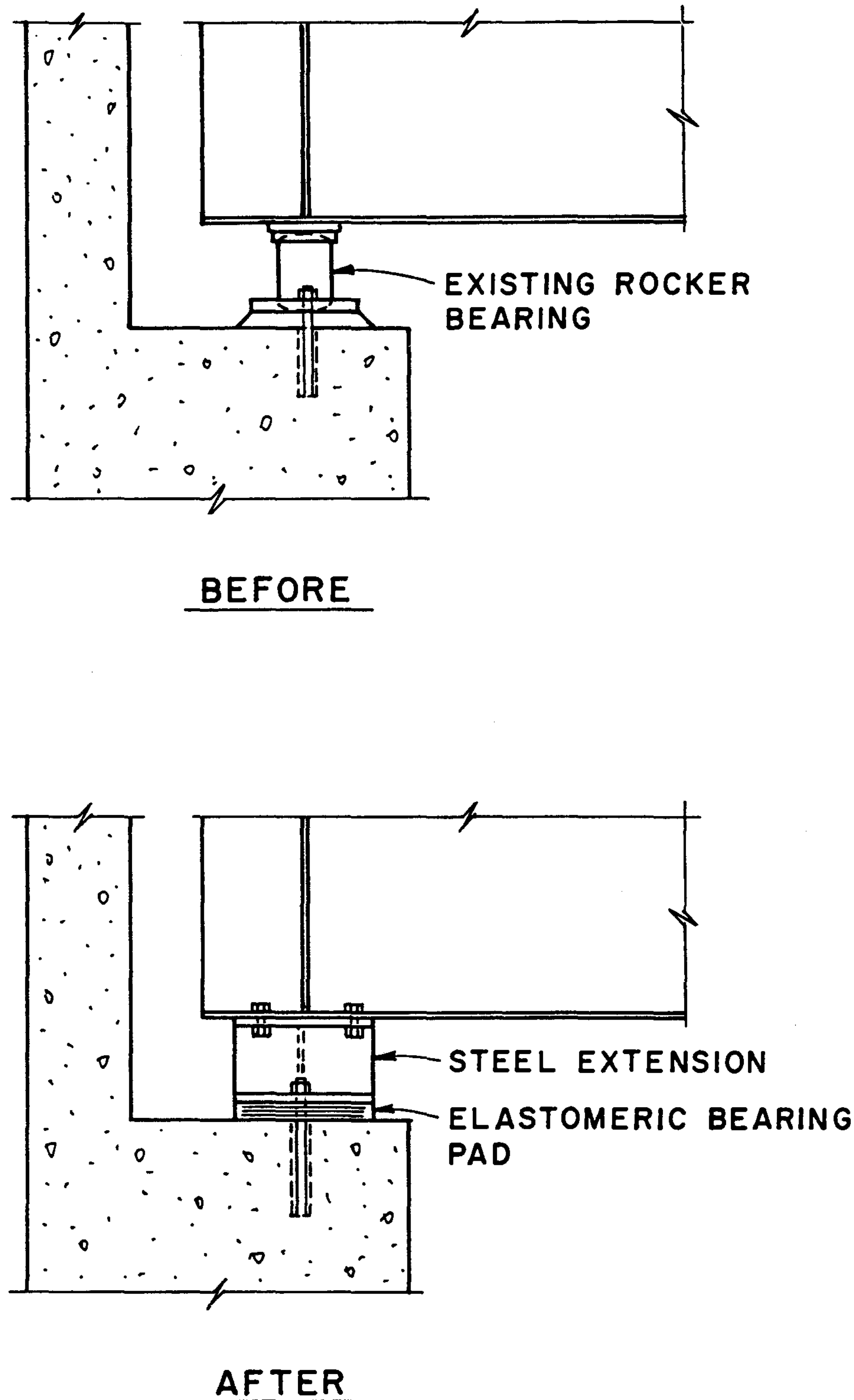


FIGURE 42. REPLACEMENT OF ROCKER BEARINGS—STEEL EXTENSION

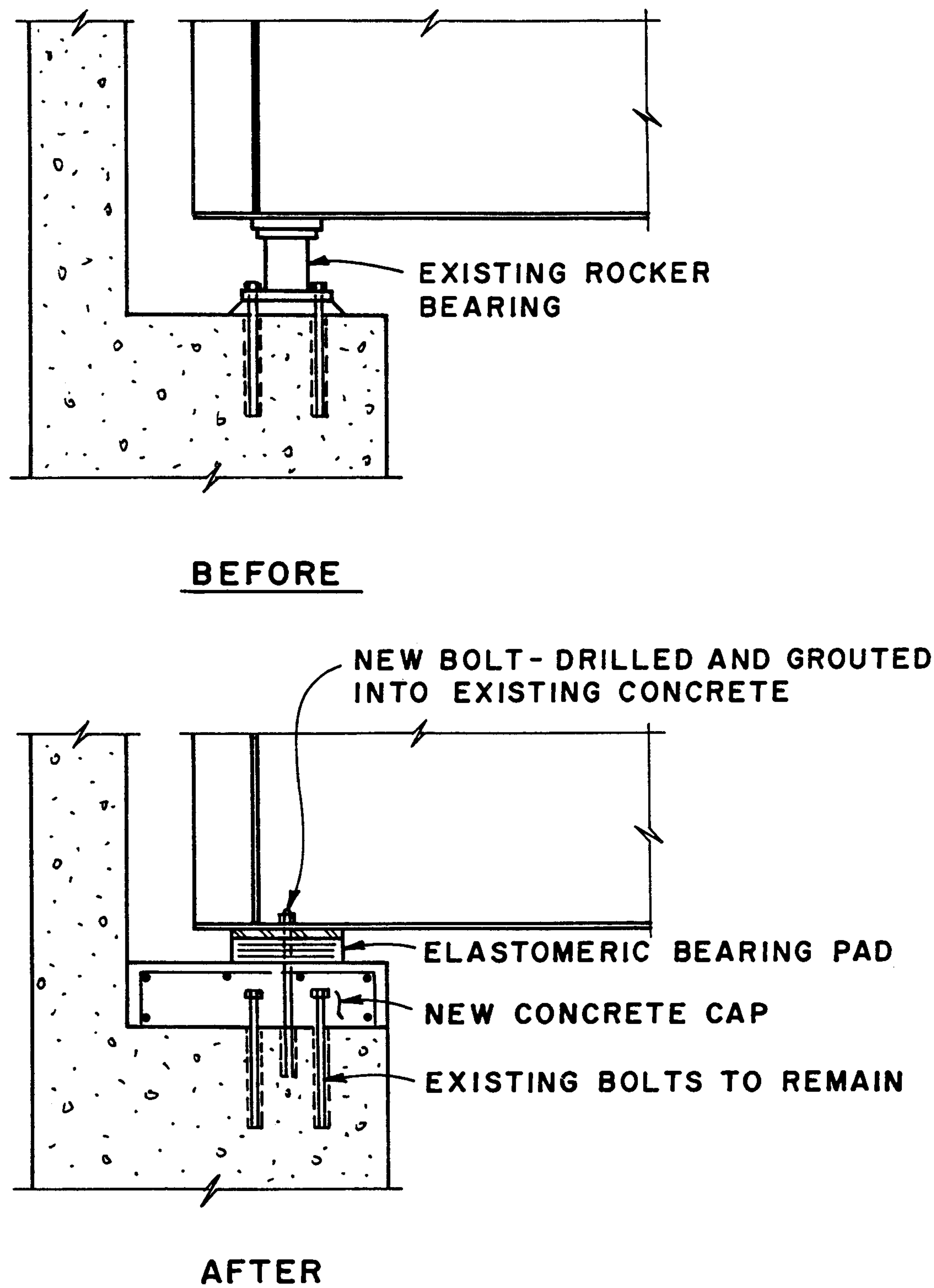


FIGURE 43. REPLACEMENT OF ROCKER BEARINGS—CONCRETE CAP

C5.3.5 REPLACEMENT OF BEARINGS

Steel rocker bearings are particularly vulnerable to damage during an earthquake. This has been demonstrated several times in the past. This type of bearing is a prime candidate for replacement by more seismically resistant bearings such as elastomeric bearing pads or for strengthening by other means. This applies to "fixed" as well as "expansion" bearings. Some retrofitting projects designed by the California Department of Transportation are directed toward the replacement of these vulnerable bearing systems.(4)

In one method, high rocker bearings are replaced by a prefabricated steel bearing assembly and elastometic bearing pads. The steel bearing assembly was necessary to maintain the proper elevation of the superstructure and to provide for the rotational and translational movement at the bearing. The details for this retrofit scheme are shown in Figure 42.

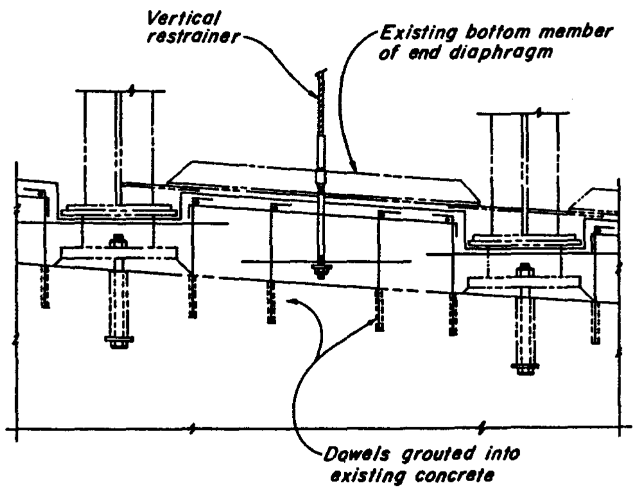
Another possible solution to replacing steel rocker bearings is shown in Figure 43. In this case a concrete cap is used to build up the elevation difference between a replacement elastomeric bearing and the original high steel rocker bearing. With this method of replacement, the concrete cap can be constructed at a higher elevation between girders to provide a transverse shear key. In addition, vertical motion restrainers can be anchored in the new concrete cap.

At "fixed" bearings it is often appropriate to completely embed existing rocker bearing pedestals in concrete as shown in Figure 44. This will prevent shear failure and toppling of the bearings. In addition, if spans were to become displaced from the bearings the concrete cap would prevent collapse. Again the concrete cap can double as a shear key and anchorage for vertical motion restrainers.

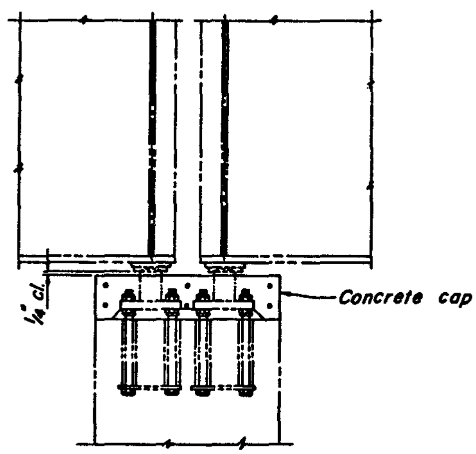
C5.3.6 SPECIAL EARTHQUAKE-RESISTANT BEARINGS AND DEVICES

Special earthquake-resistant bearings and devices utilize the concepts of isolation, energy absorbtion, and/or restraint to limit seismic forces and displacements to acceptable levels. Ideally, in addition to performing under normal service conditions, an earthquake resistant bearing should be capable of resisting seismically induced forces, restricting relative displacements within the bridge, dissipating energy, and returning the structure to its pre-earthquake position. Conceptually, a bearing system having these capabilities might be composed of the components shown in Figure 45. Vertical support would be provided by a flexible bearing and/or sliding support isolator. In the case of a "fixed" bearing, a fuse would be used to prevent movement under service conditions, but would fail during a large earthquake. During rapid movement, a motion induced arrester would engage an energy dissapator or stopper. Excessive relative displacements would be prevented by a restrainer with a gap to allow limited displacements. Following an earthquake, the flexible support would provide a restoring force to bring the structure back to its pre-earthquake position. Because it is difficult to conceive of a self-contained bearing with all of these capabilities, it is useful to think in terms of a "bearing system" that may be composed of bearings and other devices. Although in practice it is difficult, if not impossible, to design a cost-effective ideal bearing system, this concept will be useful in assessing actual bearing system designs.

A considerable amount of interest currently exists in the improvement of bearing systems to provide greater earthquake resistance. In New Zealand and Japan many innovative ideas have been implemented into the construction of bridges.(5,6) These



MODIFIED
FIXED ROCKER BEARING
PART ELEVATION



MODIFIED FIXED ROCKER
BEARING
AT PIER

FIGURE 44. "FIXED" BEARING RETROFIT BY EMBEDMENT IN CONCRETE

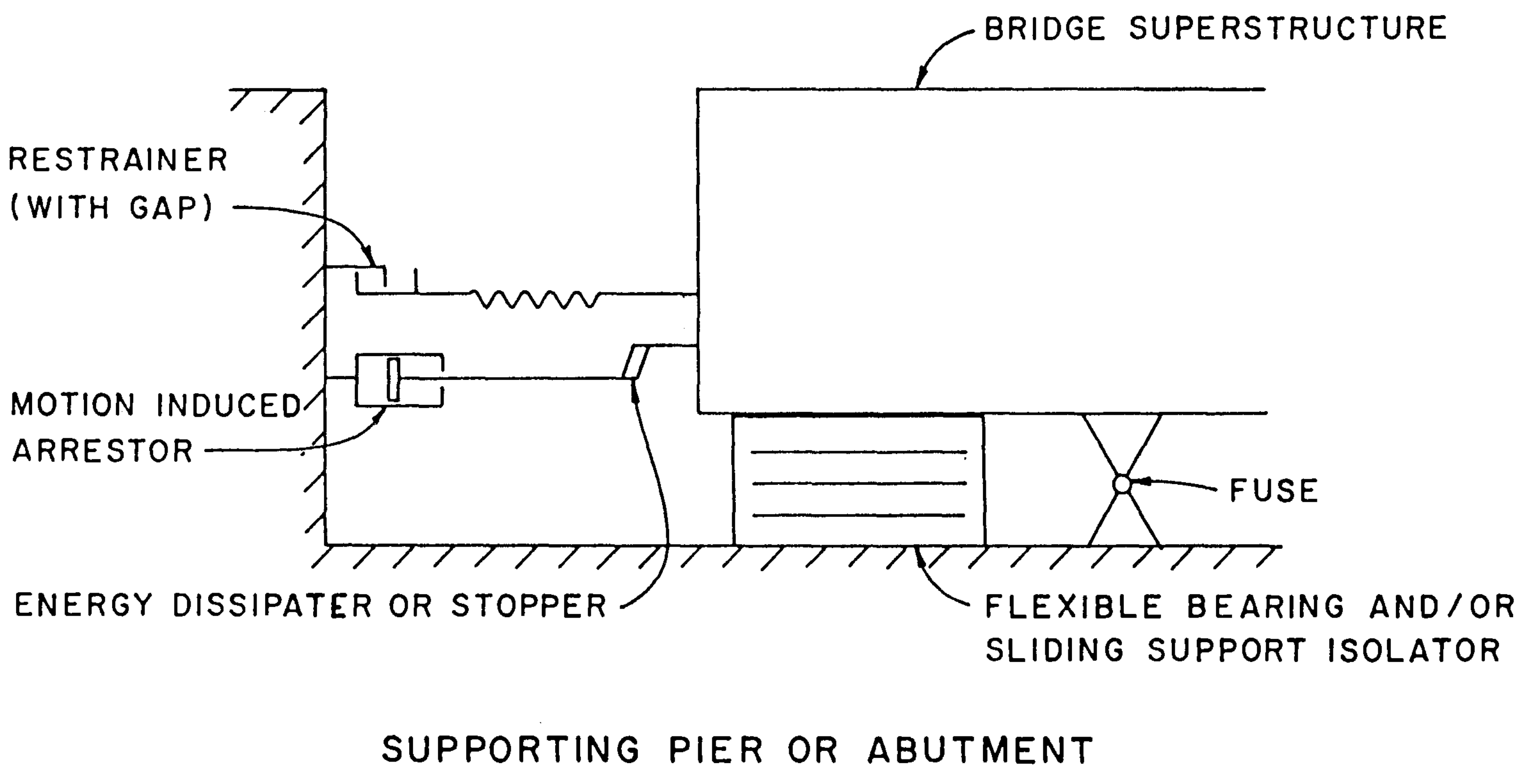


FIGURE 45. AN "IDEAL" BEARING DEVICE

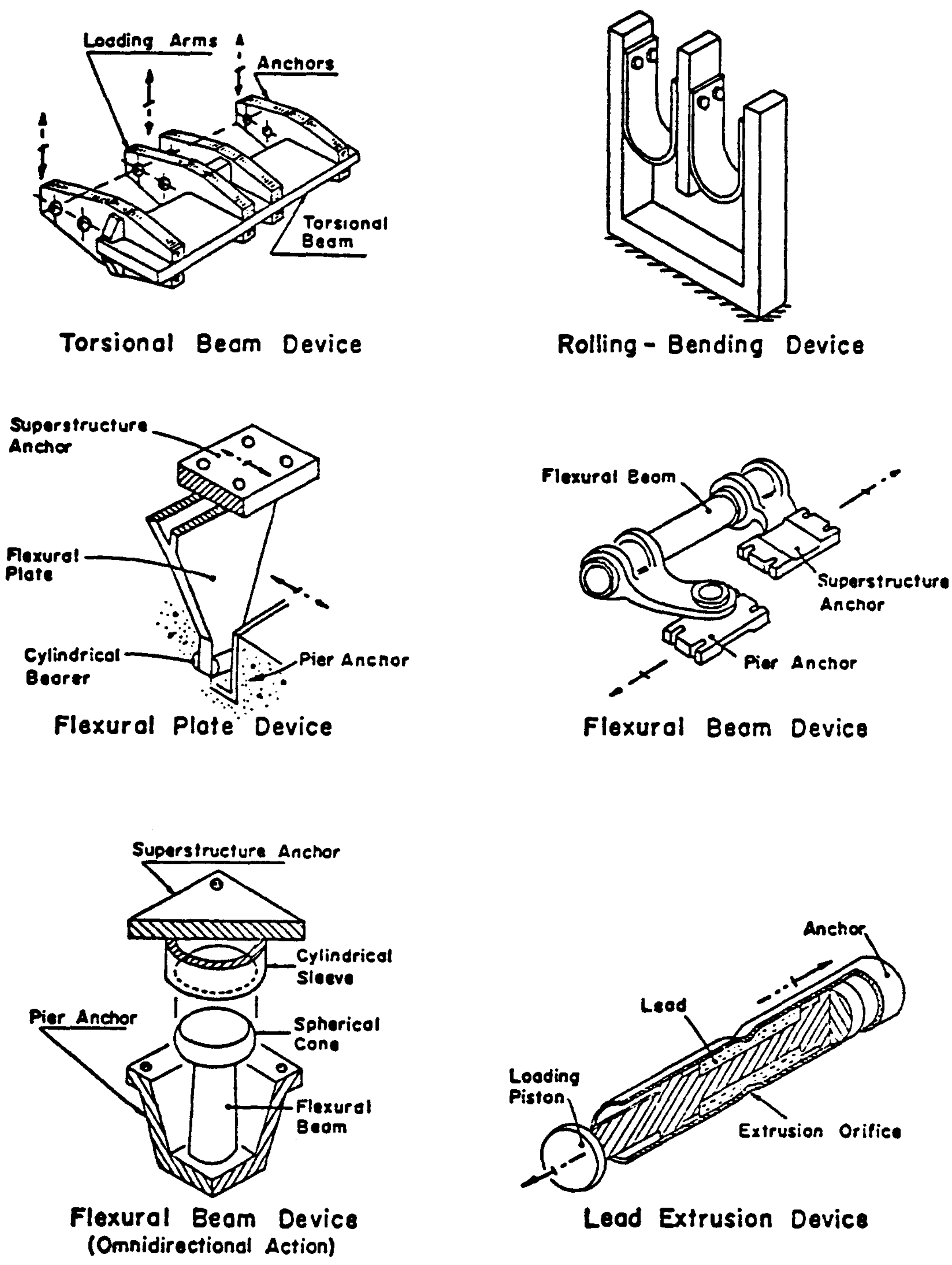


FIGURE 46. NEW ZEALAND ENERGY DISSIPATION DEVICES

ideas utilize the principals for restraint, isolation, and energy dissipation to modify structural behavior during earthquakes. In each case these bearing systems must also provide for the normal functions of bridge bearings. A few examples are discussed below.

New Zealand has constructed several bridges utilizing special energy-dissipating devices. Some of the devices initially considered are shown in Figure 46. With the exception of the lead extrusion device, all of these devices rely on the post-elastic behavior of steel and the hystertic damping that will occur during reversing cycles of yielding. The devices shown are used to connect the bridge superstructure to the substructure and are usually installed in parallel with elastomeric bearing pads. At low levels of lateral load such as may occur in a moderate earthquake or due to wind, the devices will remain elastic and restrain movement at the bearings. During strong seismic shaking, the devices will yield, allowing translation at the bearings. This application would be used at "fixed" bearings, since no translation can occur under normal conditions. When the devices yield, the load transmitted from the superstructure to the substructure will be limited to the ultimate capacity of the devices. In addition, energy will be dissipated during yielding which will tend to damp the seismic response.

One disadvantage of these devices is that provisions must be made to accommodate the displacements that will occur at the bearings. Also, many of the devices work in one direction only and care must be taken to provide for necessary movements or restraints, as the case may be, in other horizontal directions.

The New Zealand Ministry of Works is currently using an elastomeric bearing pad which has had a circular core removed and replaced by lead.(7) This concept is shown in Figure 47. Because lead has a low recrystalization temperature it can deform many times under gradual movement such as would occur due to temperature change or creep. In addition the resistance to movement is less than half of that which would occur under rapid movement. This makes it possible to utilize this device as an expansion bearing under certain circumstances. Under rapid movement such as would occur during a strong earthquake, the lead would resist greater loads and dissipate energy. Therefore, an expansion bearing which would normally carry a small lateral load would have an increased role in resisting earthquake loadings. In addition there would be a reduction in total earthquake load due to energy dissipation. New Zealand is currently replacing old bearings with this type as a retrofit technique. These bearings are currently patented in New Zealand, Japan, and the United States.

In Japan a somewhat similar philosophy about the performance of bridge expansion bearings has been adopted. The Japanese, however, have relied on viscous damping such as would be obtained from oil dampers to achieve this type of performance. With these devices the resistance to temperature movement and creep is very small although there have been problems with maintenance. A viscous damping device is used on the new Dumbarton Bridge across the southern end of San Francisco Bay in California. This device, which is shown in Figure 48, allows the expansion joint to open and close during normal temperature movement but limits the relative movement of the joint during an earthquake. The limitation of load transfer to the substructure due to temperature movement was achieved by the Japanese by forcing a rigid post to move through a pot of viscous material as shown in Figure 49. During an earthquake the rapid movement of the post would be resisted by the viscous material, and load would be transfered to the substructure. This philosophy of bearing system design has become so popular in Japan that at least one manufacturer is marketing a patented device which works on the same principle. This device, is shown in Figure 50.

Some of the oil-damper systems can leak and require maintenance and inspection.

