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Seismic Retrofitting Manual for Highway Bridges

Research and Development
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FOREWORD

This manual is an interim revision of the Federal Highway Administration publication *Seismic Retrofitting Guidelines for Highway Bridges,* which was published in 1983 as report no. FHWA/RD-83/007. In the 10 years since the preparation of the 1983 guidelines, the state-of-the-art in seismic retrofitting has changed dramatically. This revision reflects experience gained with the use of these guidelines, as well as new knowledge acquired through research and earthquake reconnaissance studies that have been conducted since 1983.

This manual is a product of the FHWA's comprehensive Seismic Research Program for bridges and highways that was initiated in 1992 as a result of the Intermodal Surface Transportation Efficiency Act of 1991. The manual is considered to be an interim revision because the field of seismic hazard assessment and retrofitting is changing rapidly at this time, and a number of research programs funded by the Federal Highway Administration, the California Department of Transportation, and others, are still in progress. Another edition of this retrofitting manual will be prepared in approximately 5 years, when the results of these research programs and studies are known and have been tested in field applications and demonstration projects.

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16. Abstract

This manual is an interim revision of the Federal Highway Administration (FHWA) publication *Seismic Retrofitting Guidelines for Highway Bridges,* which was published in 1983 as report no. FHWA/RD-83/007. It describes an evaluation procedure for retrofitting seismically deficient highway bridges and outlines various measures to upgrade these structures.

The guidance provided in this manual is comprehensive in nature and nationally applicable to bridges in different seismic zones. It includes a revised preliminary screening procedure'; two methods of detailed evaluation (which include the capacity/demand ratio method and the lateral strength method); and expanded sections on retrofit measures for bearings, columns, footings, foundations, and soils. A new section on the application of seismic isolation to bridge retrofit is also included. These recommendations are generally compatible with those for the seismic design of new bridges contained in Division I-A of the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* and should be used in conjunction with these specifications.

The manual is termed an interim revision because the subject matter is still evolving at this time. It is expected to be reissued when the results of on-going research studies are known and have been tested in field applications and demonstration projects.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	aaes	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (oi 'metric ton')	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.78	lux	lx
fl	foot-Lamberts	3.428	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	8.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	aaes	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or 'metric ton')	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

PREFACE

This manual is an interim revision of the Federal Highway Administration (FHWA) publication *Seismic Retrofitting Guidelines for Highway Bridges,* which was published in 1983 as report no. FHWA/RD-83/007. In the 10 years since the preparation of the 1983 FHWA guidelines, the state-of-the-art in seismic retrofit has advanced substantially. This revision reflects experience gained with the use of the 1983 guidelines as well as new knowledge acquired through research and earthquake reconnaissance studies.

This manual is considered to be an interim revision because the field of seismic hazard assessment and retrofitting is still evolving at this time and a number of research programs, funded by the FHWA and the California Department of Transportation (Caltrans), are still in progress. Another edition of this manual will be prepared in approximately 5 years, when the results of these major research studies are known and have been tested in field applications and demonstration projects.

The principal reference sources for this revision are as follows:

Seismic Retrofitting Guidelines for Highway Bridges, Federal Highway Administration Report No. FHWA/RD-83/007, 1983, 205 pp.

Seismic Design and Retrofit Manual for Highway Bridges, Federal Highway Administration Report No. FHWA-IP-87-6, 1987, 290 pp.

Seismic Design References, California Department of Transportation, 1991.

In addition, sections 3.3.3, 3.5, 6.2, 6.3, 7.2, and appendix B were extracted from:

Priestley, M.J.N., Seible, F., and Chai, Y.H., *Design Guidelines for Assessment Retrofit and Repair of Bridges for Seismic Performance,* Report No. SSRP-92/01, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, 1992, 266 pp.

This revision also reflects recent changes in seismic design philosophy and performance criteria that have been proposed for the design of new highway bridges under projects sponsored by AASHTO through the National Cooperative Highway Research Program (NCHRP Project 20-7, Task 45, for the revision of current seismic design criteria and NCHRP Project 12-33 for limit state design specifications), and by the Applied Technology Council (Project ATC-32, for Caltrans seismic bridge design specifications).

The recommendations provided in this manual are intended to be used in conjunction with the seismic design specifications for new bridges, as contained in Division I-A of the 15th Edition (including Interim Specifications for 1993, 1994, and 1995) of *the* AASHTO *Standard Specifications for Highway Bridges.*

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LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations used in this report:

AASHTO	American Association of State Highway and Transportation Officials
ATC	Applied Technology Council
Caltrans	California Department of Transportation
FI-IWA	Federal Highway Administration
NCHRP	National Cooperative Highway Research Program
NIST	National Institute of Standards and Technology
UCSD	University of California at San Diego

Symbols used in this report:

A	Acceleration coefficient
A_b	Area of reinforcing bar
A_g	Gross column area (section A.5.4)
A_p	Area of prestressing wire wrap (section 62.3)
A_r	Area of cable restrainers (sections 3.3.3.4 and 5.2.4)
AVR	Abutment vulnerability rating (section 2.3)
BCR	Benefit-to-cost ratio (section 1.7)
b_{eff}	Effective joint width (section B.5)
b_{max}	Maximum transverse column dimension
b_{min}	Minimum transverse column dimension
C/D	Capacity/Demand ratio
CVR	Column vulnerability rating (section 2.3)
C	Column axial compression
c	Lesser of clear cover or half the clear spacing between adjacent longitudinal reinforcing (section AS. 1)
D	Column diameter (section 6.2.3)
$D_{(c)}$	Displacement capacity (section A.6)
$D_{(d)}$	Displacement demand (section A.6)
D'	Reinforcing steel pitch diameter (sections 6.2.3 and B.2.2)
D_r	Deflection capacity of restrainer (section 5.2.4)
D_y	Restrainer deflection at yield (section 5.2.4)
D_g	Gap in the restrainer system (section 5.2.4)
D_l	Longitudinal earthquake deflection (section 5.2.4)
D_t	Transverse earthquake deflection (section 5.2.4)
d_b	Reinforcing bar diameter
E	Modulus of elasticity
	Seismic hazard rating (section 2.3)
F	Framing factor used for calculation of column, pier, and footing vulnerabilities (section 2.3.1.1)
	Lateral force used in frame analysis for the lateral strength method (section 3.3.3)

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

F_y	Steel yield stress
f'_c	Concrete compressive stress
f'_{cc}	Confined concrete strength (sections 6.2.3.2(a) and B.2.2)
f_ℓ	Column confinement pressure (section 6.2.3)
f_{pu}	Ultimate stress for prestressing wire
f_s	Steel stress
G	Gap opening at joints (section 5.2.4)
g	Acceleration due to gravity
	Gap between retrofit measure and critical bridge section (section 6.2.3)
H	Average height of columns supporting the bridge deck to the next expansion joint (section 3.3.3.4)
IC	Importance Classification
I	Moment of inertia
K	Column, deck, or frame stiffness
k_s	Reinforcing steel constant (section AS. 1)
Loss_n	Probable loss before (n = b) or after (n = r) retrofitting
L	Length of bridge deck to next expansion joint (section 3.3.3.4)
	Restrainer length (section 5.2.4)
LVR	Liquefaction vulnerability rating (section 2.3)
L_c	Effective column height
ℓ_a	Effective anchorage length (section A.5.1)
ℓ_e	Embedment length (section B.2.2)
ℓ_p	Plastic hinge length (section 6.2.3)
ℓ_s	Splice length (sections 6.2.3 and B.2.1)
M_{cap}	Moment capacity
M_n	Nominal moment capacity (section B.2.2)
M_y	Moment at first yield (section B.2.2)
N	Minimum required seat length
N_r	Number of cable restrainers required (section 5.2.4)
n	Number of cable restrainers present (section 3.3.3.4)
P	Column axial force
	Priority index (section 2)
P_c	Column compressive axial load (section A.5.4)
P_s	Amount of main reinforcing steel expressed as a percent of the column cross-sectional area
p	Risk, expressed as an annual probability of exceedance (section 3.5.2)
Q	Column shear vulnerability factor (section 2.3)
	Displacement or force demands (section 3.4)
R	Bridge rank (section 2.3)
	Force reduction factor (section 3.5.2)
	Moment capacity ratio (section 3.3.3.2)
	Radius (section 5.2.4)

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

R_c Nominal ultimate displacement or force capacity (section 3.4)
r Calculated C/D ratios
SPC Seismic Performance Category
S Soil site coefficient
$S_{A(T)}$ Design acceleration response spectrum (section B.6)
$S_{D(T)}$ Displacement response spectrum (section B.6)
$S_{a(p)}$ Base shear coefficient (section 3.5.2)
$S_{a(R)}$ Design elastic spectral response level (section 3.3.3.2)
$S_{a(r)}$ Site assessment response spectrum (section 3.5.2)
s Spacing of prestressed wire wrap (section 6.2.3)
T Fundamental period of vibration
T_b Maximum force that can be developed in a column bar without transverse confinement present (section B.2.2)
T_0 Period corresponding to peak spectral response for a given site (section 3.5.2)
t Thickness of steel jacket (section 62.3)
t_a Fiberglass or epoxy jacket active wrap thickness (section 62.3)
t_p Fiberglass or epoxy jacket passive wrap thickness (section 6.2.3)
V Column shear force or shear capacity
Bridge vulnerability rating (section 2)
V_1 Vulnerability rating for connections, bearings, and seat widths (section 2.3)
V_2 Vulnerability rating for columns, foundations, abutments, and soils (section 2.3)
V_c Shear carried by concrete (section B.4)
V_e Equivalent elastic lateral strength (section 3.5)
V_f Equivalent lateral mechanism strength (section 3.3.3.5)
Equivalent yield strength (section 3.5)
V_L Bearing vulnerability to longitudinal movement (section 2.3)
V_n Nominal shear strength (section B.4)
V_p Shear carried by axial compression (section B.4)
V_t Shear carried by truss mechanisms (section B.4)
V_T Bearing vulnerability to transverse movement (section 2.3)
v Shear stress
W Bridge width (section 5.2.4.1)
Weight
Δ Displacement
Δ_y Calculated yield displacement (section 3.3.3.5)
ε Strain
ε_{su} Fracture strain of prestressing wire
μ Ductility indicator
Coefficient of friction (section 7.3)
ϕ Capacity reduction factor
Curvature (section B.2.2)
ρ Volumetric ratio of reinforcing steel (sections 6.2.3, A.5.4, B.4)

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

σ_u Effective lap splice bond stress (section B.2.2)

θ Rotation (section B.2.2)

χ Coefficient related to steel reinforcing bar grade (χ=6 for grade 40 reinforcement and χ=9 for grade 60 reinforcement) (section 6.2.3)

CHAPTER 1

INTRODUCTION

1.1 GENERAL

It has become apparent in recent years that many bridges in the United States are inadequate to resist seismic loadings. Several bridges have collapsed in Alaska, California, and Oregon as a result of recent earthquakes. Furthermore, some of these failures occurred at relatively low levels of ground motion. Although the risk of bridge collapse is lower in the Central and Eastern United States, ground motions of sufficient magnitude to cause bridge damage have been estimated to have a l-in-10 chance of occurring within the next 50 years, in 37 of the 50 States and Puerto Rico.(1) Fifteen of these States plus Puerto Rico are subject to relatively high levels of ground shaking. It is therefore necessary that an effort be made to identify seismically deficient bridges, evaluate the consequences of seismic damage, and initiate a program for reducing this seismic risk.

Seismic retrofitting of existing bridges is one method of mitigating the risk that currently exists. However, the goals and economics of retrofitting may differ from those of new construction. The options of doing nothing and thus accepting the risk of failure, and of abandoning or replacing the bridge, may also be considered. This requires that both the importance and degree of vulnerability of the structure be evaluated. Important bridges with high vulnerability in high seismic zones should be given first 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 often limited to preventing unacceptable failure. This implies that a considerable amount of structural damage during a major earthquake is acceptable provided collapse of the bridge is prevented. However, for important bridges, the ability of the bridge to carry emergency traffic immediately following an earthquake may require a higher level of performance with less structural damage. The threshold of damage that will constitute unacceptable failure may therefore 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 and subjectivity of retrofitting decisions and the many nonengineering factors involved, a considerable amount of judgment will be required.

Since cost is also a major issue, it is important in low-to-moderate seismic zones that seismic retrofitting be considered whenever nonseismic rehabilitation of the bridge is planned or when bearing deficiencies exist in a structure. Mobilization and traffic control costs represent a major part of the total seismic retrofitting cost, and therefore it may be considerably more cost-effective to perform seismic retrofitting at the same time as other (nonseismic) rehabilitation.

This manual recommends that whenever practical, deficient components should be strengthened to new design standards. At first sight, this may appear to be 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 economic situations 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 at least strengthen such components to lower standards rather than to reject retrofitting altogether. Selection of acceptable levels of strengthening requires the judgment of the engineer, taking into consideration the performance of the remainder of the structure.

There are some secondary factors that may 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 is undesirable to strengthen a ductile component if load would then be transferred to a nonductile or brittle component. This should be the case even if calculations indicated an overall increase in seismic capacity.

Maintenance and inspection of retrofitted components should also be considered during the retrofit design stage. Many years may pass before a structure is subjected to an earthquake. The retrofit measure should be designed so that it can be maintained in a condition to function as planned when and if a significant earthquake does occur.

1.2 PURPOSE

This manual offers procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Specifically it contains:

- A preliminary screening process to identify and prioritize 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 of alternative seismic retrofitting measures, including cost and ease of installation.

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

This manual does 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 primary focus of this manual is directed towards the engineering factors.

Seismic retrofitting of bridges is a relatively new activity for most bridge engineers and is still an art requiring considerable engineering judgment. This manual presents the current state-of-the-art, but should not be interpreted in such a way as to restrict 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-level earthquake. Damage is unacceptable if it results in:

- Serious injury or the loss of life.

- Collapse of all or part of the bridge.

- Loss of use of a vital transportation route.

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 can be designed to limit damage so that, as far as possible, it occurs in easily accessible areas, particularly for low- or moderate-sized earthquakes. In this way, bridges can be repaired following an earthquake, if necessary, and restored 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.

Some particularly important bridges may need to be retrofitted to higher standards than required by the governing design code for new highway construction. This is because the current code does not usually recognize the need to limit damage in critical structures to a level that permits immediate access during post-earthquake recovery. In such cases, a rigorous evaluation should follow preliminary screening in order to justify the additional effort and cost required to retrofit these structures to a higher standard.

1.3 BACKGROUND

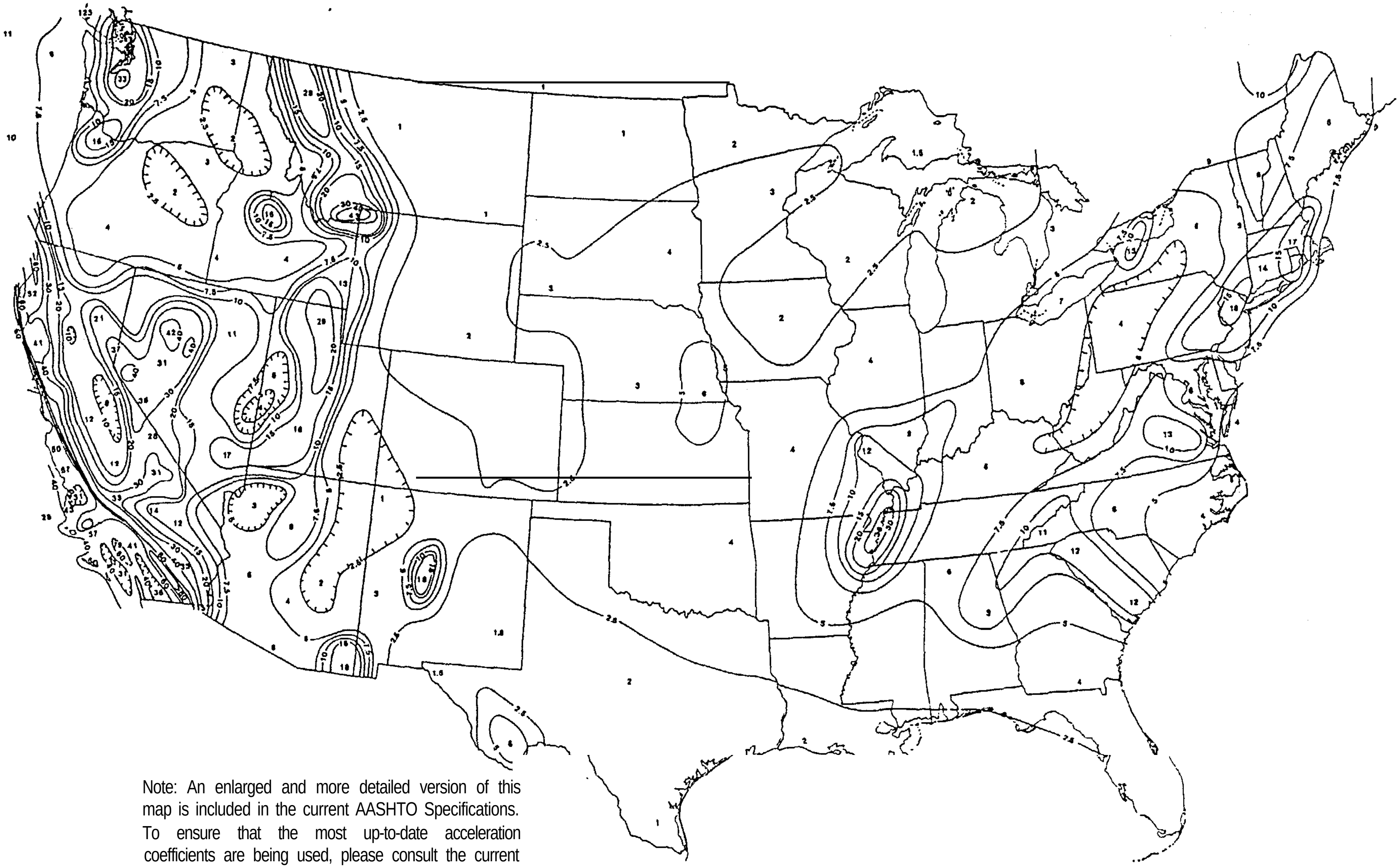
This manual is intended to be used in conjunction with the seismic design requirements for new highway bridges contained in Division I-A of the American Association of State Highway and Transportation Officials *Standard Specifications for Highway Bridges* (15th Edition, 1992, including Interim Specifications for 1993, 1994, and 1995), hereafter referred to as the AASHTO Specifications.[1] The AASHTO Specifications 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 AASHTO Specifications. Contour maps of Acceleration Coefficients from the current edition of these specifications are shown in figures 1 and 2. These maps were originally prepared by the U.S. Geological Survey and are subject to review. The most up to date version should be used as published in the current AASHTO Specifications and Interims.

The AASHTO Specifications also consider the importance of the structure in societal/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 Classifications are used along with the Acceleration Coefficient to assign bridges to one of four Seismic Performance Categories (SPC), A through D. The complexity of analysis and design requirements vary according to the Seismic Performance Category. It should be noted that, for the purpose of this manual, the Seismic Performance Categories have been modified to reflect recent changes in philosophy concerning the degree to which bridge importance should influence the selection of design and retrofit measures (see section 1.5).

The AASHTO Specifications for new bridges are based on 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 are intended to 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 AASHTO Specifications also 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 seat 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 and displacement of abutments and columns resulting from spatial variation in the ground motion.

This approach to the determination of seismic forces and displacements has been adapted in this manual to the special needs of seismic retrofitting.



Note: An enlarged and more detailed version of this map is included in the current AASHTO Specifications. To ensure that the most up-to-date acceleration coefficients are being used, please consult the current AASHTO Specifications.

Figure 1. Acceleration coefficients – continental United States (expressed as percent of gravity – adapted from 1988 NEHRP provisions).

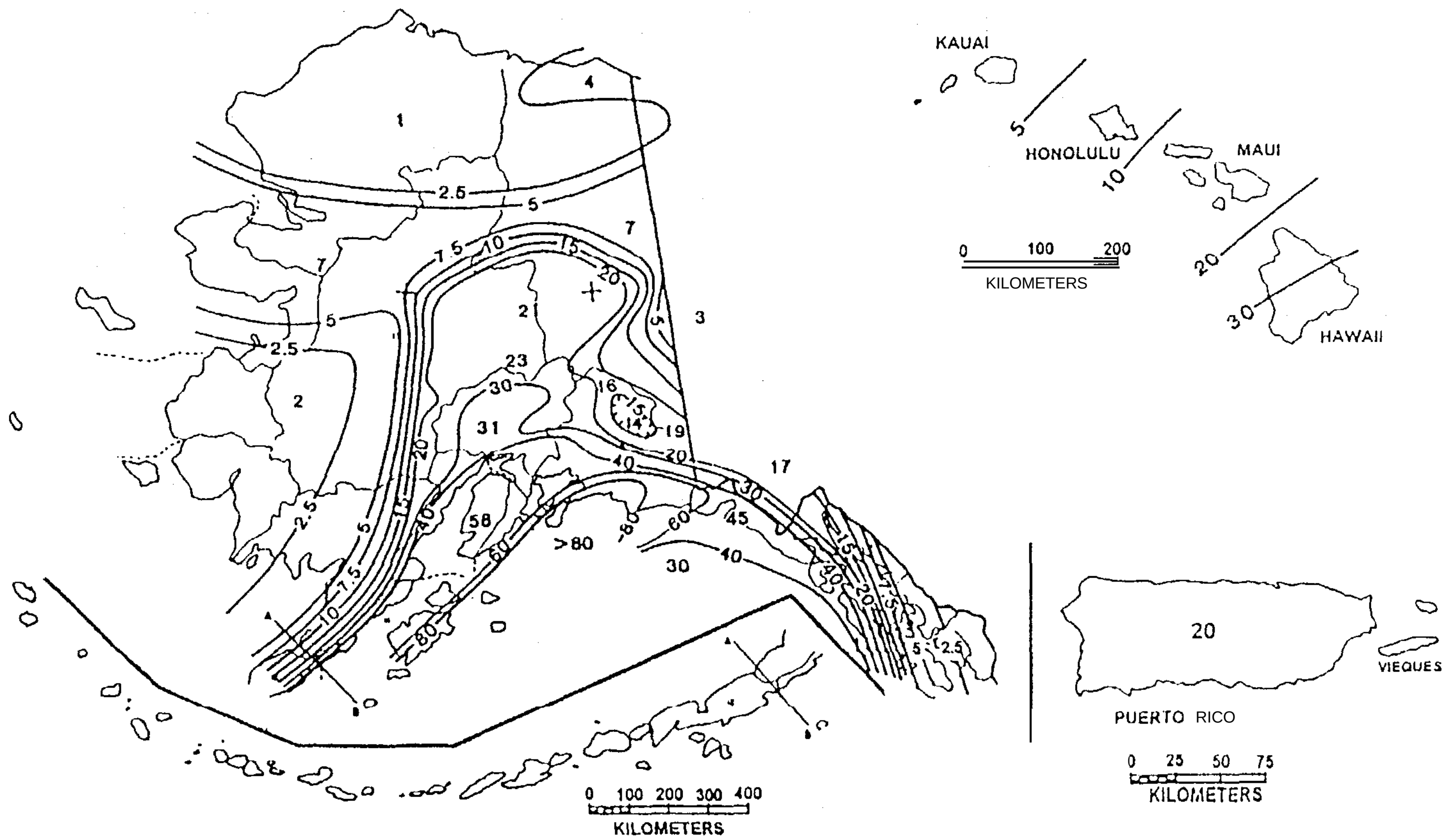


Figure 2. Acceleration coefficients – Alaska, Hawaii, and Puerto Rico (expressed as percent of gravity – adapted from 1988 NEHRP provisions).

This manual is based on an earlier set of guidelines and design manuals published by the Federal Highway Administration in 1983 and 1987. These earlier works are titled *Seismic Retrofitting Guidelines for Highway Bridges* and *Seismic Design and Retrofit Manual for Highway Bridges*.[3,11] The present report essentially updates this earlier work by modifying or adding knowledge based on experience gained and research completed in the intervening 10 years. Much of the original work remains valid today and has been preserved in this revised edition.

Nevertheless, this manual is only intended as an interim revision to the 1983 and 1987 documents. Considerable activity in research and field implementation is currently under way in California, funded by the California Department of Transportation (Caltrans). This activity is rapidly expanding the state-of-the-art to the point where "accepted practice" is difficult to identify at this time. Furthermore, the Federal Highway Administration is funding a multi-year research program in the seismic vulnerability of existing bridges in the Eastern and Central United States through the National Center for Earthquake Engineering Research. As a consequence, the retrofit guidelines contained in this manual are expected to be revised again within the next 5 years. In the meantime, the material presented herein should be a useful update to the earlier guidelines even though the state-of-the-art is still evolving and subject to change.

1.4 APPLICABILITY

This manual is intended for use on highway bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 150 m (500 ft). Suspension bridges, cable-stayed bridges, arches, long-span trusses, 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. This is particularly true for truss spans. Although specifically developed for highway bridges, this manual may also have applicability to other types of bridges.

Minimum requirements for evaluation and upgrading vary based on the Seismic Performance Category (SPC) of a particular bridge (section 1.5).

Bridges in SPC A do not have to be considered for seismic retrofitting. Bridges in SPC B need only be screened, evaluated, and strengthened based on the vulnerability of their bearings, joint restrainers, and support widths. However, a comprehensive program of retrofitting should be established for all bridges classified in Seismic Performance Categories C and D. Screening, evaluation, and retrofitting will include all major components subject to failure during a strong earthquake (bearings, substructures, and foundations). The effects of soil failure, such as liquefaction, are also considered for bridges in categories C and D, and for certain bridges in category B.

1.5 BRIDGE CLASSIFICATION

Before seismic retrofitting can be undertaken for a group of bridges, they may first be classified according to their Seismic Performance Category (SPC). As noted in section 1.3, the SPC is determined by a combination of seismic hazard and structure importance.

Seismic hazard is reflected in the Acceleration Coefficient (A) values that are assigned to all locations covered by the AASHTO Specifications. When multiplied by the acceleration due to gravity (g), the product (A · g) represents the likely peak horizontal ground acceleration that will occur due to an earthquake sometime within a 475-year period. More rigorously, this acceleration has a 10 percent probability of being exceeded within a 50-year timeframe.(1)

Bridge importance is not so readily quantified. Two Importance Classifications are specified: essential and standard. "Essential" bridges are those which may continue to function after an earthquake or which cross routes that may continue to operate immediately following an earthquake. All other bridges are classified as "standard," The determination of the Importance Classification of a bridge is necessarily subjective and consideration should be given to societal/survival and security/defense requirements.

The societal/survival evaluation addresses a number of socioeconomic needs and includes, for example, the need for access for emergency relief and recovery operations immediately following an earthquake.

Security/defense requirements may be evaluated using 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 military installations, industries, and resources not covered by the Federal-aid primary routes.

An "essential" bridge is therefore one that satisfies one or more of the following conditions:

- A bridge that is required to provide secondary life safety; e.g., a bridge that provides access to local emergency services such as hospitals. This category also includes those bridges that cross routes which provide secondary life safety, and bridges that carry lifelines such as electric power and water supply pipelines.
- A bridge whose loss would create a major economic impact; e.g., a bridge that serves as a major link in a transportation system.
- A bridge that is formally defined by a local emergency plan as critical; e.g., a bridge that enables civil defense, fire departments, and public health agencies to respond immediately to disaster situations. This category also includes those bridges that cross routes which are defined as critical in a local emergency response plan and those that are located on identified evacuation routes.
- A bridge that serves as a critical link in the security/defense roadway network.

Based on the above considerations for seismic hazard and importance, four Seismic Performance Categories are defined as shown in table 1.

Table 1. Seismic performance category.

Acceleration Coefficient	Importance Classification	
	Essential	Standard
$A \leq 0.09$	B	A
	C	B
$0.09 < A \leq 0.29$	C	C
$0.29 < A$	D	C

These SPC's are assigned differently from those in the AASHTO Specifications for new design, where no allowance for structure importance is made in seismic zones with acceleration coefficients less than 0.29. In view of the high cost of retrofitting, it is important to be able to distinguish between "essential" and "standard" structures; this is especially so in low-to-moderate seismic zones. Such a distinction also enables a more rational allowance to be made for the nature of the seismic hazard in the Central and Eastern United States where the maximum credible earthquake is expected to be significantly larger than the "design" earthquake (475-year event). This implies that if an essential bridge in the East is to remain fully operational following a large earthquake, it will need to be retrofitted to a standard higher than that required by the current specification for new construction. This observation is reflected in the assignment of SPC's for essential bridges in table 1.

1.6 THE RETROFI'ITING 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; others include bridge closure, bridge replacement, or acceptance of the risk of seismic damage. Bridge closure or replacement is usually not justified by seismic deficiency alone and will generally only be considered when other deficiencies exist. Therefore, for all practical purposes, a choice may 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 judgment. It is therefore helpful to divide the process into three major stages. These are:

- Preliminary screening.
- Detailed evaluation.
- Design of retrofit measures.

Each of these stages is outlined below and described in further detail in subsequent chapters. Figure 3 is a flow chart which illustrates the retrofit process for each SPC.

1.6.1 PRELIMINARY SCREENING

Preliminary screening of an inventory of bridges is recommended to identify those bridges which are seismically deficient and those in the greatest need of retrofitting. This is particularly useful when a comprehensive retrofitting program is to be implemented.

This manual describes a method for developing a Seismic Rating System which may be used to prioritize bridges on a highway system according to their need for seismic hazard reduction. Factors considered in the seismic rating process include structural vulnerabilities, seismic and geotechnical hazards, and bridge importance or criticality. In this way, 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 factors as well as engineering issues, highly-rated bridges may not necessarily be retrofitted. On the other hand, bridges with a lower rating may need to be retrofitted immediately.

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. Two examples will serve to illustrate the influence that this consideration may have on a decision to retrofit a 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-rated bridges B and C, which are vulnerable to seismic loading, but to a lesser degree than bridge A. This situation is shown in figure 4. Assume that no convenient detour to this route exists and that each bridge can be economically retrofitted. Because retrofitting of the higher-rated 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 rated, should also be considered for retrofitting.