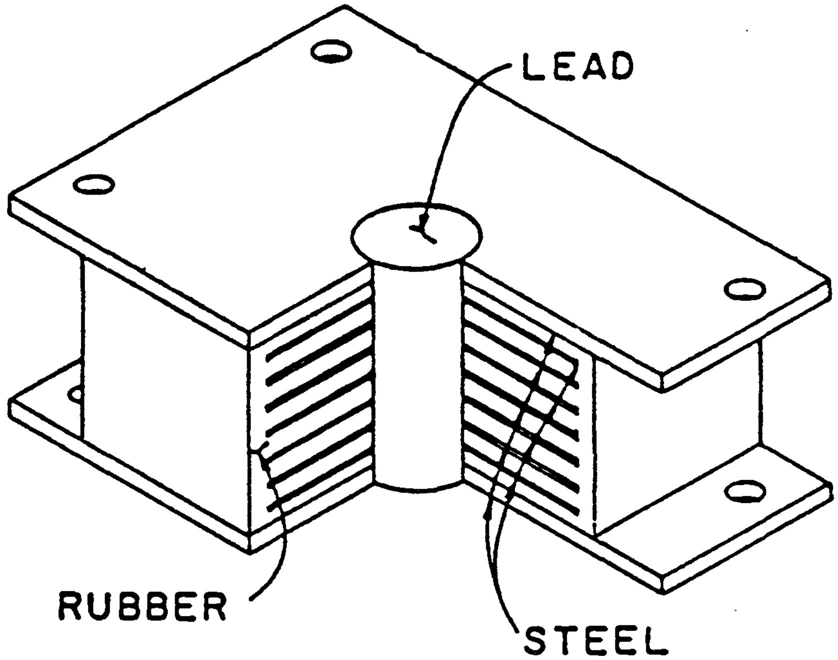


FIGURE 47. ELASTOMERIC BEARING PAD WITH A LEAD CORE

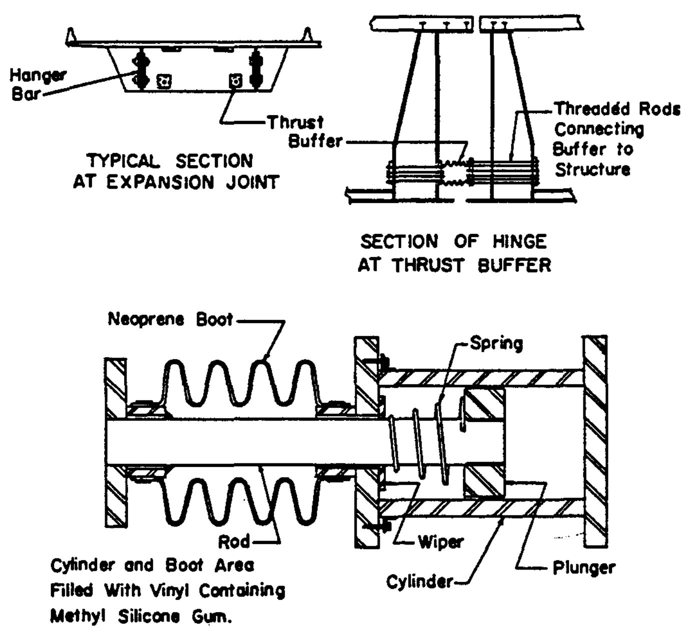


FIGURE 48. VISCOUS DAMPING DEVICES USED ON THE NEW DUMBARTON BRIDGE

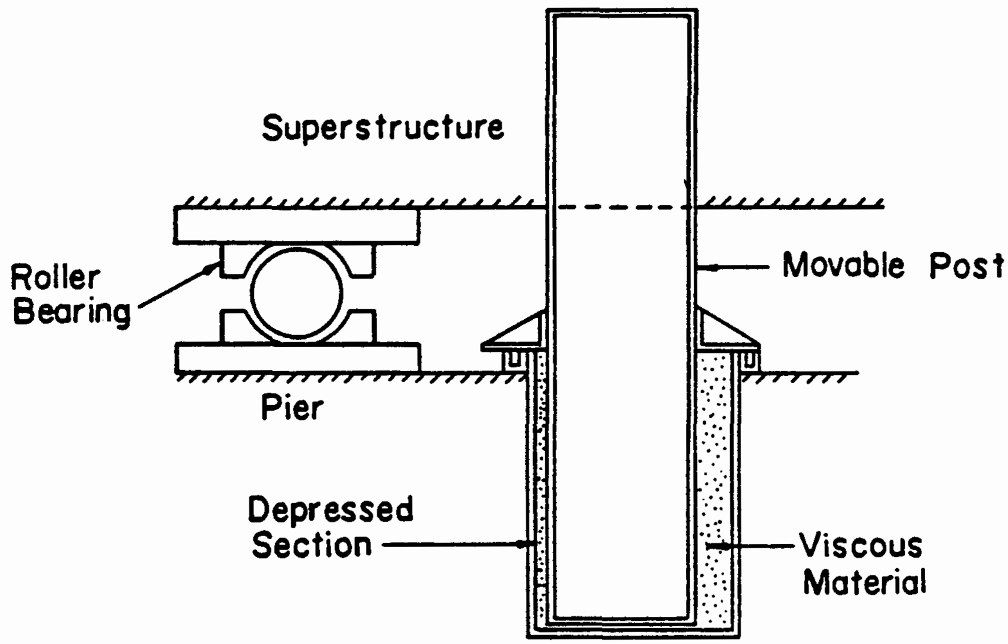


FIGURE 49. JAPANESE "EARTHQUAKE STOPPER" DEVICE USED ON THE OHTAGAWA RAILWAY BRIDGE

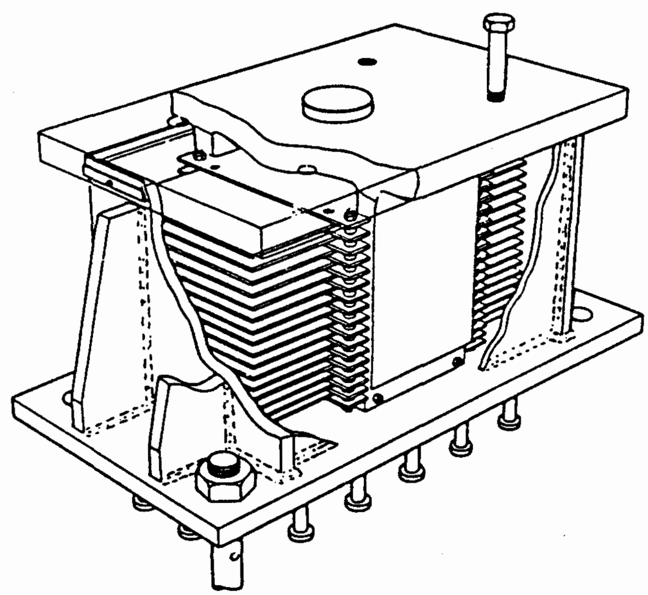


FIGURE 50. VISCOUS SHEARING TYPE DAMPER

Since failure of these systems can occur due to normal wear, a positive back-up system such as elastic restrainers should be used to prevent catastrophic failure.

An expansion bearing design concept developed and tested during a study for the Federal Highway Administration employs an elastomeric bearing pad surfaced with a special material designed to slide during seismic loading.[8] Under normal conditions the bearing will perform as a normal elastomeric bearing pad. At high displacements the sliding will limit the load transferred to the supports, protect the pad from being destroyed, and maintain the reliability of vertical support. This bearing concept is shown in Figure 51.

The behavior of most special bearing systems is highly nonlinear. In the absence of special design procedures that will account for this behavior, a nonlinear analysis should be used when designing any of these systems. Since earthquake-resistant bearings and devices are a relatively new concept, there are not many special design procedures available. A good general guideline for designing earthquake-resistant bearings and devices is given in an FHWA report entitled "Increased Seismic Resistance of Highway bridges Using Improved Bearing Design Concepts."[8] The New Zealand Ministry of Works and Development has published a more specific booklet entitled "The Design of Lead-Rubber Bearings for Use as Energy Dissipators in Bridges."[7] This publication contains several design charts which are intended to be used to obtain approximate design forces and displacements. Seminar notes (10) have also be prepared that adapt New Zealand design procedures to U.S. Seismic Design criteria.

C5.4 REINFORCED CONCRETE COLUMNS, PIERS, AND FOOTINGS

Very few of the retrofit concepts for reinforced concrete columns, piers, and footings presented have actually been used. Most engineers feel these retrofit measures would be less cost effective than retrofit measures used at the bearings. Many of the concepts presented are also very controversial, such as reducing or increasing the flexural reinforcement. Both of these retrofit measures have the potential for producing detrimental effects if not properly designed. Other concepts, such as increased tranverse confinement, are less controversial as to their effectiveness, but present practical problems with respect to construction and cost.

Several retrofitting details for reinforced concrete columns, piers, and footings are presented below as a source of ideas. The engineer should be aware that he is working at the limits of the current state-of-practice when he designs a column retrofit.

Because the effectiveness of retrofitting columns, piers, and footings has not been totally demonstrated by testing or experience, the design requirements for this type of retrofit are fairly restrictive. It is anticipated that as the state-of-practice is advanced, these design requirements will be modified.

C5.4.1 FORCE-LIMITING DEVICES

Several types of force-limiting devices have been used in Japan and New Zealand. These devices are designed to limit the seismically induced forces that can be transferred to the columns, piers, and footings. These were discussed in Section C5.3.6.

C5.4.2 INCREASED TRANSVERSE CONFINEMENT

Several methods of increasing the transverse confinement of columns through retrofitting have been proposed.

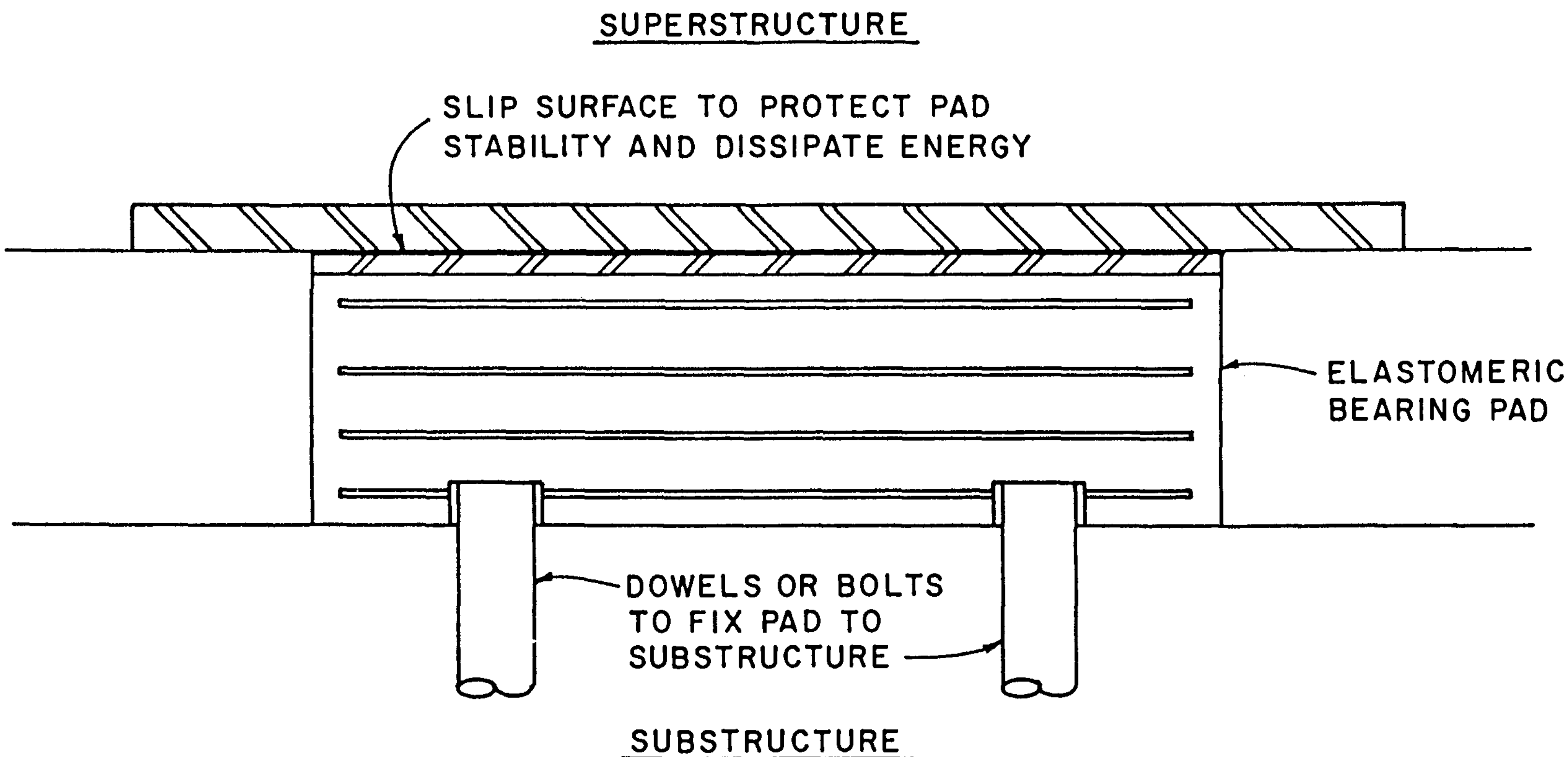


FIGURE 51. SLIDING ELASTOMERIC BEARING PAD

One proposal which utilizes conventional half-inch steel reinforcing prestressed on the outer face of the column is shown in Figure 52. The prestress force is provided by threading the ends of the bars so these can be connected together with a specially designed turnbuckle also shown in Figure 52. The steel bars would be spaced at 3-1/2 inches on center, which would provide confinement equivalent to new construction in most cases. The steel would be protected with a layer of pneumatically applied concrete.

Another proposal to use quarter-inch prestressing wire wrapped under tension around the column is shown in Figure 53. The wire and anchorages would also be protected by pneumatically applied concrete.

A solid-steel shell placed around an existing column as shown in Figure 54 has also been proposed as a retrofit method to increase concrete confinement in columns. A small space would be left between the column and the shell that would be grouted solid. The steel shell could be painted or it could be constructed of a weathering type of steel.

Design requirements for this type of column retrofit have been adapted from the "Seismic Design Guidelines for Highway Bridges."(3) Retrofit methods should be carefully detailed so that transverse confinement will remain effective throughout the duration of the seismic loading. In addition, the designer should be aware of the potential for increased column strength which may overload other components.

C5.4.3 REDUCED FLEXURAL REINFORCEMENT

Reduction of longitudinal reinforcement in a column is a controversial retrofit method, and therefore must be applied only in an extreme condition. The requirement that the minimum column capacity/demand ratio be improved by at least 50% to a minimum value of 0.75 implies that the inital column shear capacity is well below the shear demand created by column yielding. This is a rare condition that will be limited to a relatively small number of actual columns. Even when this condition does exist, reduction in flexural reinforcement should be considered only when it is not feasible to use other retrofit methods. Reduction in flexural reinforcement should never be used when the loss of flexural capacity will result in the formation of a collapse mechanism. The simplest method for reducing flexural reinforcement is to cut some of the longitudinal reinforcing bars as shown in Figure 55. Care should be taken when using this detail to assure that the footing can accommodate the forces transmitted from the shear keys.

C5.4.4 INCREASED FLEXURAL REINFORCEMENT

Increasing the flexural reinforcement in a column can create problems if this retrofit measure is not properly designed. The design requirements are intended to prevent the transfer of failure to other components. Increased reinforcement will not be totally effective unless the column can be made to respond ductily. Adequate transverse confinement will assure ductile behavior. When this confinement is not provided, the response modification factor cannot exceed 2. This value reflects the reduced ductility of an unconfined column. Retrofit methods used by the Japanese to increase the flexural strength of reinforced concrete building columns are shown in Figures 56 through 58.

C5.4.5 INFILL SHEAR WALL

Figure 59 illustrates the use of an infill concrete shear wall to retrofit a multi-