The opposite effect could occur if bridge B had a high seismic rating but could not be economically retrofitted. Because bridge B is in series with bridges A and C,

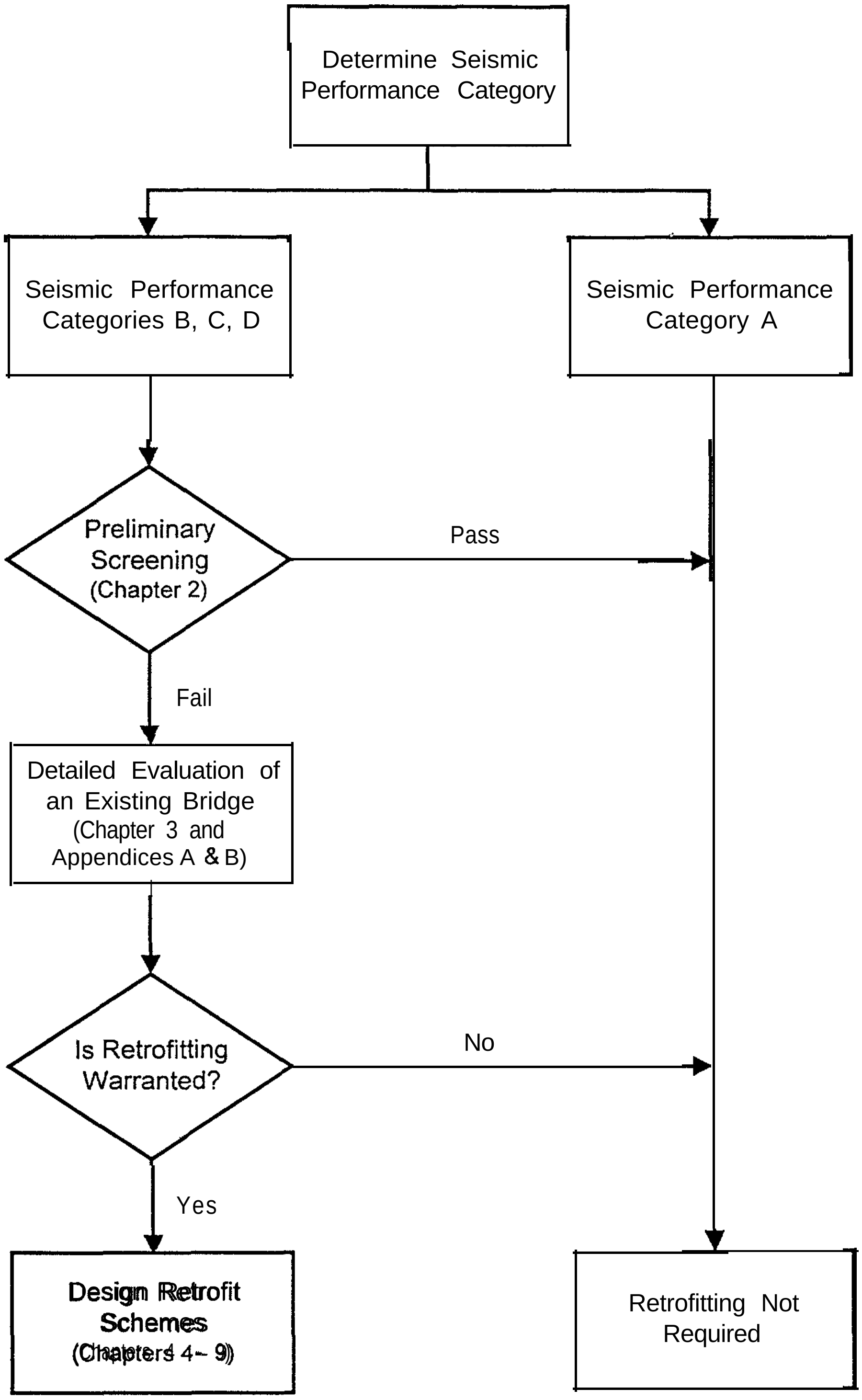


Figure 3. Seismic retrofitting process.

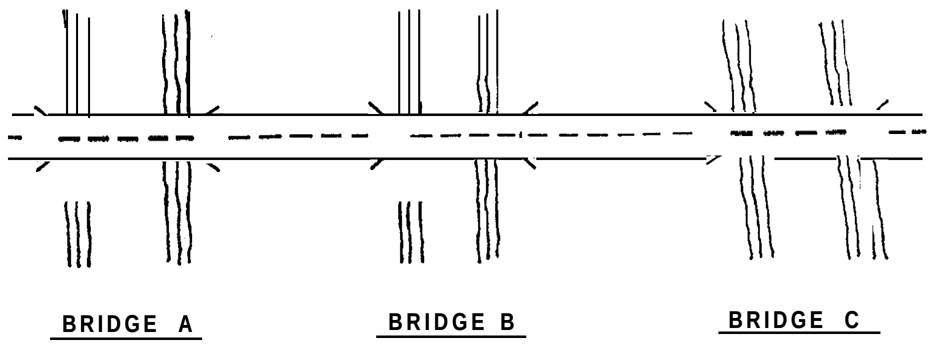


Figure 4. Bridges in series.

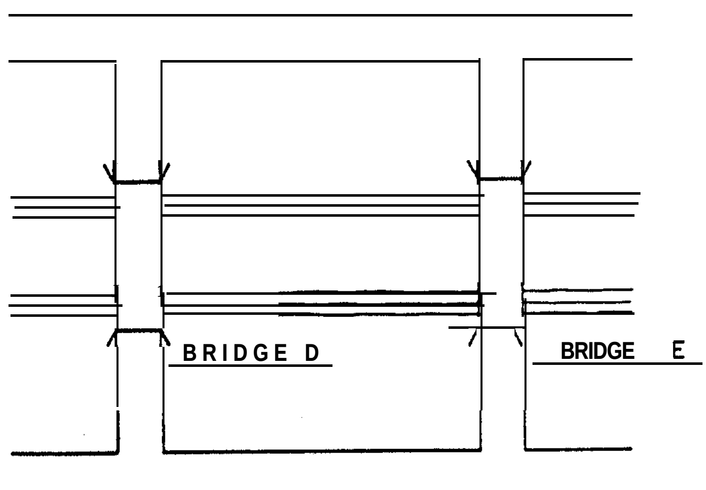


Figure 5. Bridges in parallel.

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 5. If bridge D is rated at a lower priority than bridge E, it is possible that bridge D could be more economical to retrofit if less strengthening 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 5 years of service life remaining. An unusually high seismic vulnerability may, however, be a justification to accelerate closure or replacement of such a bridge.

A bridge in poor physical condition that is already scheduled for structural or functional 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 judgment 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 may be kept in mind.

1.6.2 DETAILED EVALUATION

Two alternative methods for the detailed evaluation of existing bridges are currently available. One is based on a quantitative assessment of the “capacities” and “demands” of individual components of a bridge structure. The other evaluates the lateral strength of the bridge as a new structure.

The first method was proposed in the 1983 FHWA Retrofit Guide and has been used in a modified form by Caltrans since the early 1980’s. In this method, the results from an elastic spectral analysis are used to calculate the force and displacement “demands” which are then compared with the “capacities” of each of the components to resist these forces and displacements. For columns, ultimate capacities are modified to reflect the ability of a column to resist post-elastic deformations. Capacity/Demand (C/D) 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 1.0 indicates that component failure may occur during the design earthquake and retrofitting may be appropriate. In many respects, this evaluation method is similar to rating a bridge for live load.

An overall assessment of the consequences of local component failure is 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 should be assessed by performing a detailed re-evaluation of the retrofitted bridge. This is because strengthening one component may result in an even less-desirable failure mode elsewhere in the structure.

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 potential 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 emergency traffic, which is 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 another area in which engineering judgment is required.

Once it has been determined to consider retrofitting, acceptable methods may be selected from those suggested in chapters 4 through 9 of this manual. If the seismic response of the structure is affected, then a reanalysis should be performed and new set of component C/D ratios calculated. The new C/D ratios will reflect a change in the size of the earthquake that will cause serious damage. A decision to use any 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, in large part, on judgment.

1.6.3 DESIGN OF RETROFIT MEASURES

This manual describes retrofit measures for the types of bridge components which have performed poorly during past earthquakes. Detailed design of these measures may be performed using the guidelines contained in this manual in conjunction with the AASHTO Specifications. If possible, components which are selected for retrofitting should be strengthened to conform to the specifications for new construction, even though the structure may otherwise be seismically deficient.

1.7 THE ECONOMICS 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:

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

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

Loss, = probable loss before retrofitting
Loss, = probable loss after retrofitting

In a bridge structure, unacceptable damage is a function of the importance of the bridge. Since bridge damage or failure will result in losses, the probable losses are directly 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. This manual describes the calculation of C/D ratios which can be used to determine the size of a damaging earthquake. By determining the probability of occurrence of this earthquake, the benefit-to-cost ratio of retrofitting may then 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 can be calculated, threshold values at which retrofitting is recommended should be determined by engineering judgment and experience with retrofitting.

Instead, current practice uses a subjective assessment of benefits and costs based on past experience and engineering judgment. For example, Caltrans began its retrofitting program by installing bearing and expansion joint restrainers since these devices were 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 one of the most feasible measures of bridge retrofitting. Most of these devices are relatively simple to install and cost only a very small percentage of the replacement value of the structure. Retrofitting measures for other components such as columns, footings, and abutments have also been developed, but field experience is limited at this time. Construction procedures for retrofitting these components are more involved and are 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 relative 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 workers becoming more familiar with construction techniques associated with retrofitting, which 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 and work-zone safety can add significantly to construction costs. It is important to consider retrofit details and construction practices that will minimize these costs.

1.8 MANUAL OUTLINE

Chapters 2 through 9 contain detailed information about each of the major steps in the retrofit process. Chapter 2 covers the preliminary screening of bridges. Chapter 3 describes two alternative procedures for the detailed evaluation of existing bridges. These methods include a quantitative evaluation of the C/D ratios for individual bridge components, and an alternative method based on assessment of a structure's lateral strength. Both methods are further described in appendices A and B, respectively.

The procedures for evaluating bridges for retrofitting also include the identification and assessment of retrofit measures. Several potential retrofitting measures and retrofit design requirements are discussed in chapters 4 through 9. Example problems are included in appendices C, D, and E, which will help illustrate the use of the manual in planning the retrofitting of a typical highway bridge.

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. It is recommended that a preliminary screening process be established for this purpose for all bridges classified as Seismic Performance Categories B, C, and D. The flow chart shown in figure 6 illustrates a preliminary screening procedure as it might apply to bridges in different Seismic Performance Categories.

In general, the Seismic Rating System described in this chapter may be used as a basis for selecting bridges for more detailed quantitative evaluation as described in chapter 3. The Seismic Rating System attempts to consider both the technical aspects of the problem and the administrative, economic, and/or political considerations. To do so, the system first requires the calculation of bridge seismic rank based on engineering factors, which is then followed by the assignment of a priority index based on rank, socioeconomic (importance) factors, and other issues. It is noted that the cost of retrofitting is not directly included in this preliminary screening procedure, but it is recognized that cost will determine the final outcome. It is expected that economic factors will play their part in the assignment of the priority index.

2.2 SEISMIC INVENTORY OF BRIDGES

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

- The structural characteristics needed to determine the vulnerability rating as described in section 2.3.1.1.
- The seismicity and soil conditions at the bridge site needed to determine the seismic hazard rating as described in section 2.3.1.2.

This information may be obtained from the bridge owner's records, the Federal Highway Administration's National Bridge Inventory, "as-built" plans, maintenance records, the regional disaster plan, on-site bridge inspection records, and other sources. The form shown in figure 7 is provided as an example for collecting and recording some of this information. The completed form should be filed with the existing bridge records.

Structure "importance" is a key factor in rating bridges for seismic retrofit and since establishing importance was required when assigning an SPC, such information will also need to be carried forward into the Seismic Rating System.

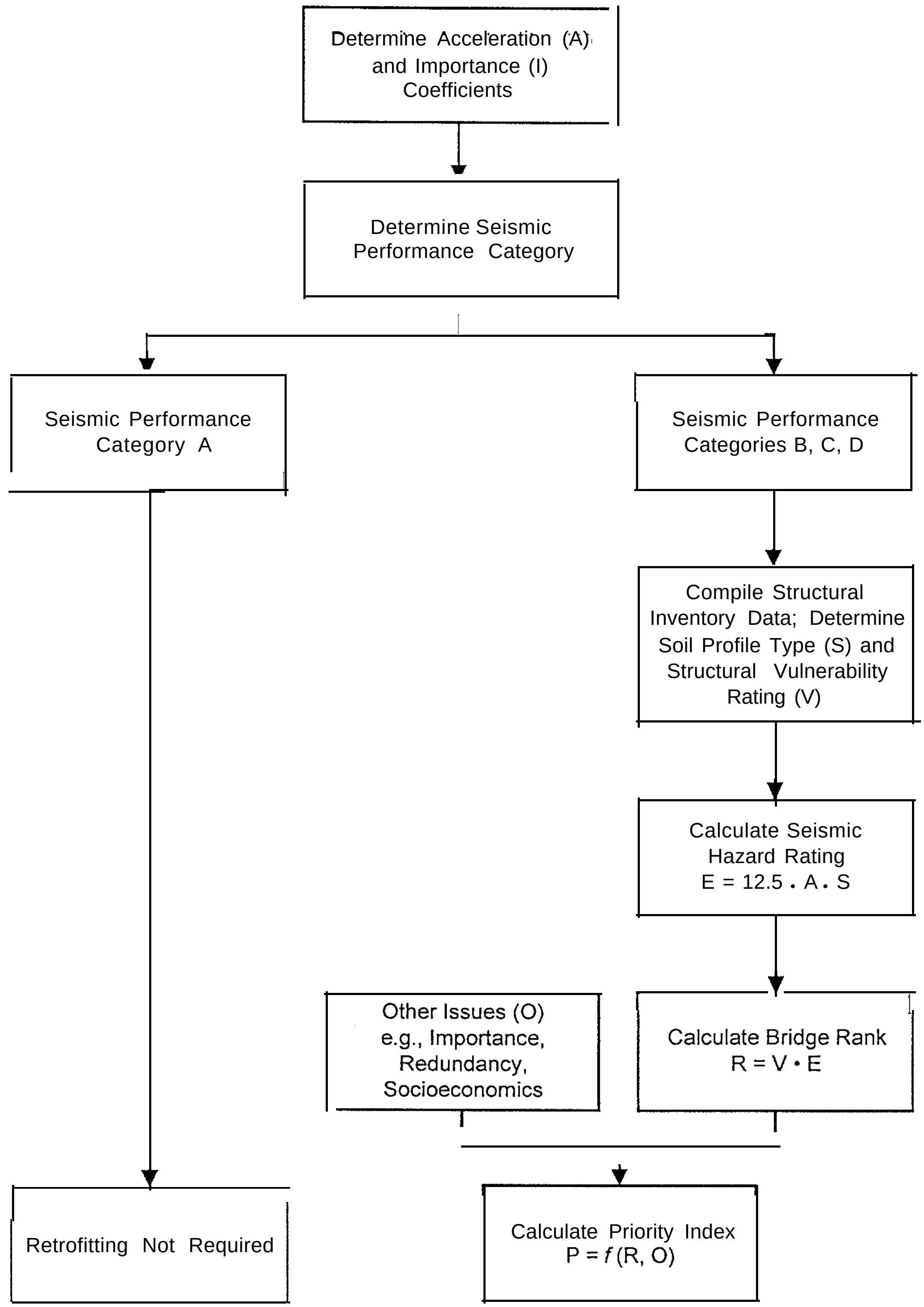


Figure 6. Preliminary screening process.

BRIDGE SEISMIC INVENTORY DATA FORM

GENERAL:

Bridge Name__________________ BIN Number__________________

Location__________________

ADT__________ Detour Length__________ Essential Bridge: Yes- No__

Alignment: Straight- Skewed- Curved- Remarks__________________

Length__________ Feature carried__________________

Width__________ Feature crossed__________________

Year Built________

Seismically Retrofitted: Yes No Description/Date__________________

Geometry: Regular__ Irregular__ Remarks__________________

SITE:

Peak Acceleration__________________

Soil Profile Type: I___ II- III - IV -

SEISMIC PERFORMANCE CATEGORY: A,, B- C - D -

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 (Long)__________________

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 Guidelines: Yes__ No___

Foundation Type__________________

ABUTMENTS:

Type__________________

Height______________

Foundation Type__________________ Location: Cut Fill

Wingwalls: Continuous Discontinuous Length

Approach Slabs: Yes No_____ Length______________

SEISMIC RANK:

Vulnerability Ratings

Connections, Bearings and Seatwidths . . . . (V,)

Other Components: CVR,______ AVR_______, LVR________ . . (V_2)_____

Overall Rating . . . . . . , ... (V)

Seismic Hazard Rating: (E)______________

Seismic Rank: (R = V x E) ______________

Figure 7. Sample bridge seismic inventory form.

2.3 SEISMIC RATING SYSTEM

To calculate the seismic rating of a bridge in order to develop retrofit priorities, consideration should be given to structural vulnerability, seismic and geotechnical hazards, and the socioeconomic factors affecting importance. This is accomplished first, by making independent ratings of each bridge in the areas of vulnerability and seismic hazard, and second, by considering importance (societal and economic issues) and other issues (redundancy and nonseismic structural issues) to obtain a final, ordered determination of bridge retrofit priorities.[33, 34]

The rating system is therefore composed of two parts: the first is quantitative, the second is qualitative. The quantitative part produces a seismic rating (called the bridge rank) based on structural vulnerability and seismic hazard. The qualitative part modifies the rank in a subjective way that takes into account such factors as importance, network redundancy, nonseismic deficiencies, remaining useful life, and other similar issues for inclusion in an overall priority index. Engineering and societal judgment are thus the key to the second stage of the screening process. This leads to a priority index which is a function of rank, importance, and other issues; i.e.,

$$P = f\,(\text{R, importance, nonseismic, and other issues...}) \tag{2-1}$$

where P = priority index
R = rank based on structural vulnerability and seismicity.

In summary, *bridge rank* is based on structural vulnerability and seismic hazard, whereas *retrofit priority* is based on bridge rank, importance, nonseismic deficiencies, and other factors such as network redundancy.

2.3.1 CALCULATION OF BRIDGE RANK

As noted above, the bridge rank, R, is based on a structural vulnerability rating, V, and a seismic hazard rating, E. Each rating lies in the range 0 to 10 and the rank is found by multiplying these two ratings together; i.e.,

$$R = V \cdot E \tag{2-2}$$

It follows that R can range from 0 to 100, and the higher the score, the greater the need for the bridge to be retrofitted (ignoring, at this time, all other factors). Recommendations for assigning values for V and E are described below.

2.3.1.1 Vulnerability Rating (V)

Although the performance of a bridge is based on the interaction of all its components, it has been observed in past earthquakes that certain bridge components of four general types are more vulnerable to damage than others. These are: (a) the connections, bearings, and seats; (b) columns and foundations; (c) abutments; and (d) soils. Of these, bridge bearings seem to be the most economical to retrofit. For this reason, the vulnerability rating to be used in the Seismic Rating System is determined by examining the connections, bearings, and seat details separately from the remainder of the structure. A separate vulnerability rating, V_1, is calculated for these components. The vulnerability rating for the remainder of the structure, V_2, is determined from the sum of the vulnerability ratings for each of the other components which are susceptible to failure. The overall rating for the bridge is then given by the maximum of V_1 and V_2. A flow chart summarizing the process is provided in figure 8.

The determination of these vulnerability ratings requires considerable engineering judgment. In order to assist in this process, a methodology for determining these ratings is given in sections 2.3.1.1(a) and 2.3.1.1(b).

Vulnerability ratings may assume any value between 0 and 10. A rating of 0 means a very low vulnerability to unacceptable seismic damage, a value of 5 indicates a moderate vulnerability to collapse or a high vulnerability to loss of access, and a value of 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.

For bridges classified as SPC B, the vulnerability ratings for bearings, joint restrainers, and support lengths need to be calculated along with a rating for liquefaction effects for bridges on certain sites. Experience has shown that most connection, bearing, and seat deficiencies can be economically corrected.

For bridges classified as SPC C or D, vulnerability ratings are also generated for the columns, abutments, and foundations. Experience with retrofitting these components is much more limited than for bearings. They are generally more difficult to retrofit and doing so may not be as cost-effective.

A comparison of the above two vulnerability ratings, V_1 and V_2, 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.

2.3.1.1(a) Vulnerability Rating for Connections, Bearings, and Seatwidths, V_1

Bearings are used to transfer loads from the superstructure to the substructure and between superstructure segments at in-span hinge seats. For the purpose of this discussion, bearings are considered to include restraints provided at these locations, including shear keys, restrainer bars, and the like. Bearings may be “fixed” bearings, which do not provide for translational movement, or expansion bearings, which do permit such movements, as shown in figure 9(a). A bearing may provide for translation in one orthogonal direction, but not in the other.

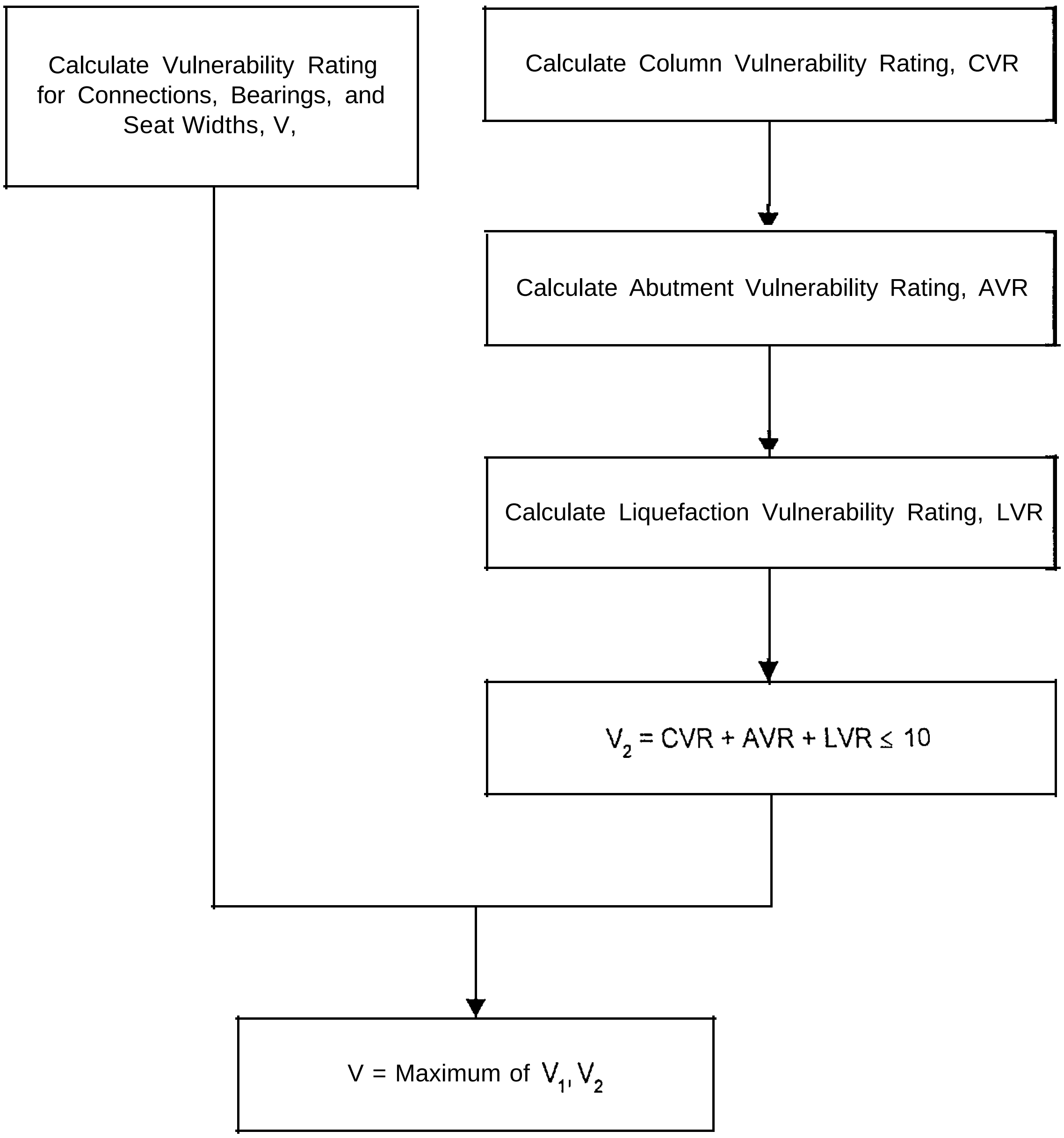


Figure 8. Flow chart for calculation of bridge vulnerability rating (V).

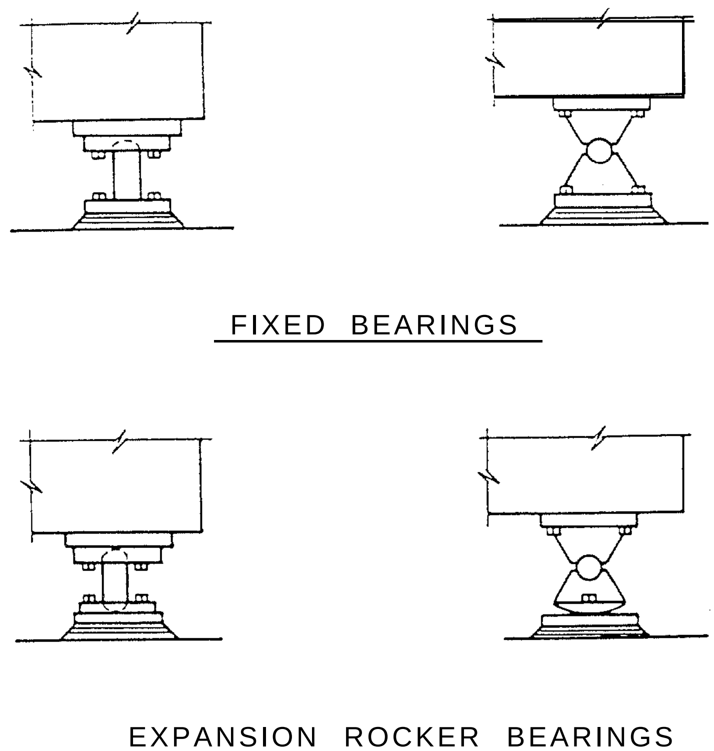


Figure 9(a). Seismically vulnerable bearings.

There are basically five types of bearings used in bridge construction. These are:

(1) The rocker bearing, which is generally constructed of steel and permits translation and rotational movement. It is considered to be the most seismically vulnerable of all bridge bearings because it usually has a large vertical dimension, is difficult to restrain, and can become unstable after a limited movement and overturn.

(2) The roller bearing, which is also usually constructed of steel. It is stable during an earthquake, except that it can become misaligned and horizontally displaced.

(3) The elastomeric bearing pad, which has become popular in recent years. It is constructed of a natural or synthetic elastomer and may be internally reinforced with steel shims. It relies on the distortion of the elastomer to provide for movement. This bearing is generally stable during an earthquake, although it has been known to "walk out" under severe shaking due to inadequate fastening.

(4) The sliding bearing, in which one surface slides over another and which may consist of almost any material from an asbestos sheet between two concrete surfaces to PTFE (teflon and similar materials) and stainless-steel plates.

(5) High-load, multi-rotational bearings such as pot, disk, and spherical bearings. These engineered bearings usually have adequate strength for earthquake loads, but have failed in their connections in past earthquakes.

Transverse restraint of the superstructure is almost always provided at the bearings. Common types of restraint are shear keys, keeper bars, or anchor bolts. Restraints 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 may be subjected to very high forces. In vulnerable structures, collapse may occur 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 calculated. The AASHTO Specifications require a minimum support length at all bearings in newly constructed bridges.[2] Because it is very difficult to predict relative movement, the minimum support lengths, N, as required by the AASHTO Specifications, may be used here as the basis for checking the adequacy of longitudinal support lengths. See also section A.3.

Support skew has a major effect on the performance of bridge bearings. In this manual, skew is defined as the angle between the support centerline and a line normal to the bridge centerline. Rocker bearings have been the most vulnerable in past earthquakes. At highly skewed supports, these bearings may overturn during even moderate seismic shaking. In such cases, it is necessary to consider the potential for collapse of the span, which will depend to a large extent on the geometry of the bearing seat. Settlement and vertical misalignment of a span due to an overturned bearing may be a minor problem, resulting in only a temporary loss of access which can be restored, in many cases, by backfilling with asphalt or other similar material. The potential for total loss of support should be the primary criteria when rating the vulnerability of the bearings.

A suggested step-by-step method for determining the vulnerability rating for connections, bearings, and seatwidths (V_1) is detailed in the flow chart of figure 9(b) and is as follows:

<u>Step 1</u>: Determine if the bridge has satisfactory bearing details. These bridges include:

a. Continuous structures with integral abutments.

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

(1) Either (a) the skew is less than 20° (0.35 rad), or
(b) the skew is greater than 20” (0.35 rad) but less than 40° (0.70 rad) and the length-to-width ratio of the bridge deck is greater than 1.5.

(2) Rocker bearings are not used.

(3) The bearing seat under the abutment end-diaphragm is continuous in the transverse direction and the bridge has more than three girders.

(4) The support length is equal to, or greater than, the minimum required support length (section A.3).

If the bearing details are determined to be satisfactory, a vulnerability rating, V_1, of 0 may be assigned and the remaining steps for bearings omitted.

<u>Step 2</u> Determine the vulnerability to structure collapse or loss of bridge access due to transverse movement, V_T.

Before significant transverse movement can occur, the transverse restraint must fail. In the absence of calculations showing otherwise, assume that nominal bearing keeper bars or anchor bolts will fail in bridges in SPC C and D. Also assume that nominally reinforced, nonductile concrete shear keys will fail in bridges in SPC D.

When the transverse restraint is subject to failure, girders are vulnerable to loss of support 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 supported near the edge of a bearing seat regardless of whether the bearings are on individual pedestals or not.

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

Steel rocker bearings have been known to overturn transversely, resulting in a permanent superstructure displacement. All bridges in SPC D are vulnerable to this type of failure. Bridges in SPC C are vulnerable only when the support skew is greater than 40° (0.70 rad). When bearings are vulnerable to a toppling failure but structure collapse is unlikely, the vulnerability rating, V_T, should be 5.