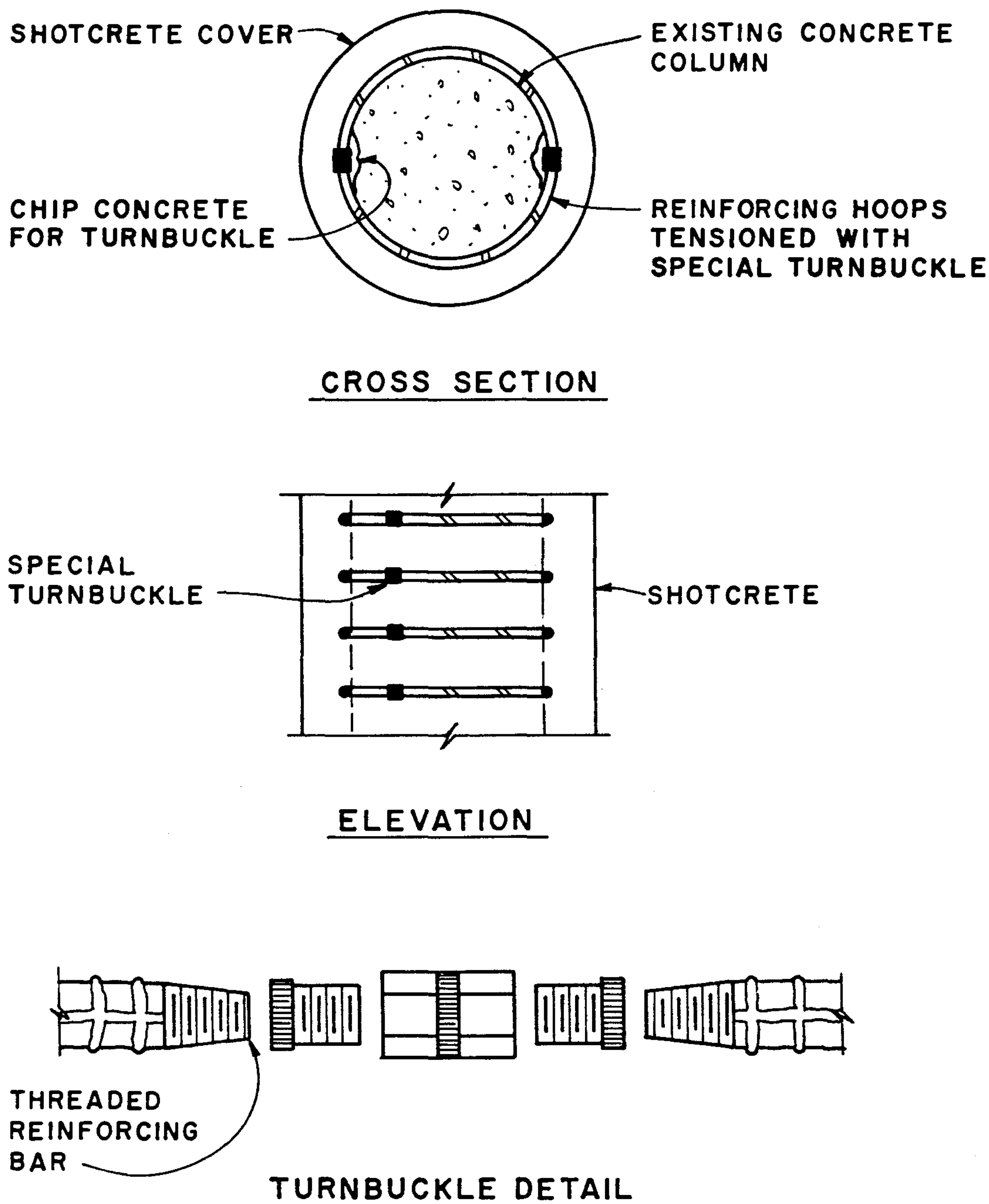


FIGURE 52. COLUMN RETROFIT TO INCREASE CONFINEMENT WITH STEEL HOOPS

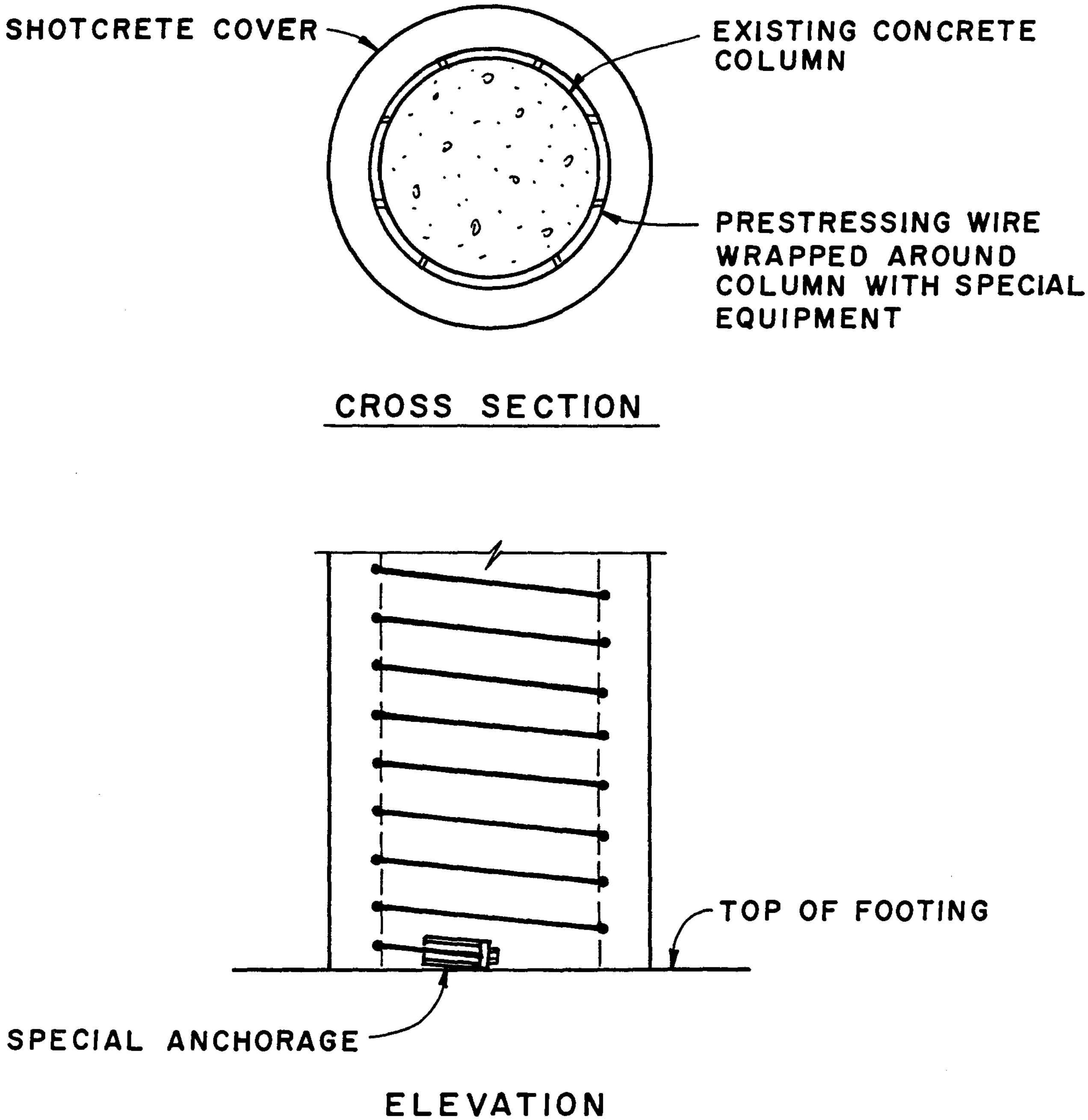


FIGURE 53. COLUMN RETROFIT TO INCREASE CONFINEMENT WITH PRESTRESS WIRE

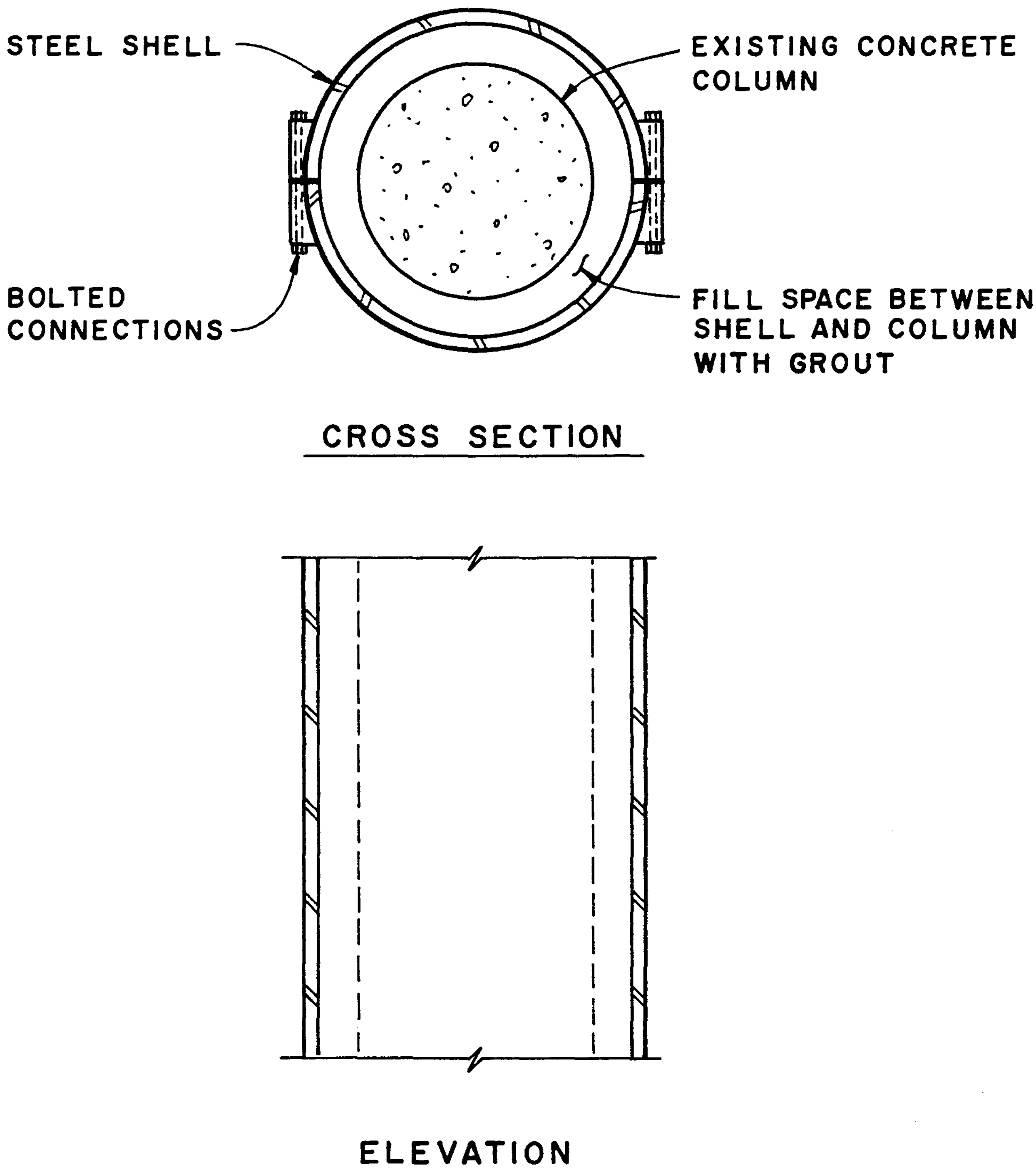


FIGURE 54. COLUMN RETROFIT TO INCREASE CONFINEMENT WITH STEEL SHELL

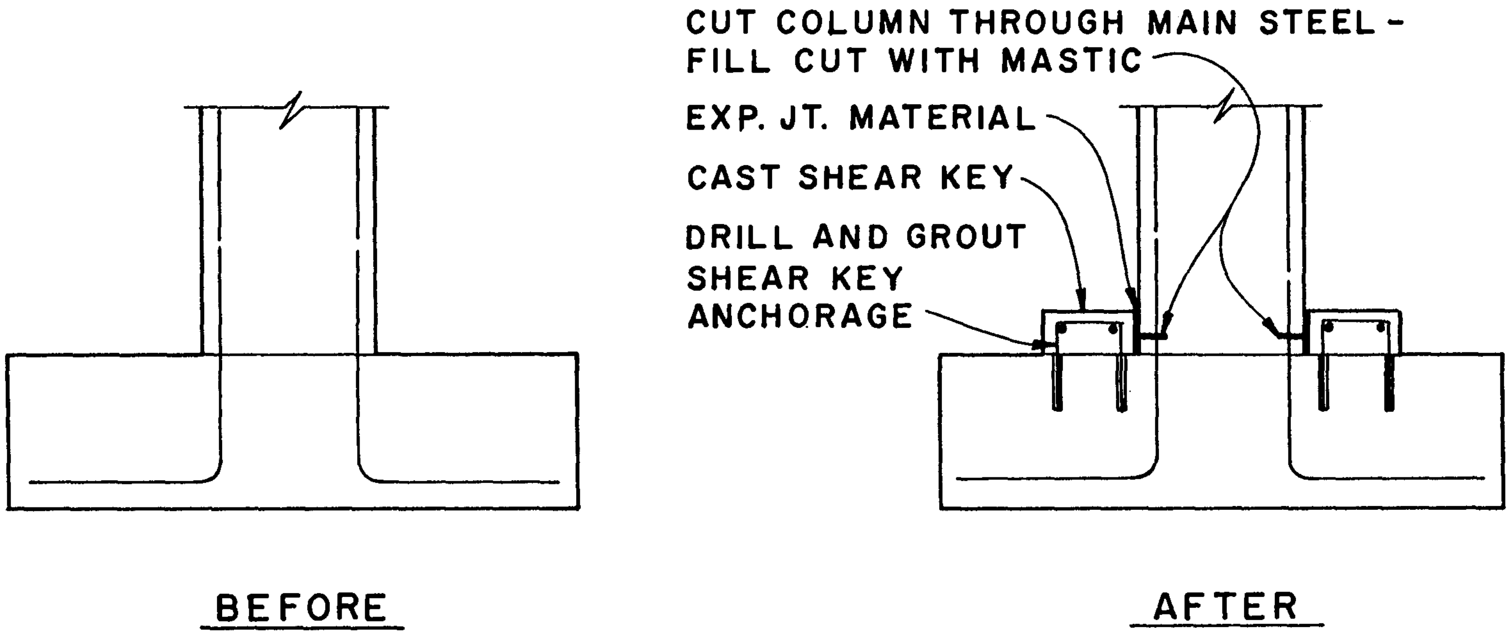


FIGURE 55. COLUMN RETROFITTING BY CUTTING MAIN REINFORCEMENT

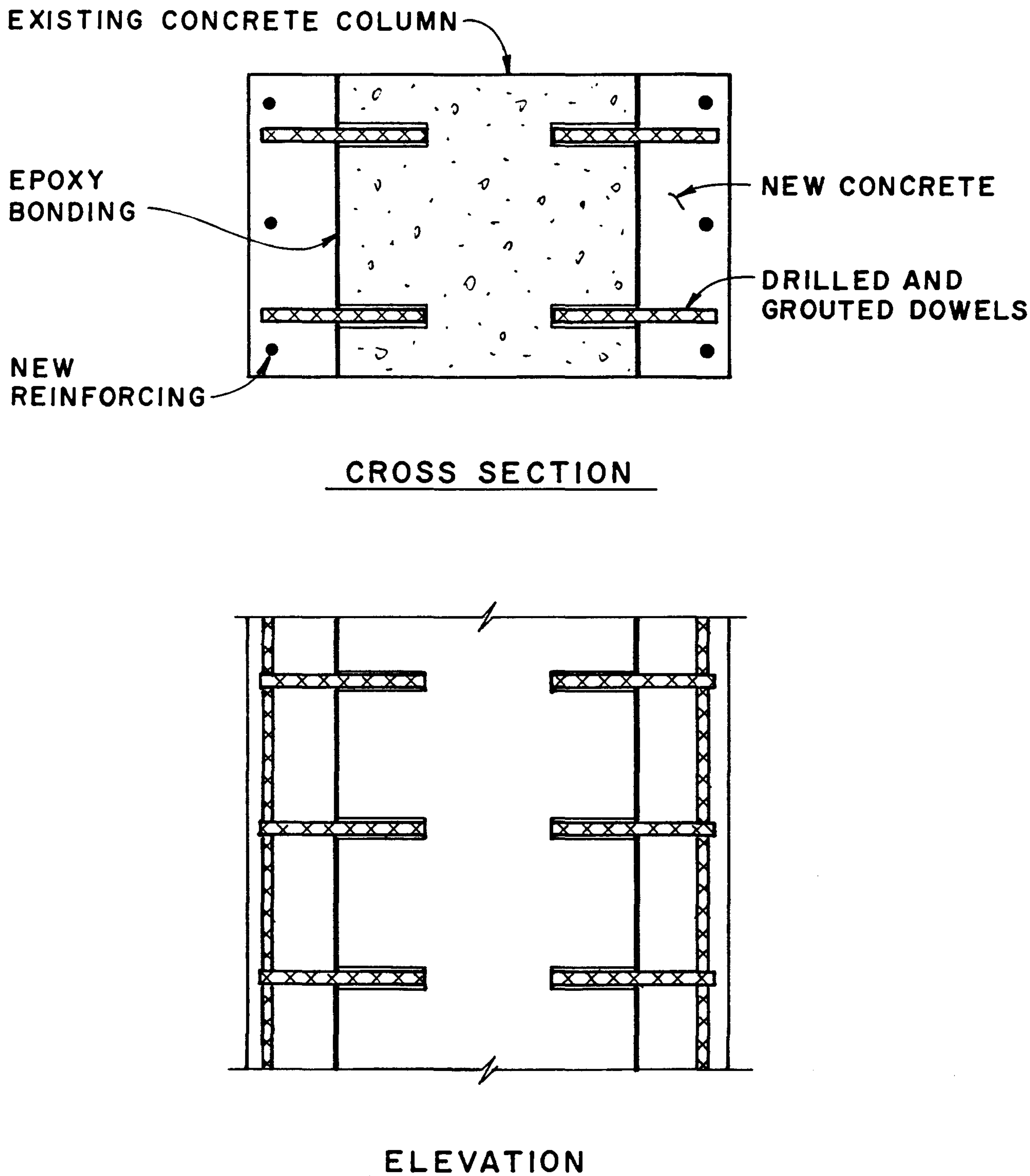
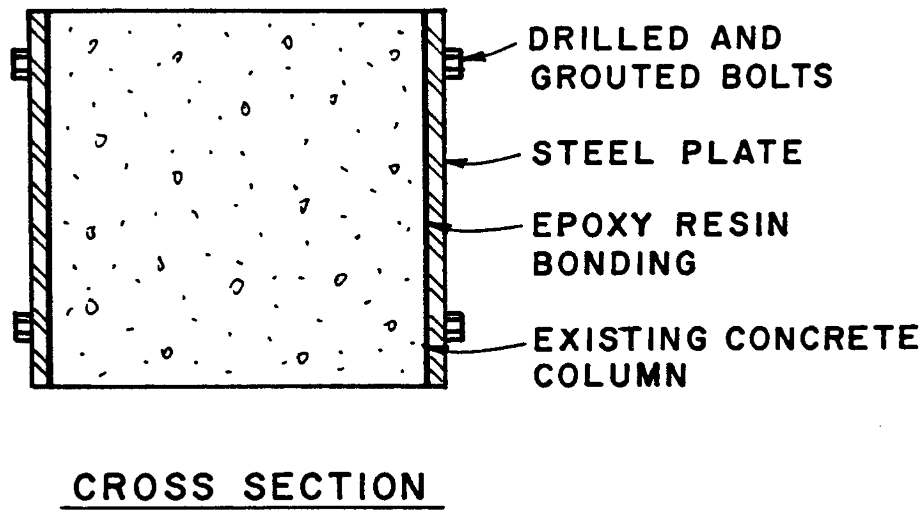


FIGURE 56. COLUMN STRENGTHENING BY CONCRETE OVERLAYS



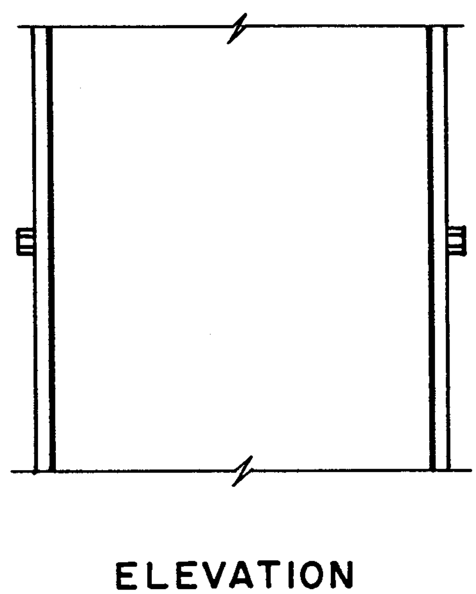


FIGURE 57. COLUMN STRENGTHENING BY STEEL PLATES

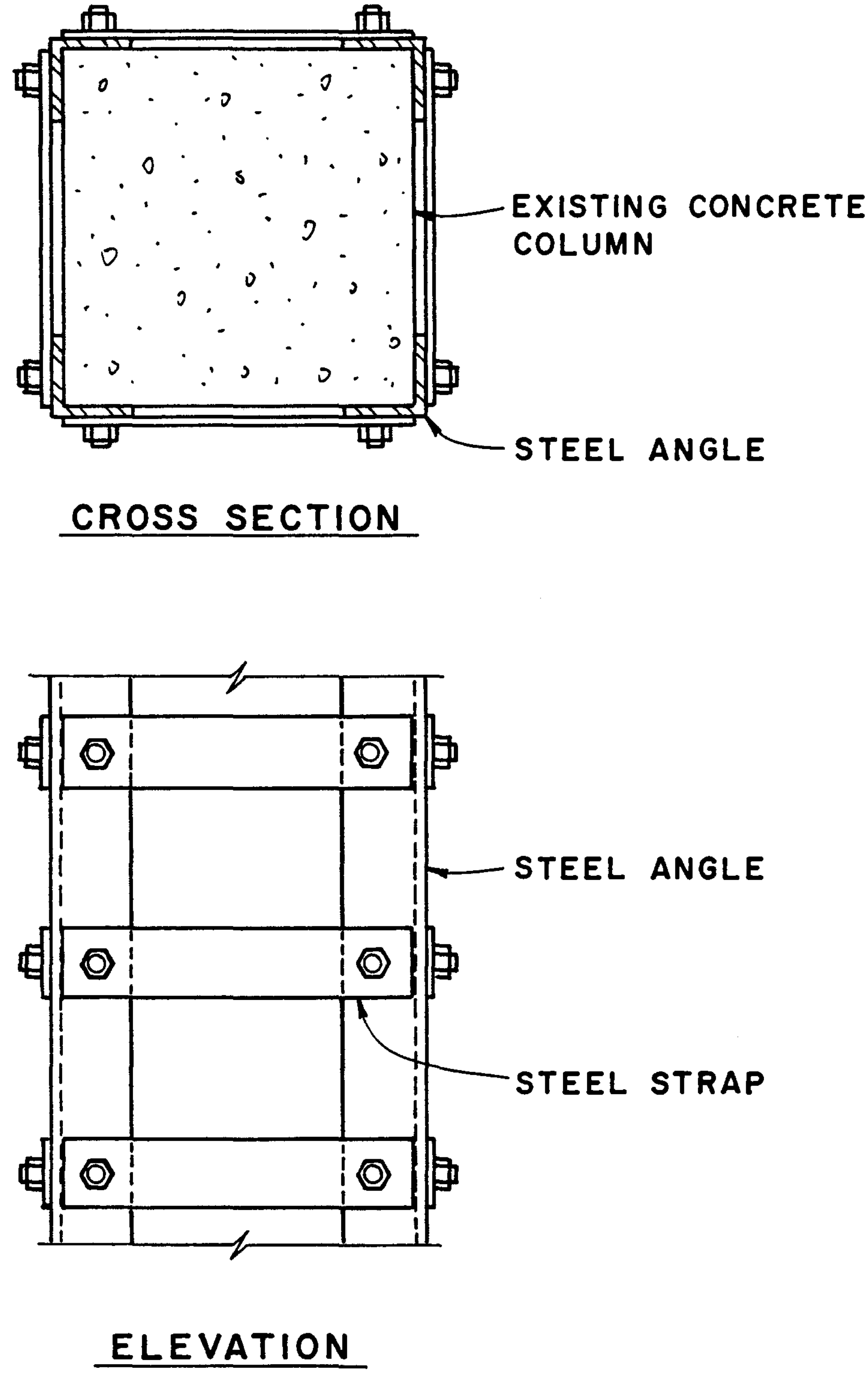


FIGURE 58. COLUMN STRENGTHENING BY STEEL ANGLES

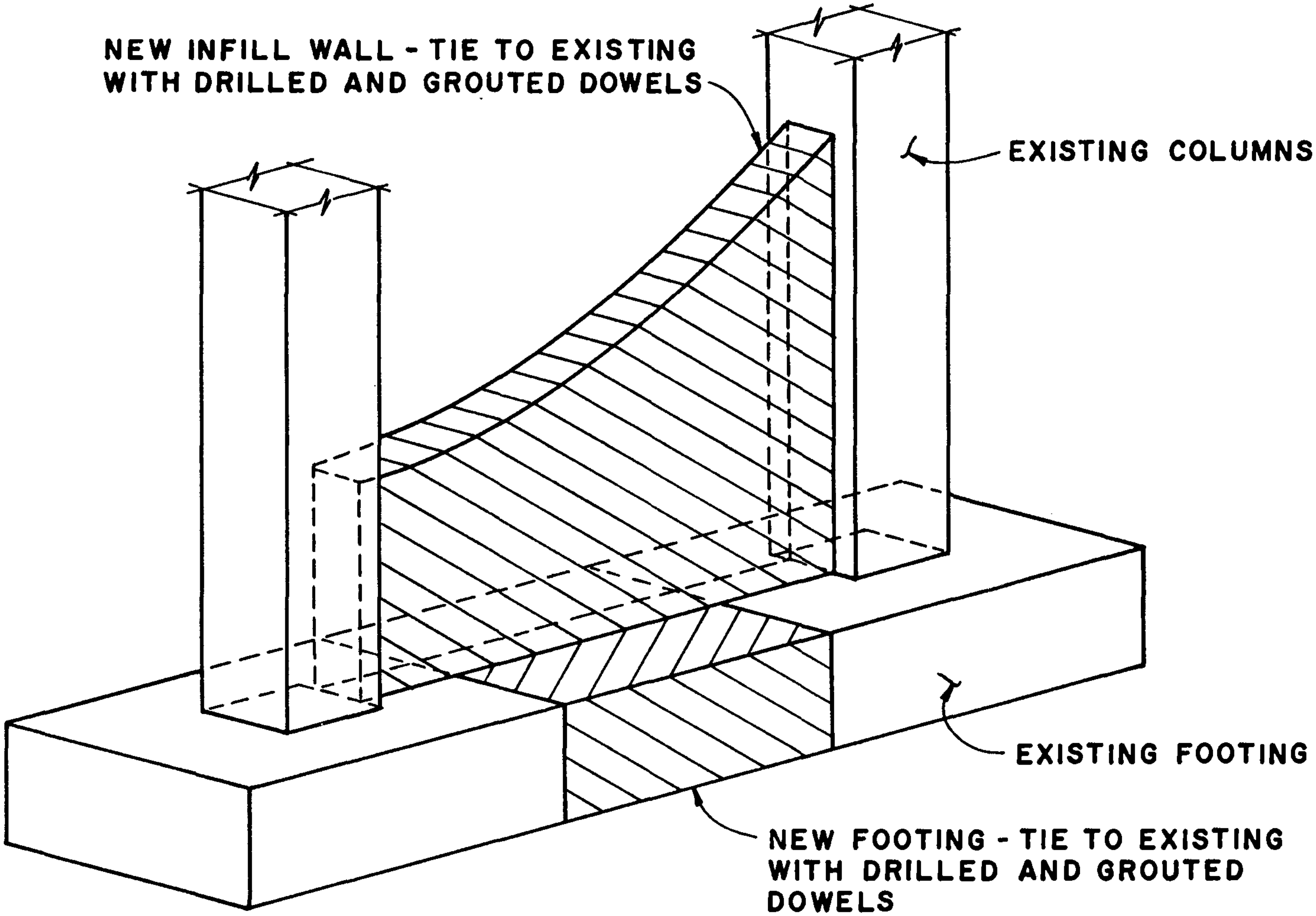


FIGURE 59. INFILL SHEAR WALL RETROFIT

column bridge bent. This type of structure modification will have a large effect on the structural strength and stiffness in the transverse direction. This will usually make it necessary to perform a separate analysis to obtain design forces.

C5.4.6 STRENGTHENING OF FOOTINGS

Although minor soil and piling failures are undesirable, these are preferable to the failure of the structural components of a footing. Therefore, in the case of retrofitting, footings should be strengthened so that they do not fail prior to soil or piling failure.

A method for retrofitting footings to correct this deficiency is shown in Figure 60. A concrete cap of constant thickness is cast directly on top of the footing. Continuity with the existing footing would be provided by steel dowels grouted in drilled holes. Negative moment capacity would be provided by a top layer of conventional reinforcement and prestress tendons. The collar would strengthen the footing to resist uplift forces and provide an extra measure of confinement at the base of the column and the top of the footing to prevent anchorage failures.

C5.5 **ABUTMENTS**

Abutment retrofitting is also rarely performed because it too is considered to be less cost effective than bearing retrofit. The loss of accessibility of a bridge due to approach settlement can usually be quickly repaired by the addition of fill material. Only when the time delay required to make such repairs is unacceptable should an abutment retrofit to mitigate earthquake-induced settlements be made.

In some cases, restrainers may be added at the abutments to prevent failure of the bearings. In these cases, the abutment should be strengthened to resist the additional forces that will be produced.

The design requirements for abutment retrofit are intended to mitigate the effects of differential movements at the abutment that could cause a loss of accessibility to the bridge.

C5.5.1 SETTLEMENT SLABS

Settlement slabs must carry traffic over a portion of the approach fill that has settled. In an extreme case the slab may have to span its entire length. Therefore slabs should be designed to carry deadload and liveload using the same criteria used to design a simple span-slab bridge. Under earthquake conditions, a slab length of 10 feet would usually provide an acceptable ramp.

The settlement slab should be tied to the bridge abutments to prevent it from pulling loose and becoming ineffective. The design force for this tie should be the maximum force during the design seismic loading. This is approximated as the sum of the coefficient of friction between the soil and the slab plus the acceleration coefficient times the deadload of the settlement slab.

Design Force = (Coefficient of Friction + Acceleration Coefficient) x Slab Deadload

It should be pointed out that this connection should be free to rotate so that moment will not be transferred to the abutment backwall when the approach fill settles.

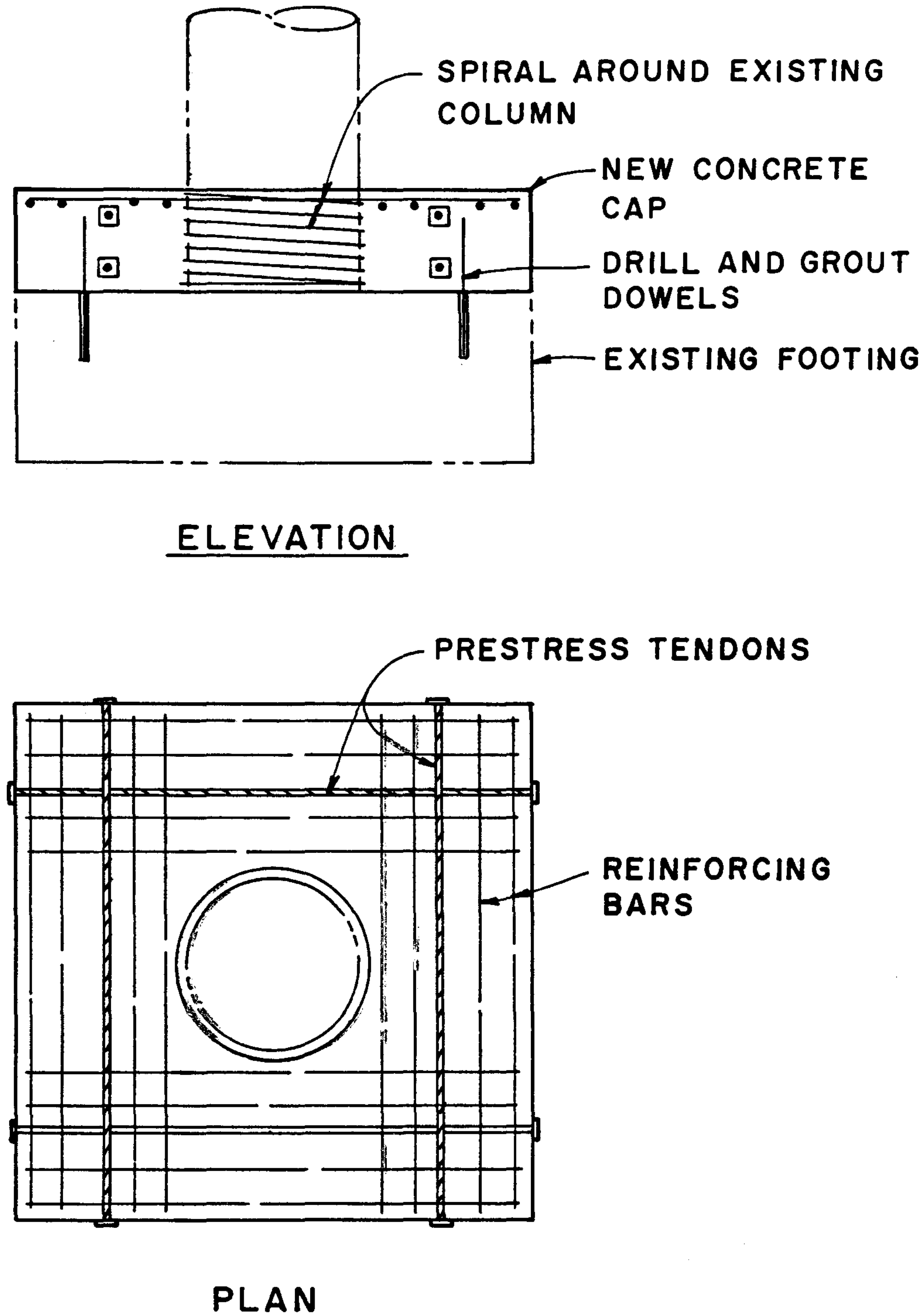


FIGURE 60. FOOTING RETROFIT BY ADDING CONCRETE CAP

Figures 61 and 62 show two different types of settlement slabs that have been used in the past.

C5.5.2 SOIL ANCHORS

Soil anchors similar to those shown in Figure 63 may be used as a retrofit measure. Forces may be produced in soil anchors due to the seismic forces transferred from the superstructure or the inertia forces generated in the abutment backfill. Because the backfill may be subject to movement during an earthquake, the anchors should extend into the backfill a sufficient distance so as not to be affected.

C5.6 **LIQUEFACTION AND SOIL MOVEMENT**

In certain indispensable bridges it will be necessary to preserve the ability of the bridge to carry emergency traffic following an earthquake. When severe liquefaction is expected, modification and strengthening of the bridge alone will not be effective in accomplishing this objective. In these cases a soil stabilization approach should be undertaken to reduce the probability and extent of a liquefaction failure. Several conceptual methods are proposed for accomplishing this.

The first method suggested is to lower the ground water table. This eliminates the presence of water which is one of the three items required before liquefaction can occur. This possibility and expense of accomplishing this will depend on the site. Obviously, some type of gravity drainage would be preferred to mechanical methods, although mechanical methods such as well points are not out of the question in a major structures of unusual importance. Drainage can cause settlement of the surrounding soil and the effect of this settlement on the existing bridge should be assessed before this method is used.

Densification of the soil can also be effective in reducing the potential for liquefaction. Since the process of liquefaction involves the compaction of loose soil, it follows that preconsolidation can reduce the risk of liquefaction. However, consolidation of only the surface layer can impede drainage and actually be detrimental. Soil densification through the use of vibrofloatation or sand-compaction piles improves drainage if porous material is used and therefore is the perferred method. Preconsolidation can result in significant settlements, and care should be taken to protect the existing structure from damage. Often excessive settlements during construction will make soil densification an impractical retrofit method.

A method which will improve drainage without disrupting the existing structure is to install a network of gravel drains as shown in Figure 64.[7] These drains will allow water to escape during an earthquake and thus prevent the build-up of pore pressure which can reduce the shear strength of the soil. Settlement will be likely during an earthquake, but large lateral movements resulting from shear strength loss will be greatly reduced.

The use of a highly porous overburden or surcharge can also greatly reduce liquefaction potential with minimal disruption to the existing structure. The increased intergranular forces resulting from the overburden will necessitate higher pore pressures to offset these forces and cause liquefaction. The permeablility of the overburden will not aggrevate the build-up of pore pressure. In addition, the overburden will result in some preconsolidation which will reduce the chances of liquefaction. However, the settlements that will accompany this preconsolidation should be considered when using this approach.

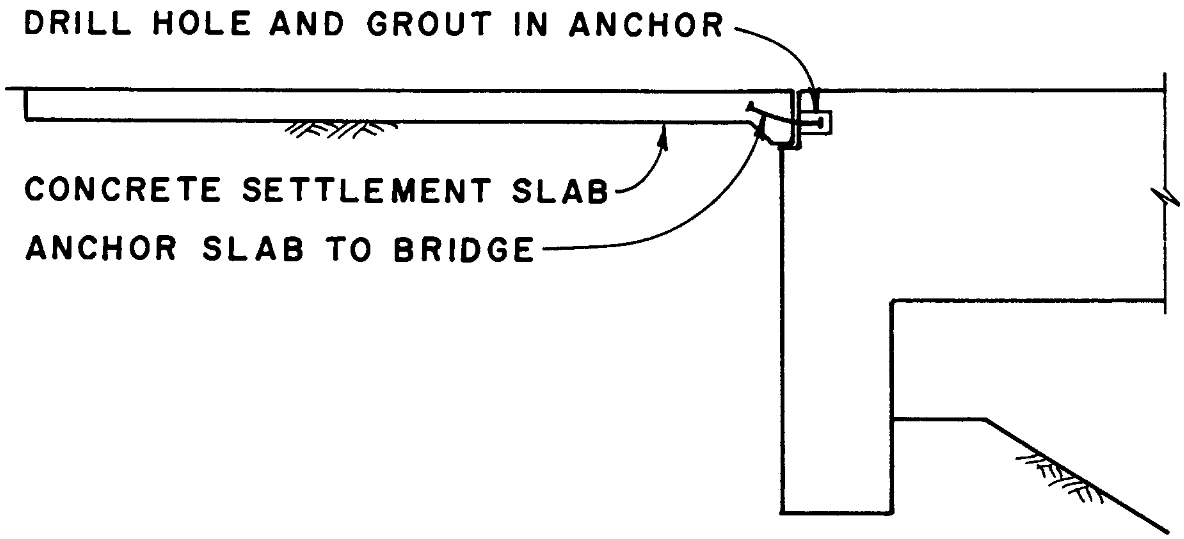


FIGURE 61. SETTLEMENT SLAB—CALIFORNIA STYLE

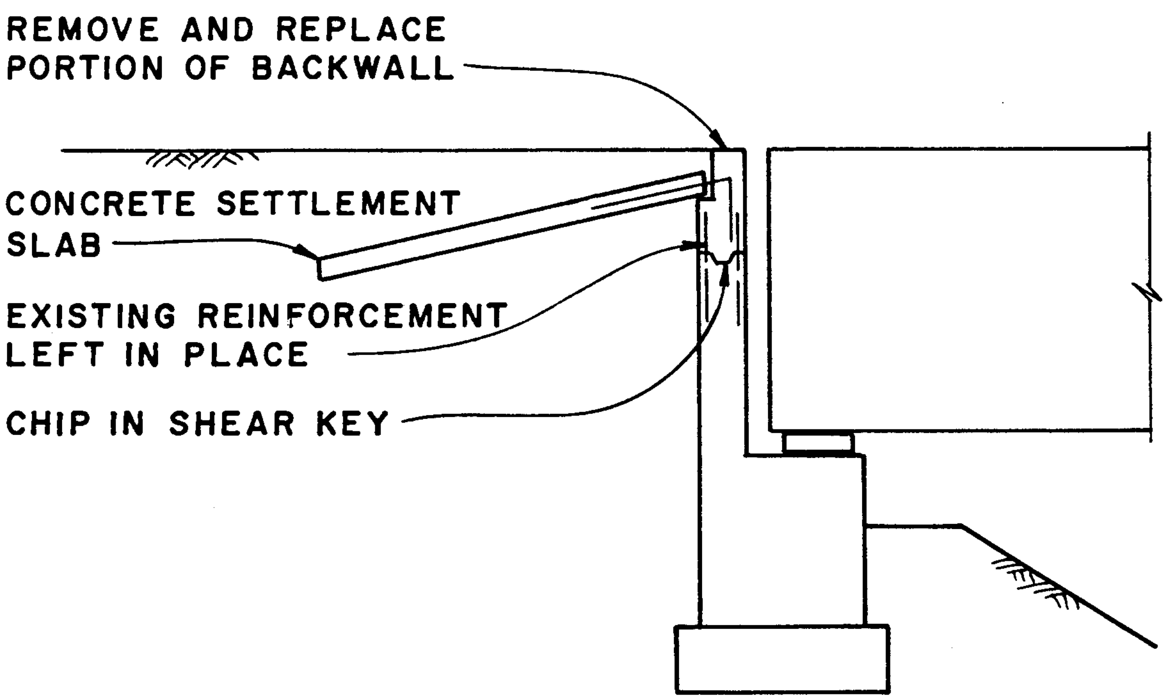


FIGURE 62. SETTLEMENT SLAB—NEW ZEALAND STYLE

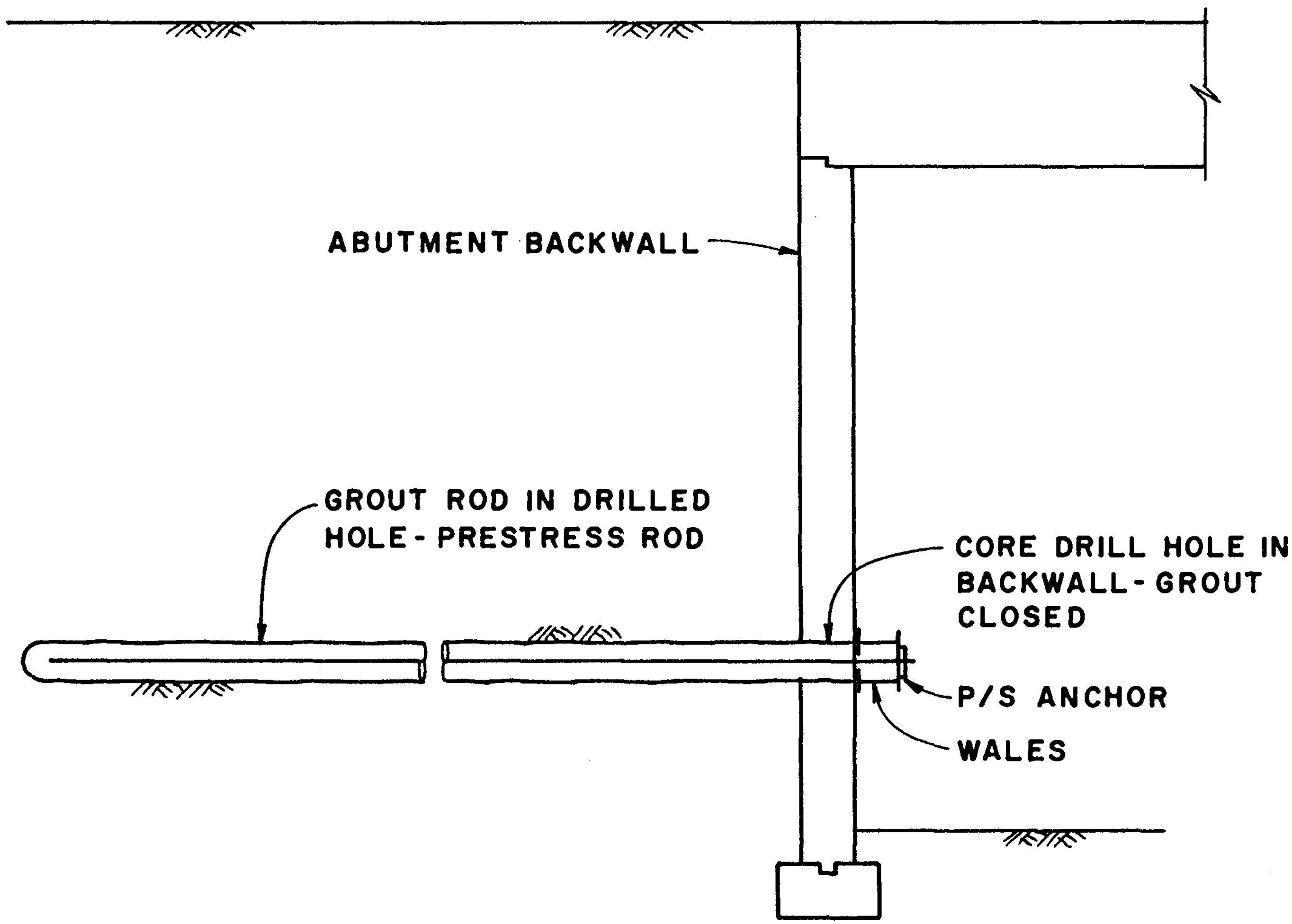


FIGURE 63. RETROFIT OF ABUTMENT WITH SOIL ANCHORS

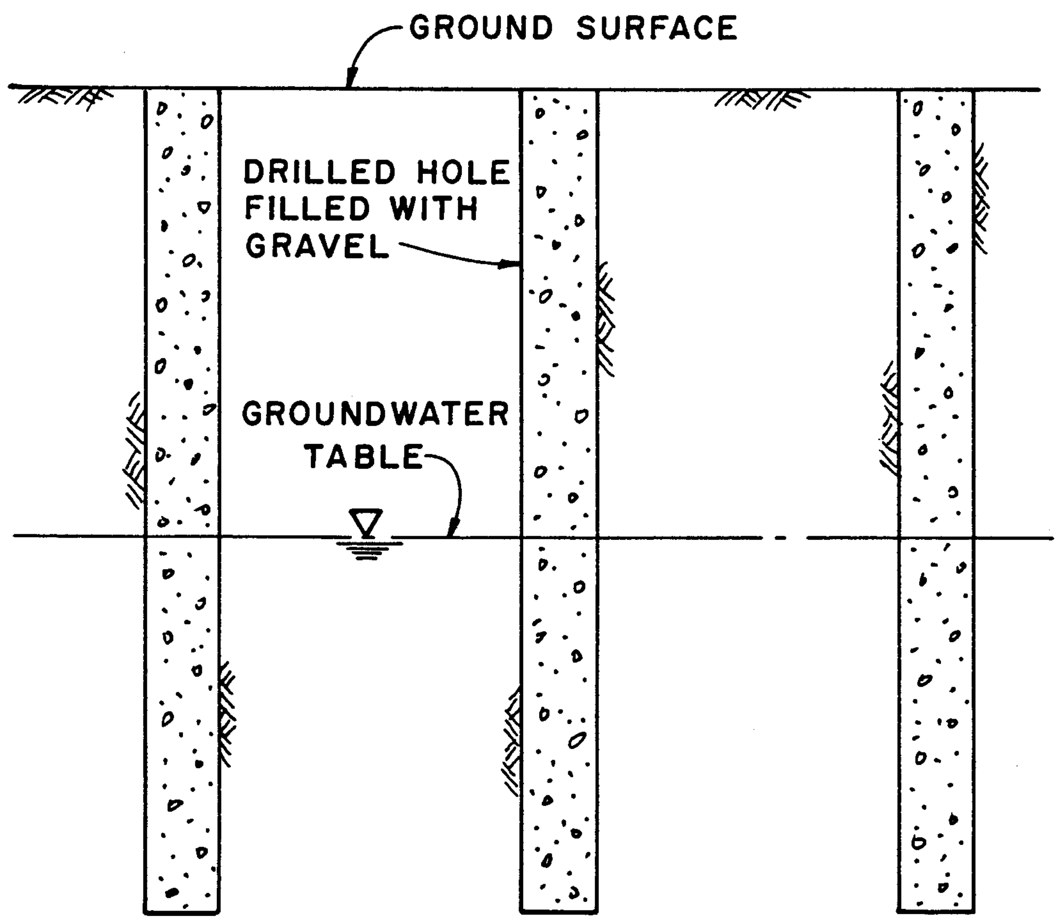


FIGURE 64. GRAVEL DRAIN SYSTEM

The use of chemicals or grouts to increase the shear strength of soil is also a possible solution. If not properly designed, these methods may reduce soil permeability and aggrevate the build-up of pore pressure. Therefore, design and construction should be performed by qualified individuals.

The previous paragraphs have discussed some possible methods of site stabilization. Because of the many variables and possible disadvantages associated with these methods, primarily due to excessive settlements during construction, it is recommended that these methods be used with caution and that expert assistance be obtained whenever any of them are contemplated.

In addition to site stabilization, strengthening of the structure will often be necessary. The strengthening methods used will depend on the configuration of the structure and components most susceptible to damage. These will usually involve methods for tying superstructure sections together and connecting the superstructure to the bents. In some cases, column retrofitting should be considered. Attempts to stabilize the abutments through the use of anchors would probably not be very effective. Because abutment tilting usually does not result in collapse, this type of failure is not considered to be critical. The use of settlement slabs may be in order, however, if undelayed access to the bridge is important.

Longitudinal restrainers should be provided at the bearings to prevent a loss of support. If bents are not tied to the superstructure, the movements of the foundation can easily pull the support out from under the bearings as shown in Figure 65. It would be preferable to fail the column in flexure rather than to lose this support. Therefore, the superstructure should be anchored to the bent, and the design load in the anchors should be at least enough to fail the bent. Care should be taken to provide a sufficient gap in the restrainers so that normal temperature movement or moderate earthquakes will not result in a column failure.

Transverse and vertical restrainers at the expansion joints tend to prevent the superstructure from buckling and should be used along with longitudinal restrainers. When expansion joints occur at the bents, these restrainers should provide a positive tie to the substructure.

Because ductile failures of the bents are required to accommodate large movements, bent retrofitting may be necessary to assure that a brittle failure does not occur. Extra transverse reinforcement or reduction of flexural capacity are two possible retrofitting techniques for accomplishing this.

A rough estimate of the liquefaction potential of a bridge site can be obtained by following the procedure for preliminary screening discussed in Section C2.3.1. When retrofitting to mitigate liquefaction failures is being considered, a more detailed evaluation of liquefaction potential and the probable extent of liquefaction should be made by a qualified geotechnical specialist using one of the many currently available techniques for more accurately evaluating liquefaction potential. An excellent discussion of liquefaction at bridge sites is presented in The Federal Highway Administration reports entitled "Determination of Seismically Induced Soil Liquefaction Potential at Proposed Bridge Sites" (Report Nos. FHWA-RD-77-127 and FHWA-RD-77-128).(5) These reports also discuss design procedures for vertical gravel drains as a method of site stabilization.

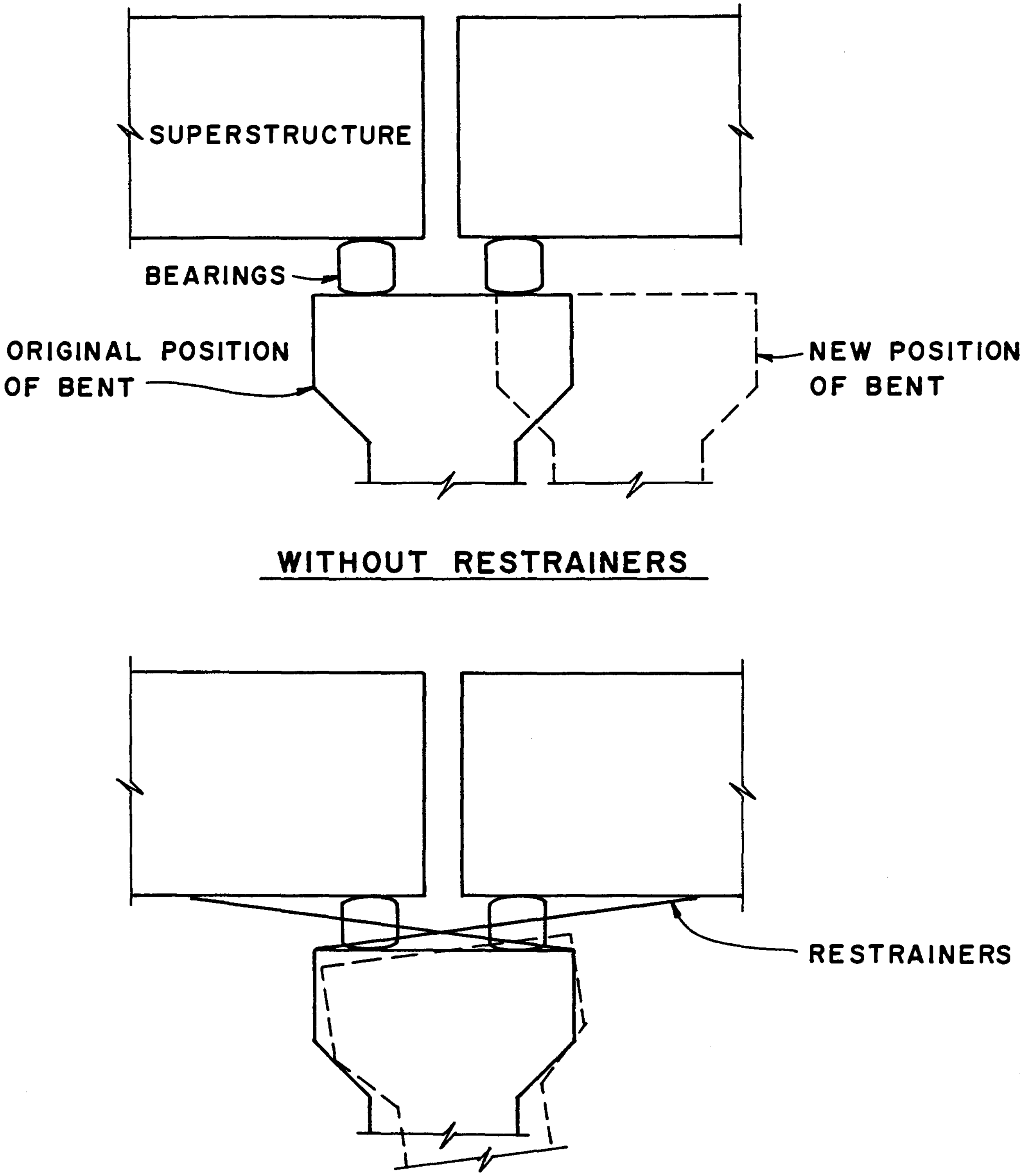


FIGURE 65. EFFECT OF RESTRAINERS AT BENT DURING LIQUEFACTION FAILURE

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1. Degenkolb, O. H., "Retrofitting of Existing Highway Subject to Seismic Loading—Practical Considerations," Proceedings of a Workshoip on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November, 1979, pp. 343-359.

2. Imbsen, R. A., Nutt, R.V., and Penzien, J., "Seismic Response of Bridges—Case Studies," Report No. UBC/EERC-78-14, Earthquake Engineering Research Center, University of California, Berkeley, June, 1978.

3. Applied Technology Council, Seismic Design Guidelines for Highway Bridges, Report No. ATC-6, October, 1981.
   or:
   Applied Technology Council, Seismic Design Guidelines for Highway Bridges, Federal Highway Administration, Report No. FHWA/RD-81/081, Washington, D.C., October, 1981.

4. Mancarti, G.D., "New Concepts in Earthquake Retrofitting of Highway Bridges," paper presented at the Conference of Northwest Bridge Engineers, 1981.

5. Blakeley, R.W.G., "Analysis and Design of a Bridge Incorporating Mechanical Energy Dissipating Devices for Earthquake Resistance," Proceedings of a Workshoip on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November 1979, pp. 313-342.

6. Yamadera, N. and Uyemae, Y., "Special Considerations and Requirements for the Seismic Design of Bridges in Japan," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Report No. ATC-6-1, November, 1979, pp. 295-312.

7. "The Design of Lead-Rubber Bearings for Use as Energy Dissipators in Bridges", New Zealand Ministry of Works and Development, Report No. CDP 818/A, 1982.

8. Imbsen, R.A. and Schamber, R.A., "Increased Seismic Resistance of Highway Bridges Using Improved Bearing Design Concepts," Federal Highway Administration, 1982.

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10. "Seismic Base Isolation using Lead Rubber Bearings," Seminar Notes on Applications in Design Procedures, Dynamic Isolation Systems, Berkeley, CA., May, 1983.

11. Robinson, R.R., Longinow, A., and Chu, K.H., "Seismic Retrofit Measures for Highway Bridges," Volumes I and II, Federal Highway Administration, Department of Transportation, April, 1979.

APPENDIX A

WORKED EXAMPLE PROBLEM

A.1 INTRODUCTION

This example problem illustrates how the provisions of the Retrofit Guidelines are applied to a realistic bridge structure. In this problem the Seismic Rating of the bridge is determined using the suggested method for calculating the Vulnerability Rating that is presented in Chapter 2 of the Commentary. In addition a detailed evaluation of the existing bridge is performed. The detailed evaluation procedure is used to identify and evaluate potential seismic retrofitting measures. Finally, the most cost effective retrofit scheme is selected and the retrofit details are designed.

A.2 DESCRIPTION OF THE EXAMPLE BRIDGE

The bridge to be examined is a typical freeway overcrossing of the type that was being constructed prior to improved seismic design provisions. It carries a major city street over an urban freeway in a region that falls within the 0.4 contour on the Accceleration Coefficient map shown in Figure 1 of the Retrofit Guidelines. The bridge is therefore classified as Seismic Performance Category D.

The superstructure is a concrete box girder. One portion of the bridge is prestressed and the other is conventionally reinforced. A mid-span expansion joint is located between the prestressed and reinforced sections. The 470 foot long superstructure is divided into four spans which are continuous over three 2 column bents as shown in Figure A-1. The diapram type abutments are cast monolithic with the superstructure and the entire structure is supported on spread footings.

As is the case with most existing bridges of this vintage and type, the expansion joint is unrestrained and supported on a relatively narrow bearing seat. The details of the expansion joint are shown in Figure A-2. Concrete columns are confined by steel hoops which are inadequate for seismic resistance. At two of the bents, column steel is spliced within a zone of potential plastic hinging. The column details are shown in Figure A-3.

A.3 SEISMIC RATING (Section 2.3)

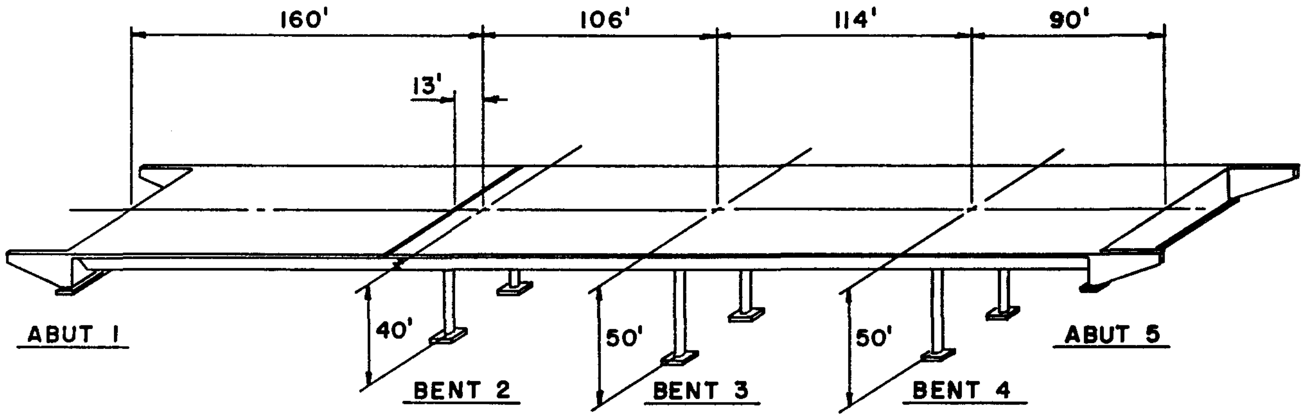
The seismic rating system is used to identify the bridges which are in the greatest potential need of retrofitting. In practice, all bridges within a region would be rated, and their ratings compared in order to identify the bridges that would be evaluated in greater detail. In this example, the seismic rating for the example bridge is calculated in order to illustrate the procedure that is followed in seismically rating bridges.