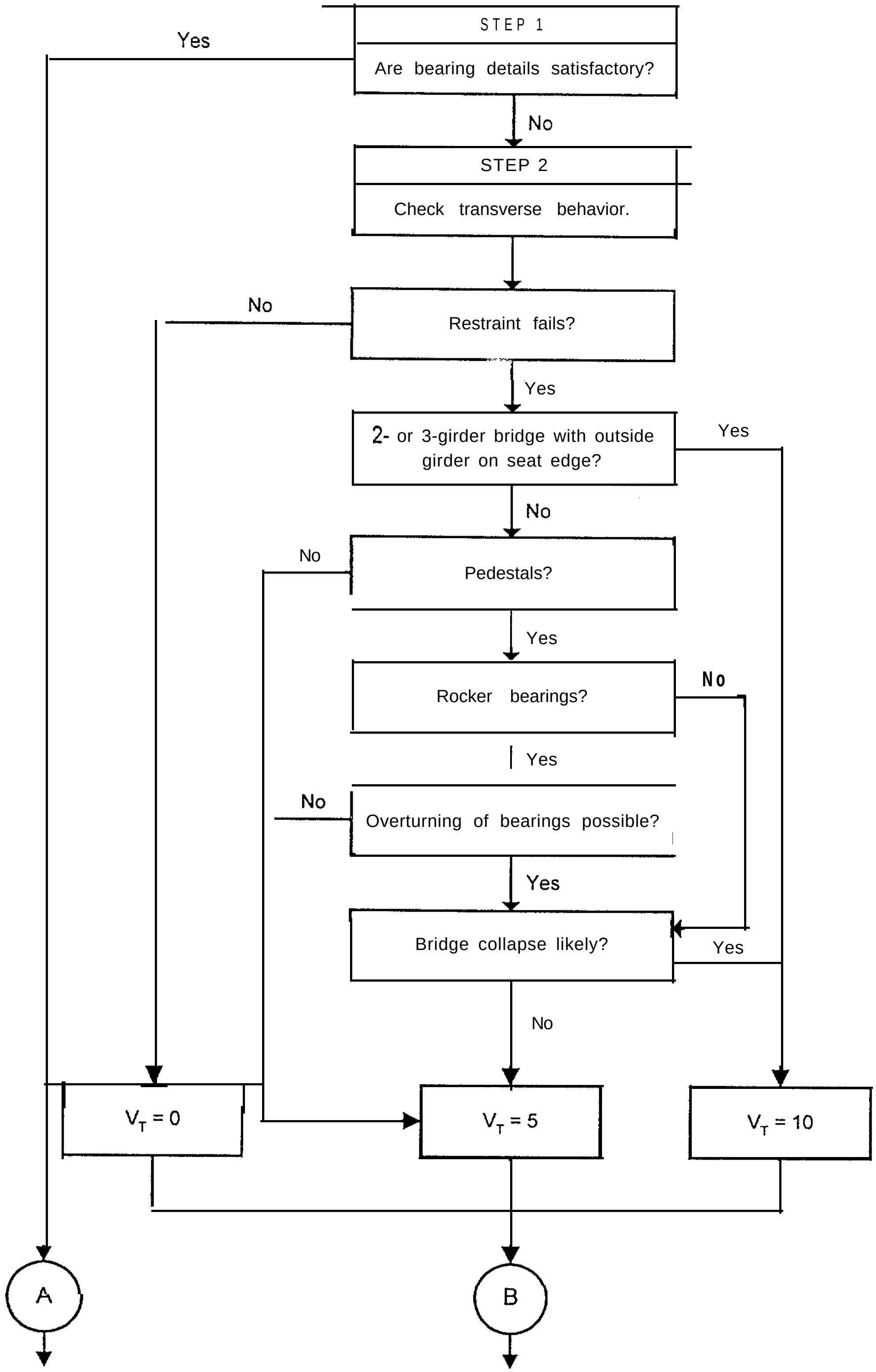


Figure 9(b). Flow chart for calculation of vulnerability rating for connections, bearings, and seat widths (V,).

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

If the longitudinal support length measured in a direction perpendicular to the support is less than one times, but greater than one-half times, the required longitudinal support length, the vulnerability rating, V_L, shall be assigned a value of 5. If, in addition, rocker bearings are present and are vulnerable to overturning, a value of 10 for V_L should be used. If the longitudinal support length is less than one-half of the required longitudinal support length, then a vulnerability rating, V_L, of 10 should be assigned regardless of bearing type.

Step 4: Calculate vulnerability rating for connections, V,, from values V_T and V_L; i.e., V_1 = maximum value of V_T and V_L.

2.3.1.1(b) Vulnerability Rating for Columns, Abutments, and Liquefaction Potential, V_2

The vulnerability rating for the other components in the bridge that are susceptible to failure, V_2, is calculated from the individual component ratings as follows:

$$V_2 = CVR + AVR + LVR \leq 10$$

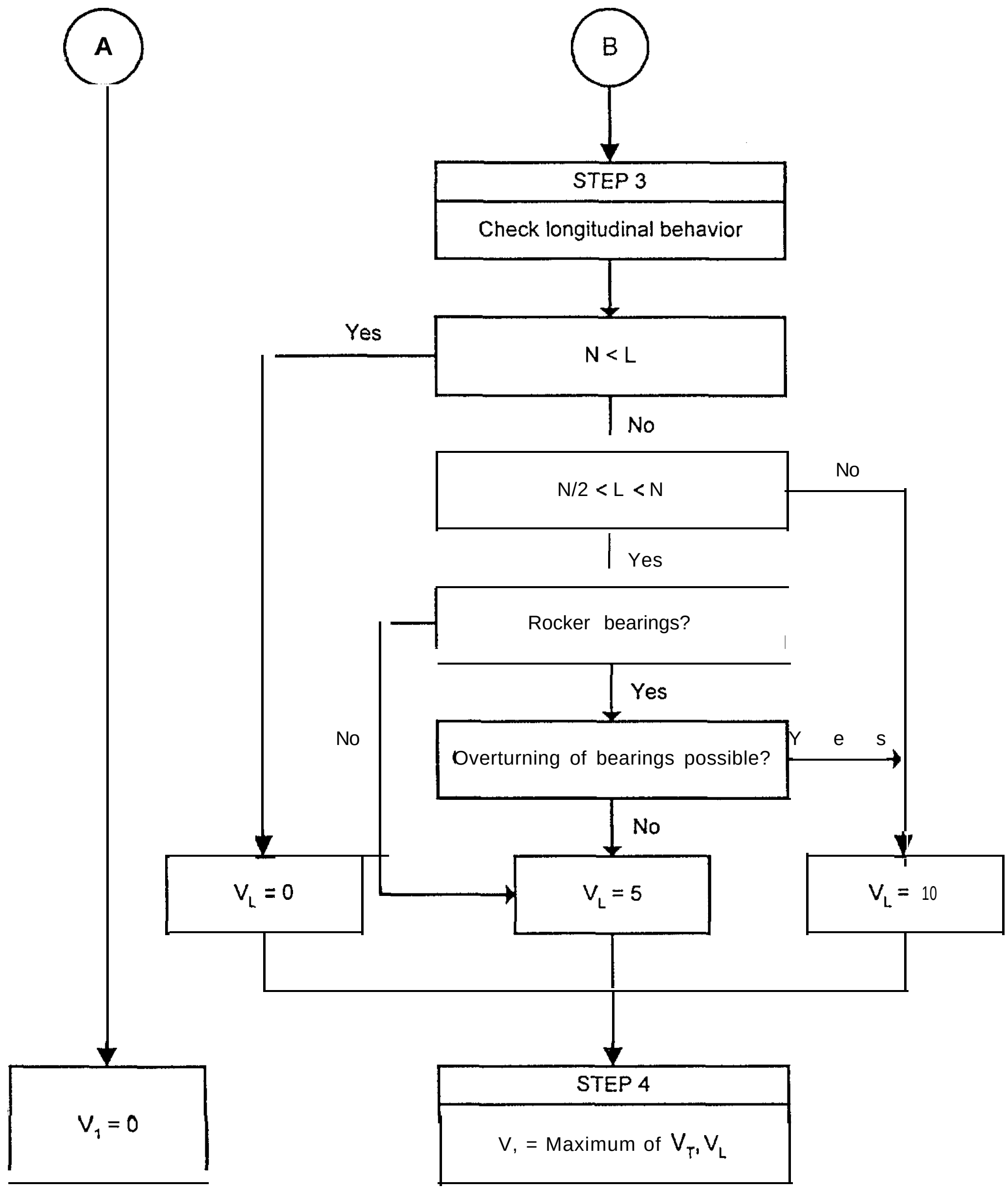
where

CVR = column vulnerability rating
AVR = abutment vulnerability rating
LVR = liquefaction vulnerability rating

Suggested methods for calculating of each of these component ratings are given in the following sections.

A. Column Vulnerability Rating, CVR

Columns have failed in past earthquakes due to lack of adequate transverse reinforcement and poor structural details. Excessive ductility demands have resulted in degradation of column strength in shear and flexure. In several collapses in past earthquakes, columns have failed in shear, resulting in column disintegration and substantial vertical settlement. Column failure may also occur due to pullout of the longitudinal reinforcing steel, mainly at the footings. Fortunately, most bridge column failures occur during earthquakes which generate high ground accelerations of relatively long duration. However, this is not always the case, as illustrated by the collapse of the Cypress Street Viaduct in Oakland, California, during the 1989 Loma Prieta earthquake. [This short-duration earthquake generated peak rock accelerations of about 0.1g close to the bridge. Soft soils amplified this figure to approximately 0.25g, which is a relatively moderate value. Nevertheless, the viaduct collapsed due to poor detailing that permitted shear failures to occur in the connections between the upper and lower decks.]



Note: L = actual seat width and N = seat width required by AASHTO, Division I-A

Figure 9(b) continued. Flow chart for calculation of vulnerability rating for connections, bearings, and seat widths (V,).

Table 2(a). Values for R.

Factor	R
Acceleration coefficient, $A < 0.4$	3
Skew $\leq 20^\circ$ (0.35 rad)	2
Continuous superstructure, integral abutments of equal stiffness and length-to-width ratio < 4	1
Grade 40 (or below) reinforcement	1

has since been checked against column failures in the Northridge earthquake (1994) and was found to be a reliable indicator of column damage.[35] However, the columns in this empirical data set are short-medium height and equation 2-3b may not apply to tall and/or slender columns. In these cases, special studies should be undertaken to estimate Q, R, and CVR.

Step 4b: Column vulnerability due to flexural failure at splices.

To account for flexural failure at column splices, the following CVR should be used for single-column bents supporting super-structures longer than 90 m (300 ft), or for superstructures with expansion joints where the column longitudinal reinforcement is spliced at a potential plastic hinge location:

for $A < 0.4$, CVR = 7.

for $A \geq 0.4$, CVR = 10 (only when microzoning is considered).

Step 4c: Column vulnerability due to foundation deficiencies.

The following CVR should be used for single-column bents supported on piled footings that are unreinforced for uplift, or for poorly confined foundation shafts. This step is only applicable if microzoning yields values of A greater than or equal to 0.4.

for $0.4 \leq A \leq 0.5$, CVR = 5.

for $A > 0.5$, CVR = 10.

Step 4d: Assign overall column vulnerability rating, CVR.

Set the column vulnerability rating, CVR, to the highest value calculated for CVR in steps 4a, 4b, and 4c.

The following step-by-step procedure may be used to determine the vulnerability of columns, piers, and footings.

Step 1: Assign a column vulnerability rating, CVR, of 0 to bridges classified as SPC B.

Step 2: Assign a vulnerability rating, CVR, of 0 if bearing keeper bars or anchor bolts can be relied upon to fail (section 2.3.1.1(a), step 2), eliminating the transfer of load to the columns, piers, or footings.

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

Step 4: If none of the above apply (steps 1 through 3), check the column for shear, splice details, and foundation deficiencies, and assign an appropriate value for the column vulnerability rating, i.e., CVR should be assigned the highest value calculated from the following steps:

Step 4a: Column vulnerability due to shear failure.

$$CVR = Q - R \tag{2-3a}$$

where

$$Q = 13 - 6\left(\frac{L_c}{P_s F b_{max}}\right) \tag{2-3b}$$

L_c = effective column length.

P_s = amount of main reinforcing steel expressed as a percent of the column cross-sectional area.

F = framing factor:

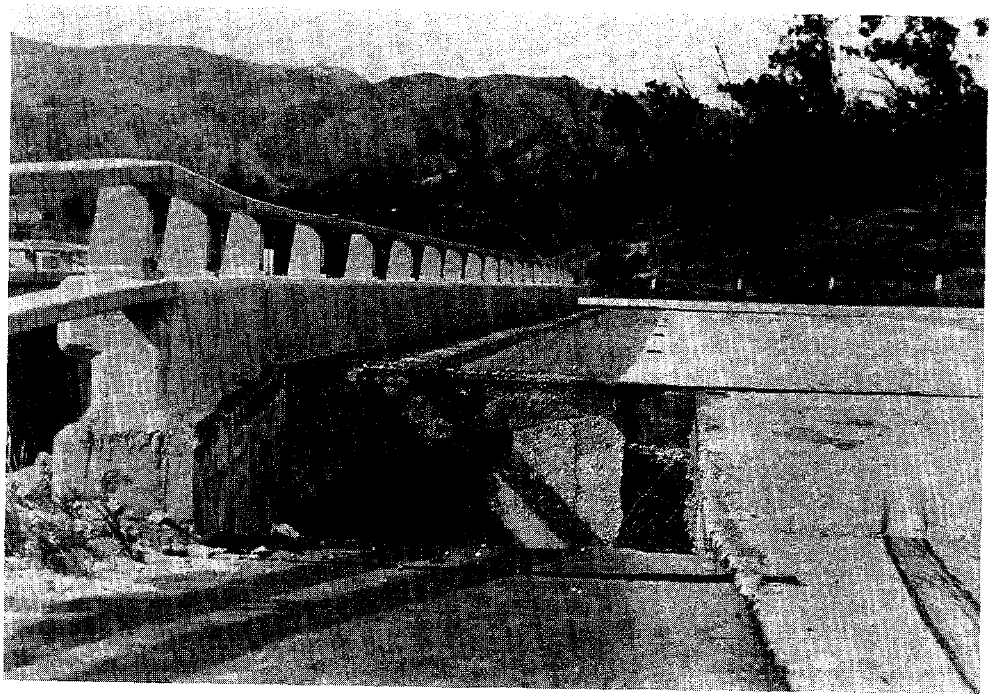
- 2 for multi-column bents fixed top and bottom.
- 1 for multi-column bents fixed at one end.
- 1.5 for box girder superstructure with a single-column bent fixed at top and bottom.
- 1.25 for superstructures other than box girders with a single-column bent fixed at top and bottom.

b_{max} = maximum transverse column dimension.

R = the number of points to be deducted from Q for factors known to reduce susceptibility to shear failure, as shown in table 2(a).

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

Note that equation 2-3b was empirically derived based on observations of column shear failure in bridges during the San Fernando earthquake in 1971. The derivation is given in appendix B of the 1983 Retrofit Guidelines.[3] This expression



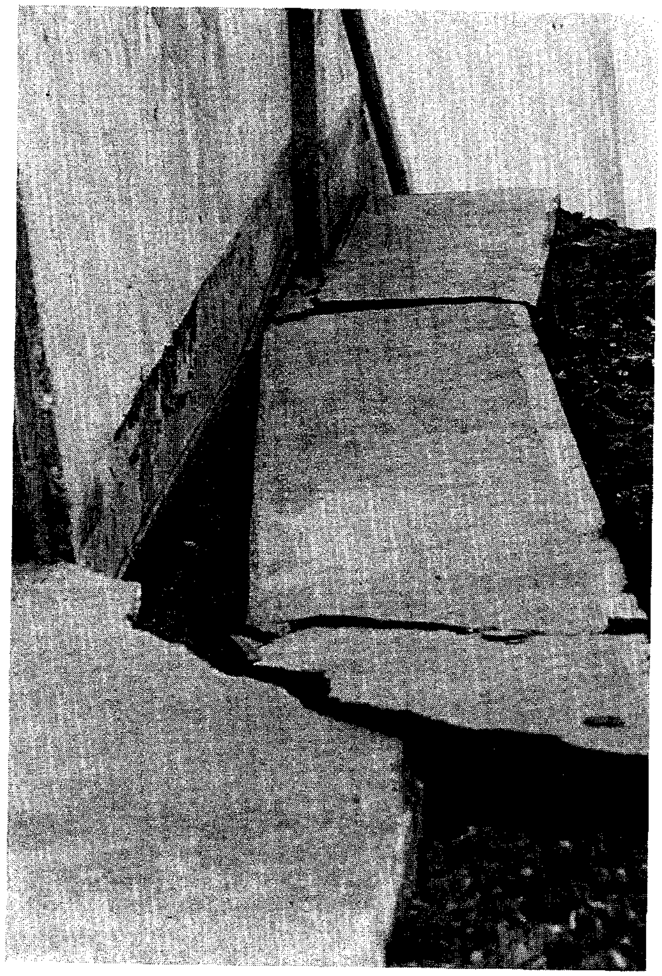


Figure 10. Abutment and approach fill settlement caused by the 1971 San Fernando earthquake.

B. Abutment Vulnerability Rating, AVR

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.

One of the major problems observed in past earthquakes has been the settlement of approach fill at the abutment as shown in figure 10. Elms reports that in past earthquakes in New Zealand and New Guinea, these settlements have been on the order of 10 to 15 percent of the fill height.(4) However, observations of damage during the San Fernando and other California earthquakes suggest far less settlement.(5) Bridges within the damage area of the San Fernando earthquake experienced average fill settlements on the order of 3 to 5 percent. This difference in observed behavior is assumed to be due to differences in abutment types (wall vs. spill-through), construction of fills, and groundwater levels.

Additional fill settlements are possible in the event of structural failures at the abutments due to excessive seismic earth pressures or seismic forces transferred from the superstructure. Certain abutment types, such as spill-through abutments and those without wing walls, may be more vulnerable to this type of damage than others. Except in unusual cases, the maximum abutment vulnerability rating, AVR, will be 5. The following step-by-step procedure for determining the vulnerability rating for abutments is based on engineering judgment and the performance of abutments in past earthquakes,

Step 1. If bridges are classified as SPC B, assign a vulnerability rating, AVR, 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. 1 percent of the fill height when $0.19 < A \leq 0.29$.

b. 2 percent of the fill height when $0.29 < A \leq 0.39$.

c. 3 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 150 mm (6 in), assign a vulnerability rating, AVR, for the abutment of 5. Otherwise assign a value of 0 for AVR.

Step 3. Bridges classified as SPC D, with cantilever earth-retaining abutments and skews greater than 40” (0.70 rad), where the distance between the seat and the bottom of the foundation footing exceeds 3 m (10 ft), should be assigned a vulnerability rating, AVR, of 5. If all of these conditions are not present, assign a value of 0 for AVR unless excessive fill settlement implies a higher value (step 2 above).

Table 2(b). Potential for liquefaction-related damage.

Soil Susceptibility to Liquefaction	Acceleration Coefficient, A				
	$A \leq 0.09$	$0.09 < A \leq 0.19$	$0.19 < A \leq 0.29$	$0.29 < A \leq 0.39$	$A > 0.39$
low	low	low	low	low	low
moderate	low	low	moderate	major	severe
high	low	moderate	major	severe	severe

Step 3: In general, bridges subjected to severe liquefaction-related damage shall be assigned a vulnerability rating, LVR, of 10. This rating may be reduced to 5 for single-span bridges with skews less than 20° (0.35 rad) or for rigid box culverts.

Step 4: Bridges subjected to major liquefaction-related damage shall be assigned a vulnerability rating, LVR, of 10. This rating may be reduced to between 5 and 9 for single-span bridges with skews less than 40° (0.70 rad), and for rigid box culverts and continuous multispan bridges with skews less than 20° (0.35 rad), provided one of the following conditions exists:

a. Reinforced concrete columns that are monolithic with the superstructure and have a CVR less than 5 and a height in excess of 8 m (25 ft).

b. Steel columns (except those constructed of non-ductile material) that are in excess of 8 m (25 ft) high.

c. Columns that are not monolithic with the superstructure, provided that gross movements of the substructure will not result in instability.

Step 5: Bridges subjected to moderate liquefaction-related damage shall be assigned a vulnerability rating, LVR, of 5. This rating should be increased to between 6 and 10 if the vulnerability rating for the bearings, V_1, is greater than or equal to 5.

Step 6: Bridges subjected to low liquefaction-related damage shall be assigned a vulnerability rating, LVR, of 0.

2.3.1.2 Seismic Hazard Rating (E)

As a measure of seismic hazard, the peak ground acceleration in rock or competent soil is used, modified by the site coefficient to allow for soil amplification effects. The seismic hazard rating is therefore defined as follows:

$$E = 12.5 \cdot A \cdot S \leq 10 \qquad (2\text{-}4)$$

C. Liquefaction Vulnerability Rating, LVR

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

a. A quantitative assessment of liquefaction susceptibility.

b. The magnitude of the acceleration coefficient.

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

The vulnerability of different types of bridge structures to liquefaction has been illustrated by failures during past earthquakes, such as the 1964 Alaskan earthquake, as reported by Ross, et al., and various Japanese earthquakes, as reported by Iwasaki, et al.[6,7] The observed damage has demonstrated that bridges with continuous superstructures and supports can withstand large translational displacements and usually remain serviceable (with minor repairs). However, bridges with discontinuous superstructures and/or non-ductile supporting members are usually severely 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 the following conditions:

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

b. Where similar soils underlie abutment fills or are present as continuous seams, which could lead to abutment slope failures.

Moderate susceptibility is associated with foundation soils that are, on average, medium dense soils; e.g., compact sands.

Low susceptibility is associated with foundation soils that are, on average, dense soils.

Step 2: Use table 2(b) to determine the potential for liquefaction-related damage where susceptible soil conditions exist.

For all sites where $A > 0.39$, engineering judgment should be applied to determine the possibility of greater damage.

where A = acceleration coefficient as given in figures 1 and 2
S = site coefficient as given in table 3.

Table 3. Site coefficients, S.

Soil Profile Type	Site Coefficient
I	1.0
II	1.2
III	1.5
IV	2.0

It will be seen that E ranges from 0.625 (A = 0.05, S = 1) to 10 (A = 0.4, S = 2).

In locations where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, or where the profile does not fit any of the four types, the site coefficient shall be based on engineering judgment. Soil Profiles are defined below:

Soil Profile Type I

A soil profile composed of rock of any description, either shale-like or crystalline in nature, or of stiff soils where the soil depth is less than 60 m (200 ft) and the soils overlying rock are stable deposits of sands, gravels, or stiff clays, shall be taken as Type I.

Soil Profile Type II

A soil profile with stiff cohesive or deep cohesionless soil where the soil depth exceeds 60 m (200 ft) and the soil overlying the rock are stable deposits of sands, gravels, or stiff clays, shall be taken as Type II.

Soil Profile Type III

A soil profile with soft to medium-stiff clays and sands, characterized by 9 m (30 ft) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils, shall be taken as Type III.

Soil Profile Type IV

A soil profile with soft clays or silts greater than 12 m (40 ft) in depth shall be taken as Type IV.

2.3.2 CALCULATION OF PRIORITY INDEX

Once a rank has been calculated for each bridge based on equation 2-2, the bridges may be listed in numerical order of decreasing rank. This order now needs to be modified to include such factors as bridge importance, network redundancy, nonseismic deficiencies, remaining useful life, and the like.

Guidance on assigning importance was given in section 1.5 and some discussion of network redundancy and nonseismic rehabilitation was provided in section 1.6 (and figures 4 and 5) under the heading "Preliminary Screening." If a bridge is part of a highly redundant highway network with alternate bridges or routes, the likelihood that these alternate facilities may also be damaged must be considered. If, for example, an overpass can be bypassed by using the on- and off-ramps, then a relatively convenient detour may be nearby, provided these access ramps remain operational. If, on the other hand, the structure in question is a critical river crossing, the nearest detour may be several miles away, but the possibility of it also being damaged may not be so great. Nevertheless, the higher priority should be given to the river crossing because of the lack of alternate routes. In general, it will not be possible to develop a single number by which to scale the seismic rank (equation 2-2) to obtain the priority index. Instead, reordering the rank by subjective means using a combination of engineering and societal judgment will be necessary. By this means, an attempt can be made to include all of the technical and societal issues that influence the prioritization of bridges for seismic retrofitting.

CHAPTER 3

DETAILED EVALUATION OF EXISTING BRIDGES

3.1 GENERAL

This chapter describes two alternative procedures for evaluating the seismic vulnerability of existing bridges. One procedure is based on a quantitative assessment of the "capacities" of, and "demands" on, individual components, where capacities include member force resistances and displacement capabilities, and demands include force effects and displacement effects. The second procedure evaluates the lateral strength of the bridge as a single structural system. Both methods are described in this chapter together with supporting material in appendices A and B, respectively. The capacity/demand ratio method was first proposed in the 1983 FHWA Retrofit Guide and used in a modified form by Caltrans since the early 1980's.[3] As noted later in this chapter, it is perhaps the easier of the two methods to apply, but can lead to conservative estimates of bridge capacity and retrofit schemes that may be more expensive than necessary.

The selection of the best retrofitting technique for an existing bridge requires that the potential deficiencies of the bridge be evaluated in detail. Retrofitting techniques that will improve performance must be identified and assessed for their feasibility and effectiveness. Retrofitting measures are described in chapters 4 through 9. The requirements for evaluating a bridge for retrofitting will vary depending on the location, configuration, and type of bridge. A flow chart detailing this procedure is shown in figure 11.

3.2 REVIEW OF BRIDGE RECORDS AND SITE INSPECTION

Regardless of the evaluation method that is to be used, the first step in the assessment process is the determination of the in situ condition of the bridge. This involves a review of the "as-built" plans, and construction and maintenance records, then followed by a site inspection.

3.2.1. BRIDGE RECORDS

The "as-built" plans, construction and maintenance records, and materials and design specifications should be reviewed as a starting point for a seismic evaluation. 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 information on 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. When information about the in situ properties of the materials is not available, the AASHTO *Manual for Maintenance Inspection of Bridges* may be used as a guide for determining typical ranges of material properties.[8] Maintenance records and bridge inspection reports may also contain information about the actual condition of the structural materials or

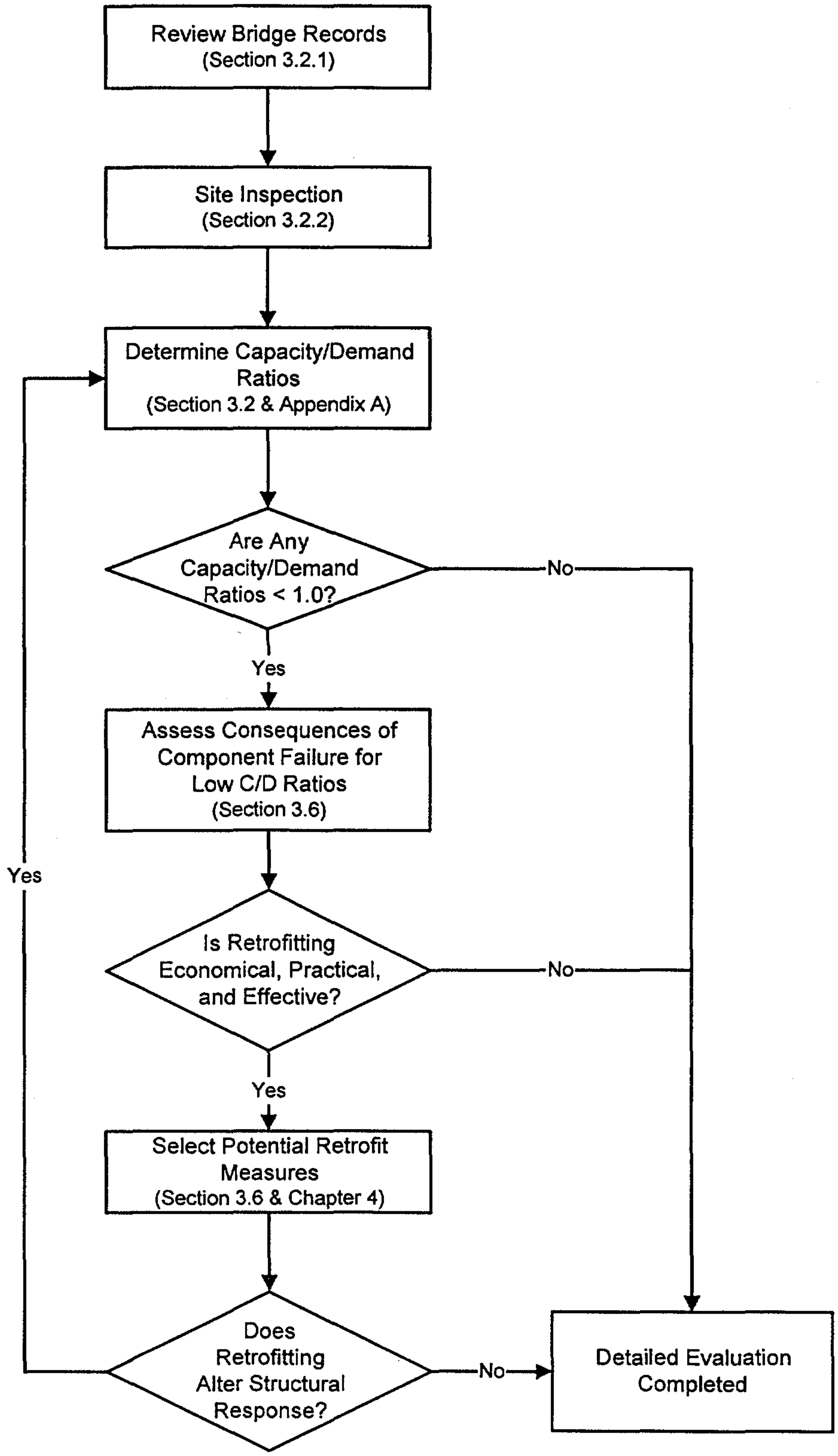


Figure 11. Procedure for evaluating a bridge for seismic retrofitting by the capacity/demand ratio method.

components. In addition, structural modifications may have been made which are not shown on the plans, but which may be noted in these reports.

Additional information may also be obtained from the original design calculations and construction records, although these documents are sometimes difficult to obtain. Bridge rating calculations to determine live load capacity may also contain useful information about the condition and strength of the materials present in the bridge. Annual scour inspection records may also be a useful source of relevant data.

3.2.2 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, if possible, to talk with the bridge maintenance and inspection personnel. The items which should be noted in the field inspection are:

- 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.
- Unusual erosion of soil at or near the foundation.
- Horizontal or vertical movement or tilting of the abutments, columns, or piers.
- Any deviations from the plans and specifications, such as nonstructural items not shown on the plans (e.g., continuous barrier rail) or modifications made since the bridge was constructed and thus not shown on the as-builts. Such items include additional roadway lanes and utilities, and adjacent retaining walls, buildings, rail lines, and the like. Some of these will affect the lateral stiffness of the bridge, whereas others may prevent the application of certain retrofit strategies.

Current Federal legislation requires that most bridges be inspected biennially as part of the National Bridge Inspection Standards. In general, these inspections are intended to monitor deterioration of the structure as it may affect the live load rating and are not specifically directed toward seismic evaluation. It is recommended that a separate, seismic-related inspection of important bridges be made to detect vulnerable conditions, and to direct maintenance personnel to monitor these conditions in their routine inspections. Other bridges may be given a less exhaustive inspection during their biennial review and these conditions may be monitored in future routine inspections.

3.3 EVALUATION METHODS

3.3.1 GENERAL

The most commonly used method for the detailed assessment of evaluation of seismic performance is based on elastic modal analysis and the estimation of component strengths and capacities. This approach leads to the calculation of C/D ratios for each component. A calculated C/D of less than 1.0 indicates the need for retrofit. This procedure is thus based on an element-by-element evaluation rather than on the performance of a bridge as a single structural system. The method is straightforward, but tends to overemphasize individual component behavior while ignoring the interaction between different components and their respective actions (forces and moments). It can give erroneous results in some instances, depending on the reinforcement volume in the member. Results can be very conservative, which may lead to unnecessarily expensive retrofit schemes and in certain unusual circumstances, it may give unconservative results.

An alternative approach is to examine the lateral strength of the bridge as a system, or at least individual segments of the bridge as a system, and determine, through an incremental collapse analysis, the load-deformation characteristics of the bridge up to collapse. The fraction of the design earthquake that can be resisted without collapse is then an indicator of the need for retrofitting and the extent of strengthening required. This procedure therefore determines the strength and ductility of the critical collapse mechanism, but it can also be used to identify the onset of damage when serviceability criteria may be important. The method emphasizes deformation capacity rather than strength. Strength is important, but it is less important than the ability to sustain substantial deformations without collapse. It is believed that fewer bridges assessed under this procedure will be found in need of retrofit than by the C/D ratio method. When retrofit is required, it should be less extensive. The increased level of effort required of the designer will then be offset by reduced retrofit costs in the field.

Both methods are described in this chapter and further detailed in appendices A and B. Analysis methods are summarized in this section and the overall procedures are explained in sections 3.4 and 3.5.

3.3.2 ANALYSIS PROCEDURE FOR CAPACITY/DEMAND RATIO METHOD

In this procedure, seismic demands are either calculated from an elastic spectral analysis as described below in sections 3.3.2.1 through 3.3.2.2, or set equal to minimum values that are described, for example, in sections A.2 and A.3. Capacities are taken as the nominal strength and/or displacement capacities of the components, without modification by capacity reduction factors, ϕ.

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.

Procedures for calculating seismic C/D ratios for various bridge components are further described in appendix A.

3.3.2.1 Selection of Spectral Analysis Method

Minimum requirements for the selection of an analysis method for a particular bridge type are given in table 4(a). Applicability is determined by the "regularity" of a bridge, which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span-to-span or support-to-support (abutments excluded). The requirements for regular bridges are shown in table 4(b). Any bridge not satisfying the requirements of table 4(b) is considered to be "irregular." A more rigorous, generally accepted analysis 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 and seat widths are considered in the evaluation process, and the force and displacement demands may be taken as the minimums prescribed in sections A.2 and A.3, respectively.

Table 4(a). Analysis procedure.

Seismic Performance Category	Regular Bridges with 2 through 6 Spans	Irregular Bridges with 2 or More Spans
B	Not required	Use procedure 2
C, D	Use procedure 1 or 2	Use procedure 3

The analysis procedures designated in table 4(a) are based on elastic analysis of the structure using the following methods:

Procedure 1: Uniform-load Method.
Procedure 2: Single-Mode Spectral Method.
Procedure 3: Multimode Spectral Method.

Table 4(b). Regular bridge requirements.

Parameter	Value				
Number of Spans	**2**	**3**	**4**	**5**	**6**
Maximum subtended angle (curved bridge)	90°	90°	90°	90°	90°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	–	4	4	3	2

Details of these procedures are given in section 4 of Division I-A of the AASHTO Specifications.[2] Some notes on the adaptation of these procedures to bridge evaluation are presented below.

3.3.2.2 Application of Elastic Spectral Methods

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 are considered, one in each direction, and the results are combined according to the rules given in section 3.3.2.4. At this time, the effect of vertical ground motion is not explicitly considered. However, this practice may change when the results become available of the failure analyses for the bridges that collapsed during the 1994 Northridge earthquake. Relatively high vertical accelerations were recorded during this earthquake, but their influence on bridge performance is uncertain at this time.

3.3.2.2(a) 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 C/D ratios of components cannot be used routinely at this time because of the difficulties and uncertainties that are involved. Some of these uncertainties include the modelling of inelastic material properties and the determination of appropriate time histories to be used in the analysis. 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 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.[9,10] 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 and boundary conditions at the foundations and abutments 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. In a retrofit project, the main objective of the abutment spring is to model the correct load distribution between the abutments and columns, so as to allow a reasonable evaluation to be made of the seismic demand on the columns and to assess the safety margin (collapse potential) of the overall bridge. In establishing the abutment stiffness, attention should be given to examining the bridge-to-abutment connections to ensure that the assumed stiffness is compatible with the load level distributed to the abutments and the corresponding reduction in load at the columns. Selection of appropriate spring stiffnesses for the abutment may be obtained by following the procedure outlined in figure 12. Methods for calculating initial elastic spring stiffnesses are discussed in the FHWA manual on the seismic design and retrofit of highway bridges.[11] If, when an analysis is performed, the ultimate force capacity of the abutments is exceeded by more than 10 percent, 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. In this way, a secant elastic stiffness is used to account for the inelastic action.

The modelling 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 contact between adjacent structural sections. This is often ignored in a multimodal response spectrum analysis

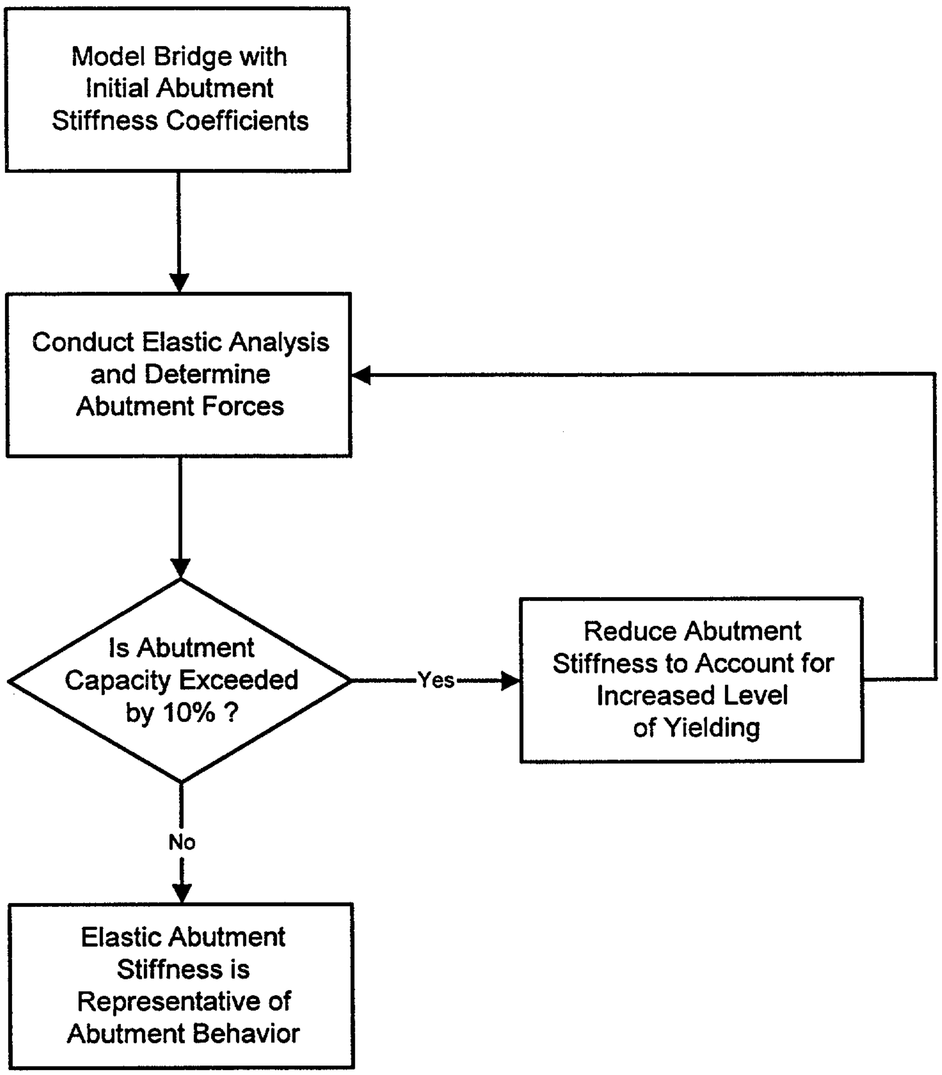


Figure 12. Interative procedure for determining abutment flexibility effects.

when expansion joints are modelled to have total freedom of movement in any direction.

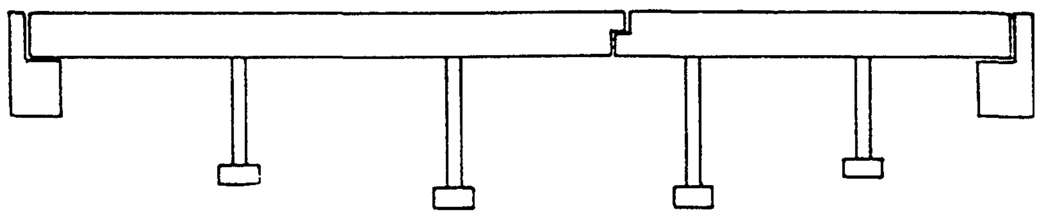
In expansion joints fitted with longitudinal motion restrainers, motion is restricted in both directions. Most restrainers are unidirectional, being only effective in preventing joint separation. Restrainers are usually designed to engage after movement of a small distance, which is provided to allow expansion joints to function properly. Joint closure prevents movement in the opposite direction. The behavior of expansion joints is very complex; nonlinear computer programs have been written to model this behavior.(12) In the case of an elastic analysis, however, it has been common practice to use tension and compression models in each of two orthogonal directions of the bridge. In the tension model, the joints are free to move and the restrainers are represented by springs with stiffnesses equal to the tensile stiffness of the restrainers. In the compression model, the joints are locked against translation, but are free to rotate. The actual situation is somewhere in between these two limits. Analytical case studies have shown that this approach often yields results considerably different than a more sophisticated nonlinear analysis.(10) Usually, the elastic force results are greater than nonlinear results, but intuitively appear to be reasonable in magnitude. Therefore, elastic analysis is often considered to be conservative for design and evaluation. Fortunately, the method of modelling restrained expansion joints for an elastic analysis appears to have a small effect on the elastic response of the remainder of the structure.

The difficulty involved in accurately analyzing the response of expansion joints is one of the reasons that the AASHTO Specifications specify minimum support lengths and motion restrainer forces.

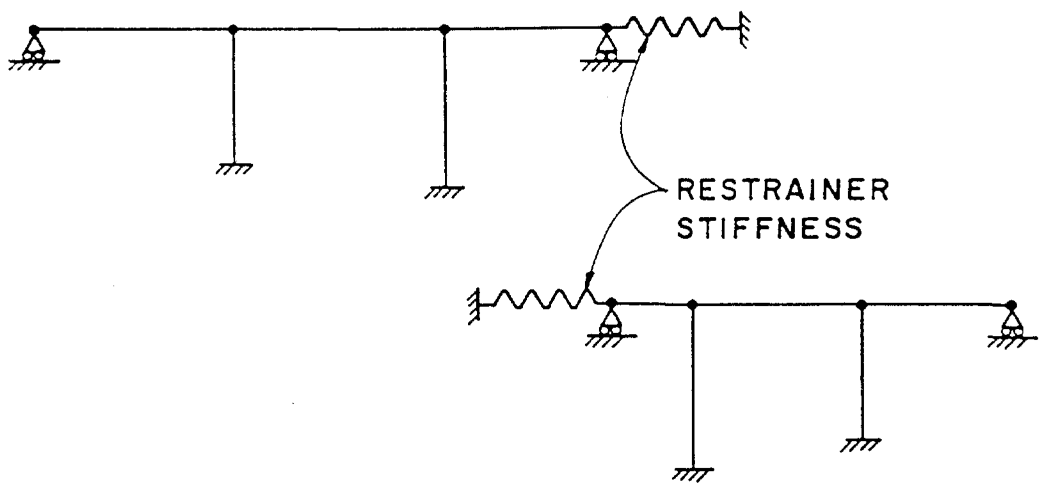
3.3.2.2(b) Single-Mode Spectral Method (Simplified Approach)

The single-mode spectral method of analysis is allowed in certain situations as shown in table 4(a). 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, readily determined mode of vibration. A description of this method is given in the AASHTO Specifications.

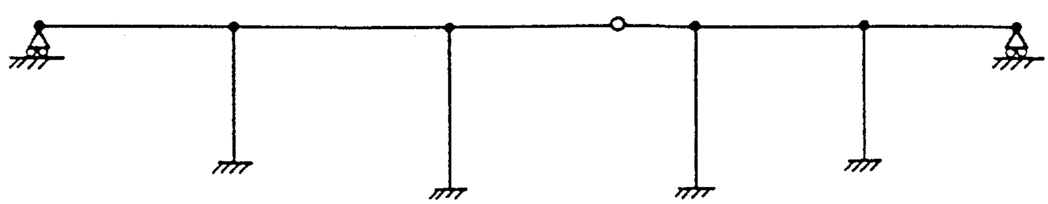
Bridges with intermediate expansion joints will have at least two significant modes of vibration and therefore an adaptation must be made if the single-mode spectral approach is to be used. This can be done by performing separate analyses for the tension and compression models which represent the structure first with the joints opening and then with the joints closing. Tension and compression models for the bridge shown in figure 13(a) are noted in figures 13(b) and (c), respectively. Note that the tension model shown in figure 13(b) is a variation of the one described above in section 3.3.2.2(a). In this case, the elastic force in the expansion joint restrainer should be taken to be the lesser force derived from the analyses of the two segments shown. The column and foundation forces due to a longitudinal earthquake loading would be the greater of the forces obtained from the analyses of the two models.



(a) Structure Configuration



(b) Models for Restrainer Forces



(c) Model for Other Forces

Figure 13. Typical structure idealization, single-mode spectral method.

3.3.2.2(c) Multi-Mode Spectral Method

More complex structures in higher seismic performance categories should be analyzed using the multimode 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 does require a basic conceptual understanding of the theory of structural dynamics. Proper modelling of a bridge system for a multimode spectral analysis is different from the modelling required to perform a static analysis. This is due to inertia effects which 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 lumped masses to be included is critical to the analysis. Too few masses will result in unsatisfactory results, and too many will increase the computer costs unnecessarily. As a rule of thumb for the type of structures covered by this manual, 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 modelling considerations related to abutments and expansion joint hinges were discussed above in section 3.3.2.2(a).

Several general-purpose computer programs are available which may be used to perform a multimode spectral analysis of a bridge. Also, the computer program SEISAB was specifically developed for the seismic analysis of bridge structures with funding from the National Science Foundation.[13] This user-oriented program has many features that assist the implementation of the AASHTO Specifications. The worked example problem presented in appendix C illustrates the use of this program for bridge seismic evaluation.

3.3.2.3 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 3.3.2.1. 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. Their contribution may be tested by performing two analyses with significantly different foundation stiffnesses that bound the actual properties and then checking the sensitivity of the results to this parameter. Elastic forces and displacements due to loading along each perpendicular axis should then be combined as specified in section 3.3.2.4 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 taken as a chord connecting the two abutments. Care should be exercised when a curved bridge is almost a half-circle in plan to be sure that the correct abutment springs are being used. This is because in a bridge with this geometry, the transverse restraint at the abutment is acting along the longitudinal axis of the bridge (if this axis is defined as the chord connecting the two abutments).

3.3.2.4 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 to 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 3.3.2.3 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 percent of the absolute value of the results from the analysis of loading in the first perpendicular (longitudinal) direction to 30 percent 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 percent of the absolute value of the results from the analysis of loading in the second perpendicular direction (transverse) to 30 percent of the absolute value of the corresponding results from the analysis of loading in the first perpendicular direction (longitudinal).

3.3.3 ANALYSIS PROCEDURE FOR LATERAL STRENGTH METHOD

3.3.3.1 General

Despite the widespread use of elastic modal analysis methods, there are a number of concerns with its application to the evaluation of bridges. The most important of these concerns is the use of response modification factors which may be a very poor indicator of the actual member ductility demand. As a consequence of these uncertainties and the current lack of suitable inelastic time history analysis methods, a simple frame-by-frame method of assessment might be considered as an alternative approach. In this method, the bridge is separated into individual segments (or frames) between expansion joints, each of which is separately assessed. An incremental collapse mechanism approach is applied to each frame to identify the critical elements and define the load-deformation curve for the frame. From an assessment of the natural period of the segment and the design response spectrum, the equivalent elastic response is estimated, and the fraction of the design earthquake level capable of being resisted without collapse is then calculated.

This section describes such a method, first by considering transverse response, followed by longitudinal response. Analysis for superstructure displacements at the expansion joints and P-Δ effects in the columns are also considered. This description is taken from reference 18.

The method requires the knowledge of member strengths and deformation capacities. Procedures for their determination are summarized in appendix B.

3.3.3.2 Frame-by-Frame Analysis of Transverse Response

Consider a typical frame consisting of three bents, where each bent has two columns framing into a cap beam at the top, and into footings at the bottom, as shown in figure 14. The three bents have different heights (and hence stiffnesses) and the columns have different reinforcement patterns at the different bents. Neglect any torsional rotation in plan due to restraint by adjacent frames. Each bent then displaces an equal amount at a given level of seismic response. However, when there are large variations in pier stiffness or there is restraint by an abutment at one end of a frame, the high probability of non-uniform transverse response, due to torsional effects, should be included in the assessment.

The assessment of the transverse response of this frame is as follows:

Step 1: Initial Stiffness–Form a simple lateral model for each bent. Apply a reference lateral force, F, at the height of the center of mass to calculate displacement, Δ, at the bent cap and induced moments, as shown in figure 15, for each bent. Column stiffness and cap beam stiffness should both be based on cracked-section properties for this analysis, since significant cracking in both members are expected. Foundation rotational and translational stiffness should also be modelled. The stiffness of bent i is then given by:

$$K_i = F/\Delta_i \tag{3-1}$$

and the natural period of the frame may be approximated as:

$$T = 2\pi\sqrt{\frac{W}{g\sum K_i}} \tag{3-2}$$

where W is the total weight of the frame superstructure, plus 50 percent of the column weight. Live load is normally ignored because it is unlikely to couple with superstructure inertia. The design elastic spectral response level, $S_{a(R)}$, can now be found (see section 3.5).

Step 2: Gravity Moments–Moments due to gravity load are calculated by making the same assumptions as to member stiffness that are made for seismic analysis, since it is the distribution of gravity moments at the onset of yield under lateral forces that is of interest. These moments will normally be based on dead load only, though an allowance for probable live load could be made. These moments, M_D, are shown in figure 15(b).

Step 3: Bent Member Moment Capacities–Moment capacities at critical sections in the column and cap beam should be calculated to enable a moment capacity envelope to be drawn, as shown in figure 15(c) for M_{cap}. In preparing these envelopes, the following points should be considered:

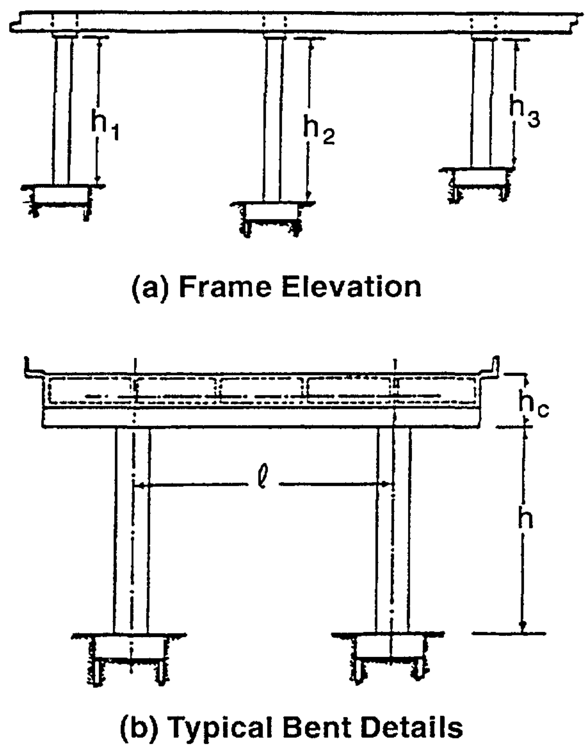


Figure 14. Frame assessment example (from ref. 18).

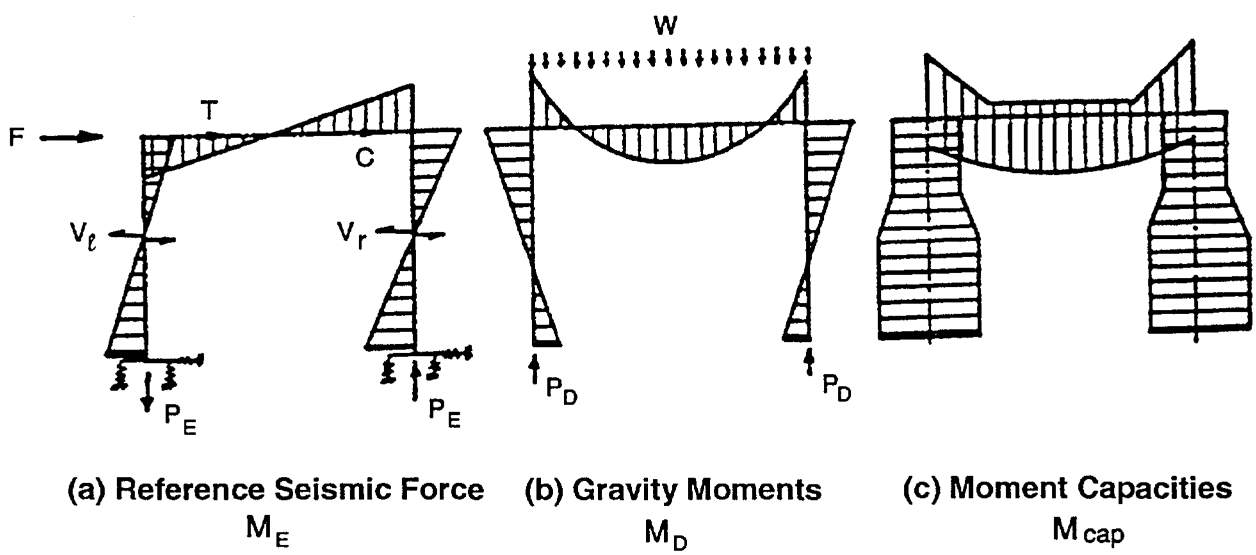


Figure 15. Moment patterns for initial assessment (from ref. 18).

a. Moment capacities should be based on an estimate of the member axial force applicable to the development of the full collapse mechanism. This may require some iteration. For example, the column axial forces will be, from figures 15(a) and (b):

$$P = P_D \pm P_E \tag{3-3}$$

where P_E is the value applied at the full mechanism lateral force level, F_M. Similarly the cap beam will be subjected to an axial tension $T = V_\ell$ at the left end, and axial compression, $C = V_r$, at the right end, where V_ℓ and V_r are the left and right column shears, and $V_\ell + V_r = F_m$.

Great accuracy in estimating P_E, T, and C is not warranted, however, since a small error in P_E will result in increased flexural capacity of one column and an almost equal decrease in flexural capacity of the other column, with little or no influence on mechanism capacity, and very little influence on ductility. Thus, initial values for column moment capacity may be based on gravity loads alone and used to calculate P_E values in a single step iteration. These values are added to (or subtracted from) the gravity loads for the final estimates of moment capacity.

b. The critical section may not be adjacent to the intersection of members. For example, premature termination of cap beam top steel may result in a critical section some distance from the column face. In assessing strength in regions with terminating reinforcement, development length effects must be included as a gradual increase in flexural capacity. Effects of tension shift resulting from indirect flexure-shear cracks should also be considered when shear stress levels are greater than $0.17\sqrt{f'_c}$ MPa ($2\sqrt{f'_c}$ psi).

c. The influence of lap splices in modifying moment capacity should be checked in accordance with section B.3. The ability of joints to sustain the member moment capacities should be investigated in accordance with section B.7. Note that the cap beam/column joints would only need to be checked for capacity to sustain the lower of the moment capacities of the beam and column framing into the joint.

Step 4: First Hinge Formation–The location and level of seismic force associated with formation of the first plastic hinge can be found from the information in figure 15, by finding the minimum value of:

$$R = \frac{M_{cap} - M_D}{M_E} \tag{3-4}$$

for all sections. The corresponding level of seismic force for the bent is:

$$F_1 = R \cdot F \tag{3-5a}$$

with corresponding displacement:

$$\Delta_1 = R \cdot \Delta \tag{3-5b}$$

At this stage, the bent is responding essentially elastically. It has already been determined whether joint capacity exceeds member-end moment capacity. However, the capacity of all members to sustain the shears associated with first hinge formation should be checked. Flexural capacity will be determined in accordance with sections B.2 and B.3, using probable, rather than nominal, material strengths. To prevent brittle shear failure at this stage, it should be determined that:

$$V_{cap} \geq 1.1\ V_1 \tag{3-6}$$

where V_{cap} is the initial shear capacity (i.e., for $\mu = 1$ at all sections) and V_1 is the section shear force corresponding to F_1, including gravity load shears, where appropriate, such as in the cap beam.

If equation 3-6, in conjunction with the capacities determined in accordance with section B.6, are not satisfied at one or more sections, brittle shear failure may occur and the initial elastic load-deflection given by equation 3-5 must be proportionately reduced to correspond to the minimum shear strength. The system has no ductility and the equivalent elastic strength is equal to the lateral force corresponding to the brittle shear strength.

Step 5: Second Hinge Formation–If equation 3-6 is satisfied at all sections, the bent possesses some ductility. It is now assumed for the purpose of discussion that the minimum value of equation 3-4 occurs at the left end of the cap beam, under positive moment. A second stage of analysis is carried out on the modified structure with a hinge inserted at point 1, as shown in figure 16(a). Again, the chosen level of incremental force, ΔF_1, is arbitrary and is used to define the incremental stiffness and the moment pattern.

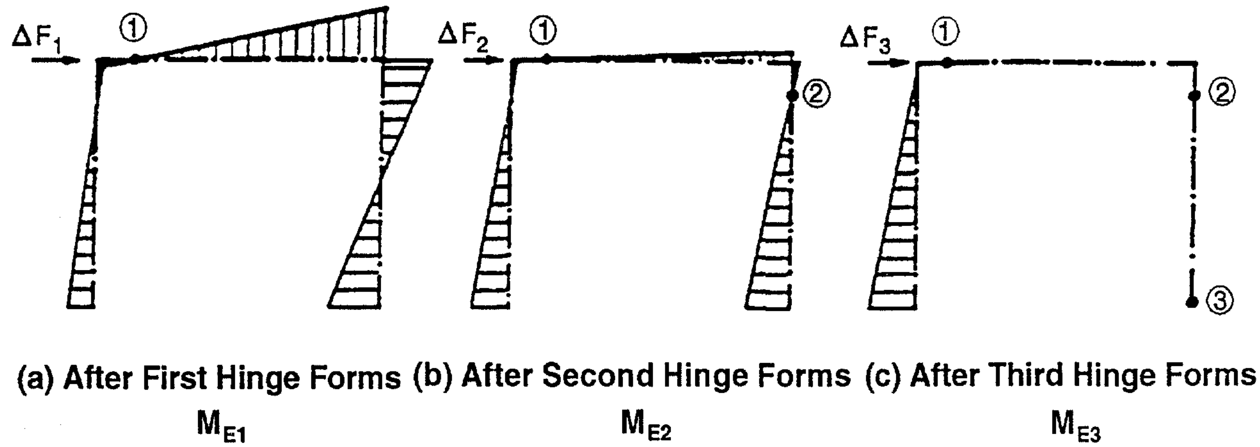


Figure 16. Moment patterns for modified bent (from ref. 18).

Initially assume that hinge 1 in the cap beam has elasto-plastic moment/ rotation characteristics, and the total moments at first hinge formation are termed M_1, where:

$$M_1 = M_D + R \cdot M_E \tag{3-7}$$

The moments induced by the arbitrary force increment, ΔF_1, are denoted M_{E1}. The location of the second hinge is determined by searching for the minimum value of:

$$R_1 = \frac{M_{cap} - M_1}{M_{E1}} \tag{3-8}$$

for all sections. In this case, it can be assumed that this occurs at the top of the right-hand column. The total seismic resistance at this stage is:

$$F_2 = F_1 + R_1 \cdot \Delta F_1 \tag{3-9a}$$

with corresponding displacement:

$$\Delta_2 = \Delta_1 + R_1 \cdot \Delta_{(\Delta F1)} \tag{3-9b}$$

where $\Delta_{(\Delta F1)}$ is the incremental displacement corresponding to ΔF_1.

The ability of the bent to sustain the formation of the second hinge must be checked. Shear forces must again be checked against capacity using equation 3-6, substituting V_2 for V_1. At hinge 1, the plastic rotation, θ_{p1}, must be checked against plastic rotational capacity in accordance with equation B-13. The corresponding member ductility, $\mu_{\Delta 1}$, may be checked using equation B-15, where ℓ is the distance from hinge 1 to the point of contraflexure on the cap beam at lateral force level F_2. The shear capacity of the plastic hinge at point 1 should be assessed using the degraded shear strength model of figure B14. Joint shear strength adjacent to point 1 will decrease with ductility in accordance with figure B18, and thus should be rechecked.

If the shear or elastic rotation capacity of any section cannot support the new hinge formation, an ultimate capacity in terms of displacement and force can be found by interpolating between Δ_1 and Δ_2. The mechanism analysis is then complete.

If the flexural strength at point 1 is assessed to increase with increasing rotation (as a result of strain hardening) or to decrease with increasing rotation (as a result of splice failure), the above approach needs to be slightly modified. After calculating the plastic rotation, θ_{p1}, and corresponding ductility, $\mu_{\Delta 1}$, the revised hinge flexural capacity is found from figure B14, and the value of R from equation 3-4 should be modified in proportion. As a consequence, the value of M_1 in equation 3-7 and R_1 in equation 3-8 will change. A quick iterative procedure will provide the final solution for equation 3-9, in order to define F_2 and Δ_2.