A.3.1 VULNERABILITY RATING (Section 2.3.1 AND C2.3.1)

The procedure suggested in Section C2.3.1 is used to calculate the vulnerability rating. Because the structure is classified as SPC D all components will be considered.

A. Bearings

<u>Step 1:</u> Because the bridge superstructure is discontinuous at the expansion joint the bridge does not qualify as having non vulnerable bearing details.



SUPERSTRUCTURE PROPERTIES

A_x = 90.8 SQ. FT

I_{11} = 168 FT^4

I_{22} = 20600 FT^4

I_{33} = 536 FT^4

LOADING

A = 0.4

SOIL TYPE = II

ADDITIONAL DEADLOAD = 2.46 K/FT

FIGURE A-1: EXAMPLE BRIDGE

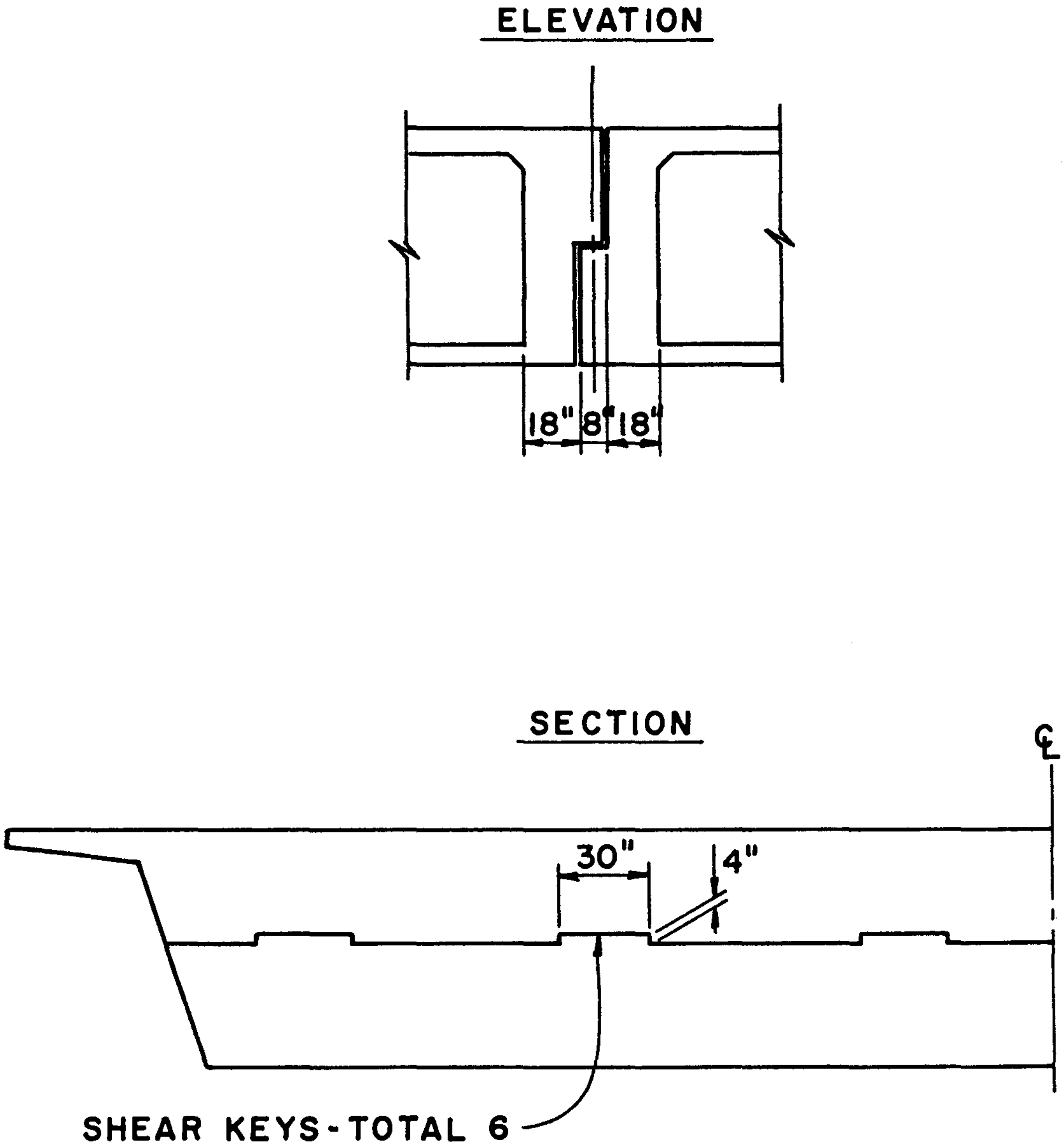


FIGURE A-2: EXAMPLE BRIDGE - EXPANSION JOINT DETAIL

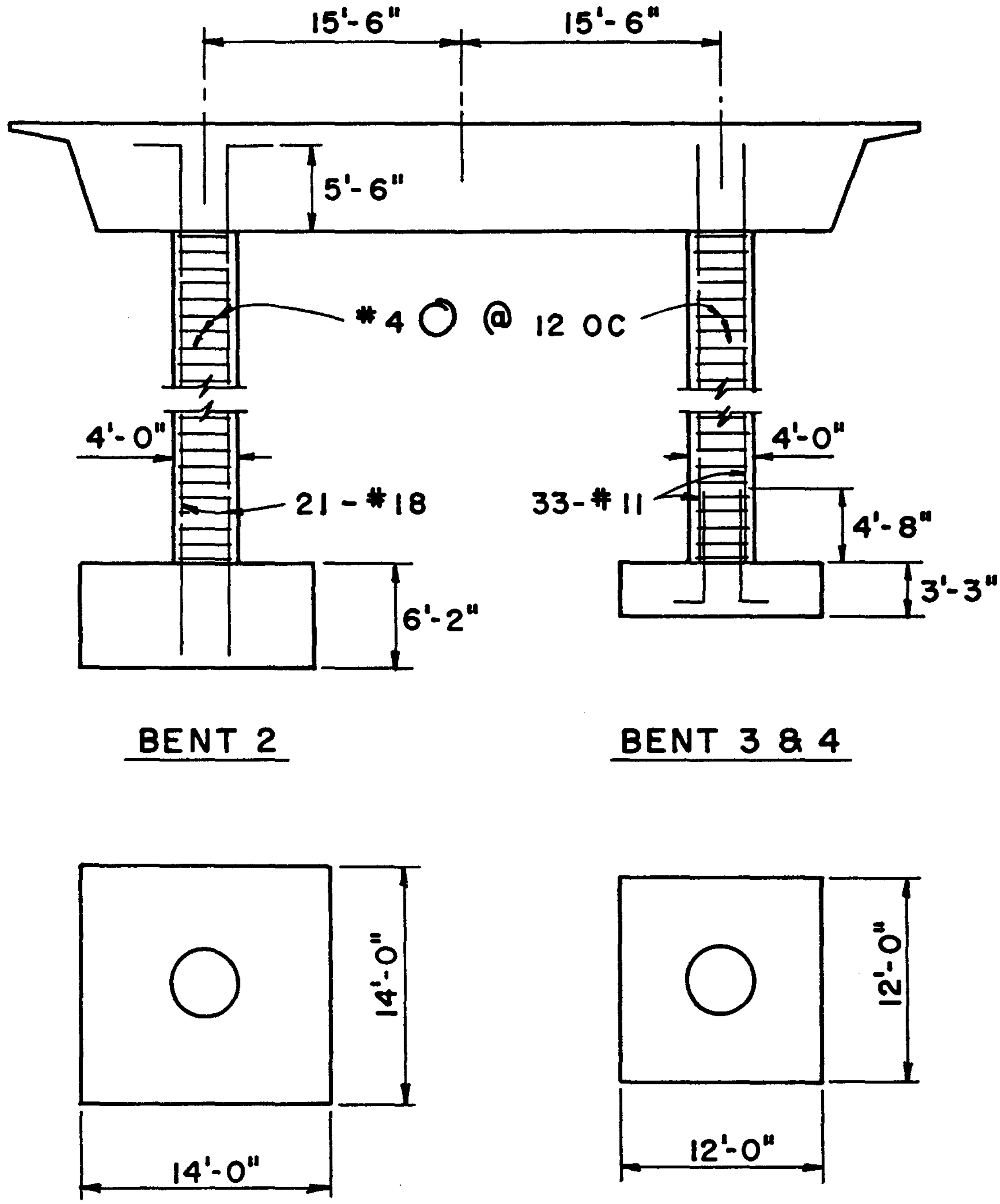


FIGURE A-3: EXAMPLE BRIDGE - COLUMN DETAILS

Step 2: Although the concrete shear keys are subject to failure the bearing seat is continuous in the transverse direction and therefore not subject to a serious failure resulting from transverse movement.

Step 3: Calculate the minimum support length

$$L = 470 \text{ ft.}$$

$$H = (40 + 0) \div 2 = 20 \text{ ft.}$$

Therefore:

$$N(d) = 12 + 0.03L + .12H$$

$$= 12 + 0.03(470) + .12(20)$$

$$= 28.5 \text{ inches}$$

$$\frac{N(c)}{N(d)} = \frac{8}{28.5} = .28 < .50$$

Therefore:

Vulnerability Rating (bearings) = 10

B. **Columns, Piers, and Footing**

Step 1: Does not apply

Step 2: Does not apply

Step 3: Does not apply

Step 4: Calculate the BVR for the shortest and most heavily reinforced columns which are the columns in Bent 2.

$$BVR = 13 - 6 \left(\frac{Lc}{P_s Fb_{max}}\right)$$

$$= 13 - 6 \left(\frac{40}{4.6(2)(4)}\right)$$

$$= 6.5$$

Because A $\leq$ 0.4 and the support skew is less than 20^{o}, the maximum reduction of 4 can be made

$$CVR = BVR - 4 = 6.5 - 4 = 2.5 \simeq 3$$

Step 5: Does not apply

Step 6: Does not apply

Therefore:

Vulnerability Rating (columns) = 3

C. **Abutments**

Step 1: Does not apply

Step 2: Does not apply because the freeway passing under the bridge is in cut.

Step 3: Does not apply

Therefore:

Vulnerability Rating (Abutments) = 0

D. **Liquefaction**

Step 1: The soil at the site is dense to very dense unsaturated sand and gravel. Therefore, the site has a low susceptibility to liquefaction.

Step 2: Low liquefaction related damage is likely.

Step 3: Does not apply

Step 4: Does not apply

Step 5: Does not apply

Therefore:

Vulnerability Rating (liquefaction) = 0

Maximum Vulnerability Rating = 10

A.3.2 SEISMICITY RATING (Section 2.3.2)

$$A = 0.4$$

$$\text{Seismicity Rating} = 25(0.4) = 10$$

A.3.3 IMPORTANCE RATING (Section 2.3.3)

The Importance Classification (IC) of the bridge is I. Therefore, the Importance Rating is between 6 and 10. The structure is located in a major metropolitan area of moderate population density. Although there is considerable traffic on the freeway, there is a diamond interchange at the overcrossing through which freeway traffic can be easily detoured. The freeway is of local importance, but is not a major regional evacuation route. The major city street does not provide exclusive access to any emergency response facilities or hospitals although major water and gas lines are carried on the bridge. Therefore, based on judgement.

Importance Rating = 8

<u>Overall Seismic Rating</u>

Vulnerability Rating	x	weight	=	10 x 3.33	=	33.3
Seismicity Rating	x	weight	=	10 x 3.33	=	33.3
Importance Rating	x	weight	=	8 x 3.33	=	<u>26.6</u>
				Seismic Rating	≈	93

The seismic ratings of several bridges should be compared to determine which bridges are in greatest need of seismic retrofitting. A score of 93 is relatively high and indicates that the example bridge should probably be evaluated in greater detail to determine the most appropriate retrofitting measures.

A.4 DETAILED EVALUATION (Chapter 3)

The existing bridge is evaluated in detail to identify structural weaknesses and to select the most economical retrofitting measure. The first step in this procedure is the determination of Capacity/Demand (C/D) ratios for the components of the existing bridge.

A.4.1 CAPACITY/DEMAND RATIOS - EXISTING BRIDGE (Chapter 4 and Section 3.4)

Capacity/Demand ratios are calculated for the applicable components shown in Table 3.

Analysis Procedure (Section 4.2)

Analysis procedure 2, a multi-modal spectral analysis, is required for this bridge. Although any one of a number of computer programs can be used to perform such an analysis, the SEISAB computer program which is a user oriented program specifically developed for bridge seismic analysis, was used for this problem. The program was developed to assist in the implementation of the Seismic Design Guidelines and automatically combines orthogonal elastic forces as described in Section 4.4. Foundation stiffnesses at the abutments were selected using the procedure outlined in Section 4.2.1 of the Commentary. The following pages include the input coding and the applicable portion of the output listing from this program for the existing bridge. Output is in units consistant with the input, which in this case are kips and feet. Output forces and moments should be interpreted according to the convention shown in Figures A-4, and A-5.

Minimum Bearing Force Demands (Section 4.5)

The minimum force demand for the transverse shear key at the expansion joint

SEISAB INPUT DATA FILE - EXISTING BRIDGE


```
C ***********************************************************************
C *                                                                     *
C *                         WORKED EXAMPLE                              *
C *                                                                     *
C *       RESPONSE SPECTRUM ANALYSIS OF THE EXISTING BRIDGE             *
C *                                                                     *
C ***********************************************************************
C
SEISAB 'WORKED EXAMPLE - NO RETROFITTING'
RESPONSE SPECTRUM
C
C ---- ALIGNMENT DATA ----
C
ALIGNMENT
STATION 00 + 00
COORDINATES N 0.0 E 0.0
BEARING N 00 35 27 E
C
C ---- SPAN DATA ----
C
SPANS
LENGTHS 159.4 106.0 114.0 88.6
I11 168.0
I22 20600.0
I33 536.0
AREA 90.8
WEIGHT 2.46
C
C ---- DESCRIBE DATA BLOCK ----
C
DESCRIBE
C
C ** INDIVIDUAL COLUMN PROPERTIES **
C
COLUMN 'TYPE 1' "4 FT. ROUND COLUMN"
SEGMENTS 1
I11 25.1
I22 12.6
I33 12.6
AREA 12.6
C
C ** BENT CAP PROPERTIES **
C
CAP 'TYPE 1'
AREA 29.3
I11 103000.
I22 103000.
I33 103.
C
C ** WALL PROPERTIES **
C
WALL 'TYPE 1' "ABUTMENT 1 BACKWALL"
HEIGHT 9.8
AREA 140.0
I11 292.0
I22 72.9
I33 36600.
WALL 'TYPE 2' "ABUTMENT 5 BACKWALL"
HEIGHT 9.9
AREA 143.0
I11 297.0
I22 74.2
I33 38600.
C
C ---- ABUTMENT DATA ----
```

SEISAB INPUT DATA FILE - EXISTING BRIDGE

```
C
ABUTMENT STATION 00 + 00
ELEVATION 1371.74  1402.94
BEARING S 85 38 00 W, N 78 30 00 W
WIDTH NORMAL 56.0 56.0
CONNECTION PIN PIN
WALL 'TYPE 1' AT 1
WALL 'TYPE 2' AT 5
C
C ---- BENT DATA ----
C
BENT
BEARING N 89 36 58 W, N 89 36 58 W, N 79 52 00 W
ELEVATION TOP 1382.96 1389.95 1397.50
ELEVATION BOTTOM 1338.6 1338.6 1345.6
CAP 'TYPE 1' AT 2,3,4
COLUMN SKEWED LAYOUT 'TYPE 1' 31.0 'TYPE 1' AT 2,3
COLUMN SKEWED LAYOUT 'TYPE 1' 31.4 'TYPE 1' AT 4
C
C ---- EXPANSION JOINT DATA
C
HINGE
AT 1 146.4
BEARING N 89 36 58 W
WIDTH NORMAL 47.0
C
C ---- FOUNDATION STIFFNESS ----
C
FOUNDATION
AT ABUTMENT 1
KF1 67200.
KF2 20000.
KM1 1.0E+12
KM2 1.0E+12
KM3 1.0E+12
AT ABUTMENT 5
KF1 5300.
KF2 20000.
KM1 1.0E+12
KM2 1.0E+12
KM3 1.0E+12
C
C ---- LOADS DATA ----
C
LOADS
RESPONSE SPECTRUM
ACCELERATION COEFFICIENT 0.4
FINISH
```

SELECTED COMPUTER OUTPUT FROM SEISAB EXISTING BRIDGE

WORKED EXAMPLE - NO RETROFITTING

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	FREQUENCY	PARTICIPATION FACTORS X	Y	Z
1	1.411E 00	7.088E-01	1.404E 01	8.908E-03	4.995E-02
2	9.529E-01	1.049E 00	-6.460E-01	7.676E-02	1.378E 01
3	6.928E-01	1.443E 00	6.480E-03	6.719E 00	3.451E 00
4	4.665E-01	2.143E 00	5.077E 00	6.801E-02	1.264E 00
5	3.336E-01	2.997E 00	-8.272E-02	-6.803E-01	2.081E 00
6	3.088E-01	3.239E 00	-6.608E-01	-2.074E 00	7.370E 00
7	2.717E-01	3.681E 00	3.751E-01	8.936E 00	-1.480E 00
8	2.503E-01	3.996E 00	-6.608E 00	7.953E-01	-9.250E-01
9	2.189E-01	4.569E 00	4.750E-02	-8.051E 00	-2.240E-01
10	2.023E-01	4.942E 00	-1.518E 00	-1.805E-02	-4.616E-01
11	1.843E-01	5.426E 00	1.659E-01	3.702E 00	1.943E 00
12	1.324E-01	7.554E 00	2.130E-02	1.261E-01	2.128E-01

ABUTMENT, HINGE AND JOINT RELATIVE RMS DISPLACEMENTS

ITEM	LOAD CASE	LONGITUDINAL OPENING OR CLOSING
ABUT. 1	1	1.881E-07
	2	1.767E-08
ABUT. 5	1	2.141E-07
	2	1.891E-08
HINGE 1	1	4.433E-01
	2	1.790E-02

SELECTED COMPUTER OUTPUT FROM SEISAB EXISTING BRIDGE

WORKED EXAMPLE - NO RETROFITTING

RESPONSE SPECTRUM RESULTS (CONTINUED)

RMS COLUMN FORCES

COL	CASE	LONGITUDINAL MOMENT	LONGITUDINAL SHEAR	TRANSVERSE MOMENT	TRANSVERSE SHEAR	AXIAL FORCE
BENT 2						
1 BOT	1	4.788E 02	2.277E 01	7.180E 03	3.189E 02	3.099E 02
1 BOT	2	1.695E 04	7.544E 02	8.670E 02	3.942E 01	1.042E 03
1 TOP	1	4.928E 02	2.016E 01	6.659E 03	3.022E 02	3.089E 02
1 TOP	2	1.618E 04	7.363E 02	8.546E 02	3.800E 01	1.042E 03
2 BOT	1	5.666E 02	2.839E 01	6.989E 03	3.102E 02	3.235E 02
2 BOT	2	1.695E 04	7.543E 02	9.249E 02	4.195E 01	1.039E 03
2 TOP	1	6.338E 02	2.519E 01	6.473E 03	2.940E 02	3.224E 02
2 TOP	2	1.617E 04	7.362E 02	8.952E 02	3.980E 01	1.039E 03
BENT 3						
1 BOT	1	2.296E 02	1.115E 01	5.618E 03	2.217E 02	2.567E 02
1 BOT	2	8.648E 03	3.345E 02	6.451E 02	2.530E 01	5.342E 02
1 TOP	1	2.538E 02	6.871E 00	5.342E 03	2.020E 02	2.558E 02
1 TOP	2	8.213E 03	3.199E 02	6.261E 02	2.402E 01	5.341E 02
2 BOT	1	2.775E 02	1.320E 01	5.483E 03	2.164E 02	2.627E 02
2 BOT	2	8.643E 03	3.342E 02	7.207E 02	2.853E 01	5.245E 02
2 TOP	1	3.440E 02	1.063E 01	5.212E 03	1.971E 02	2.618E 02
2 TOP	2	8.204E 03	3.197E 02	7.011E 02	2.659E 01	5.244E 02
BENT 4						
1 BOT	1	3.472E 02	1.309E 01	5.545E 03	2.181E 02	2.128E 02
1 BOT	2	3.920E 03	1.508E 02	2.269E 02	9.266E 00	2.473E 02
1 TOP	1	2.627E 02	9.774E 00	5.337E 03	1.981E 02	2.125E 02
1 TOP	2	3.691E 03	1.412E 02	2.094E 02	7.504E 00	2.472E 02
2 BOT	1	4.658E 02	1.972E 01	5.437E 03	2.138E 02	1.565E 02
2 BOT	2	3.916E 03	1.505E 02	1.387E 03	5.366E 01	2.387E 02
2 TOP	1	4.892E 02	1.665E 01	5.234E 03	1.942E 02	1.561E 02
2 TOP	2	3.682E 03	1.410E 02	1.343E 03	5.118E 01	2.386E 02

SELECTED COMPUTER OUTPUT FROM SEISAB EXISTING BRIDGE

WORKED EXAMPLE - NO RETROFITTING

RESPONSE SPECTRUM RESULTS (CONTINUED)

ABUTMENT, HINGE AND JOINT RMS FORCES

ITEM		LOAD CASE	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. LONGITUDNL	W/R TO ITEM C.L. TRANSVERSE
ABUT.	1	1	1.881E 03	1.920E 02	3.222E 02	1.863E 03
		2	1.767E 02	1.121E 03	1.108E 03	2.443E 02
ABUT.	5	1	2.141E 03	2.692E 02	3.661E 02	2.127E 03
		2	1.891E 02	1.035E 03	1.048E 03	8.723E 01
HINGE	1	1	0.000E-01	1.009E 02	1.009E 02	3.646E-01
		2	0.000E-01	7.211E 02	7.211E 02	2.604E 00

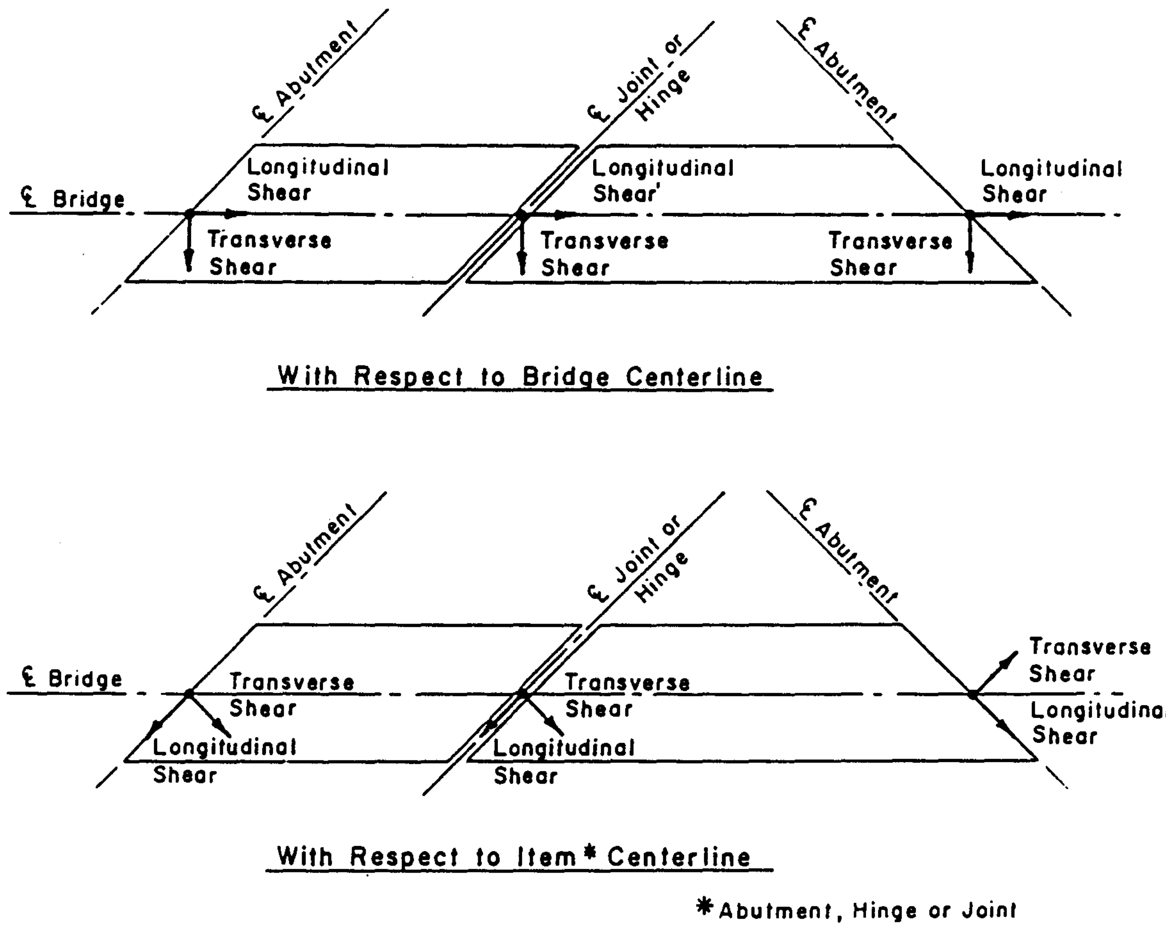


FIGURE A-4: POSITIVE SIGN CONVENTION FOR ABUTMENT, HINGE AND JOINT FORCES

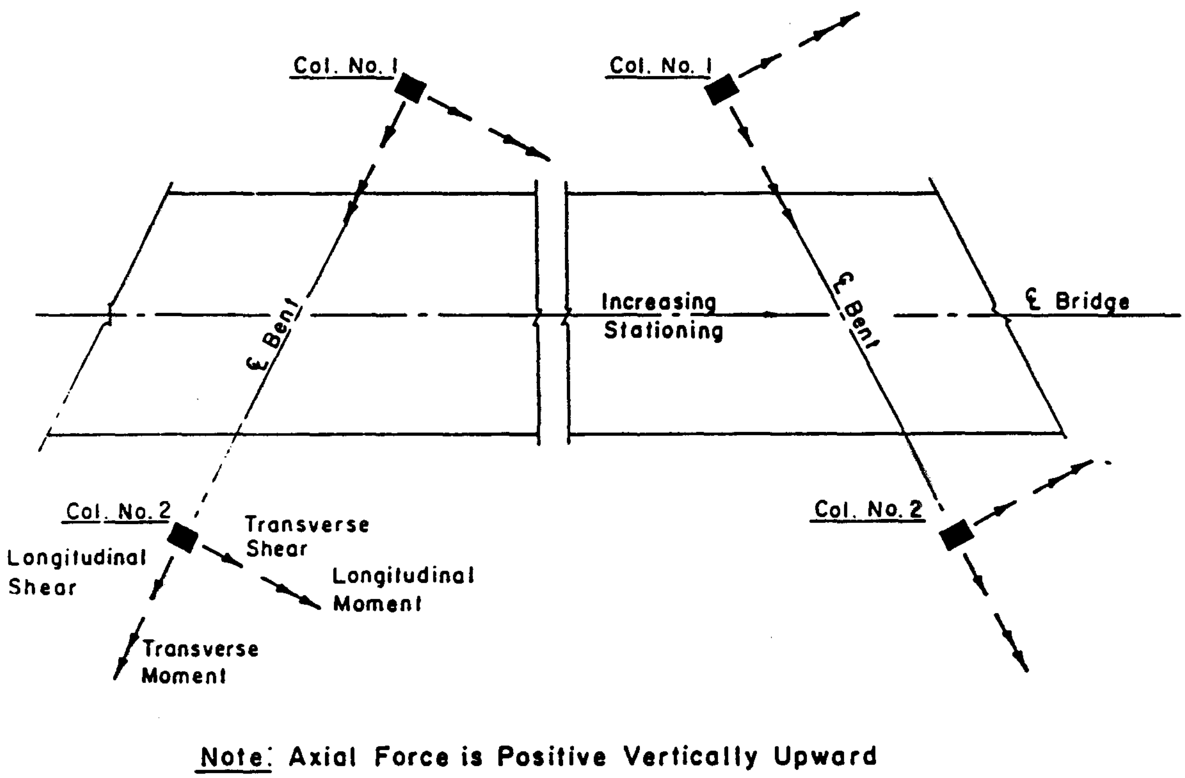


FIGURE A-5: POSITIVE SIGN CONVENTION FOR COLUMN FORCES AND MOMENTS

is calculated by considering the equivalent static load to be acting only on the suspended portion of the first span.

Superstructure Weight	=	90.8(.150) + 2.46
	=	16.1 kips/ft.
Minimum Force Demand	=	.20(16.1(147 ÷ 2))
	=	237 kips

Minimum Support Lengths (Section 4.6)

$$N(d) = 12 + 0.03(470) + .12(20) = 28.5 \text{ inches}$$

Capacity/Demand Ratio at the Expansion Joints and Bearings (Section 4.7)

Displacement C/D Ratio (Section 4.7.1) - Expansion joint

Method 1:

$$N(c) = 8 \text{ inches}$$

$$r_{bd} = \frac{N(c)}{N(d)} = \frac{8}{28.5} = .28$$

Method 2:

Assume that of the 8 inches of total seat length, 3 inches may be considered ineffective because it is the cover on expansion joint reinforcement.

$$\Delta_s(c) = 8 - 3 = 5 \text{ inches}$$

$$\Delta_i(d) = 3.3 \text{ inches (temperature, etc.)}$$

$$\Delta_{eq}(d) = 4.4 \text{ ft.} = 5.3 \text{ inches (from computer output)}$$

$$r_{bd} = (5 - 3.3) \div 5.3 = .32$$

Force C/D Ratio (Section 4.7.2)

$$V_b(d) = 721 \times 1.25 = 901 \text{ kips} > 237 \text{ kips}$$

$$V_b(c) = \frac{6(30)(4 + 8)(6 \quad 3250)}{1000} = 739 \text{ kips}$$

The shear resistance is provided by the six, 30 inch long shear keys. Concrete shear stress of $6\sqrt{f_c'}$ can be developed over the 4 inch high and 8 inch wide surface between the shear key and the superstructure.

$$r_{bf} = \frac{739}{901} = .82$$

Capacity/Demand Ratios at Columns, Piers, and Footings (Section 4.8)

Step 1: Elastic Moment Demands (Load Case 2 Controls)

The elastic moment demands are calculated by combining the moments about the principal axes of the columns to obtain the maximum moments. Load Case 2 has the highest demands. Deadload moments, which are also included in the calculations, have been obtained from a separate analysis. Moments at the base of the footing were obtained by adding the moment created by the shear at the top of the footing to the moment at the top of the footing. Elastic moment demands are summarized in Table A-1

TABLE A-1: MAXIMUM ELASTIC MOMENT DEMANDS (kip-feet)

Location	Component	Transverse Moment		Longitudinal Moment		Elastic Moment Demand
		EQ	DL	EQ	DL	
B-2 (C-2) Top	Column	900	60	16200	0	16200
B-2 (C-2) Bottom	Column	930	30	17000	0	17000
B-2 (C-2) Bottom	Footing	1200	40	21700	0	21700
B-3 (C-1) Top	Column	630	210	8210	0	8250
B-3 (C-1) Bottom	Column	650	160	8650	0	8700
B-3 (C-1) Bottom	Footing	730	180	9690	0	9730
B-4 (C-1)Top	Column	210	60	3690	0	3700
B-4 (C-1) Bottom	Column	230	20	3920	0	3930
B-4 (C-1) Bottom	Footing	260	20	4410	0	4420

Step 2: Ultimate Moment Capacities

Ultimate moment capacities for the columns are obtained from the computer generated column interaction diagrams shown in Figures A-6, and A-7. Ultimate moment capacities for the footing are obtained from interaction diagrams for the footings which are also shown in Figures A-6, and A-7. The development of coordinates on the footing interaction diagram for bent 2 is illustrated in Figure A-8. The development of these diagrams for bent 3 and 4 footings is similar.

Because elastic moment demands are primarily in the plane of the bent, moment capacities will be calculated for bending in this plane. This requires a consideration of the variation in axial load due to bent

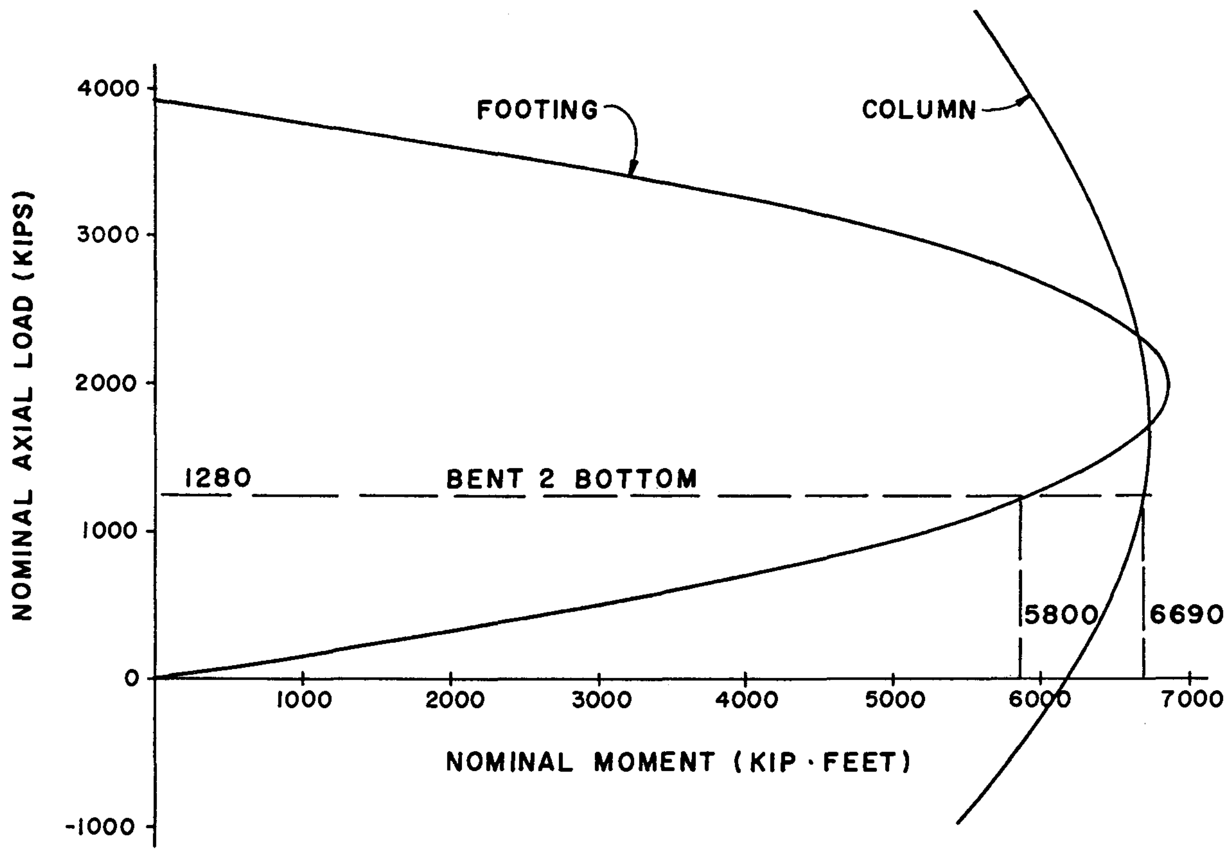


FIGURE A-6: BENT 2 COLUMN AND FOOTING INTERACTION DIAGRAMS

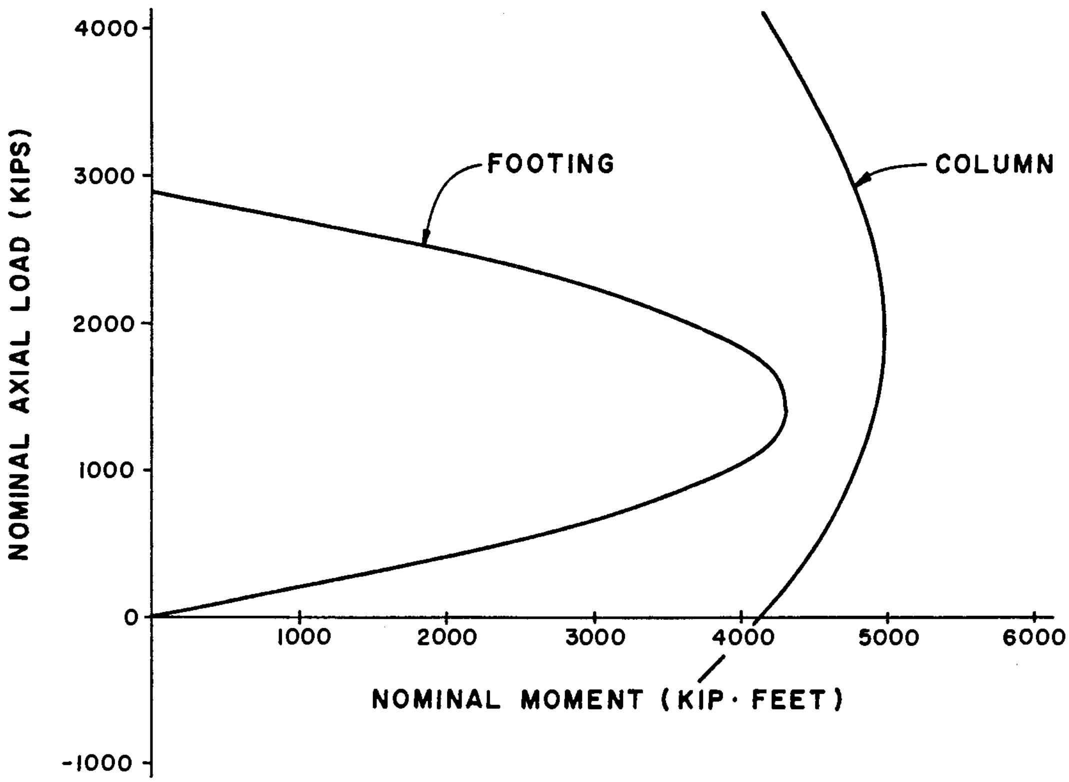


FIGURE A-7: BENT 3 AND 4 COLUMN AND FOOTING INTERACTION DIAGRAMS overturning as outlined in the iterative procedure presented in Section 4.8.2.B of the Seismic Design Guidelines. The steps of this procedure are as follows.

Step 1. Overstrength Moment Capacities at Axial Load Corresponding to Deadload

Table A-2 summarizes the overstrength column and footing moment capacities taken from the interaction diagrams. An example for the bottom of bent 2 is shown in Figure A-6. Bent 3 and 4 have identical capacities.

TABLE A-2: COLUMN AND FOOTING OVERSTRENGTH MOMENTS

Bent	End	Axial Force due to Dead Load	$1.3\ M_u$ Column	$1.3\ M_u$ Footing
2	Top	1190	8680	
2	Bottom	1280	8700	7540
3 & 4	Top	960	6180	
3 & 4	Bottom	1060	6290	5320

Step 2. Column Shear Forces*

Bent 2: $V_u = (8680 + 7540) \div 46.2 = 351$ kips

Bent 3 & 4: $V_u = (6180 + 5320) \div 53.3 = 216$ kips

* Because the ultimate moment is less at the footing than at the column base, the footing moments and the distance between the superstructure soffit and the base of the footing are used to calculate column shears.

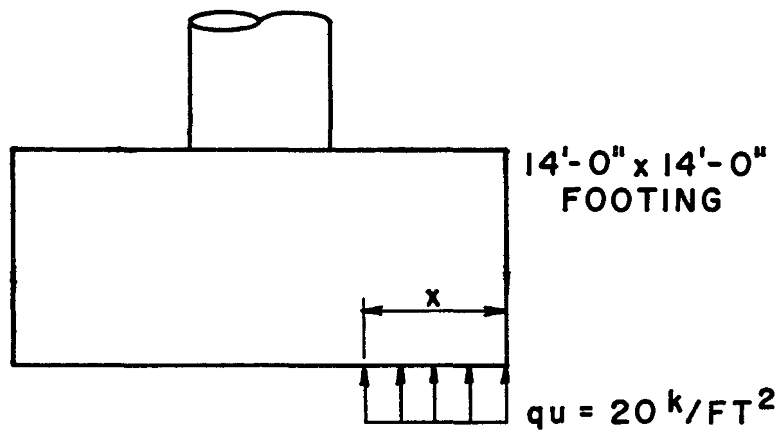
Step 3. Axial Forces Due to Overturning in the Transverse Direction

Bent 2: Axial Force $= 2(351)(46.2) \div 31 = +1046$

Bent 3 & 4: Axial Force $= 2(216)(53.3) \div 31 = +742$

Step 4. Revised Overstrength Moment Capacities

The axial loads due to overturning calculated in Step 3 are used to obtain new overstrength moment capacities from the interaction diagrams. Table A-3 summarizes these revised moment capacities.



ASSUMED ULTMATE LOAD AT
BENT 2 FOOTING

DEVELOPMENT OF EQUATIONS OF NOMINAL CAPACITY:

AXIAL LOAD CAPACITY = $(20)(14)x = 280x$

MOMENT CAPACITY = $(280x)(7-x/2) = 1960x - 140x^2$

x	AXIAL FORCE	MOMENT
2'	560	3360
4'	1220	5600
6'	1680	6720
8'	2240	6720
10'	2800	5600
12'	3360	3360
14'	3920	0

FIGURE A-8: DEVELOPMENT OF FOOTING INTERACTION SURFACE AT BENT 2

TABLE A-3: **REVISED COLUMN AND FOOTING OVERSTRENGTH MOMENTS (Iteration 1)**

Bent	End	Axial Force due to Dead Load + Overturning	1.3 M_u Column	1.3 M_u Footing
2	Top	144	8140	-
2	Top	2236	8700	-
2	Bottom	234	8220	1540
2	Bottom	2326	8660	8610
3 & 4	Top	218	5610	-
3 & 4	Top	1702	6480	-
3 & 4	Bottom	318	5730	2210
3 & 4	Bottom	1802	6500	5260

These moment capacities are used to calculate revised shear forces at the bent.

Shear at Bent 2 = (8140 + 1540) ÷ 46.2 + (8700 + 8610) ÷ 46.2

= 584 kips

Shear at Bents 3 & 4 = (5160 + 2210) ÷ 53.3 + (6480 + 5260) ÷ 53.3

= 359 kips

These bent shears are not within 10 percent of the bent shears (twice the column shear) calculated in step 2. Therefore the axial forces due to overturning must be recalculated.

Bent 2 Axial Force = 584(46.2) ÷ 31= + 870 kips

Bent 3 & 4 Axial Force = 359(53.3) ÷ 31= + 620 kips

These axial loads are used to recalculate the overstrength moments which are summarized in Table A-4

TABLE A-4: REVISED COLUMN AND FOOTING OVERSTRENGTH MOMENTS (Iteration 2)

Bent	End	Axial Load due to Dead Load + Overturning	$1.3\ M_u$ Column	$1.3\ M_u$ Footing
2	Top	320	8290	-
2	Top	2060	8730	-
2	Bottom	410	8350	2740
2	Bottom	2150	8720	8830
3 & 4	Top	340	5700	-
3 & 4	Top	1580	6460	-
3 & 4	Bottom	440	5790	2850
3 & 4	Bottom	1680	6480	5450

New shear forces at the bents are calculated using these moments.

Shear at Bent 2 = (8290 + 2740) ÷ 46.2 + (8730 + 8720) ÷ 46.2

= 616 kips

Shear at Bents 3 & 4 = (5700 + 2850) ÷ 53.3 + (6460 + 5450) ÷ 53.3

= 384 kips

The newly calculated bent shears are within 10 percent of the previously calculated shears and therefore no further iteration is needed.

Step 3: Ultimate Moment Capacity/Elastic Moment Demand Ratios

The most critical combinations of the unfactored nominal ultimate moment capacities and elastic moment demands are used to calculate r_{ec} and r_{ef} at each bent. The possible values of r_{ec} and r_{ef} are summarized in Table A-5.

TABLE A-5: ULTIMATE MOMENT CAPACITY/ELASTIC MOMENT DEMAND RATIOS

Bent	End	Axial Load	Column Demand	Column Capacity	r_{ec}	Footing Demand	Footing Capacity	r_{ef}
2	Top	Min.	16200	6380	.39	-	-	
2	Top	Max.	16200	6720	.41	-	-	
2	Bottom	Min.	17000	6420	.38	21700	2110	.10
2	Bottom	Max.	17000	6710	.39	21700	6790	.31
3	Top	Min.	8250	4380	.53			
3	Top	Max.	8250	4970	.60			
3	Bottom	Min.	8700	4450	.51	9730	2190	.23
3	Bottom	Max.	8700	4980	.57	9730	4190	.43
4	Top	Min.	3700	4380	1.18			
4	Top	Max.	3700	4970	1.34			
4	Bottom	Min.	3920	4450	1.13	4420	2190	.50
4	Bottom	Max.	3920	4980	1.27	4420	4190	.95

Step 4: Calculate C/D Ratios for Possible Plastic Hinging Cases at the Bottom of the Columns

Bent 2 - Case II (r_{ec} = .38 and r_{ef} = .10):

1. Anchorage (Section 4.8.1) - Straight anchorage

$$\ell_a(c) = 74 - 3 = 71 \text{ inches}$$

For anchorage in the footing, assume the large cover (62 inches) has a confining effect equal to transverse steel with equivalent tensile strength. In this case, twice the area of the cover divided by half the number of longitudinal bars is considered. Concrete Tensile Strength = $7.5\sqrt{f'_c}$ = 430 psi.

$$k_{tr} = 430(62)(2)/(21 \div 2)(600)(2.25)$$

$$= 3.76 > 2.5$$

$$\ell_a(d) = \frac{((60000 - 11000) \div 4.8)\ 2.25}{3250\ (1 + (2.5)(2.16 \div 2.25)\ + 2.5}$$

$$= 68 \text{ inches}$$

Therefore Case B applies. Calculate the negative moment capacity of the footing using a concrete tensile strength of 430 psi ($7.5\sqrt{f'_c}$).

$$\text{Negative Moment Capacity} = 430(168(74)^2 \div 6) \div 12000$$

$$= 5490 \text{ kip-ft.}$$

This capacity is sufficient to resist the weight of the overburden. Therefore,

$$r_{ca} = 1.0$$

2. Splices (Section 4.8.2) - Does not apply

3. Footing Rotation (Section 4.8.5)

   Because anchorage or splice failures will not prevent footing rotation,

   $r_{fr} = \mu r_{ef} = 4(.10) = .40$

Bent 2 - Case II: (r_{ec} = .39 and r_{ef} = .31)

1. Anchorage - Same as before

$$r_{ca} = 1.0$$

2. Splices - Does not apply

3. Footing Rotation

   $r_{fr} = \mu r_{ef} = 4(.31) = 1.24$

Bent 3 - Case II: (Two Possible combinations of r_{ec} and r_{ef} must be investigated - r_{ec} = .51 and r_{ef} = .23 plus r_{ec} = .57 and r_{ef} = .43)

1. Anchorage - Hooked Anchorage

   $\ell_a(c) = 33$ inches

   $\ell_a(d) = 0.7(1200)\ 1.38 \div\sqrt{3250}$

   $= 20$ inches

Case B applies.

$$\text{Negative Moment Capacity} = 430(144(39)^2 \div 6) \div 12000$$
$$= 1308 \text{ kip-ft.}$$

Because this capacity is sufficient to resist the weight of the overburden,

$$r_{ca} = 1.0$$

2. Splices (Section 4.8.2)

Because the clear spacing between splices = 1.5 inches < 4(1.38),

$$A_{tr}(c) = 2(.20)/(33 \div 2) = .02$$

$$A_{tr}(d) = \frac{12}{56} 1.56 = .33$$

$$\ell_s = 56 > \left(\frac{1860}{\sqrt{f'_c}}\right) d_b = 45$$

Therefore, Case A applies.

$$r_{cs} = .75\ (.51) = .38$$

$$r_{cs} = .75\ (.57) = .43$$

Notice that the minimum value for r_{cs} controls.

3. Footing

$$0.8\ r_{ef} = 0.8(.43) = .34 < .43$$

$$0.8\ r_{ef} = 0.8(.23) = .23 < .38$$

Therefore a splice failure cannot be assumed to prevent footing rotation. The minimum C/D ratio for the footing is given by

$$r_{fr} = 4(.23) = .92$$

Bent 4 - Case I (r_{ec} = 1.27 and r_{ef} = .95):

1. Anchorage - Same as bent 3

$$r_{ca} = 1.0$$

2. Splices

$$r_{cs} = .75\ (1.27) = .95$$

Bent 4 - Case II (r_{ec} = 1.13 and r_{ef} = .50)

1. Anchorage

$$r_{ca} = 1.0$$

2. Splices

$$r_{cs} = .75\ (1.13) = .85$$

3. Footing

$$0.8\ r_{ef} = 0.8(.50) = .40 < .85$$

Therefore, a splice failure cannot be assumed to prevent footing rotation. The minimum C/D ratio for the footing is given by:

$$r_{fr} = 4(.50) = 2.00$$

<u>Step 5: Calculate C/D Ratio at the Top of the Column</u>

Bent 2:

1. Anchorage

$$\ell_a(c) = 66 \text{ inches}$$

$$\ell_a(d) = 1200\ (2.25) \div \sqrt{3250} = 47 \text{ inches}$$

Therefore Case B applies

$$r_{ca} = 1.0$$

2. Splices - Does not apply

3. Confinement (Section 4.8.4)

$$\rho(c) = .20(3.14)(44) \div 3.14\ (24)^2(12)$$

$$= .0013$$

$$\rho(d) = .45\ \left(\frac{3.14(24)^2}{3.14(22)^2} - 1\right) \frac{3250}{60000}$$

$$= .0046$$

$$\frac{P_c}{f_c'\ Ag} = \frac{2150}{3.25(3.14)(24)^2} = .37$$

$$k_1 = \frac{.0013}{.0046\ (0.5 + 1.25(.37))}$$

$$= .29$$

$$k_2 = \frac{0.2}{(12 \div 48)} = 0.8$$

Because transverse steel is poorly anchored, an iterative solution for μ is required.