Step 6: Subsequent Hinge Formation–The procedure continues with further structural modification as shown in figures 16(b) and (c) until shear failure occurs, the plastic rotation capacity of a hinge is exceeded, or the lateral force/displacement curve indicates a reduction of strength with increasing displacement.

The procedure outlined above is carried out for the three bents of figure 14 independently. The displacement at which the first frame reaches its ultimate displacement is then taken as the maximum displacement capacity for that frame. The composite force deformation response for the frame as a whole is found by adding the bent forces for given displacements, as shown in figure 17. The resulting frame mechanism strength, V_f, and yield displacement, Δ_y, are calculated from the intersection of the initial slope and final slope of the response. The displacement ductility is calculated from the ratio of ultimate to yield displacement, and the equivalent elastic lateral force is then found from the method outlined in section 3.5, using the calculated ductility from figure 17 in conjunction with equations 3-16 and 3-17 and the calculated initial period.

3.3.3.3 Frame-by-Frame Analysis of Longitudinal Response

Analysis for the capacity of the frame in the longitudinal direction proceeds in essentially the same fashion as above. However, if the superstructure is monolithic with the columns, some consideration of the superstructure flexural capacity is warranted. Under transverse response, there will be little difficulty in determining the appropriate flexural capacity of the cap beam, but in those cases where the

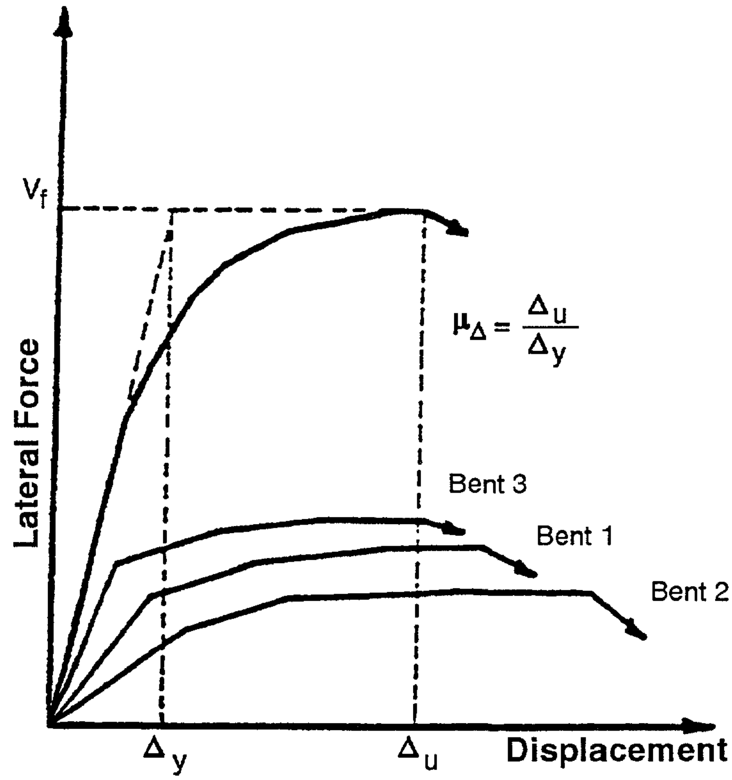
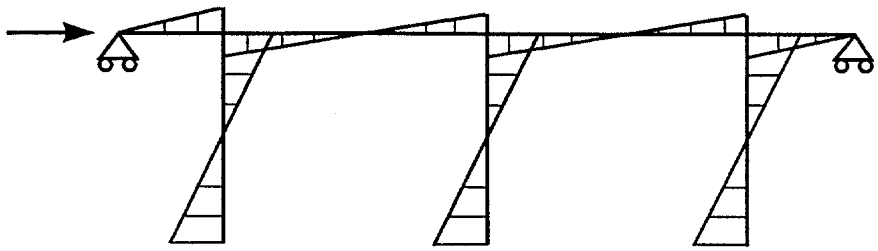


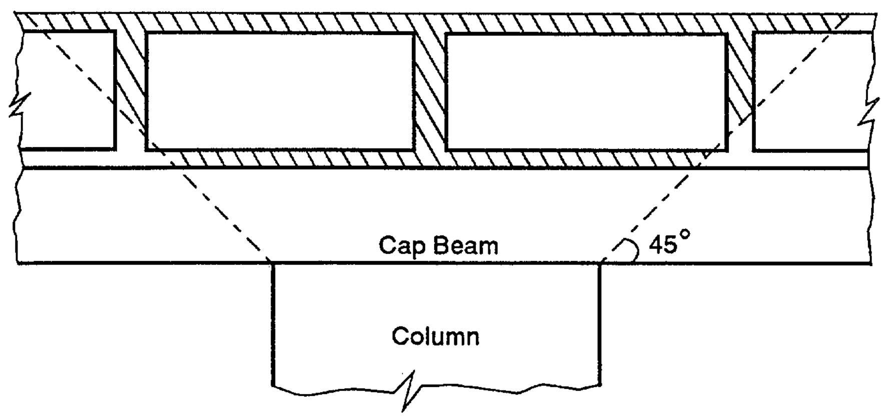
Figure 17. Composite lateral force/deflection response (from ref. 18).

superstructure is monolithic with the cap beam, as might be the case with a cast-in-place concrete box girder bridge, some judgment will be required in assessing how much transverse reinforcement in the superstructure can contribute to the cap beam flexural response. Under longitudinal response, even if the superstructure is monolithic, it is unlikely that the full superstructure section will contribute to resisting moments induced by column flexure. As shown in figure 18(a), there will be a requirement for moment reversal on opposite sides of the cap beam under longitudinal seismic action. The torsion induced in the cap beam by this moment change in the superstructure will cause considerable torsional rotation, and possibly torsional failure of the cap beam, reducing the capacity of the cap beam to resist these superstructure moments. It is thus recommended that superstructure capacity be checked using a reduced effective section width based on a 45° spread of influence from the intersection of the column with the cap beam soffit, as shown in figure 18(b).

Note that most older bridges are only designed for vertical (gravity) loads and the reinforcement in the superstructure is usually terminated where it is not required to resist the flexural moments due to gravity loads. Although the moments due to lateral loads decrease with distance from the cap beam (see figure 18(a)) and the effective width of the superstructure that resists these moments increases, the capacity may decrease faster then the demand decreases because of the termination of



(a) Longitudinal Moments



(b) Effective Superstructure Section

Figure 18. Longitudinal response of superstructure girders (from ref. 18).

reinforcement. In this case, a 45° spread in two directions may be necessary to adequately assess the capacity of the superstructure. In situations where an in-span hinge is located close to a bent, almost the full column moment must now be resisted by the superstructure.

If the flexural capacity of the shaded area of the superstructure is insufficient to develop the moment capacity of the column, then superstructure hinging can be expected. Because of the undesirability of such a mechanism and the uncertainty of its consequences, it is recommended that an ultimate compression strain no greater than 0.004 be adopted when assessing the ductility of superstructure hinging. As plastic rotation develops in the superstructure, the effective width is likely to increase above that indicated in figure 18(b). The consequence of this will be increased torsional moments in the cap beam, which should then be assessed for torsional capacity.

3.3.3.4 Analysis for Movement at Expansion Joints

The assessment of the adequacy of the expansion joints between bridge segments or between end spans and abutments is a matter of some controversy. It is clear that the unseating of bridge spans with short seat lengths has been a major cause of bridge collapse in past earthquakes; but the computation of expected displacements across these joints, and how this is affected by the presence of joint restrainers and their structural characteristics, still requires the use of assumptions of doubtful validity. A number of different methods have been used, some of which have been previously discussed under the C/D ratio method (section 3.3.2). These are summarized in the following sections.

3.3.3.4(a) Empirical Assessment

Seating widths are compared to specified minimum values, which are related to span length and pier height. Minimum seat lengths of:

$$N = 305 + 2.5L + 10H \text{ (mm)} \qquad (3\text{-}10a)$$
$$N = 12 + 0.03L + 0.12H \text{ (in)} \qquad (3\text{-}10b)$$

are required for the most important structures in the highest seismicity region, where L is the length of bridge deck to the next expansion joint, and H is the average height of columns supporting the bridge deck to the next expansion joint (m in equation 3-10a and ft in equation 3-10b). If the existing seating length does not satisfy this criterion, retrofit is required.

The procedure does not consider the influence of existing restrainers in limiting displacements and ignores the influence of foundation characteristics and structural period, though the latter aspect is somewhat included as a consequence of the height term, H, in equations 3-10a and 3-10b.

3.3.3.4(b) Caltrans Design Approach

Each segment of the superstructure on either side of the joint is separately considered, with the other segment assumed to be stationary. Longitudinal displacements resulting from longitudinal first mode response and gap openings due to transverse response are computed and combined based on the stiffness and mass of the segment under consideration. If the computed displacements are less than the available safe joint opening, no further assessment is needed. If the computed displacement is more, but restrainers have been installed, the computed displacement is reduced by an amount:

$$\Delta = \frac{nF_yA_r}{K_u} \tag{3-11}$$

where n is the number of restrainers present; F_y is the yield stress; A_r is the area of each restrainer; and K_u is the longitudinal stiffness of the segment under consideration, ignoring the restrainer stiffness. If Δ does not reduce the calculated displacements to the existing safe joint opening, retrofit will be required.

This approach makes several assumptions about the means by which joint openings occur that are based more on experience and intuition than on rational analysis. It should be noted that the reduction in segment stiffness resulting from formation of column plastic hinging is not incorporated into the analysis.

3.3.3.4(c) Elastic Modal Analysis

In more recent assessments, Caltrans has used elastic modal analysis to determine the forces and displacements expected at expansion joints. However, this is not normally carried out at the diagnostic stage, which makes simplified assumptions about joint connectivity and applies more to "as-retrofitted" analyses. But since the method would appear to be applicable (within the limits imposed by its assumptions, as subsequently noted) to an "as-built" assessment, it will also be considered here.

The stiffness of existing expansion joints is represented by strut models whose stiffness is based on that of the installed restrainers, if any. A global analysis (or semi-global analysis involving a reduced number of frames for very long structures) is carried out and modal forces and displacements are calculated by appropriate modal combination rules, such as square root of sum of squares (SRSS) or complete quadratic combination (CQC), to determine the adequacy of the joints. Since the restrainers are required to remain elastic, these forces are not reduced by ductility considerations. If the displacements or restrainer forces are excessive, retrofit may be required. If the restrainers are to be redesigned, the procedure in section 3.3.3.4(b) is used.

The weakness of the elastic modal analysis approach is that it cannot account for the inelastic behavior of the columns. If one or both of the segments are at yield, the incremental stiffness will drop to a very low value, and an elastic restraining system should be able to restrain additional relative displacements with low force levels. It is thus probable that the relative displacements and forces will be greatly overestimated by the elastic modal analysis approach when ductile response of the piers in the adjacent segments is expected.

At joints in compression, the assumption of strut and tie models, based on restrainer properties, will result in relative displacements across joints that may be meaningless, with some showing closing displacements that are greater than the existing movement gap.

3.3.3.4(d) Dynamic Inelastic Analysis

A better estimate of expected relative displacements will be achieved when dynamic inelastic time-history analysis techniques are considered. However, sophisticated joint models are needed to simulate the effects of friction, initial gap, and different tension and compression stiffnesses. Recent research has been directed towards development of improved models for this purpose, but no universally accepted method is available at this time.

3.3.3.4(e) Analysis for Relative Ground Motions

The lack of coherence and synchronized ground motion at different piers along the length of a long bridge makes even the results from dynamic inelastic time history analysis of doubtful validity unless these effects are considered in the form of differing input accelerations at different piers. For long bridges with a number of separate frames and restricted movement gaps at internal and end joints, it seems unlikely that a fully resonant longitudinal response could develop, since different frames and different piers within a frame will not only deform out of phase, but will also be restrained from large joint openings by compression in some of the joints.

An alternative approach for assessing the performance of joints involves determining relative displacements and restrainer forces, corresponding to the foundations of adjacent bents being subjected to relative displacements caused by seismic wave propagation in the longitudinal direction, as shown in figure 19. A seismic ground wave with displacement amplitude equal to the predicted maximum and with a wave length appropriate to the site material is imposed on the foundations of the pier, and the joint opening and restrainer force, T, is then calculated. Note that the foundation displacement will be strongly dependent on site conditions. Soft soils will have large ground-displacement amplitudes and short wave lengths, thus inducing maximum restraining forces. Advice from a geotechnical engineer should be sought in determining the appropriate soil deformations.

In all cases, force transfer from restrainers into the superstructure should be carefully considered. The connections and the superstructure must both be capable of resisting the tensile strength of the restrainers.

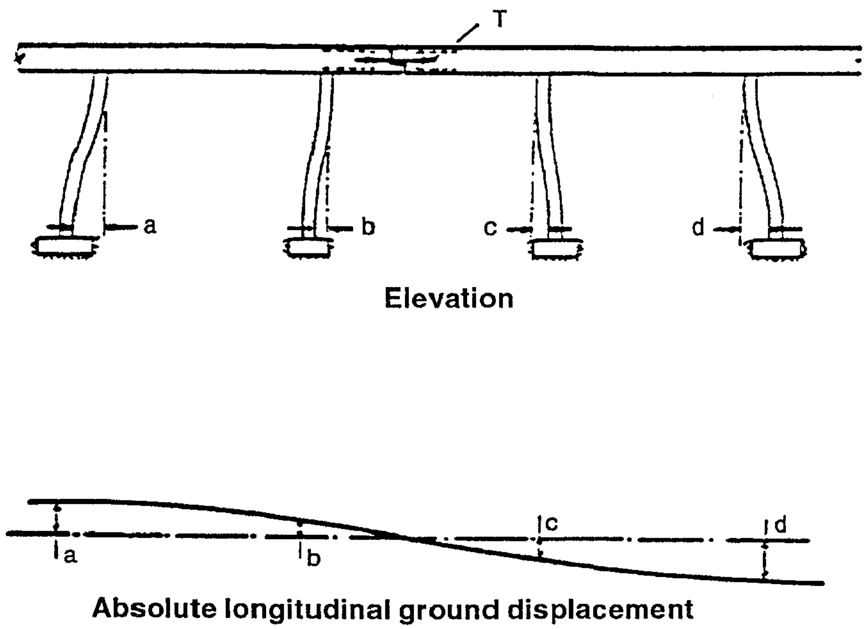


Figure 19. Restrainer forces from relative foundation movement (from ref. 18).

3.3.3.5 Analysis for P-Δ Effects

Failure is expected in a bridge column when the total moment, including the P-Δ moments, exceeds the moment capacity of the flexural plastic hinge. The P-Δ effects can also influence the response by increasing displacements above those calculated by simple first-order analyses, as presented in the previous sections. For design of new structures, P-Δ effects can be compensated for by designing for an enhanced moment capacity:

$$M_D = M_V + \Delta M \tag{3-12}$$

where M_V is the moment based on specified lateral design forces, and:

$$\Delta M = \left(\frac{1+\mu}{2}\right) P \cdot \Delta_y \tag{3-13}$$

for a fixed-fixed column, where P is the column axial force, including seismic effects; Δ_y is the calculated yield displacement; and μ is the required displacement ductility. Equation 3-13 is based on "equal energy" considerations, and should be conservative for typical bridge periods. If the column is fixed-pinned, then ΔM is twice that given by equation 3-13.

In assessing performance in accordance with the "equivalent elastic lateral strength" approach outlined in section 3-5, P-Δ effects may be included by reducing the equivalent lateral mechanism strength, V_f, calculated in accordance with section 3.5 by the ratio:

$$V_{P\Delta} = V_f \frac{(M - \Delta M)}{M} \tag{3-14}$$

where M and ΔM are the moments at a critical hinge, calculated in accordance with equation 3-13.

This reduced lateral strength will be used in conjunction with equations 3-16 and 3-17 (see section 3.5.2) to predict the equivalent elastic lateral strength. Note that for flexible structures with high ductility capacity, the equivalent elastic lateral strength found from equations 3-14, 3-16, and 3-17 will be non-linearly related to μ, and a maximum may occur at less than the calculated ductility capacity.

3.4 CAPACITY/DEMAND RATIO METHOD

Bridge components that have the potential of being damaged during an earthquake should be evaluated quantitatively to determine their ability to resist the design earthquake. This should be done by calculating the seismic C/D 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 5 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. C/D 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.

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 C/D ratios will be calculated using these minimum requirements as demands.

Seismic capacities are calculated at their nominal ultimate values without capacity reduction factors, ϕ. In cases such as well detailed reinforced concrete columns, where post-elastic behavior is acceptable, C/D ratios are modified by ductility indicators to reflect the capacity of the column to withstand yielding.

Table 5. Components for which seismic capacity/demand ratios should be calculated.

Seismic Performance Category	B	C	C	D
Acceleration Coefficient	0.09<A≤0.19	0.19<A≤0.29	>0.29	>0.29
Component				
EXPANSION JOINTS AND BEARINGS				
Support Length	X	X	X	X
Forces	X	X	X	X
REINFORCED CONCRETE COLUMNS, WALLS, 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	X

In general, the available capacity is assumed to be defined as 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 and their anchorages.
- 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 C/D ratio, r, is:

$$r = \frac{R_C - \Sigma Q_i}{Q_{EQ}} \tag{3-15}$$

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 equations 6-1 and 7-1 of Division I-A of the AASHTO Specifications.
Q_{EQ}	=	The displacement or force demand for the design earthquake loading at the site.

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

C/D ratios are intended to represent 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.

Appendix A describes a methodology for determining component C/D ratios.

3.5 STRUCTURE LATERAL STRENGTH METHOD

This alternative procedure for evaluating existing bridges attempts to determine the expected lateral force/displacement characteristic of a portion of the structure and relate this to an equivalent elastic strength. In order to determine expected deformation characteristics, assessment of ductility capacity, integrity of lap splices, anchorage of reinforcement, shear capacity of connections between members, and other assessments must be made. In many cases, the procedures outlined below are based on recent experimental research; but in some cases, they are based on theoretical considerations alone, since experimental data are not yet available.(14) These cases are identified in the text.

3.5.1 ASSESSMENT LIMIT STATES

Three possible limit states could be considered:

Serviceability Limit State: For response to this limit state, different elements of the bridge might develop their strength, but no significant ductility would be required, and the bridge would be expected to be serviceable immediately following the earthquake. Any damage would essentially be of cosmetic rather than structural significance.

Damage Control Limit State: This limit state represents the extreme level of seismic response after which it would still be economically and technically feasible to repair the bridge.

Survival Limit State: Response to the survival limit state represents the extreme level of seismic response, beyond which collapse would occur.

A problem in seismic assessment clearly relates to the conditions identifying the characteristics of each limit state. The serviceability limit state may be considered reasonably straightforward in that normally, a simple elastic analysis will identify the critical element or section in terms of strength and the corresponding system level of seismic response therefore can be quickly determined.

The damage control limit state is probably the most important in terms of seismic assessment and, as such, is considered in detail in the following. It is taken to be the limit state beyond which lateral resistance diminishes with increasing displacement.

Although the survival limit state is of critical concern, its determination has received comparatively little attention. It is almost self-evident to mention that this limit state corresponds to the condition where the bridge is no longer able to support its gravity loads and therefore collapses; but this is a very valuable and effective way of defining the survival limit state. Reduction of the lateral resistance of a bridge by a given percentage (say 20 percent or 50 percent) is sometimes taken to represent this condition, but this is clearly inadequate. The structure may still be stable with very low residual strength, since lateral response displacements essentially have an upper bound in any given seismic event. Collapse will occur when gravity load capacity is reduced below the existing gravity loads, for example, as a result of column shear failure or total collapse of a column plastic hinge. Alternatively, collapse may result from a stability failure, such as when the P-Δ moments exceed the residual capacity of the bridge columns, as shown in figure 20.

If the ultimate displacement, Δu, determined from the intersection of the resistance and P-Δ curves, exceeds the maximum expected in the assessment level earthquake, then a stability failure is not expected. Although assessments based on such an approach should not be used as the basis for a decision as to whether or not to retrofit a bridge, they are appropriate for prioritizing structures for retrofitting.

3.5.2 EQUIVALENT STRENGTH

Once the strength and ductility of the critical collapse mechanisms have been determined, the equivalent elastic response may be estimated by consideration of "equal-displacement" or "equal-energy" approaches, depending on the natural period of response. An annual probability of failure may then be determined by comparison of the equivalent elastic response with site-specific elastic response spectra. This procedure is illustrated in figures 21 and 22.

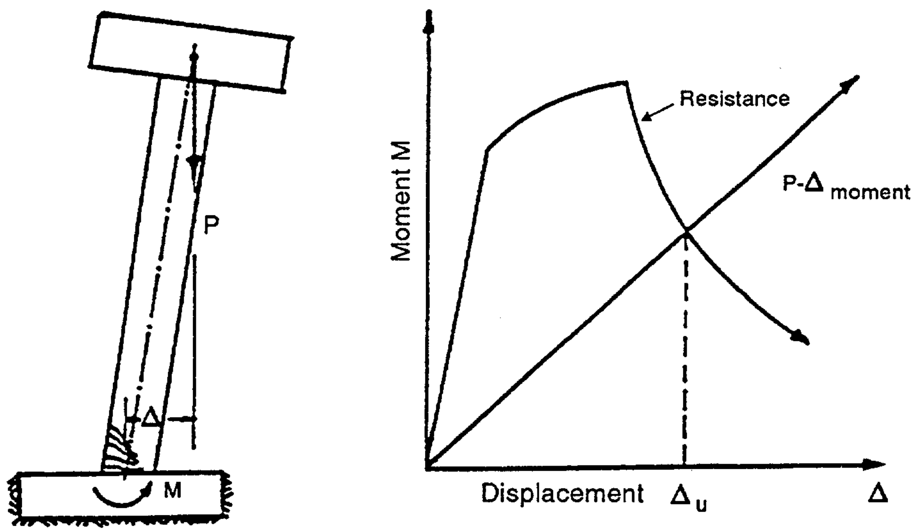


Figure 20. P–Δ collapse of a bridge column (from ref. 18).

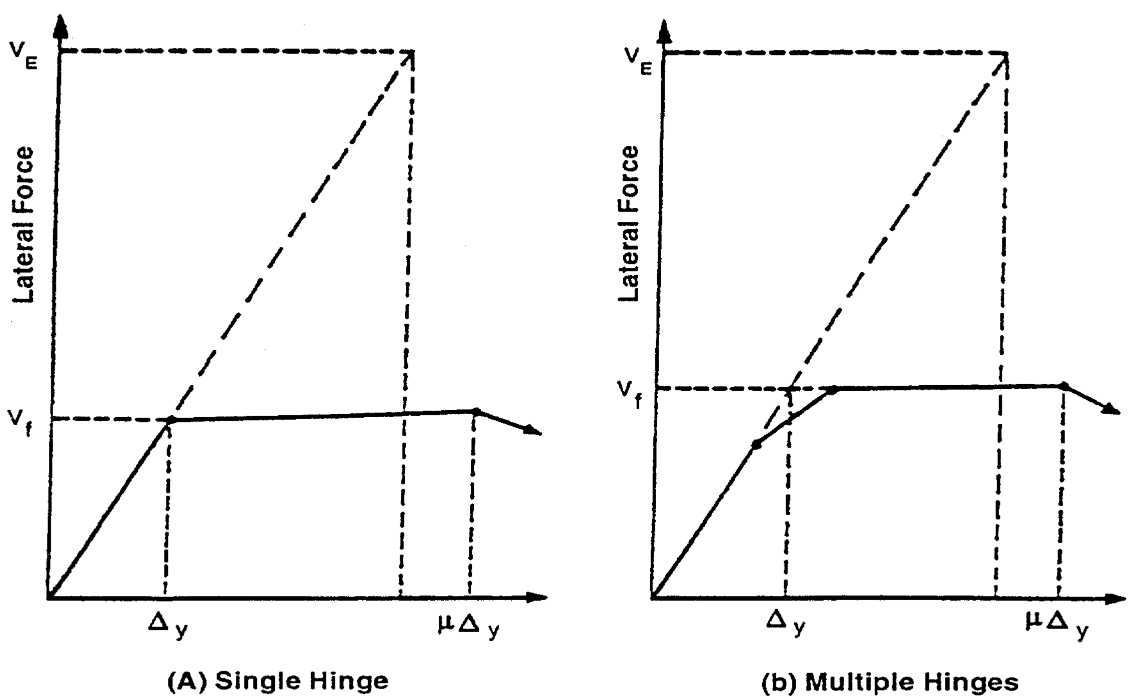
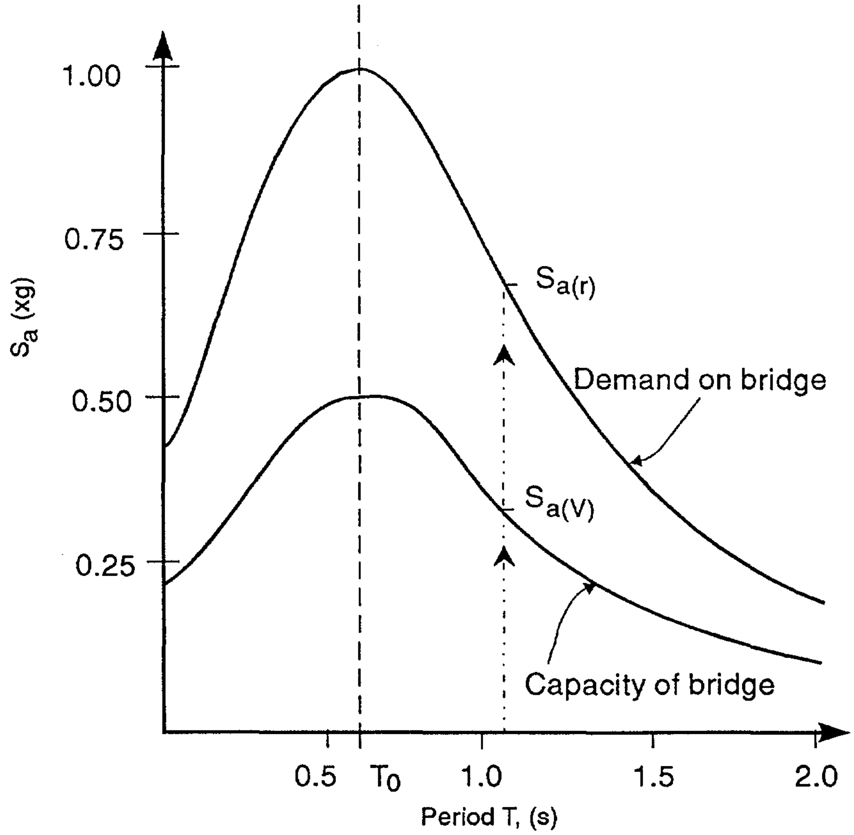
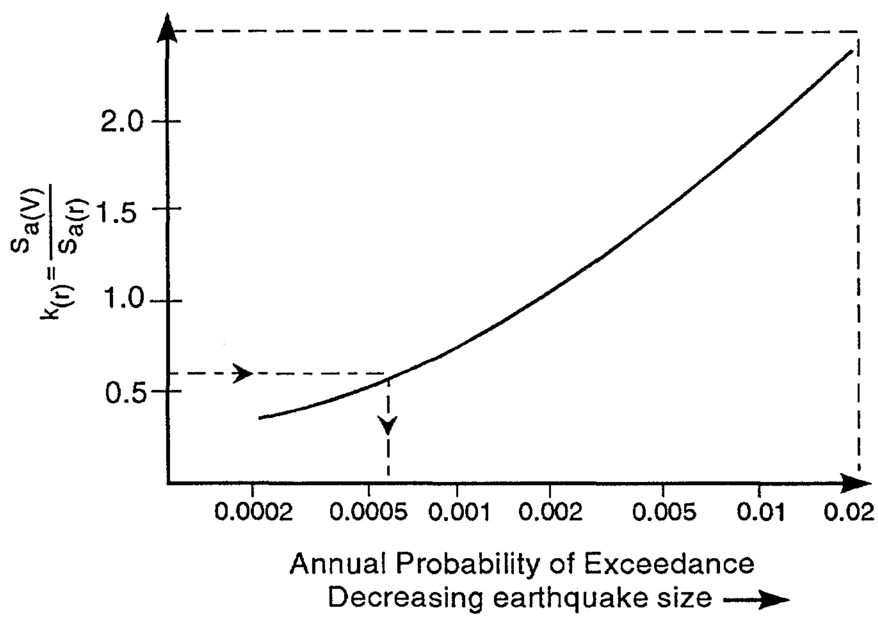


Figure 21. Equivalent elastic lateral strength (from ref. 18).



(a) Response Spectra



(b) Spectral Response vs. Probability of Exceedance

Figure 22. Determination of seismic risk from limit state spectral capacity (from ref. 18).

In figure 21, lateral force/deflection curves are established by consideration of sequential hinge formation and rotational capacity of plastic hinges. Figure 21(a) shows the curve appropriate for the formation of a single hinge, with equivalent yield strength V_f. The appropriate definition of V_f for a system with multiple hinges is shown in figure 21(b). In both cases, the equivalent elastic lateral strength is given by

$$V_E = R \cdot V_f \tag{3-16}$$

It will be recognized that R is equivalent to the force reduction factor commonly used in design, and is primarily related to the displacement ductility capacity $\mu = \Delta_u/\Delta_y$. For long-period structures, $R = \mu$ is appropriate, implying an "equal displacement" approach. For shorter-period structures, the "equal-energy" approach is appropriate, although this is still unconservative for very short-period structures. It is recommended that the expression:

$$R = 1 + 0.67(\mu - 1)\frac{T}{T_o} \le \mu \tag{3-17}$$

be adopted to assess equivalent elastic lateral strength, where T is the elastic fundamental period of vibration, and T_o is the period corresponding to peak spectral response for the site (see figure 22(a)). Equation 3-17 implies a gradual increase in R from $R = 1$ at $T = 0$ to $R = \mu$ for $T \ge 1.5T_o$, and produces a conservative envelope of the relationship between R, μ, and T. Note that the value of $R = 1$ at $T = 0$, independent of μ, is theoretically correct, since very stiff structures are subjected to the actual ground accelerations regardless of ductility level.

Figure 22 shows the relationship between the spectral ordinate (base shear coefficient) $S_{a(V)}$, corresponding to the equivalent elastic lateral strength V_E and the site assessment response spectrum $S_{a(r)}$, which has a known annual probability of exceedance, r (e.g., 10 percent probability of exceedance in 50 years). The relationship between $S_{a(V)}$ and $S_{a(r)}$ may be expressed as:

$$S_{a(V)} = k_{(r)} \cdot S_{a(r)} \tag{3-18}$$

If $S_{a(V)}$ can be identified for the critical collapse mechanism, as described above, then the associated probability of exceedance, r, may be found from the inversion of equation 3-18, i.e.,

$$k_{(r)} = \frac{S_{a(V)}}{S_{a(r)}} \tag{3-19}$$

and the relationship between $k_{(r)}$ and r, as illustrated in figure 22(b). This procedure has also recently been suggested for frame buildings.