Try k_3 = .35 (Corresponds to μ = 2.7)

$$\mu \doteq 2 + 4 \left(\frac{.29 + .80}{2}\right) .35$$

$$= .28 \qquad \text{o.k.}$$

$$r_{cc} = \mu r_{ec} = 2.8(.39) = 1.09$$

Bent 3:

1. Anchorage

$$\ell_a(c) = 66 \text{ inches}$$

$$\ell_a(d) \doteq \frac{((60000 - 11000) \div 4.8)\ 1.38}{3250\ (1 + 2.5\ \frac{1.44}{1.38})}$$

$$= 70 \text{ inches}$$

Case A applies

$$r_{ca} = \frac{66}{70} (1.18) = 1.11$$

2. Splices - Does not apply

3. Confinement

$$k_1 = \frac{.0013}{.0046\ (0.5 + 1.25(.270))}$$

$$= .34$$

$$k_2 = \left(\frac{6}{12/1.38}\right) = .69$$

Try k_3 = .35 (Corresponds to μ = 2.7)

$$\mu = 2 + 4 \left(\frac{.34 + .69}{2}\right) .35$$

$$= 2.7 \qquad \text{ok}$$

$r_{cc} = 2.7\ (.53) = 1.43$

Bent 4:

1. Anchorage

$$r_{ca} = \frac{66}{70}\ (1.18) = 1.11$$

2. Splices - Does not apply

Step 6: Calculate C/D Ratios for Column Shear (Section 4.8.3)

Bent 2: (Transverse Bending) Notice that footing rotation will govern the maximum shear. Therefore use the nominal footing overstrength moment plus an effective length measured to the base of the footing.

$$V_u(d) = \frac{8730 + 8830}{46.2}$$

$$= 380 \text{ kips}$$

$$V_e(d) = 754 \text{ kips}$$

$$V_i(c) = v_c db + \frac{A_{tr} f_{yt} d}{s}$$

$$= .114(41.7)(48) + \frac{.4(60)(41.7)}{12}$$

$$= 312 \text{ kips}$$

Because column axial stress may fall below $.10f_c'$ and transverse steel is ineffective,

$V_f(c) = 0$

Therefore, Case A applies

$$r_{cv} = \frac{312}{754} = .41 > .38 \text{ (The value of } r_{ec}\text{)}$$

$$r_{cv} = .38$$

Bent 3: (An anchorage failure at the top of the column and rotation of the footing at the bottom of the column will limit the maximum shear.)

$$V_u(d) = \frac{(66/70)\ 6460 + 5450}{53.3}$$

$$= 217 \text{ kips}$$

$$V_e(d) = 335 \text{ kips}$$

$$V_i(c) = 312 \text{ kips}$$

$$V_f(c) = 0$$

Therefore, Case B applies

$$\mu = 2 + (.75(4)) \left(\frac{312 - 217}{312 - 0}\right)$$

$$= 2.9$$

$$r_{cv} = 2.9(.57) = 1.65$$

Bent 4:

$$r_{ec} = 1.13 > 1.0$$

$$V_e(d) = 151 \text{ kips}$$

$$r_{cv} = \frac{312}{151} = 2.1$$

Capacity/Demand Ratio for Abutments (Section 4.9)

Abutment C/D ratios are based on the displacements from an analysis.

Transverse Displacement

$$d(c) = 3 \text{ inches}$$

Abutment 1:

$$d(d) = 0.7 \text{ inches}$$

$$r_{ad} = \frac{3}{0.7} = 4.3$$

Abutment 5:

$$d(d) = 0.6$$

$$r_{ad} = \frac{3}{0.6} = 5.0$$

Longitudinal Displacement

$d(c) = 6$ inches

Abutment 1:

$d(d) = 0.3$ inches

$$r_{ad} = \frac{6}{0.3} = 20.0$$

Abutment 5:

$d(d) = 4.8$ inches

$$r_{ad} = \frac{6}{4.8} = 1.25$$

Capacity/Demand Ratio for Liquefaction (Section 4.10)

Because the preliminary screening (Seismic Rating System) indicated that low liquefaction related damage was likely, a C/D ratio does not have to be determined.

A.4.2 IDENTIFICATION AND ASSESSMENT OF POTENTIAL RETROFIT MEASURES (Section 3.5)

Table A-6 summarizes the C/D ratios which are less than one for the existing bridge.

TABLE A-6: CAPACITY DEMAND RATIOS FOR THE EXISITING BRIDGE

Component	Notation	As-Built Bridge
Expansion Joint	r_{bd} r_{bf}	.28 .82
Bent 2 (Overall)	r_{cv}	.38
Bent 2 (Bottom)	r_{fr}	.40
Bebt 3 (Bottom)	r_{fr}	.92
Bent 3 (Top)	r_{ca}	.50

The expansion joint displacement is critical because it has the lowest C/D ratio and may result in a partial collapse of the bridge. This may be economically corrected by retrofitting the joint with longitudinal expansion joint restrainers. Because the transverse shear keys are also inadequate as indicated by the C/D ratio for bearing force, transverse pipe restrainers should also be included in any retrofitting.

A potentially serious failure is indicated by the C/D ratio for shear at bent 2. The shear failure in this case will be sudden and can result in a rapid disintigration in the ability of the column to support axial load. The seriousness of this particular shear failure is compounded because the column is located adjacent to the expansion joint which increases the probability of a partial collapse. Therefore, the consequences of a shear failure in bent 2 are unacceptable and warrent further consideration of retrofitting. Because shear failure is initiated by forces transverse to the centerline of the bridge, an infill shear wall at bent 2 would be a relatively economical retrofitting measure. This type of retrofiit would also eliminate the potential of the footing rotation failure at bent 2.

The next lowest C/D ratio occurs at the column steel anchorage in bent 3. There are several factors that make this potential failure of secondary concern. The primary effect will be a loss of flexural strength at the top of the column. Because of the bent redundancy, this will not result in the formation of a collapse mechanism for this case. Therefore, this anchorage failure in itself is not considered unacceptable. However, the footing rotation failure at bent 3 would threaten the stability of this bent when combined with the previously discussed anchorage failure. However, because the footing C/D ratio is fairly high, and the stiffening of bent 2 will greatly reduce bent 3 forces, retrofitting is not proposed.

Because the infill shear wall at bent 2 would significantly effect the dynamic response of the structure, another analysis is required. Computer input and output files for the retrofitted bridge are included on the following pages. An abbreviated reevaluation of the C/D ratios for the most critical components in the retrofitted bridge shows that the C/D ratios at bent 3 are greatly improved by the modified response.

Capacity/Demand Ratio at the Retrofitted Expansion Joint

Displacement C/D Ratio

Method 2:

$\Delta_s(c) \quad = \quad 5$ inches

$\Delta_i(d) \quad = \quad 3.3$ inches

$\Delta_{eq}(d) \quad = \quad 0.6$ inches

$$r_{bd} = \frac{(5 - 3.3)}{0.6} = 2.8 > 1 \quad \text{ok}$$

Capacity/Demand Ratios at the Columns, Piers, and Footing

Bent 3 (Bottom)

$r_{ef} \quad \simeq \quad 2190 \div (2570 + 74(3.25)) = .80$

$r_{fr} \quad = \quad 4(.80) = 3.2$

Bent 3 (Top):

SEISAB INPUT DATA FILE - RETROFITTED BRIDGE

```
C ***********************************************************************
C *                                                                     *
C *                          WORKED EXAMPLE                             *
C *                                                                     *
C *          RESPONSE SPECTRUM ANALYSIS OF THE EXISTING BRIDGE          *
C *                                                                     *
C ***********************************************************************
C
SEISAB 'WORKED EXAMPLE - RETROFITTED'
RESPONSE SPECTRUM
C
C ---- ALIGNMENT DATA ----
C
ALIGNMENT
STATION 00 + 00
COORDINATES N 0.0 E 0.0
BEARING N 00 35 27 E
C
C ---- SPAN DATA ----
C
SPANS
LENGTHS 159.4 106.0 114.0 88.6
I11 168.0
I22 20600.0
I33 536.0
AREA 90.8
WEIGHT 2.46
C
C ---- DESCRIBE DATA BLOCK ----
C
DESCRIBE
C
C ** INDIVIDUAL COLUMN PROPERTIES **
C
COLUMN 'TYPE 1' "4 FT. ROUND COLUMN"
SEGMENTS 1
I11 25.1
I22 12.6
I33 12.6
AREA 12.6
COLUMN 'INFILL' "12 INCH INFILL WALL"
SEGMENTS 1
AREA 51.2
I11 60.0
I22 27.4
I33 7135.
C
C ** RESTRAINER PROPERTIES **
C
RESTRAINER 'TYPE 1' "CALIFORNIA CABLE RESTRAINER"
LENGTH 8.0
AREA .0215
E 2590000.
C
C ** BENT CAP PROPERTIES **
C
CAP 'TYPE 1'
AREA 29.3
I11 103000.
I22 103000.
I33 103.
```

SEISAB INPUT DATA FILE - RETROFITTED BRIDGE

```
WALL 'TYPE 1' "ABUTMENT 1 BACKWALL"
HEIGHT 9.8
AREA 140.0
I11 292.0
I22 72.9
I33 36600.
WALL 'TYPE 2' "ABUTMENT 5 BACKWALL"
HEIGHT 9.9
AREA 143.0
I11 297.0
I22 74.2
I33 38600.
C
C ---- ABUTMENT DATA ----
C
ABUTMENT STATION 00 + 00
ELEVATION 1371.74  1402.94
BEARING S 85 38 00 W, N 78 30 00 W
WIDTH NORMAL 56.0 56.0
CONNECTION PIN PIN
WALL 'TYPE 1' AT 1
WALL 'TYPE 2' AT 5
C
C ---- BENT DATA ----
C
BENT
BEARING N 89 36 58 W, N 89 36 58 W, N 79 52 00 W
ELEVATION TOP 1382.96 1389.95 1397.50
ELEVATION BOTTOM 1338.6 1338.6 1345.6
CAP 'TYPE 1' AT 2,3,4
COLUMN 'INFILL' AT 2
COLUMN SKEWED LAYOUT 'TYPE 1' 31.0 'TYPE 1' AT 3
COLUMN SKEWED LAYOUT 'TYPE 1' 31.4 'TYPE 1' AT 4
C
C ---- EXPANSION JOINT DATA
C
HINGE
AT 1 146.4
BEARING N 89 36 58 W
WIDTH NORMAL 47.0
RESTRAINER NORMAL LAYOUT 'TYPE 1' 7.5 25.0 7.5 'TYPE 1' AT 1
C
C ---- FOUNDATION STIFFNESS ----
C
FOUNDATION
AT ABUTMENT 1
KF1 67200.
KF2 20000.
KM1 1.0E+12
KM2 1.0E+12
KM3 1.0E+12
AT ABUTMENT 5
KF1 5300.
KF2 20000.
KM1 1.0E+12
KM2 1.0E+12
KM3 1.0E+12
C
C ---- LOADS DATA ----
C
LOADS
RESPONSE SPECTRUM
ACCELERATION COEFFICIENT 0.4
FINISH
```

SELECTED COMPUTER OUTPUT FROM SEISAB RETROFITTED BRIDGE

WORKED EXAMPLE - RETROFITTED

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

			PARTICIPATION FACTORS		
MODE	PERIOD	FREQUENCY	X	Y	Z
1	7.473E-01	1.338E 00	-1.134E 00	4.707E 00	1.226E 01
2	5.383E-01	1.858E 00	-1.063E 01	-2.940E 00	4.195E 00
3	5.135E-01	1.947E 00	-6.079E 00	3.982E 00	-9.177E 00
4	3.288E-01	3.041E 00	9.129E-02	-4.839E-01	-3.638E-01
5	2.711E-01	3.688E 00	6.302E-01	8.120E 00	-6.084E-01
6	2.637E-01	3.793E 00	-9.077E 00	5.929E-01	4.382E-02
7	2.252E-01	4.440E 00	7.713E-01	-2.423E 00	3.279E 00
8	2.227E-01	4.489E 00	2.701E-02	3.374E 00	-1.295E 00
9	2.185E-01	4.576E 00	-2.611E-02	7.327E 00	6.859E-01
10	1.558E-01	6.417E 00	-5.065E-01	-2.263E 00	-1.879E 00
11	1.391E-01	7.188E 00	5.667E 00	-9.747E-02	-8.010E-03
12	1.307E-01	7.652E 00	7.230E-02	1.207E 00	-2.674E-01

ABUTMENT, HINGE AND JOINT RELATIVE RMS DISPLACEMENTS

ITEM	LOAD CASE	LONGITUDINAL OPENING OR CLOSING
ABUT. 1	1	2.157E-07
	2	6.897E-08
ABUT. 5	1	1.004E-07
	2	3.322E-08
HINGE 1	1	5.088E-02
	2	2.327E-02

SELECTED COMPUTER OUTPUT FROM SEISAB RETROFITTED BRIDGE

WORKED EXAMPLE - RETROFITTED

RESPONSE SPECTRUM RESULTS (CONTINUED)

RMS COLUMN FORCES

COL	CASE	LONGITUDINAL MOMENT	LONGITUDINAL SHEAR	TRANSVERSE MOMENT	TRANSVERSE SHEAR	AXIAL FORCE
BENT 2						
1 BOT	1	5.613E 04	1.273E 03	7.403E 03	3.501E 02	9.714E 02
1 BOT	2	1.040E 05	2.365E 03	1.072E 03	4.795E 01	2.745E 02
1 TOP	1	3.130E 02	1.268E 03	7.179E 03	2.985E 02	9.704E 02
1 TOP	2	3.926E 02	2.333E 03	8.020E 02	3.324E 01	2.740E 02
BENT 3						
1 BOT	1	1.792E 03	7.383E 01	2.406E 03	9.508E 01	2.021E 02
1 BOT	2	2.502E 03	1.015E 02	5.530E 02	2.322E 01	1.807E 02
1 TOP	1	1.555E 03	5.351E 01	2.151E 03	8.005E 01	2.017E 02
1 TOP	2	2.114E 03	7.382E 01	5.175E 02	1.780E 01	1.802E 02
2 BOT	1	1.703E 03	6.853E 01	2.426E 03	9.632E 01	8.251E 01
2 BOT	2	2.513E 03	1.023E 02	5.249E 02	2.199E 01	1.994E 02
2 TOP	1	1.389E 03	4.895E 01	2.168E 03	8.011E 01	8.201E 01
2 TOP	2	2.133E 03	7.430E 01	4.733E 02	1.613E 01	1.989E 02
BENT 4						
1 BOT	1	1.895E 03	7.649E 01	2.482E 03	9.928E 01	1.449E 02
1 BOT	2	2.686E 03	1.084E 02	5.608E 02	2.260E 01	1.849E 02
1 TOP	1	1.581E 03	5.399E 01	2.332E 03	8.388E 01	1.444E 02
1 TOP	2	2.267E 03	7.770E 01	4.880E 02	1.697E 01	1.846E 02
2 BOT	1	1.904E 03	7.693E 01	2.434E 03	9.726E 01	1.470E 02
2 BOT	2	2.671E 03	1.075E 02	7.816E 02	3.160E 01	1.561E 02
2 TOP	1	1.600E 03	5.456E 01	2.294E 03	8.267E 01	1.465E 02
2 TOP	2	2.238E 03	7.691E 01	6.789E 02	2.354E 01	1.558E 02

SELECTED COMPUTER OUTPUT FROM SEISAB RETROFITTED BRIDGE

WORKED EXAMPLE - RETROFITTED

RESPONSE SPECTRUM RESULTS (CONTINUED)

ABUTMENT, HINGE AND JOINT RMS FORCES

ITEM		LOAD CASE	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. LONGITUDNL	W/R TO ITEM C.L. TRANSVERSE
ABUT.	1	1	2.157E 03	1.468E 02	2.638E 02	2.146E 03
		2	6.897E 02	1.577E 03	1.577E 03	6.889E 02
ABUT.	5	1	1.004E 03	1.446E 03	1.491E 03	9.354E 02
		2	3.322E 02	2.055E 03	2.071E 03	2.082E 02
HINGE	1	1	1.420E 03	2.660E 01	2.939E 01	1.420E 03
		2	6.495E 02	9.436E 02	9.441E 02	6.487E 02
RES.	1	1	2.995E 02			
		2	3.413E 02			
RES.	2	1	3.099E 02			
		2	2.563E 02			
RES.	3	1	4.224E 02			
		2	2.082E 02			
RES.	4	1	4.706E 02			
		2	2.842E 02			

$$r_{ec} \approx 4380 \div 2130$$

$$= 2.06$$

$$r_{ca} = \frac{66}{70}(2.06) = 1.9$$

A.5 DESIGN OF RETROFIT DETAILS (Chapter 5)

The retrofitting details are designed for the force level specified in the Seismic Design Guidelines. The results of the computer analysis of the retrofitted structure are used.

A.5.1 LONGITUDINAL EXPANSION JOINTS RESTRAINERS (Section 5.3.1)

The number of longitudinal restrainers frequently is determined by a trial and error procedure because the number of restrainers will influence the total restrainer design force. This was necessary for this example. An initial analysis was performed using the number of restrainers necessary to provide the minimum restrainer force capacity. This analysis indicated a design force that was beyond the capacity of the assumed number of restainers. The analysis was reperformed using an increased number of restrainers sufficient to resist the design force from the first analysis plus an anticipated increase in design force resulting from the increased stiffness of the added restrainers. The analysis results shown are those in which the design forces equal the capacity of the assumed number of restrainers. A check of the results for a 7 cable California style restrainer (See Figure 36) indicates that one of the restrainer units is slightly overstressed as follows:

Maximum Restrainer Design Force	= 470 x 1.25	= 590 kips
Design Force (Single Cable)	= 590 ÷ 14	= 42 kips
Design Capacity (Single Cable)	= .85 (46)	= 39 kips

Although one of the 7 cable restrainer units will be slightly overstressed, the remaining three units will be at or below design stress. Therefore, four seven cable restrainer units are proposed. The need for diaphram bolsters is based on punching shear. Assuming a 12 inch square bearing plate, concrete shear stress of $2\sqrt{f'_c}$, and a capacity reduction factor of 0.85, a 30 inch thick wall is required to resist the design force given by 1.25 times the ultimate restrainer force (53 kips per cable).

$$\text{Bearing Plate Design Force} = 7(53)(1.25) = 464 \text{ kips}$$

Because the existing expansion joint diaphram is only 18 inches in thickness, 12 inch thick bolsters are required.

A.5.2 TRANSVERSE BEARING RESTRAINERS (Section 5.3.2)

The transverse design force at the expansion joint is given by

$$V_{eq} = 944 \times 1.25 = 1180 \text{ kips}$$

The design capacity of the existing concrete shear keys is

$$V_c = .85(739) = 628 \text{ kips}$$

Therefore, it is proposed that transverse pipe restrainers (See Figure 40) be added to provide the additional capacity to carry the design load.

$$\text{Required Pipe Restrainer Capacity} = 1180 - 628 = 552 \text{ kips}$$

A four inch double-extra strong pipe restrainer has a design capacity of 175 kips based on a 50 percent increase in the allowable steel shear stress. Therefore, four of these pipe restrainers will provide the required additional design capacity. Designers should be aware that concrete bearing stresses may be the controlling factor in the design of these transverse restrainers in some cases.

A.5.3 INFILL SHEAR WALL (Section 5.4.5)

If properly designed, the infill shear wall will cause bent 2 to behave like a pier. The design forces for this pier were obtained from the computer analysis and are based on the requirements of the Seismic Design Guidelines. A twelve inch thick infill wall is assumed.

$$\text{Design Shear (Load Case 2)} = 2370 \div 2 = 1185 \text{ kips}$$

$$\text{Design Moment (Load Case 2)} = 104000 \div 2 = 52000 \text{ kip feet}$$

The ultimate moment capacity of the pier, ignoring axial load is given by

$$\begin{aligned} \text{Ultimate Moment Capacity} &= A_s f_y \left(d - \frac{a}{2}\right) \\ &\simeq 84(60)(33.2).9 \\ &\simeq 150000 \text{ kip ft} \end{aligned}$$

Therefore the moment capacity is sufficient

The ultimate shear capacity of the pier is calculated according to Section 8.4.2 of the Seismic Design Guidelines

$$V_u = 2\sqrt{f_c'} + \rho_h f_y$$

$$\begin{aligned} \text{Design Shear Stress} &= 1185 \div 12(33.2(12)) \\ &= 248 \text{ psi} < 8\sqrt{f_c'} \end{aligned}$$

Therefore,

$$\rho_h = \frac{248 - 114}{60000} = .0022 < .0025$$

The minimum reinforcement ratio of .0025 will be used. Two curtains of vertical and horizontal reinforcing consisting of number 5 bars at 18 inches on center will satisfy this requirement. Dowels should be used to anchor the infill wall to the existing structure. To provide continuity, shear forces at the column-wall interface must be resisted by shear friction that is developed by these dowels.

$$\text{Shear Force} = \frac{VQ}{I}$$

where

$$Q \approx \pi(2)^2(31.5 \div 2) = 198 \text{ ft.}^3$$

$$I \approx \pi(2)^2(31.5 \div 2)^2(2) + 1(27.5)^3 \div 12$$

$$\div \quad 3120 + 1730 = 4850 \text{ ft.}^4$$

Therefore

$$\text{Shear Force} = \frac{1185(198)}{4850} \approx 48 \text{ kips/ft.}$$

This force will be resisted by shear friction if number 9 dowels at 12 inches on center are used to anchor the new construction to the existing structure.

APPENDIX B

DEVLOPMENT OF COLUMN VULNERABILITY RATING

Experience and research has shown that transverse reinforcement in the zones of yielding is important to the successful performance of reinforced concrete columns during earthquakes. Transverse reinforcement serves to confine the main longitudinal reinforcement and the concrete within the core of the column, thus preventing buckling of the main reinforcement and the severe loss of compression strength in the concrete. Transverse reinforcement is also effective as shear reinforcement and increases the shear capacity of the column.

Modern bridge design standards such as the American Association of State Highway and Transportation Officials (AASHTO) specifications and the Seismic Design Guidelines require minimum transverse confinement reinforcement and sufficient shear reinforcement to resist the shear forces developed by the formation of plastic hinges. Careful attention is given to the reinforcement details to ensure that transverse reinforcement remains effective during the cyclic loading that is characteristic of large earthquakes.

Unfortunately, prior to 1971 the transverse reinforcement placed in most bridge columns was totally inadequate by today's standards. For example, a typical pre-1971 AASHTO detail consisted of transverse hoops of 1/2-inch bars spaced at 12 inches on center. Hoops usually were lap spliced and crossties to support rectangular hoops at intermediate points often were not installed. The column damage suffered during the San Fernando earthquake demonstrated the inadequacy of this detail. Most bridges in service today have column transverse reinforcement details that are just as inadequate for seismic loading.

Although most bridge columns within the area of heavy damage in the San Fernando earthquake of 1971 had inadequate transverse reinforcement details, there was a vast difference in the way they performed. For many of the heavily damaged columns it appeared that shear was the primary mode of failure whether it was due to inadequacy of the initial shear capacity, or the degradation of shear capacity resulting from confinement failure. An example of the consequences of this type of column failure was demonstrated in the San Fernando Road Overhead shown in Figure B-1.

A method of screening existing bridges with poor column confinement details for column vulnerability is used in the proposed Seismic Rating System (see Section C2.3.1). Because shear has been observed as a critical column failure mode one step of the procedure uses a Base Vulnerability Rating (BVR) which reflects the ratio of relative shear capacity to relative expected shear force during an earthquake. The BVR can be calculated from data that can be obtained easily and rapidly from the current bridge inventory and a set of "as-built" bridge plans. The BVR was developed as an indicator of column vulnerability by observing the San Fernando earthquake.