Modified capacity design considerations must be employed to determine which of several alternative inelastic deformation mechanisms may develop. This is of considerable importance in assessment, since ductility capacity, and hence equivalent elastic lateral strength, are critically related to selection of the appropriate mechanism.

For example, initial calculations for strength of a critical member may indicate that shear strength exceeds flexural strength by 10 percent. Both shear and flexural strength are based on probable member strengths and "best-estimate" analyses. A ductile flexural mode is thus indicated, and it is supposed that the ductility capacity is assessed as $\mu = 3$, using procedures discussed in appendix B. Assuming, for simplicity, that $T \geq 1.5\ T_o$, the equivalent elastic lateral strength would thus be $V_E = 3\ V_f$, as shown in figure 23, where V_f is the strength of the flexural mechanism, indicated as line 1. However, the consequences of the flexural strength exceeding the probable value must be considered. It is conceivable that the longitudinal reinforcement has unusually high yield strength, and that the actual flexural strength exceeds $1.1\ V_f$. In this case, the shear strength will be reached before flexural strength is achieved, and the brittle failure characteristic represented in figure 23 by line 2 is predicted. The ductility of this brittle mechanism must be assessed as $\mu = 1$, and the equivalent elastic lateral strength is thus equal to the shear strength of $1.1\ V_f$. Thus, as a consequence of a small unanticipated increase in flexural strength, the equivalent elastic lateral strength has been decreased by 63 percent.

The above example is, of course, simplistic. A more realistic evaluation must consider the effects not only of excess yield strength, but of the possibility of strain hardening and the known fact that shear strength is not an independent parameter, but decreases with increasing flexural ductility. Further, the assumption of elasto-plastic flexural response is also unrealistic, implying strengths that are too high at low ductilities and too low (as a result of strain hardening) at high ductilities. A more realistic curve is shown in figure 23 by the dashed curve 3. The implication is that the equivalent lateral strength of $1.1\ V_f$ in the above example is excessively conservative, since the assessed flexural capacity will typically not be developed until displacement ductility factors of about $\mu = 2$ have been reached.

Although a precise quantification of all the effects discussed above is not warranted, or possible, in an assessment procedure, some approximate considerations can be made by bounding the effects of the uncertainties and evaluating their impact on the perceived risk.

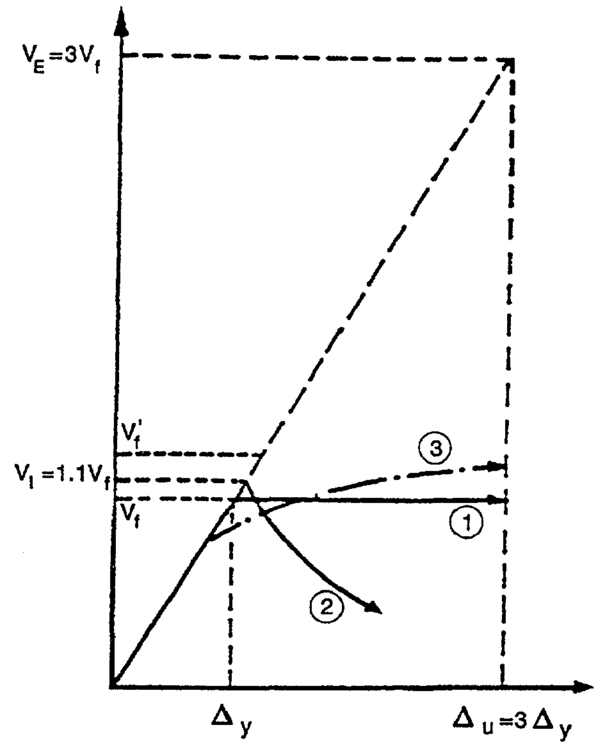


Figure 23. Equivalent elastic strength of alternative mechanisms (from ref. 18).

3.6 OVERALL ASSESSMENT AND POTENTIAL RETROFIT MEASURES

3.6.1 GENERAL

Since the C/D ratio method is likely to be the preferred method for bridge evaluation for the immediate future, the discussion here on the overall assessment and the selection of appropriate retrofit measures is directed towards the results of this method. If the lateral strength method is used, similar considerations will be necessary.

The C/D ratios calculated according to the procedures outlined in appendix A indicate the reduced load levels at which individual components may fail. The C/D ratios for the as-built condition of a bridge should be tabulated as shown in table 6. Values greater than unity indicate that the corresponding component is not likely to fail during the design earthquake, whereas values less than unity indicate a possible failure.

Beginning with the lowest C/D ratio, each value less than unity should be investigated to assess the consequences of local component failure on the overall performance of the bridge in order 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 C/D ratios should be calculated and tabulated as shown in table 6. 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 C/D ratio less than unity should be investigated in this way.

Table 6. 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 COLUMNS, WALLS, 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}	---	---	---

As noted previously, component C/D ratios below 1.0 indicate that a localized failure is likely to occur which may 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 C/D ratio less than 1.0, then retrofitting should be considered. In certain cases, it may not be economically feasible to retrofit all bridges or components that have substandard C/D ratios. In these cases, the values of the C/D 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 the bridge, replace it, retrofit to a lower standard, or accept the risk of seismic failure. A discussion of the philosophy behind retrofitting decision-making is given in section 1.6. The following discusses some items that should be considered in assessing the consequences of failure in various components.

3.6.2 ASSESSMENT OF BEARINGS AND EXPANSION JOINTS

3.6.2.1 Displacement

The C/D 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, 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 use 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.

3.6.2.2 Forces

A failure of bearing anchor bolts, keeper bars, or shear keys is usually not an unacceptable failure. However, 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 a facia or edge girder due to transverse movement may render a portion of the superstructure unusable, but may not result in a structure collapse, except possibly in a bridge with only two main girders or trusses. It may still be possible in most cases to use the remaining portion of the superstructure. In other cases, solid diaphragms 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.

3.6.3 ASSESSMENT OF COLUMNS, WALLS, 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 C/D 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 C/D ratios for each of these failure modes must be calculated when evaluating a bridge.

Column shear failure may occur suddenly and can potentially result in a collapse of the bridge, especially if the ability of the column to resist lateral loads is lost. In addition, a column which has failed in shear will rapidly degrade 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. Examples of such shear failures are shown in figures 24(a) and 24(b). In such cases, the potential for loss of life or serious injury exists. In addition, the use 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 can be 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 lose 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.

Figure 24(a). Column shear failure during the 1971 San Fernando earthquake (right).

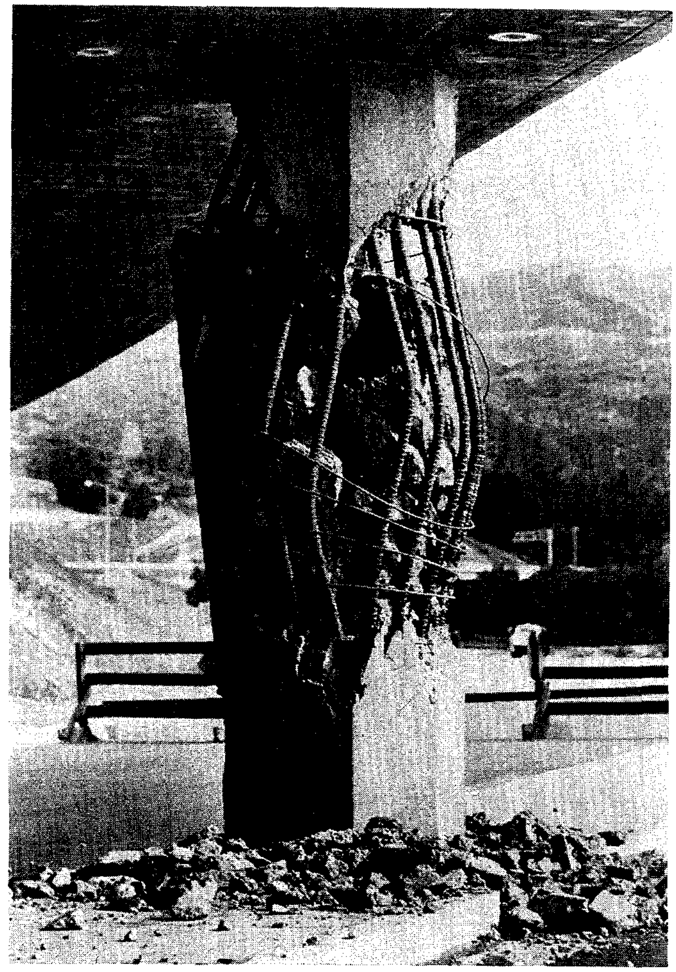


Figure 24(b). Collapse of the Cypress Street Viaduct due to shear failures in the connections between the upper and lower decks during the 1989 Loma Prieta earthquake (below).

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 be stable, even if flexural capacity is lost at the base of the columns. A collapse mechanism may occur if flexural capacity is also lost at the top of the columns or in the superstructure. In a similar structure on a straight alignment, the loss of longitudinal 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 torsional 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 the joints between simply supported spans and in-span hinges in continuous structures, will affect the overall stability of the bridge. If these 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 may 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 potential 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.

Flexural failures in footings have much the same effect on overall structure stability as do flexural failures in columns at the column/footing interface. The C/D 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 C/D 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 since 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 is 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 prevented.

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 the order of its 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 forced requires sophisticated analytical procedures which are not typically available to the engineer, this assessment will usually be dependent on engineering judgment.

3.6.4 ASSESSMENT OF ABUTMENTS

Other than the loss of girder support due to inadequate seat lengths, abutment failures seldom lead to structural collapse unless associated with liquefaction failures. Therefore, abutment failure is sometimes considered to be an acceptable risk when determining the need for seismic retrofitting. However, in the case of bridges that provide an essential function following an earthquake, the loss of accessibility resulting from an abutment failure may be unacceptable and, thus, retrofitting may be required.

It should be noted that whereas abutment failures seldom lead to structure collapse, abutments can usually be retrofitted economically. These improvements may have a favorable effect on the capacity/demand ratios for adjacent columns which in turn may reduce the cost of retrofitting these columns.

3.6.5 ASSESSMENT OF LIQUEFACTION

Liquefaction-related failure is 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. C/D 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 historical performance of various bridge systems that have been subjected to large displacements resulting from liquefaction.

The directions of relative ground movements at the various supports of bridges in past earthquakes are shown in figure 25. Movements at abutments of 0.6 m (2 ft) are common, with movements recorded as large as 2.4 m (8 ft). Such large relative movements often result in severe structural damage.

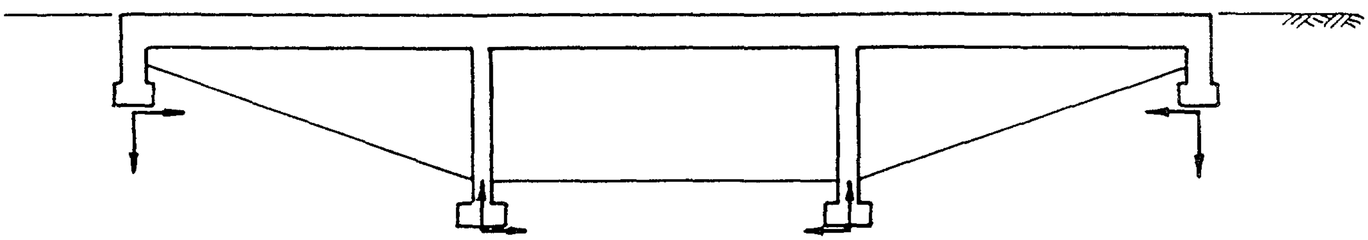


Figure 25. Relative foundation movements due to liquefaction.

The typical relative foundation movements shown in figure 25 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 about a horizontal axis. 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.

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 probability is high that the span will fall from the bent. If, as sometimes occurs, two expansion bearings exist at the same bent, the collapse of one of the two spans supported by the bent is almost assured. As might be expected, simply supported multispan structures have historically been the most susceptible to collapse due to liquefaction.

The final issue with regard to preventing liquefaction failures is the high cost of such retrofitting. In the case of severe liquefaction, the prevention of the loss of function may be extremely expensive and if this is the case, the importance of the bridge will be a major factor in determining the extent of the site remediation to be undertaken, if at all.

CHAPTER 4

SEISMIC RETROFITTING STRATEGIES

4.1 GENERAL

Following the 1971 San Fernando earthquake, the California Department of Transportation initiated a retrofitting program which was designed to reduce the occurrence of future bridge failures, and for more than a decade, retrofitting efforts were directed towards tying bridge superstructures together and preventing a loss of support at the bearings. However, the need to strengthen columns, footings, and abutments was also recognized and, by the late 1980's, serious attention was being given to column jacketing and foundation strengthening. The 1989 Loma Prieta earthquake accelerated this program not only in California, but also in Washington, Nevada, and some Eastern States. Caltrans has again taken the lead in the development and implementation of these retrofit strategies, and much of the material presented here and in succeeding chapters is drawn from Caltrans' experience.

The strategy to be used when selecting a retrofit measure should be one that reduces the probability of total collapse and/or severe structural damage to the bridge. In most instances it will not be practical or economically feasible to improve the level of performance to that of a new bridge, but there are cases (especially for critically important bridges) where a higher standard will be necessary. In some of these cases, this standard may even be higher than that for a new bridge in view of the fact that the current AASHTO performance criteria for new construction do not meet the requirements of some owners for certain bridge classifications.

In general, however, bridges in Seismic Performance Category B will only require 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. As described in chapter 3, an analysis of the existing structure is usually performed to identify deficiencies in the seismic resistance of the bridge. Retrofitting schemes which will either increase the capacity of one or more of its components, or reduce the demand on deficient components should be considered.

Several methods for retrofitting bridges have been proposed and used in practice. The use of expansion joint restrainers is the most popular and has proven to be an economical method of retrofitting, whereas measures such as column and liquefaction retrofit are, by comparison, more expensive. Chapters 5 through 9 summarize many of these concepts. Economic and practical considerations will also be important in the final selection of a retrofit scheme.

4.2 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:

- Loss of support at the bearings which will result in a partial or total collapse of the bridge.
- Excessive strength degradation of the supporting components.
- 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 performed as described in chapter 3.

As noted above, it is not always practical or economically feasible to retrofit a bridge to the same level of performance as a new bridge. It is therefore useful to identify three classes of retrofit which are related to the expected performance of the retrofitted bridge. These are as follows:

Class A retrofit:	Bridge is retrofitted to a standard higher than that required for new construction. This may be required for a bridge that is judged to be critically important.
Class B retrofit:	Bridge is retrofitted to the same standard as new construction. This is the ideal case but not always feasible for reasons of practicality or economics.
Class C retrofit:	Bridge is retrofitted to a standard that is less than that of new construction. This is less than ideal, but if the economics are sound, it is better than doing nothing.

Most retrofits conducted to date are of the Class C category. Once the cost of achieving a Class B retrofit for a given structure begins to exceed 60 percent of the cost of a new bridge, the economics begin to favor either replacement or dropping back to a Class C retrofit. However, if the structure is essential to the community, demolition and loss of use while reconstruction is undertaken may be unacceptable. It follows that loss of use due to an earthquake will also be unacceptable and a Class B, or even Class A, retrofit may be the only alternative.

4.3 RETROFIT STRATEGIES

4.3.1 GENERAL

There are two alternative strategies that a designer may adopt when faced with retrofitting a bridge. One is based on conventional strengthening techniques which increase the capacity of the structure to meet the likely demand. This is the most common approach. The other strategy is based on reducing the demand on the structure such that its existing capacity is sufficient to withstand the given earthquake. This latter approach involves the use of an earthquake protective system, such as seismic isolation, or the addition of a mechanical energy dissipation device. Although not applicable to all structures and all site conditions, retrofitting with a protective system has been shown to be a cost-effective alternative to conventional strengthening.

Retrofit measures based on conventional techniques are necessarily customized according to the component in need of strengthening. Potential measures are described in subsequent chapters, as noted below in section 4.3.2.

On the other hand, a protective system such as seismic isolation is not necessarily component-specific since the same methodology (design process and implementation) may be used to correct several deficiencies at the same time. The principles underlying the use of earthquake protective systems and their application to bridges are described in chapter 9, as noted below in section 4.3.3.

4.3.2 CONVENTIONAL RETROFIT MEASURES

Conventional retrofit measures for bearings, seats, and expansion joints are described in chapter 5. Measures for the retrofit of reinforced concrete substructures are presented in chapter 6. Foundations and difficult sites are covered in chapters 7 and 8, respectively. This material is intended to represent the current state-of-the-art, but the "art" is changing rapidly at this time. Some of the measures described in the following chapters may be discontinued in the next few years, whereas others, not even noted herein, may become standard practice. Various sources have been used to assemble this material and the principal ones include the 1983 FHWA retrofit guide, Caltrans' Memos to Designers, Caltrans Bridge Design Aids, and the NIST/UCSD report on guidelines for the assessment and retrofit of bridges.[3,15,16,17,18]

4.3.3 EARTHQUAKE PROTECTIVE SYSTEMS

The term "earthquake protective system" includes passive and active devices which are installed in a bridge to minimize the seismic demand on the members in a structure. At this time, active systems are considered to be to be attractive only for longer span structures. This is because the higher cost of active control and its required continual maintenance can be justified more easily for these larger structures. The longer spans are also more flexible and thus more suitable for control by active means. Since long-span structures are outside the scope of this manual, active control is not discussed herein. Instead, the emphasis is on passive systems which are now being used in several States as a cost-effective retrofit measure for many bridge types. Passive systems include mechanical devices, which simply dissipate energy and thus reduce response, and seismic isolation systems, which change the natural period of a bridge so that earthquake loads are significantly reduced. Seismic isolation concepts and options for design and implementation are given in chapter 9.

CHAPTER 5

RETROFIT MEASURES FOR BEARINGS, SEATS, AND EXPANSION JOINTS

5.1 GENERAL

Several bridges have failed during past earthquakes due to a loss of support at their bearings, seats, and/or expansion joints. Although frequently spectacular, these failures are also relatively simple and inexpensive to prevent by retrofitting. Because of this, most retrofitting efforts to date have been directed toward tying bridges together at their bearings and expansion joints. Several retrofitting methods have been used extensively, and these are discussed in this chapter.

It should be remembered that the use of an earthquake protective system, such as seismic isolation, may be an acceptable alternative to any of the conventional approaches described here. An overview of this option is given in section 4.3.3 and chapter 9. It should also be kept in mind that the optimum retrofit solution may well be a combination of both strategies. This will be particularly true whenever isolation cannot reasonably reduce the seismic demand to a level below the existing capacity and some strengthening is required to satisfy the demand. In these cases, the amount of strengthening required should be significantly less than if isolation had not been used, thus offsetting the costs of the isolators.

5.2 RESTRAINERS

Restrainers may be used for three different purposes when tying the various parts of a bridge together. These are as follows:

- Longitudinal joint restraint.
- Transverse bearing restraint.
- Vertical motion restraint.

These applications are discussed in sections 5.2.1 through 5.2.3, respectively. A method of equivalent static analysis is presented in section 5.2.4 and some practical design details are summarized in section 5.2.5.

5.2.1 LONGITUDINAL JOINT RESTRAINERS

5.2.1.1 General

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.

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 in-service movements at expansion joints. For joints located at piers, restrainers should provide a direct and positive tie between the superstructure and the pier, unless the 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.

5.2.1.2 Design Options and Issues

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 positions. These restrainers should have redundancy to allow for defects in the elements of the restrainer system.

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 26 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 the difficulty of access 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 27, 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 28. This arrangement should only be used, however, if movements parallel to the hinge that could shear the restrainers are minimal.

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 29. 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 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. Note that in figure 29 the restrainers are connected to the bottom flange. While this will prevent the possibility of tearing the web as mentioned earlier, it will also reduce vertical clearance under the bridge.

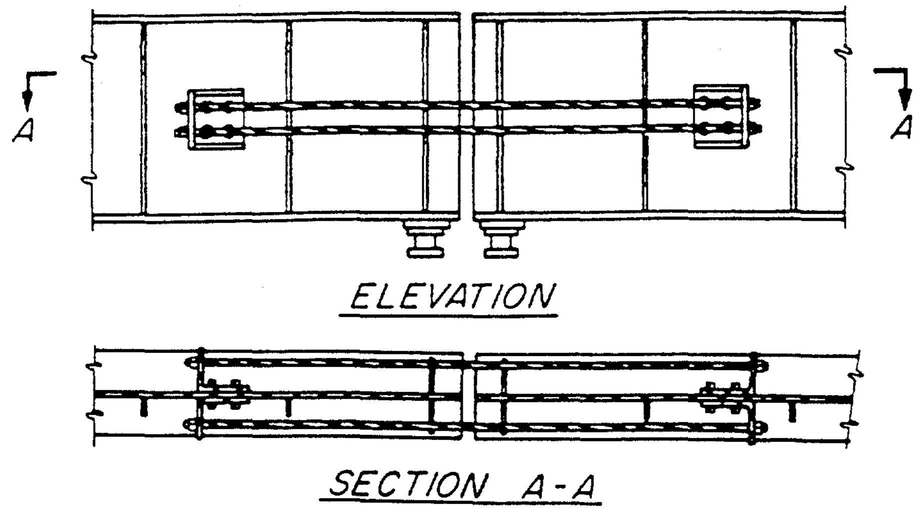


Figure 26. Undesirable restrainer detail.

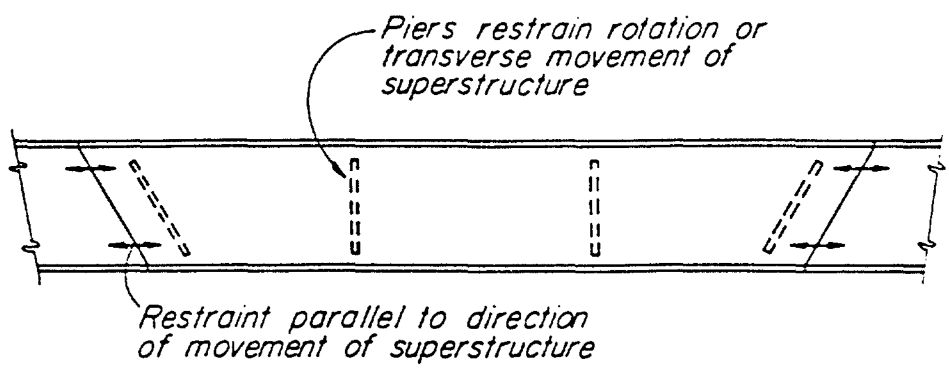


Figure 27. Restrainer orientation for transversely rigid supports.

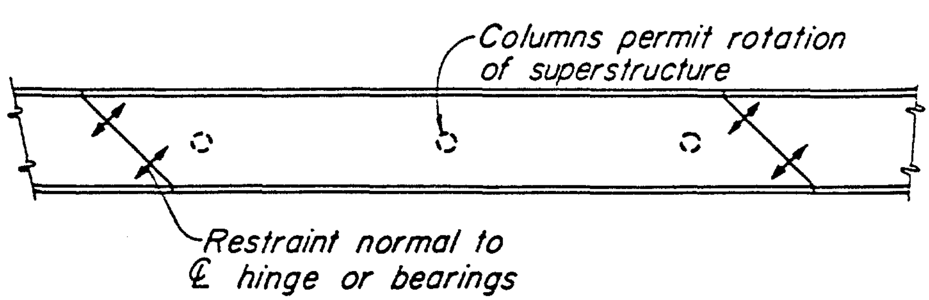


Figure 28. Restrainer orientation for transversely flexible supports.

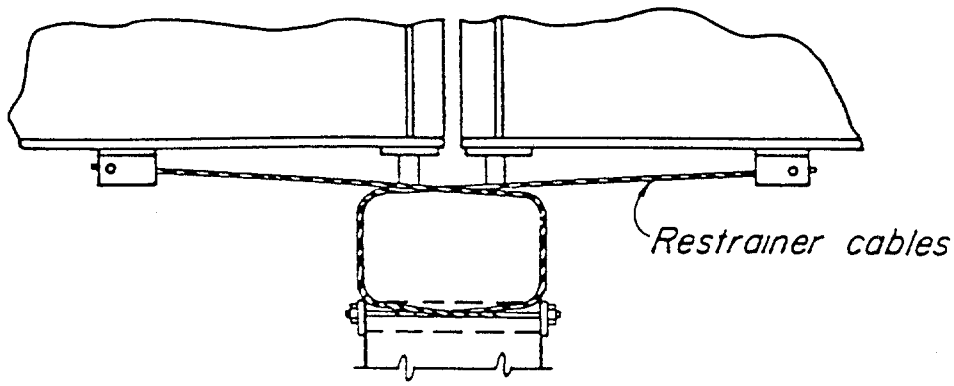


Figure 29. Restrainer at pier with a positive tie to pier.

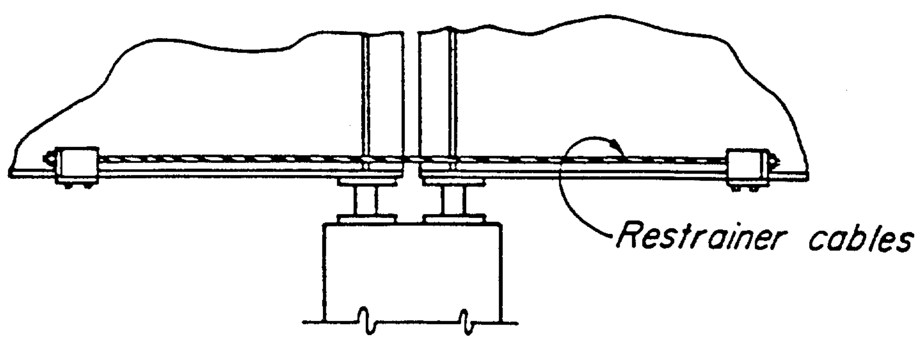


Figure 30. Restrainer at pier without positive tie to pier.

In some cases, it may be appropriate to forego the positive tie to the pier; in such cases, adjacent spans may be tied as shown in figure 30. This should be considered only when all of the following apply: (a) the cumulative openings of expansion joints are small enough to prevent the spans from becoming unseated; (b) positive ties could excessively overload the pier; and (c) 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 should 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. Caltrans has used two types of restrainer materials. The first type is 19 mm (0.75 in) 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 32 mm (1.25 in) diameter, 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 percent in 10 bar diameters before fracture.

Caltrans has 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 31 was developed by loading specimens to the specified yield stress (assumed to be $0.85f_u$ for the wire rope) for 15 cycles and then to failure. Notice that both materials are capable of elongating beyond the elastic limit. The 32 mm (1.25 in) diameter bars are stiffer, yielding at approximately 18 mm (0.7 in) of elongation over a 2.9 m (114 in) length. These bars are also more ductile and will continue to stretch to about 190 mm (7.4 in) 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 2.9 m (114 in) specimen will elongate approximately 38 mm (1.5 in) before yield. Total elongation after the initial conditioning is approximately 115 mm (4.5 in) prior to failure.

In a second series of tests, specimens were loaded by applying 25 mm (1 in) 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 32. It is interesting to note that 32 mm (1.25 in) diameter bars will withstand displacements up to about 280 mm (11 in), which is greater than that experienced in the first series of tests where loading conditions were different. The wire rope, on the

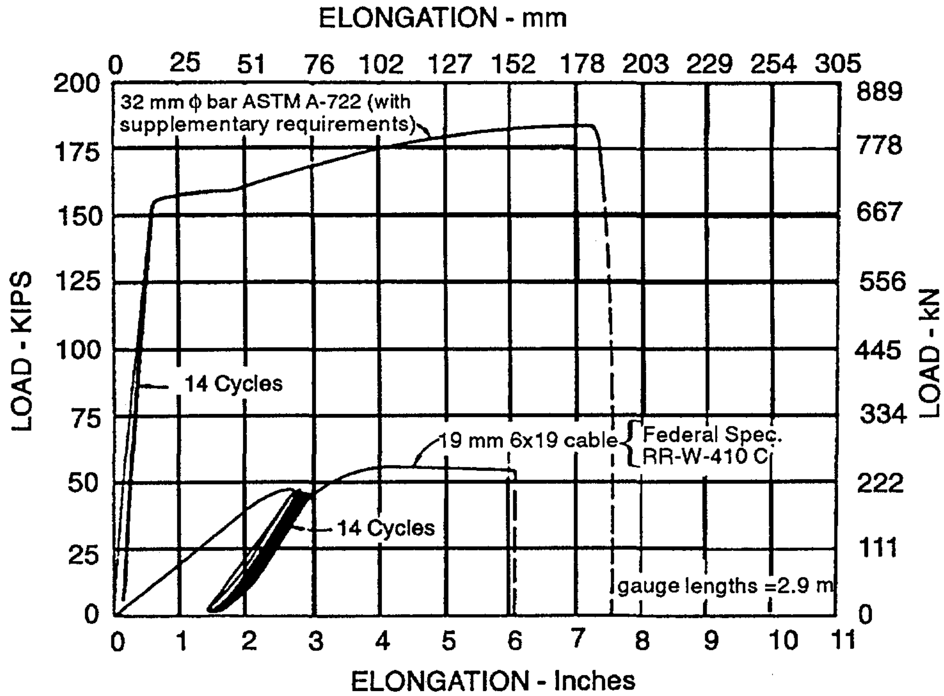


Figure 31. Restrainer material cyclic tests to yield.

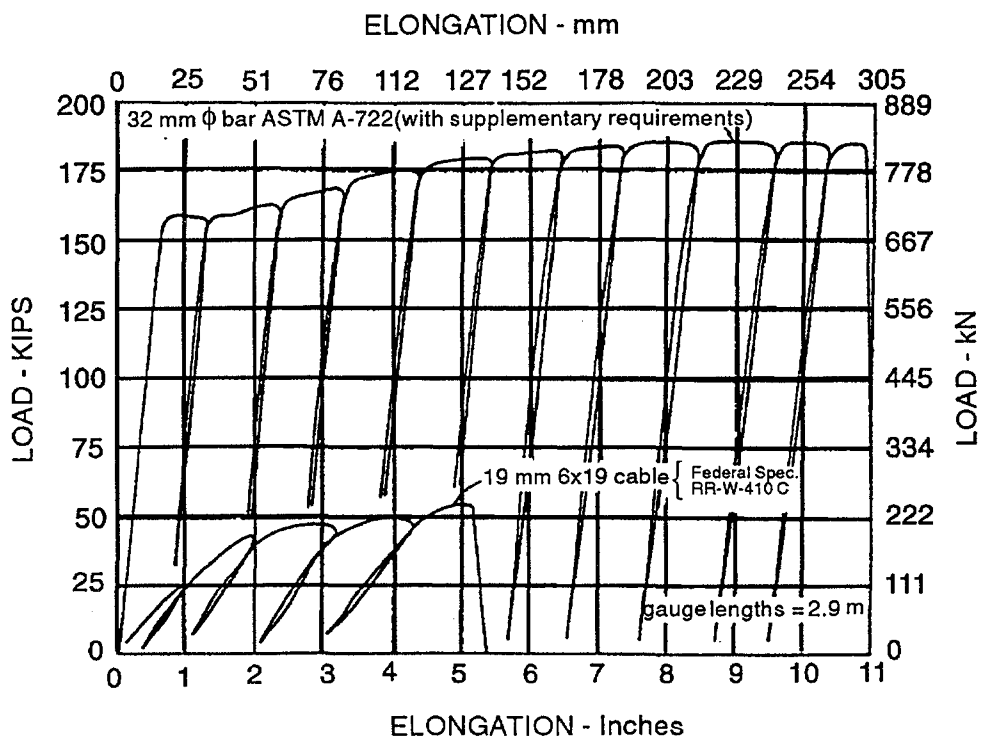


Figure 32. Restrainer material cyclic tests beyond yield.

other hand, fails at 127 mm (5 in) of elongation, which is slightly less than that demonstrated by the first series of tests.

Caltrans has no established rule as to when wire rope or bars are preferred. Since restrainers are designed to perform elastically, the extra ductility of the 32 mm (1.25 in) diameter 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.

Wire rope often has an economic advantage, since shorter lengths are possible 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 shear and flexural distortion in the bars.

Figure 33 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 diaphragms to accommodate the force transmitted from the restrainers.

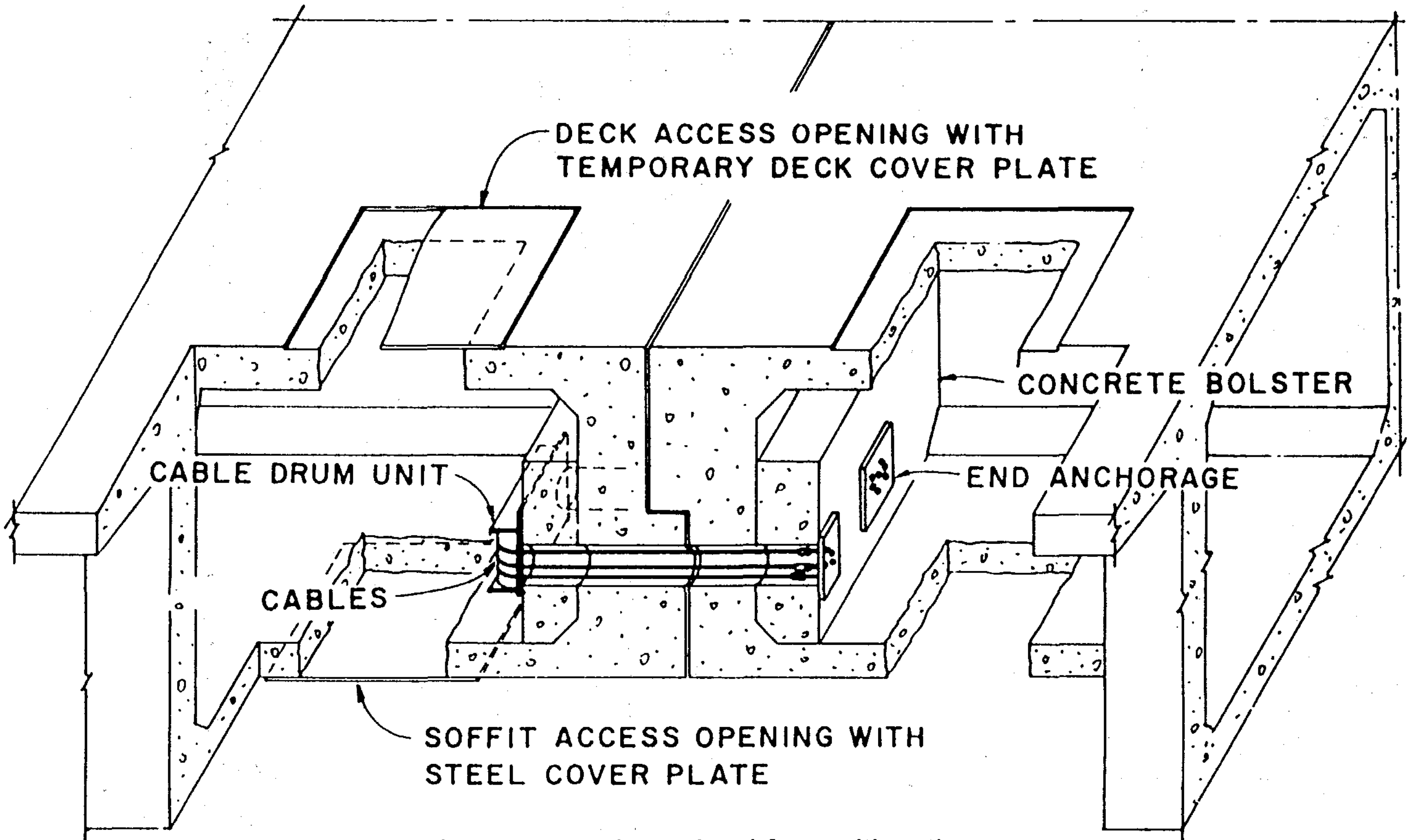
In open-web concrete bridges such as "T" beams, the lack of support at the bottom edge of the diaphragm may make it necessary to locate restrainers as shown in figure 34. This detail is usually restricted to situations where the restrainer force requirements are relatively low. When the joint is located 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 diaphragm 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 35. A direct tie to the bent is difficult when anchoring restrainers in this way.

Certain special situations permit some variation in the use of restrainer details. For example, figure 36 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 may be uneconomical because of the use of excessively long restrainers.

5.2.1.3 Design Forces

Restrainer forces and effective stiffness will generally be determined from an analysis of the structure. The elastic spectral analysis method (section 3.3.2.1) may



Note: Access may be gained from either the deck or soffit, but need not be both.

Figure 33. Longitudinal joint restrainer for concrete box girder.

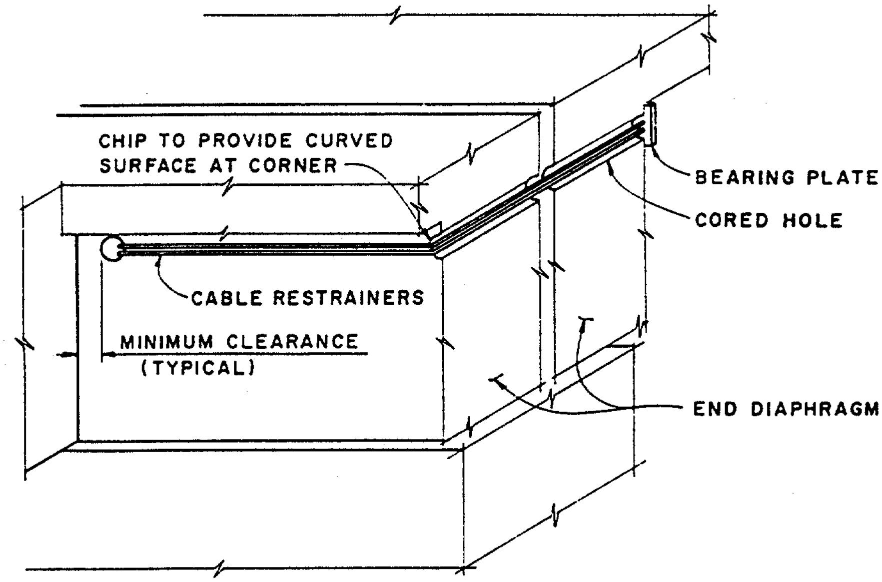


Figure 34. Expansion joint retrofit detail for concrete T-beam.

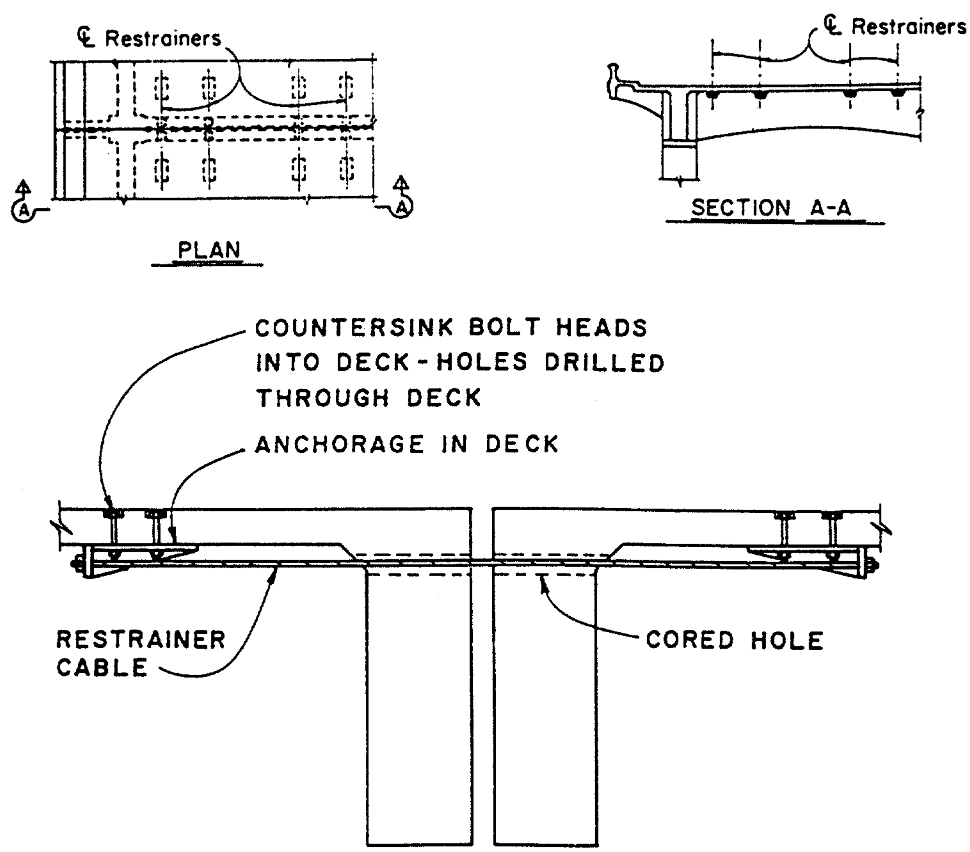


Figure 35. Expansion joint restrainers tied to the concrete deck.

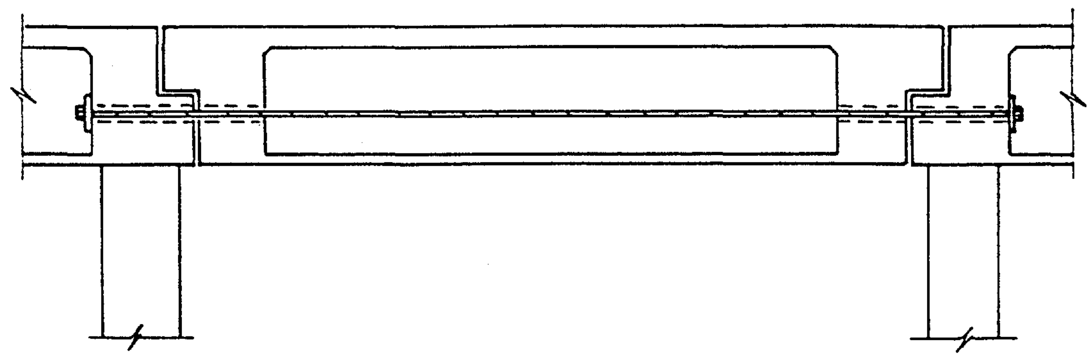


Figure 36. Restrainer retrofit of a suspended span.

be adapted to determine restrainer forces using tension and compression models or an approximate static analysis may be satisfactory in some instances (see section 5.2.4). In either case, design of the restrainer system usually follows the procedure outlined in section 5.2.4. In no case should the restrainer force capacity be less than that required to resist an equivalent horizontal static load of 0.35 times the dead load 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 required restrainer force capacity of 0.35 times the dead load 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 desirable to restrain joints having narrow bearing seats by using short, stiff restrainers designed to function below their yield 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 judgment in light of the several assumptions usually made in dynamic analysis. When higher forces seem appropriate, they should be used for design.

Connections of the restrainer to the superstructure or substructure should be capable of resisting 125 percent of the ultimate restrainer capacity. In addition, the existing structural elements subject to brittle failure should be capable of resisting 125 percent 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 percent of the nominal ultimate restrainer capacity.

5.2.2 TRANSVERSE BEARING RESTRAINERS

5.2.2.1 General

Transverse restrainers are necessary in many cases to keep the superstructure from sliding off its supports should the bearings fail in the transverse direction. Particularly vulnerable conditions exist when high concrete pedestals serve as bearing seats under individual girders, when bearing seats are narrow and highly skewed, and in two-girder bridges where the transverse distance between the bearing and the edge of the seat is small. Whenever transverse movement might lead to a loss of support, transverse restraint should be provided as a retrofit measure.

However, as discussed in section 3.6.2, transverse bearing failure alone does not always justify retrofitting. This is because total or even partial structural collapse is unlikely despite the failure. Nevertheless, retrofitting should not be ruled out, since it may be possible to prevent severe structure damage for very little cost.

5.2.2.2 Design Options and Issues

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 37. This design is based on bearing of the pipe against the walls of the cored hole. The full concrete compressive strength may be used in well-reinforced expansion joint diaphragms. However, care should be taken not to rely on the full strength of acute corners at highly skewed joints because they can easily break off.

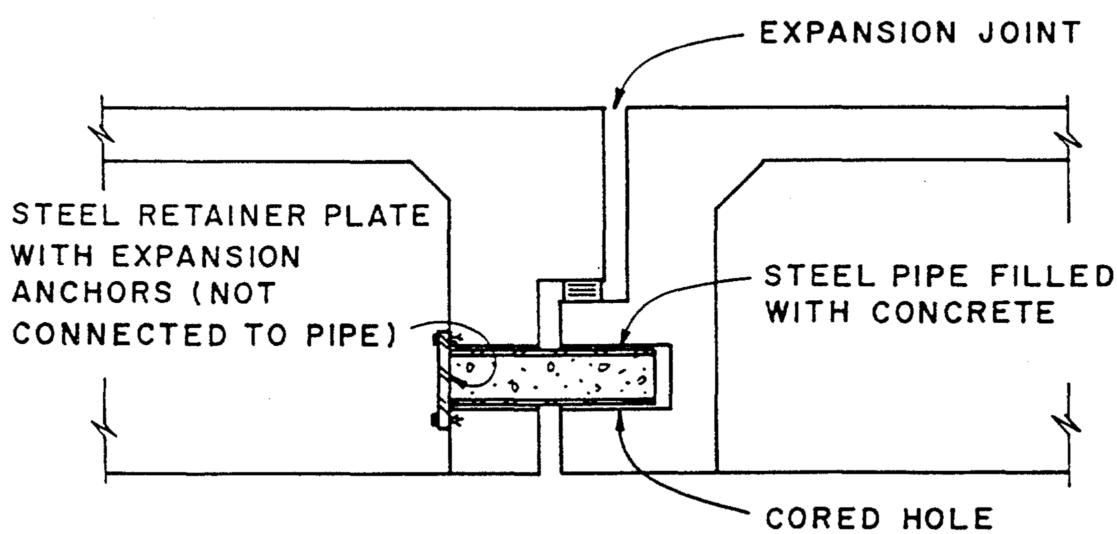


Figure 37. Transverse restrainer retrofit for a concrete bridge.

5.2.2.3 Design Forces

Transverse bearing restrainers are usually designed to resist load elastically. Analytical studies have shown that when columns yield, additional forces will be transferred to elements that are designed not to yield. In addition, installed transverse restrainers will have slightly different construction tolerances which will cause them to engage and resist load unevenly. To account for possible increased load due to these effects, the elastic forces from an analysis are increased by 25 percent for design.

The minimum transverse restrainer design capacity should be not less than that required to resist an equivalent horizontal static load of 0.35 times the superstructure dead load. 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.

5.2.3 VERTICAL MOTION RESTRAINERS

Vertical hold-down devices may be used at bearings to prevent uplift that, if free to occur, 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 forces due to Load Case 1 exceed 50 percent of the dead load 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. An example of a possible hold-down detail is shown in figure 38.

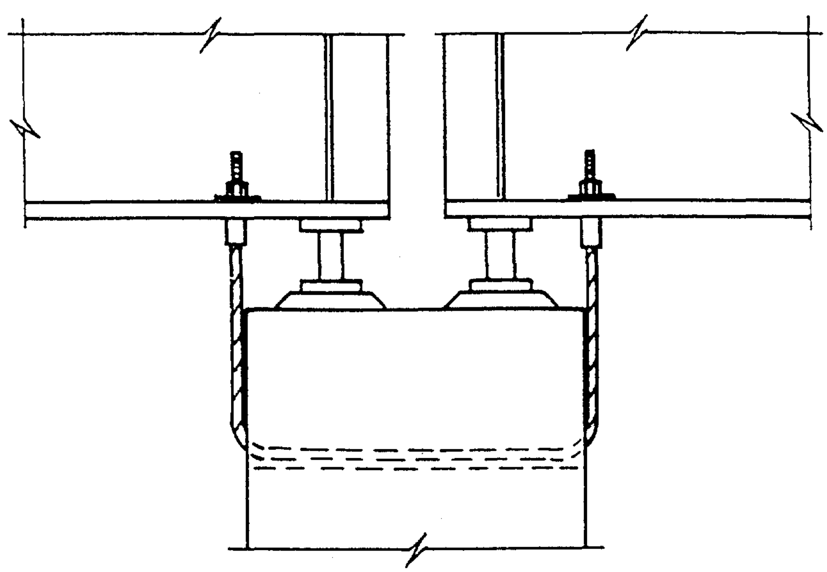


Figure 38. Vertical motion restrainer retrofit.

Vertical accelerations are not usually included in a seismic analysis, but if uplift is an issue, an analysis which includes the vertical component of input ground motion should be considered.

5.2.4 ANALYSIS

The response of a bridge that has been retrofitted with restrainers is nonlinear, even if the columns and foundations remain elastic. This is because the restrainers are initially slack and engage only after some movement has taken place. The restrainers are also essentially tension-only devices and are ineffective while the joints are closing. Furthermore, the impact which occurs at joint closure is a highly nonlinear problem that cannot be solved rigorously by simplified means.

In order to permit equivalent elastic solutions to be made, several assumptions are necessary and iterating through a succession of elastic analyses is required. Usually two dynamic models are used to bound the nonlinear response of the bridge: a "tension model" and a "compression model."[16] These two models are used because the bridge possesses different characteristics in tension compared to that in compression. As the bridge opens up at its joints, it pulls on the restrainers. In contrast, as the bridge closes at its joints, its superstructure elements go into compression. Typical modelling techniques based on the STRUDL finite element computer program are shown in figure 39.

In the tension model, the superstructure joint elements, including the abutments, are released longitudinally with truss elements to represent the restrainers, connecting them together at the joints (see figure 39). In the compression model, all of the restrainer elements are inactivated and the superstructure elements are locked longitudinally to capture the structural response in modes in which the superstructure tends to close up and go into compression, mobilizing the abutments if necessary.

The forces in the restrainers and their design may then be calculated based on the equivalent static analysis procedure provided in section 5.2.4.1.

5.2.4.1 Equivalent Static Analysis of Restrainers

To perform an equivalent static analysis for restrainer forces and deformations, the following definitions and assumptions are recommended by Caltrans:[17]

- A segment is defined as a portion of superstructure between expansion joints.
- Three separate analyses may be required to evaluate the restrainers at a particular joint, one each for the segment on either side of the joint and, for curved bridges, an evaluation of the joint opening from lateral earthquakes. 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 either case, the analysis which requires the fewer number of restrainers will govern.

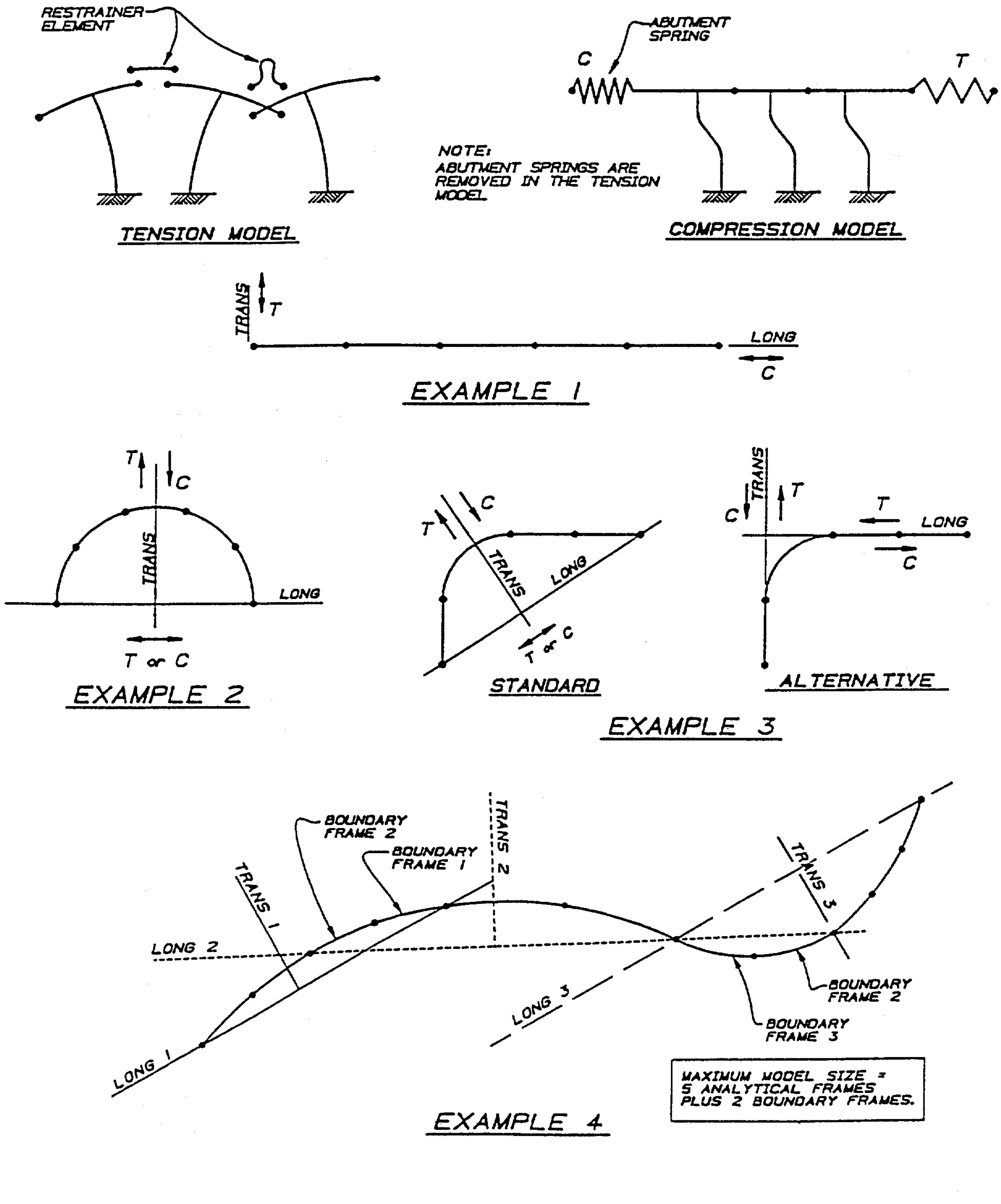


Figure 39. Examples of STRUDL modelling techniques.

- The mass to be used for computing the earthquake force shall be the mass of one segment adjacent to the joint under consideration.

- Assume one end of the restrainer is fixed and the other end is attached to the superstructure 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. This adjacent segment can only be mobilized when the gap between the segment under consideration and the adjacent segment is closed. If this joint gap 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 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 percent of the gap is compressed if the material is still in the joint. Many of these older joints have been cleaned or rebuilt and the material removed. A field inspection should be made to confirm the status of these joints.

- Multiple simple spans on bearings require an evaluation of the longitudinal adequacy of the bearings. If the bearings are not adequate to transfer the earthquake forces to the substructure, then only the restrainers can be used to determine 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 inadequate footings cannot be expected to develop large ductile forces. Whenever the applied earthquake moments exceed the nominal strength, 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 to 100 percent of the columns are damaged in this way, depending on how many columns are involved and how many inadequate details are involved. The presence of lap splices and the lack of top footing reinforcement generally increases the chances for damage at the bottom of existing columns. Of course, if the columns and footings are being (or have been) retrofitted, the actual column end conditions should be used in the restrainer analysis.

The overall procedure involves five steps as follows:

Step 1	Compute the maximum permissible restrainer deflection and limit deflection to the hinge seat width.
Step 2	Compute the maximum longitudinal earthquake deflections on both sides of the superstructure joint under consideration. For curved bridges, compute the joint opening resulting from a lateral earthquake.
Step 3	Compare the deflections from Steps 1 and 2 and determine the next course of action.
Step 4	Determine the number of restrainers required.
Step 5	Check the deflections of the restrained system and revise the restrainer and/or column assumptions if required. Repeat Steps 1 through 5 if necessary.

Detailed procedures for each step follow.

Step 1. Compute the maximum permissible restrainer deflection and compare to the hinge seat width.

Step 1a. Calculate the deflection capacity of a restrainer, D_r, as follows:

$$D_r = D_y + D_g \tag{5-1}$$

where D_r = maximum permissible restrainer deflection
D_y = restrainer deflection at yield (equation 5-2)
D_g = gap in the restrainer system

The yield deflection is given by:

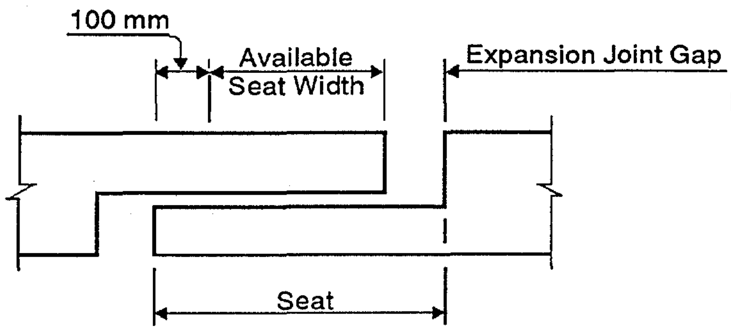
$$D_y = F_y L/E \tag{5-2}$$

where F_y = yield stress in restrainer
= 1200 MPa (175 ksi) for cables
= 825 MPa (120 ksi) for rods
L = restrainer length
E = initial modulus of elasticity of restrainer (before initial stretching)
= 69 000 MPa (10,000 ksi) for cables
= 207 000 MPa (30,000 ksi) for rods

The gap D_g is the clearance provided to accommodate thermal expansion and other nonseismic effects.

Step 1b. Compare the available hinge seat width with the maximum permissible restrainer deflection, D_r.

If the maximum permissible restrainer deflection, D_r, is greater than the available seat width (see figure 40), then the hinge could become unseated before the restrainer capacity is reached. In this case, either the seat width must be increased or D_r must be reduced by: (a) shortening the restrainers, (b) decreasing the restrainer gap, or (c) reducing the stress in the restrainers to a value less than yield.



Note: The 100 mm dimension shown provides for nominal "reasonable" allowable seat width. A larger or smaller dimension may be required. Expansion joint gap for new structures shall be the maximum estimated opening.

Figure 40. Example of Caltrans Bridge Design Aid seat detail (from ref. 17).

Step 2. Compute the maximum longitudinal earthquake deflections on both sides of the superstructure joint under consideration.

Step 2a. Compute the unrestrained system stiffness, K_u, 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 it 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.

Then K_u = unrestrained total system stiffness
= sum of stiffnesses (K) of contributing components

where K is as follows:

columns and piers	$K = 12EI/L^3$ for fixed-fixed ends
	$K = 3EI/L^3$ for fixed-pinned ends
	$K = 0$ for pinned-pinned ends
abutments	$K = 3.25W$ (m units) $= 200W$ (in units)
piles	$K = 7000$ kN/m/pile (40 k/in/pile)

where E = modulus of elasticity
I = moment of inertia
L = height of column or pier
W = normal bridge width

Note that the capacity of the abutment soils and all column and pier connections should be compared against the demand to determine if the assumed stiffnesses are appropriate. For example, the maximum force which can be transferred to the soil at an abutment is generally given by $368A_n$ ($7.7\ A_n$), where 368 (7.7) = maximum soil stress in kPa (ksf) and A_n = abutment area of mobilized soil (normal to span). This allowable soil pressure assumes an effective abutment depth of 2.4 m (8 ft). If the actual depth is D, the allowable pressure may be taken as $368(D/2.4)^2$ kPa ($7.7(D/8)^2$ ksf).

Also, if failure is expected in a column or pier, a reduced stiffness should be used to model the "failed" condition. It is not too unreasonable to assume that 50 percent of the columns or piers will be damaged in a large earthquake. It is also unlikely that all of the columns or piers will fail simultaneously. It is important to use the actual end conditions for the columns and to include the effects of substructure retrofitting, if any.

Simple spans on bearings will require a similar calculation of K_u. If failure of the bearings is expected from longitudinal forces, then the restraint offered by the substructure cannot be relied upon. In this case, the stiffness of the system will be entirely that of the restrainers.

Step 2b. Compute the longitudinal earthquake deflection for both of the segments adjacent to the joint under consideration. Assume that there are no restrainers in the system for this calculation (except for fully released segments such as simple spans). The larger deflection value is the controlling one.

Compute the longitudinal deflection:

$$D_{\ell} = ASW/TK_u \text{ (in)} \tag{5-3}$$

where D_{ℓ} = longitudinal earthquake deflection of the unrestrained system
A = acceleration coefficient (figures 1 and 2)
T = fundamental period of vibration, calculated using the weight of the segment and the unrestrained system stiffness (s)
S = site coefficient (from table 3)
W = weight of the segment (kN or k)
K_u = unrestrained system stiffness (kN/mm or k/in), from Step 2a.