The shear capacity of a column has classically been considered a function of the cross-sectional area of the column and the amount of shear reinforcement crossing a plane of diagonal tension failure. For columns with similar transverse reinforcement details, such as the pre-1971 AASHTO standard detail mentioned above, the shear capacity is approximately proportional to the cross-sectional area of the column. Therefore:

FIGURE B-1: DAMAGED COLUMN-SAN FERNANDO ROAD OVERHEAD

$$\text{Shear Capacity} \propto A_g \qquad \text{(B-1)}$$

where

A_g = Gross cross-sectional area of the column

The maximum expected shear force resulting from an earthquake in which flexural yielding takes place is given by:

$$\text{Shear Force} \simeq \frac{\Sigma M_u}{L_c} \qquad \text{(B-2)}$$

where

ΣM_u = The sum of the nominal ultimate bending moments at the top and/or bottom of the column

L_c = The effective height of the column

The sum of the nominal ultimate moments will depend on the column dimensions, the amount of longitudinal reinforcing steel, and the ultimate stresses in the material. For simplicity the column cross-section is assumed to be constant over its length and therefore the top and bottom ultimate moment capacities are equal. Most bridge columns will have axial loads approximately equal to 10% of the ultimate axial load. At this axial load the ultimate moment capacity of columns reinforced with equal strength steel is approximately proportional to the cross-sectional area of the steel and the maximum transverse column dimension. This proportional relationship is maintained if the gross cross-sectional area of the column times the percentage of main reinforcing steel is substituted for the cross-sectional area of the steel. Therefore,

$$M_u \propto A_g P_s b_{max} \qquad \text{(B-3)}$$

where

P_s = Percent main reinforcing steel

b_{max} = Maximum transverse column dimension

In many columns, the ultimate moment can only be produced at one end. To account for this, it is useful to consider a framing factor, F. This framing factor is multiplied by the ultimate moment to give an approximation of the sum of the moments at the top and bottom of the column. Therefore, the maximum earthquake shear force is given by:

$$\text{Shear Force} \simeq \frac{FM_u}{L_c} = \frac{FA_g P_s b_{max}}{L_c} \qquad \text{(B-4)}$$

where

F	=	2.0	For multicolumn bents - both ends fixed
F	=	1.0	For multicolumn bents - one end fixed
F	=	1.5	For single column bents - both ends fixed - box girder superstructure
F	=	1.25	For single column bents - both ends fixed - non-box girder flexible superstructure

TABLE B-1: BRIDGE COLUMN DAMAGE CLASSIFICATION – SAN FERNANDO EARTHQUAKE

	Bridge Name	Bridge Number	Column Data					
			L_c	b_{max}	P_s	F	$\frac{L_c}{FP_s b_{max}}$	Damage Classification
1.	Rte. 5 (Truck Lane)/405 Separation	53-1548	20'	5'	3.9%	2	.51	10
2.	San Fernando Road Overhead	53-1990R	32'	7'	4.2%	1.5	.73	8
3.	Northbound Truck Route U.C.	53-1991R	20'	4'	3.5%	1	1.43	6
4.	Foothill Blvd. U.C.	53-2016R	19'	4'	4.2%	2	.57	8
5.	Foothill Blvd. U.C.	53-2016L	22'	4'	2.9%	2	.95	5
6.	Los Angeles Aqueduct (Foothill Blvd)	53C-316	40'*	4'*	3.0%*	2	1.67	2.5
7.	West Sylmar O.H.	53-1984R/L	30'	4'	4.4%	1	1.7	2.5
8.	Bledsoe St. O.C.	53-1926	24'	5'	2.2%	2	1.09	4

*Estimated from the General Plan.

A parameter which reflects the relative likelihood of a shear failure, and thus the column vulnerability, is given by the ratio of shear capacity to the expected maximum earthquake shear force. By observing equations B-1 and B-4 it can be shown that

$$\frac{\text{Shear Capacity}}{\text{Shear Force}} \propto \frac{L_c}{P_s F b_{max}} \qquad \text{(B-5)}$$

To test the validity of this parameter as an indicator of potential column damage, it was compared with the column damage observed in the 1971 San Fernando earthquake. To do this, column damage was classified on a scale of 0 to 10 to correspond to the range of the Vulnerability Rating.

0	=	No damage observed
2.5	=	Minor damage (spalling) - does not weaken structure
5	=	Moderate damage which weakens structure but not to the extent that normal light traffic would be restricted
7.5	=	Major damage necessitating closure to traffic but no structure collapse - damage may be irreparable
10	=	Total damage - column disintegration and structure collapse

Several bridges were evaluated using these damage classifications and data from the field investigation of bridge damage. Table B-1 summarizes the results for bridges studied.

A plot of the damage classification versus the proposed parameter is shown in Figure B-2. It is evident from this graph that there is a useful correlation between the proposed parameter and the damage classification. By fitting a straight line throught the points on the graph and equating the damage classificaton to a Base Vulnerability Rating (BVR) the following relationship is achieved.

$$\text{BVR} = 13 - 6 \frac{L_c}{P_s F b_{max}} \qquad \text{(B-6)}$$

Some observations should be made about the proposed Base Vulnerability Rating. First it will be noted that columns with higher longitudinal reinforcement ratios are considered more vulnerable than similar columns with low reinforcement ratios. This is only the case for columns which experience approximately equal ductility demands. For columns designed for similar force levels in regions where seismic loading governs the column design ($A \geq .29$), this is probably approximately true. Secondly, it should be noted that all the bridges investigated were in the short-to-medium period range. It is expected that the longer the period of the structure the less important the proposed parameter is as an indicator of individual column damage since both the maximum and cumulative curvature ductility demands will be less. In general the primary mode of column failure is flexural in long period structures .

A flexural failure can be critical for single column bents. Critical failure can also result from the pullout of main longitudinal reinforcement as was the case in the Route 210/5 Separaton and O.H. during the San Fernando Earthquake. The failed column of this bridge is shown in Figure B-3. Notice how the main reinforcing bars which were anchored in a concrete pier shaft pulled cleanly out of the concrete allowing the

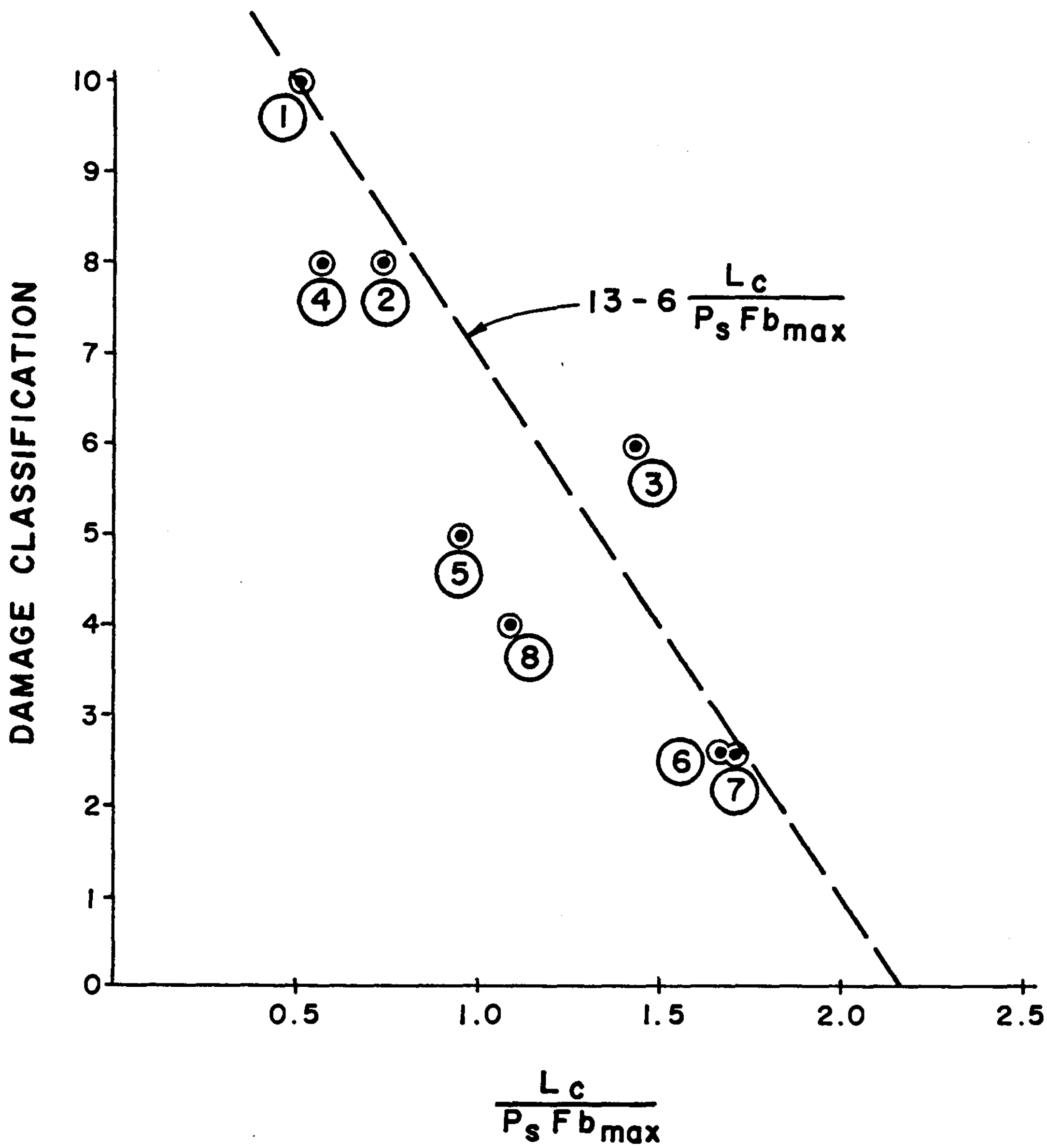


FIGURE B-2: COLUMN DAMAGE IN SHORT TO MEDIUM PERIOD BRIDGES DURING THE SAN FERNANDO EARTHQUAKE

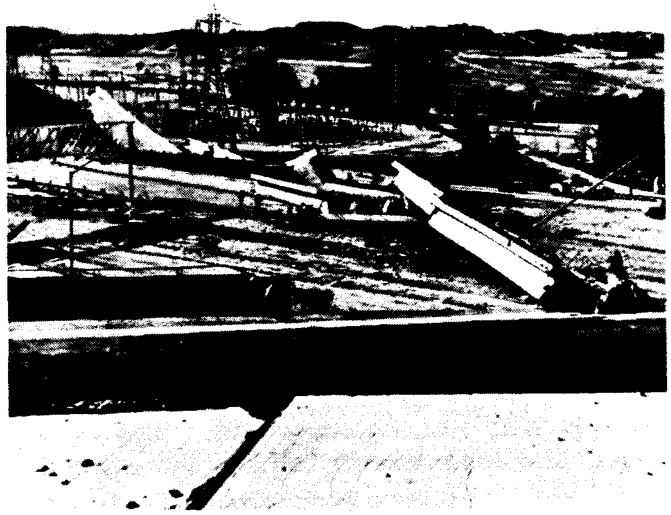


FIGURE B-3: COLUMN FAILURE RTE 210/5 SEPARATION AND O.H.

bridge to collapse. The BVR for this structure averaged approximately 4 which indicates a low susceptibility to serious column damage due to shear failure.

Certain factors besides the BVR appear to affect the performance of columns in short-to-medium period bridge structures although the BVR seems to be the most important. For example, a highly skewed structure will tend to respond in a rotational mode which introduces additional shear stresses in the columns that may accelerate a column shear failure. This appears to have been the case in bridges 3 and 4 above. This is demonstrated by the pattern of column failure in the right structure of Bridge 4 (Foothill Blvd. U.C.) shown in Figure B-4. The column to the right is severely damaged while the left column appears unaffected. The flexibility of the foundation can be important since it can reduce the effective fixity at the footing. This may have been important in the left structure at Bridge 5 (Foothill Blvd. U.C.) which has a BVR which would seem to indicate more damage than actually occurred. Variation in the foundation flexibility at the abutments may also be important. This was probably important in bridges 1 and 3 where one abutment was in fill and the other in cut. This may have caused the damage to be more severe than it ordinarily would have been. A transversely rigid and continuous superstructure on continuous diaphragm abutments will carry excess load and tend to mitigate the seriousness of column failures. Although no examples were studied, it is assumed that architectural flares will increase the sum of the moments and thus result in a greater maximum earthquake shear force. This may be taken into account in computing the Base Vulnerability Rating using a reduced effective length of the column.

Obviously the intensity and duration of the earthquake ground motion will effect the performance of the column. Greater resulting maximum and cumulative ductility demands will cause a more rapid degradation of column strength. The estimated peak ground acceleration for each of the bridges studied above was near .4g or greater. As was observed in the San Fernando and other earthquakes, a lower peak acceleration of similar duration would have less potential for damaging the columns .

Based on the past performance of columns in the San Fernando earthquake, the BVR appears to be a simple and effective method for screening potentially seismically deficient columns in short-to-middle period structures. Minor adjustments can be made to this factor to account for factors that may reduce column vulnerability as discussed above. Long period structures should not be considered vulnerable to column failures unless they have a very flexible superstructure supported on single column bents with lapped main reinforcement in the area of high moment or poorly anchored main reinforcement.

FIGURE B-4: COLUMN DAMAGE FOOTHILL BLVD, U.C.

APPENDIX C

TENTATIVE SIMPLIFIED RESTRAINER ANALYSIS–CALIFORNIA DEPARTMENT OF TRANSPORTATION METHOD

General Procedure

1. Compute the maximum permissible restrainer deflection and check the hinge seat width.
2. Compute the maximum longitudinal earthquake deflections on both sides of the superstructure joint under consideration.
3. Compare the deflections from steps 1 and 2 (above) and determine the course of action.
4. Determine the number of restrainers required.
5. Check the deflections of the restrained system and revise the restrainer and/or column assumptions if required.
   Repeat steps 1-5 if necessary.

Assumptions

- A segment is defined as a portion of superstructure between expansion joints.
- Two separate analyses will be required to evaluate the restrainers at a particular joint, one for the segment on each side of the joint. The segments should be assumed to be moving longitudinally away from the joint. Usually the lighter segment will govern the restrainer design, but if one segment is heavier and significantly stiffer it may require fewer restrainers. In this case the analysis which requires the FEWER number of restrainers will govern.
- The mass to be used for computing the earthquake force shall be the mass of the one segment adjacent to the joint under consideration.
- Assume one end of the restrainer is fixed with the mass of the segment moving away from the joint

- The longitudinal stiffness of the structure/restrainer system shall be computed by mobilizing the longitudinal stiffness of one adjacent segment in addition to the longitudinal stiffness of the segment under consideration. If the gap next to the adjacent segment is equal to or greater than the estimated earthquake deflection, then the adjacent segment cannot be expected to be mobilized. If this gap represents a significant portion of the estimated earthquake movement, then a reduced stiffness should be assumed. The abutment may be included as a part of the adjacent segment when gap considerations permit.

- Expansion joint gaps in recently constructed hinges with expanded polystyrene in the joint are not capable of transmitting any appreciable force until the joint is fully closed. Older hinges with 'expansion joint filler' in the joint may be considered closed after 50% of the gap is compressed if the material is still in the joint. Many of these older joints have been cleaned/rebuilt and the material removed. Do not assume there is material in the joint unless you know for sure it is there.

- Multiple simple-spans on bearings require an evaluation of the longitudinal adequacy of the bearings. If the bearings are not adequate to carry the earthquake forces to the substructure then only the restrainers can be utilized to compute the longitudinal stiffness of the system. Adjacent segments should not be considered when computing the stiffness of multiple simple-span systems.

- For retrofit analysis, a determination must be made in regard to column adequacy. As a general rule, older columns with widely spaced ties, lap splices in main reinforcement and inadequte footings cannot be expected to develop large ductile forces. Whenever the applied earthquake moments exceed about 80% of the yield capacity, these older columns should be assumed to have failed and a moment release introduced at that location. It is not too unreasonable to assume that 50% of the columns are damaged in this way, as it is unlikely that all of the columns will fail simultaneously.

Detailed Procedure

1. Compute the maximum permissible restrainer deflection and check the hinge seat width.

   a. Maximum permissible restrainer deflection, Dr.

   $$Dr = Dy + Dg$$

   Where -- Dr = The maximum permissible restrainer deflection.
   Dy = The restrainer deflection at yield.
   Dg = The gap in the restrainer system.

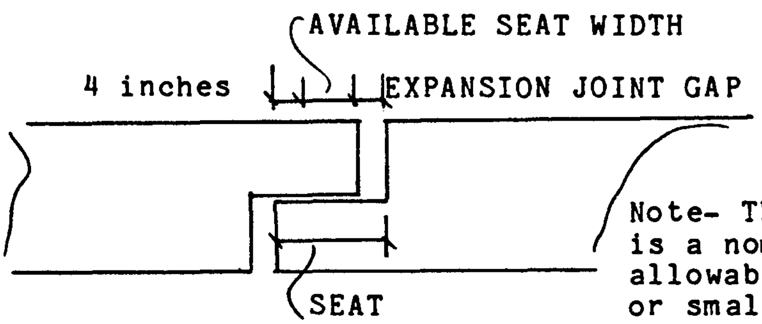
   $$\text{Yield deflection, } Dy = \frac{FyL}{E}$$

   Where -- Fy = Yield stress in restrainer
   = 176.1 ksi for cables (39.1/.222)
   = 120 ksi for rods

   L = Restrainer length

   E = Initial modulus of elasticity of restrainer (before initial stretching)
   = 10,000 ksi for cables
   = 30,000 ksi for rods

   b. Compare the available hinge seat width with the maximum permissible restrainer deflection, Dr.



Note- The 4 inch dimension shown is a nominal 'reasonable' allowable seat width. A larger or smaller dimension may be required.

If the maximum permissible restrainer deflection (Dr From 1a.) is greater than the available seat width then the hinge could become unseated before the restrainer capacity is reached. In this case, either Dr must be reduced by, (a) shortening the restrainers, (b) decreasing the restrainer gap, or (c) reducing the stress in the restrainers or the seat width must be increased.

Detailed Procedure Continued

2. Compute the maximum longitudinal earthquake deflections on both sides of the superstructure joint under consideration.

   a. Compute the unrestrained system stiffness, (Ku) of the segment nearest to the joint under consideration. Assume the segment is moving away from the joint under consideration. Consider all columns or piers which can be mobilized. The next adjacent segment (including the abutment, if present) may also be added if they can be mobilized. The segments on either side of the joint should be evaluated separately.

      DO NOT INCLUDE THE RESTRAINERS IN THIS CALCULATION EXCEPT FOR FULLY RELEASED SEGMENTS OR SIMPLE SPANS.

      Ku = The unrestrained total system stiffness.

      Where -- Ku = The equivalent stiffness of the total system considering the stiffness of all substructures mobilized and any gaps in the system.

      Stiffnesses, (K) of various componants ---

```
        Columns & Piers - K = 12EI/(L**3)   for Fixed-Fixed ends
                          K =  3EI/(L**3)   for Fixed-Pinned ends
                          K =  0.0          for Pinned-Pinned ends
              Abutments - K =  200 W
                  Piles - K =  40 k/in/pile
                    Where -- E = Modulus of elasticity
                             I = Moment of inertia
                             L = Length
                             W = The normal bridge width

                   Note -- The maximum force which can
                           be transferred to the soil
                           at the abutment is  7.7An where -
                              7.7 = Max. soil stress (ksf)
                               An = Abut. area of soil
                                    mobilized (normal)
```

** denotes exponentiation

Note - On retrofit jobs, the capacity of the columns or piers should be evaluated. If failure is expected. a reduced stiffness should be used to model the "failed" condition. It is not too unreasonable to assume that 50% of the columns or piers will be damaged, as it is unlikely that all of the columns or piers will fail simultaneously. Simple spans on bearings will require a similar analysis.
If failure of the bearings are expected from longitudinal forces, then the restraint offered by the substructure cannot be relied upon. In this case the stiffness of the system will come entirely from the restrainers.

Detailed Procedure Continued

2b. Compute the maximum earthquake deflection for both of the segments adjacent to the joint under consideration. Assume no restrainers in the system for this calculation. (Except for fully released segments such as simple spans.)

Compute the earthquake deflection, Deq = ARS(W)/Ku (in)

Where -- Deq = The earthquake deflection of the unrestrained system.
ARS = The acceleration in g. for a given period of vibration, T (sec). Where T = 0.32(W/Ku)**0.5 (Ref. Bridge Design Specifications, Figures 1.2.20 A-D and Section 1.2.20(B)).
W = Weight of the segment (k)
Ku = The unrestrained system stiffness (k/in), from 2a.

3. Compare the deflections from steps 1 and 2 and determine the course of action.

Compare the smaller of the two earthquake deflections from step 2b with the maximum permissible restrainer deflection from step 1a. If Deq is LESS than Dr, then only a minimum number of restrainers will be required. Provide at least 2 separate restrainer units across the joint. Locate these units as close as practicable to to the outside edges of the bridge.
If Deq is GREATER than Dr by a significant amount, the analysis will show that a large number of restrainers will be required. This is because the analysis will determine the number of cables required to modify the earthquake deflection, Deq to equal the restrainer capacity, Dr.

4. Determine the number of restrainers required.

Nr = Ku(Deq-Dr)/(FyAr)

Where Nr = The number of restrainers required.
Ku = The unrestrained system stiffness from 2a.
Deq = The deflection due to earthquake forces from 2b. (The minimum of the two values from each side of the joint should be used here.)
Dr = The maximum restrainer deflection from 1a.
Fy = Yield stress- 176.1 ksi for cables
120 ksi for rods
Ar = Area of one restrainer.

3/4" cables	=	0.222 sq in
1" rods	=	0.85 sq in
1 1/4" rods	=	1.25 sq in
1 1/2" rods	=	1.58 sq in

Detailed Procedure Continued

5. Check the deflection of the restrained system and revise the restrainer and/or column assumptions if required. Repeat steps 1-5 if necessary.

   a. Determine the deflection of the restrained system

      Dt = ARS(W)/Kt

      Where -- Dt = The deflection of the restrained system.
      ARS = The acceleration in g. for a given period of vibration, T (sec). Where T = 0.32(W/Kt)**0.5 (Ref. Bridge Design Specifications, Figures 1.2.20 A-D and Section 1.2.20(B)).
      W = Weight of the segment (k)
      Kt = The total restrained system stiffness (k/in)
      = Ku + Kr
      Ku = The unrestrained system stiffness (k/in)
      Kr = Fy(Nr)Ar/Dr
      Fy = Yield stress in restrainer
      176.1 ksi for cables (39.1/.222)
      120 ksi for rods
      Nr = The number of restrainers
      Ar = Area of one restrainer.

3/4" cables	=	0.222	sq in
1" rods	=	0.85	sq in
1 1/4" rods	=	1.25	sq in
1 1/2" rods	=	1.58	sq in

      Dr = The maximum restrainer deflection from 1a.

   b. Adjustment procedure

      If the deflection of the restrained system, (Dt) is not equal to the permissible restrainer deflection, (Dr), then the adjustment procedure must be used. Usually this adjustment is accomplished by changing the number of restrainers, but revision of gaps can sometimes be used for minor adjustments. Column or pier capacity under the restrained system deflection, (Dt) should be verified to assure that the initial assumptions are still valid. If not the model must be adjusted and steps 1-5 repeated.

      If Dr is GREATER than Dt, the number of restrainers may be reduced. After reduction, the new restrainer configuration should be checked to assure that Dr is not less than Dt.

      If Dr is LESS than Dt, the number of restrainers should be increased. Steps 1-5 should be repeated until Dr is equal to or greater than Dt.

APPENDIX D

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APPENDIX E

CONVERSION FACTORS TO SI METRIC UNITS

inches (in)	meters (m)	0.0254
inches (in)	centimeters (cm)	2.54
inches (in)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (mi)	kilometers (km)	1.609
degrees (°)	radians (rad)	0.0174
acres (acre)	hectares (ha)	0.405
acre-feet (acre-ft)	cubic meters (m^3)	1233
gallons (gal)	cubic meters (m^3)	3.79×10^{-3}
gallons (gal)	liters (l)	3.79
pounds (lb)	kilograms (kg)	0.4536
tons (ton, 2000 lb)	kilograms (kg)	907.2
pound force (lbf)	newtons (N)	4.448
pounds per sq in (psi)	newtons per sq m (N/m^2)	6895
pounds per sq ft (psf)	newtons per sq m (N/m^3)	47.88
foot-pounds (ft-lb)	joules (J)	1.356
horsepowers (hp)	watt (W)	746
British thermal units (Btu)	joules (J)	1055
British thermal units (Btu)	kilowatt-hours (kWh)	2.93×10^{-4}

<u>Definition</u>

newton - force that will give a 1-kg mass an acceleration of 1 m/s^2
joule - work done by a force of 1N over a displacement of 1 m
1 newton per sq m (N/m^2) = 1 pascal
1 kilogram force (kgf) = 9.087 N
1 gravity acceleration (g) = 9.087 m/s^2
1 are (a) = 100 m^2
1 hectare (ha) = 10,000 m^2
1 kip (kip) = 1000 lb

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.