For curved segments, first compute the transverse deflection at the joint due to a transverse earthquake using a separate analysis. Then compute the longitudinal opening at the joint due to this transverse deflection using:

$$G_h = D_t \frac{L}{2R} \tag{5-4}$$

where G_h = half-gap at ends of curved segment
D_t = transverse earthquake deflection of segment
L = length of segment
R = radius of segment

Equation 5-4 should not be used when L exceeds R; in such a case, a more exact relationship for G_h should be used.

For curved segments, the total gap opening at a joint from a transverse earthquake is obtained by adding the half-openings from the ends of the two adjacent segments:

$$G_t = G_{h1} + G_{h2} \tag{5-5}$$

where G_t = total gap opening due to a transverse earthquake
G_{h1}, G_{h2} = half-gap openings at joint from equation 5-4

Step 2c. Compute the maximum joint opening, D_{eq}, by combining the effects of the longitudinal and transverse loads.

$$\begin{aligned} D_{eq} &= \text{earthquake deflection of the unrestrained system} \\ &= \text{maximum of } D_\ell + 0.3(G_t) \text{ or } 0.3(D_\ell) + G_t \end{aligned} \tag{5-6}$$

Step 3. Compare the deflections from Steps 1 and 2 and determine the next course of action.

Compare the controlling earthquake deflection from Step 2c (one from each side of the joint) with the maximum permissible restrainer deflection from Step 1a. If D_{eq} is **less** than D_r, then only a minimum number of restrainers will be required. Provide at least two separate cable restrainer units (or equivalent) across the joint. Locate these units as close as practicable to the outside edges of the bridge. If D_{eq} is **greater** than D_r by a significant amount, 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, D_{eq}, to equal the restrainer capacity, D_r.

Step 4. **Determine the number of restrainers required.**

$$N_r = K_u(D_{eq} - D_r)/(F_yA_r) \tag{5-7}$$

where
N_r = number of restrainers required
K_u = unrestrained system stiffness from Step 2a
D_{eq} = maximum deflection due to earthquake forces from Step 2c (the maximum of the two values from each side of the joint should be used)
D_r = maximum restrainer deflection from Step 1a
F_y = yield stress = 1200 MPa (175 ksi) for cables, 825 MPa (120 ksi) for rods
A_r = area of one restrainer

19 mm (3/4 in) cables	=	143 mm^2 (0.222 in^2)
25 mm (1 in) rods	=	549 mm^2 (0.85 in^2)
32 mm (1-1/4 in) rods	=	807 mm^2 (1.25 in^2)
38 mm (1-1/2 in) rods	=	1020 mm^2 (1.58 in^2)

Step 5. **Check the deflection of the restrained system. Revise the restrainer and/or column assumptions if required.**

Step 5a. Determine the deflection of the restrained system. Use the maximum value obtained from equations 5-8a and 5-8b.

$$D_t = ASW/TK_t + 0.3\ (G_t) \tag{5-8a}$$

$$D_t = 0.3\ ASW/TK_t + G_t \tag{5-8b}$$

where
D_t = deflection of the restrained system
A = acceleration coefficient
T = fundamental period of vibration (s)
S = site coefficient
W = weight of the segment (kN or k)
K_t = total restrained system stiffness $K_u + K_r$ (kN/mm or k/in)
K_u = unrestrained system stiffness (kN/mm or k/in)
K_r = $F_y(N_r)A_r/D_r$
F_y = yield stress = 1200 MPa (175 ksi) for cables, 825 MPa (120 ksi) for rods
N_r = number of restrainers
A_r = area of one restrainer

19 mm (3/4 in) cables	=	143 mm^2 (0.222 in^2)
25 mm (1 in) rods	=	549 mm^2 (0.85 in^2)
32 mm (1-1/4 in) rods	=	807 mm^2 (1.25 in^2)
38 mm (1-1/2 in) rods	=	1020 mm^2 (1.58 in^2)

D_r = maximum restrainer deflection from Step 1a

Step 5b. Adjustment procedure.

If the deflection of the restrained system, D_t, is not equal to the permissible restrainer deflection, D_r, then an 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, D_t, should be verified to ensure that the initial assumptions are still valid. If not, the model must be adjusted and Steps 1 through 5 repeated.

If D_r is **greater** than D_t, the number of restrainers may be reduced. After reduction, the new restrainer configuration should be checked to ensure that D_r is not less than D_t.

If D_r is **less** than D_t, the number of restrainers should be increased. Steps 1 through 5 should be repeated until D_r is equal to or greater than D_t.

Note that two examples of this procedure are included in appendix E.

5.2.5 PRACTICAL DETAILS

5.2.5.1 Coring of Existing Concrete

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 75 mm (3 in). For holes larger than 150 mm (6 in), the edge of the hole may be flush against the adjacent surface. In addition, cored holes should be located so that a minimum of 1.2 m (4 ft) of clearance exists on at least one side along the centerline of the hole. These clearances are shown in figure 41.

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

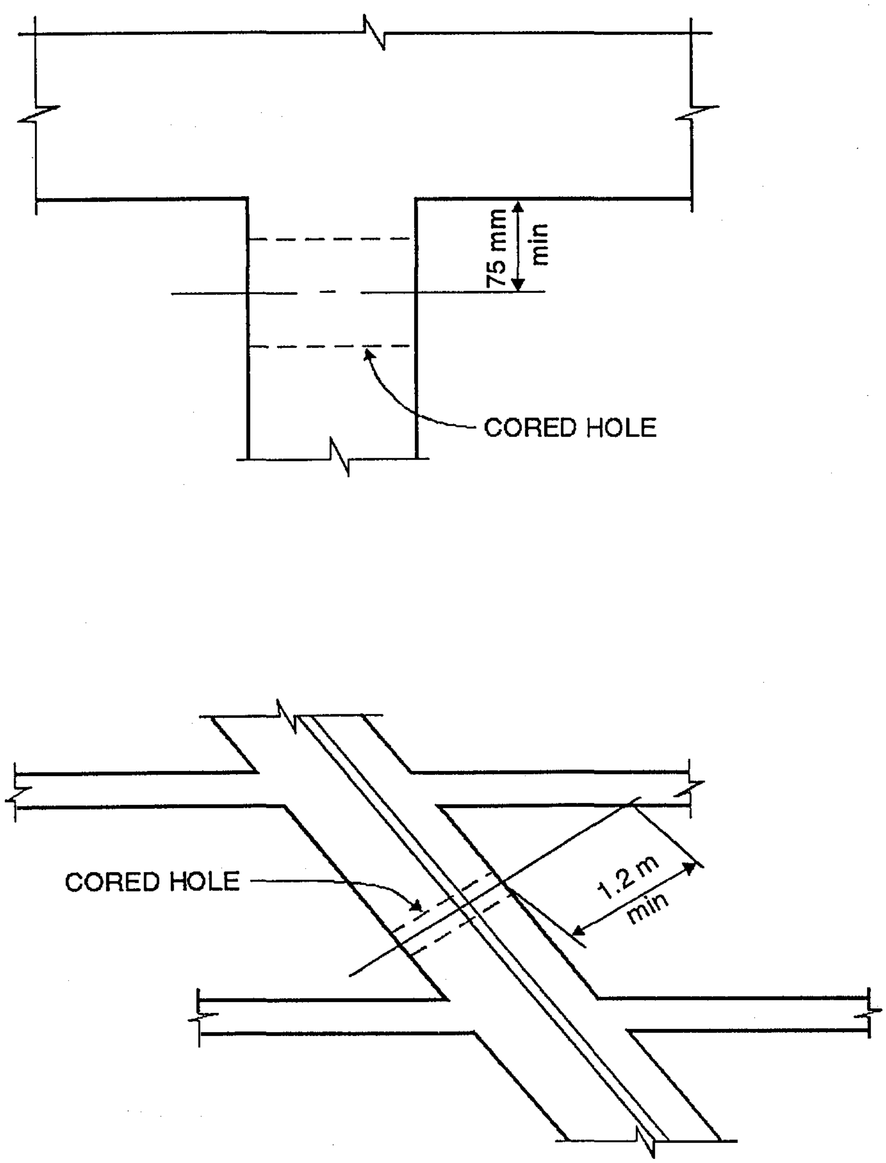


Figure 41. Required clearances for concrete coring.

5.2.5.2 Brackets and Bearing Plates

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 percent overstress in all 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 percent overstress. Concrete walls subject to a punching failure should be strengthened. This type of strengthening is often required at expansion joint diaphragms. Punching shear plate size can be selected from the chart in figure 42.

5.2.5.3 Restrainer Materials

There are two basic materials for restrainer applications. These are as follows:

<u>19 mm (3/4 in) cable</u>: Refer to Caltrans Standard Specifications, section 75-1.035 — Bridge Joint Restrainer Units, for a full description.(31) However, basic properties include:

Minimum ultimate tensile breaking strength = 205 kN (46 kips).
A_s = 143 mm^2 (0.222 in^2)
E = 96 500 MPa (14,000 ksi) (minimum specified before yielding)
= 124 100 MPa (18,000 ksi) (after initial stretching)

If Load Factor Design is being used, assume:

$$f_y = (0.85)(205) = 174 \text{ kN}$$
$$= (0.85)(46) = 39.1 \text{ kips}$$

<u>High strength bars, galvanized</u>: Refer to ASTM A-722 for supplementary requirements (the supplementary requirements specify a minimum elongation of 7 percent in 10 bar diameters). Basic properties include:

Diameter mm (in)	Area mm (in)	Ult. Strength MPa (ksi)	Yield Strength MPa (ksi)	Yield Force kN (kips)
25 (1)	549 (0.85)	1030 (150)	827 (120)	703 (102)
32 (1¼)	807 (1.25)	1030 (150)	827 (120)	1030 (150)
35 (1½)	1020 (1.58)	1030 (150)	827 (120)	1310 (190)

E = 206 850 MPa (30,000 ksi)

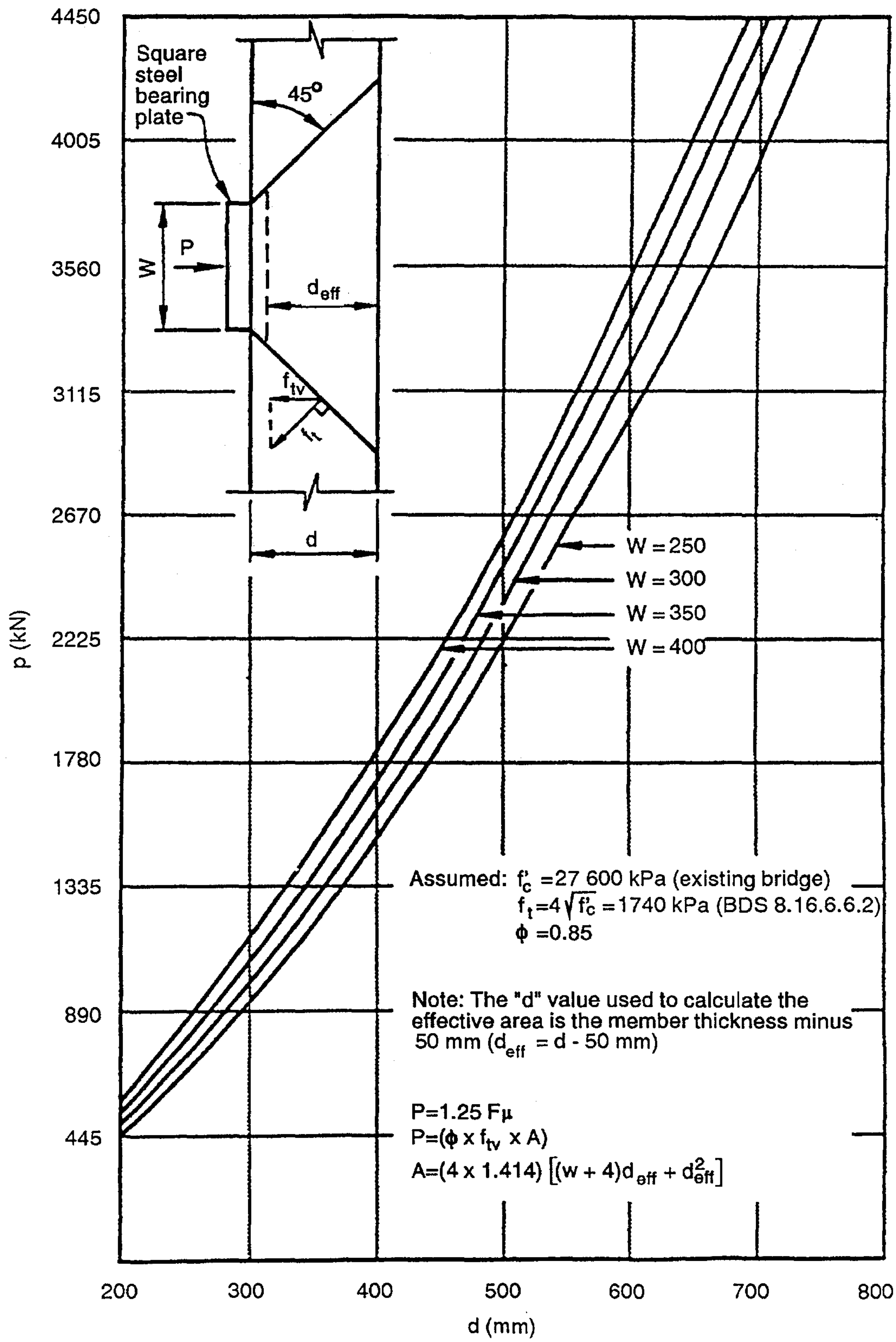


Figure 42. Resistance of concrete wall to punching (from ref. 15).

Note that galvanizing has sometimes caused field problems that have been related to the installation of high-strength rods. Two types of rods are typically used—threaded rods and smooth rods with threaded ends.

Threaded rods are galvanized after being threaded. Therefore, the rod ends must be hot-brushed immediately after galvanizing. Even after this operation, placement of end nuts is difficult. Smooth rods are usually threaded after being galvanized. After installation, the ends are coated with zinc-rich paint. Neither galvanizing nor threading compromise the strength of either type of rod nor the anchorage requirement of 90 percent of the minimum ultimate strength of the rod. If any damage to the galvanizing occurs, zinc-rich paint must be applied to the affected area.

Another area of concern is that standard locking devices are often not effective on threaded rods. Steps must be taken to prevent lock nuts from vibrating off such rods. Restrainer devices used in easily accessible areas should have bolt threads peened after installation to prevent loss of components to vandalism.

Rods longer than 9 m (30 ft) should be avoided. Stock lengths are 9 m (30 ft) and galvanizing tanks have difficulty with lengths greater than this.

5.3 BEARING SEAT EXTENSION

A bearing seat extension may be considered as a retrofit measure when it is impractical to restrain movement sufficiently to prevent loss of support at the bearings. If they can be installed at abutments, these extensions should be supported directly on the foundation, as shown in figure 43.

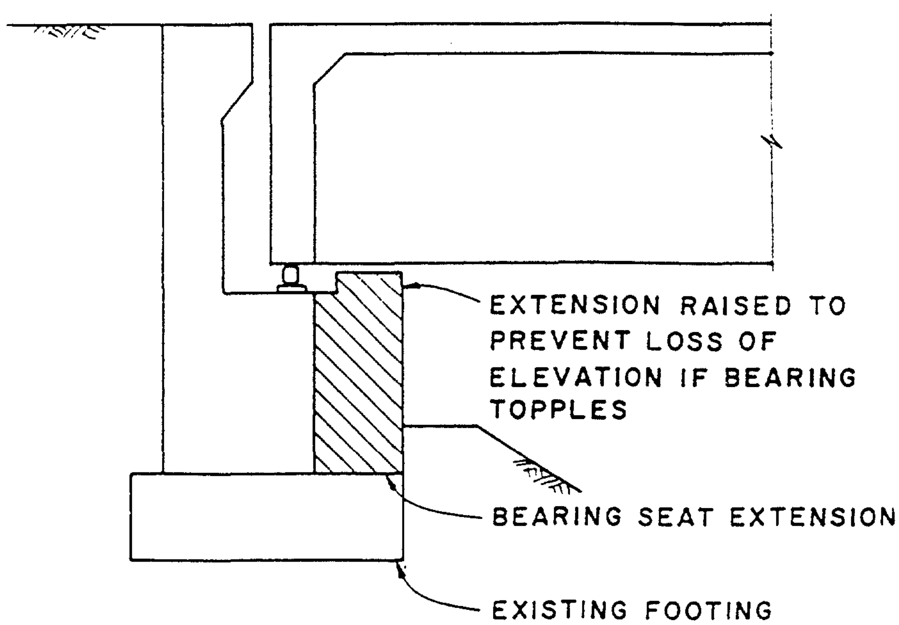


Figure 43. Seat extension at abutment.

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 falls 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.

Many older bearings are anchored by an oversized sole plate that is fastened below the bottom flange of the girder. There must be sufficient clearance for this plate to move freely above the raised portion of the extended seat and to avoid impact or interference with the seat extension.

If bearing seats are extended, their width should be increased to the minimum seat width recommended in section A.3. These recommended widths 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.

Since 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 either a vertical load of twice the dead load reaction plus the maximum live load reaction, or a vertical load equal to the dead load reaction in conjunction with horizontal load equal to the dead load reaction times the acceleration coefficient.

These design forces are representative of the large forces to which a bearing seat may be subjected during an earthquake that is large enough to cause the bearings to become unseated. Two loading conditions are therefore recommended. The first case 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 case 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.

For seats at in-span hinges, pipe extenders may be used to increase seatwidth capacity. A typical detail is shown in figure 44. Caltrans uses pipe extenders in combination with long ductile restrainers that have correspondingly large extension requirements. These are usually attached directly to the girders and are used when either the diaphragm capacity is too weak or the available space is too small to permit a more conventional restrainer (i.e., stiff) solution to be used to "lock" the hinge.

5.4 BEARING REPLACEMENT

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 were shown previously in figure 9(a). When these bearings are present, consideration should be given to replacing them.

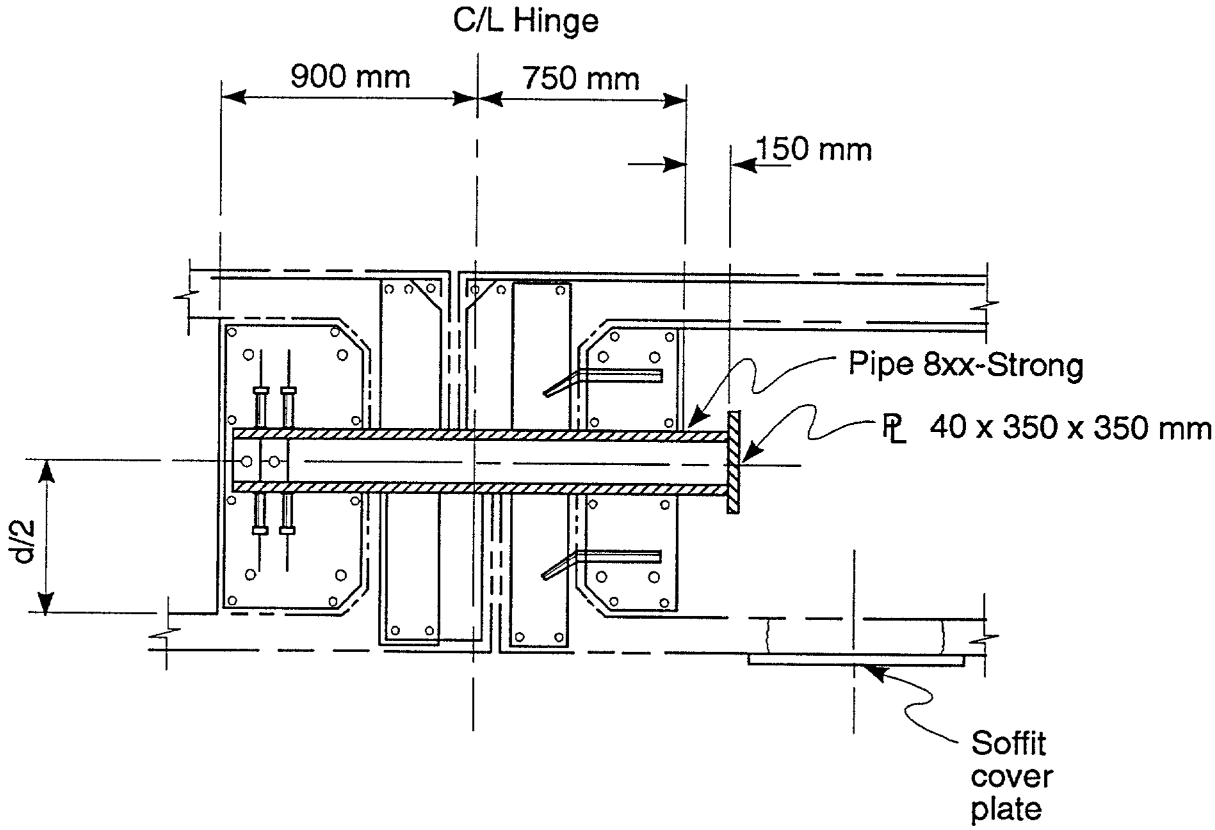


Figure 44. Pipe seat extension at in-span hinge.

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.

High rocker bearings may be replaced by a prefabricated steel bearing assembly and elastomeric bearing pads. The thickness of the steel bearing assembly is adjusted so as to maintain the proper elevation of the superstructure and to provide for the rotational and translational movement at the bearing. Some details for this retrofit scheme are shown in figure 45.

Another possible solution to replacing steel rocker bearings is shown in figure 46. In this case, a concrete cap is used to build up the elevation difference between a replacement elastomeric bearing and the original 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 into the new concrete cap.

At "fixed" bearings, it is often appropriate to completely embed existing rocker bearing pedestals in concrete as shown in figure 47. This will prevent shear failure

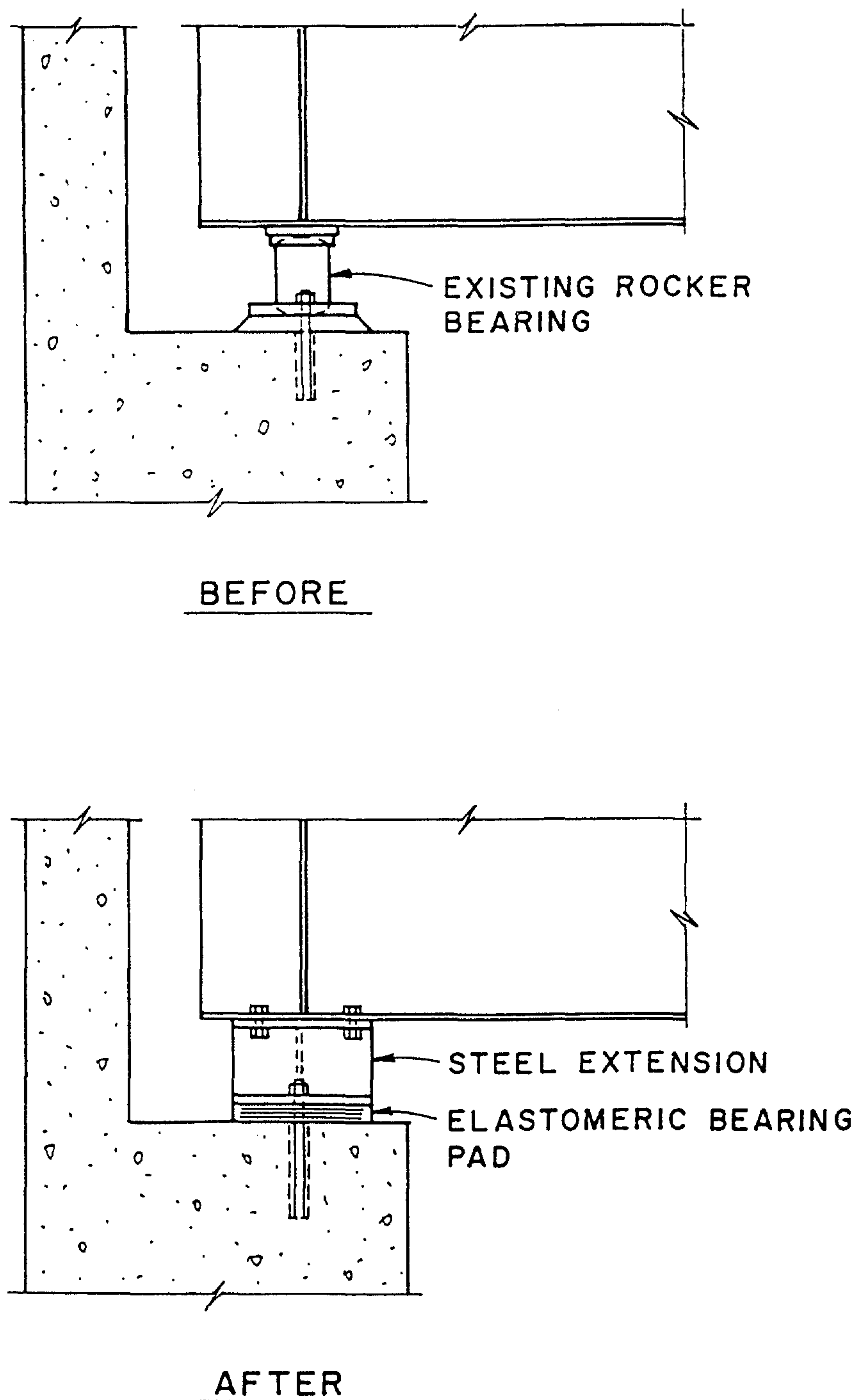


Figure 45. Replacement of rocker bearings using a steel extension.

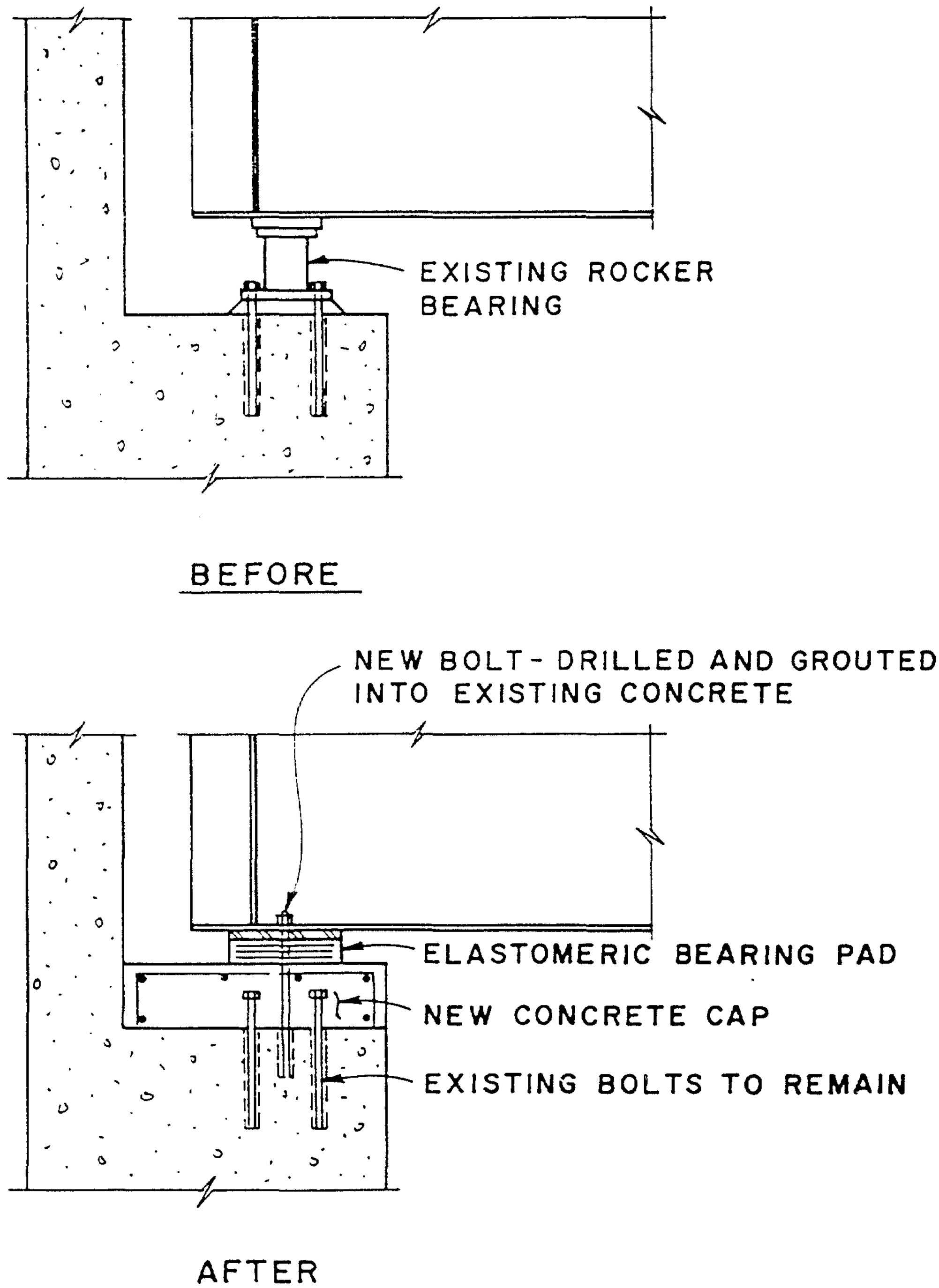
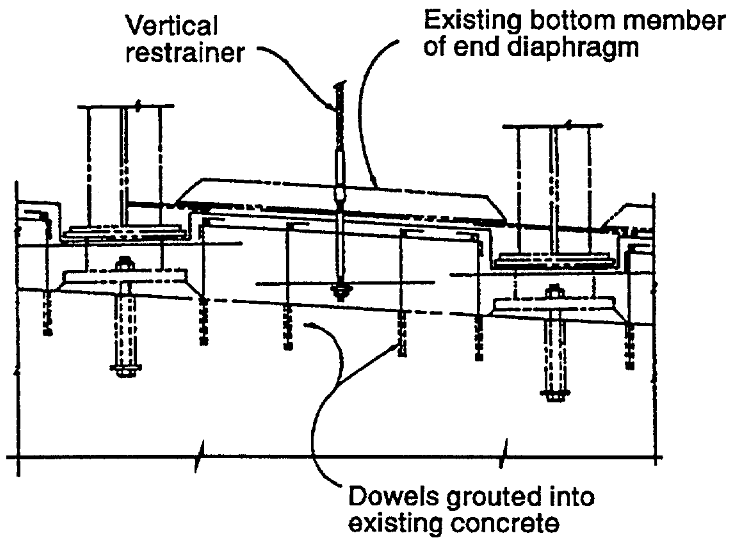
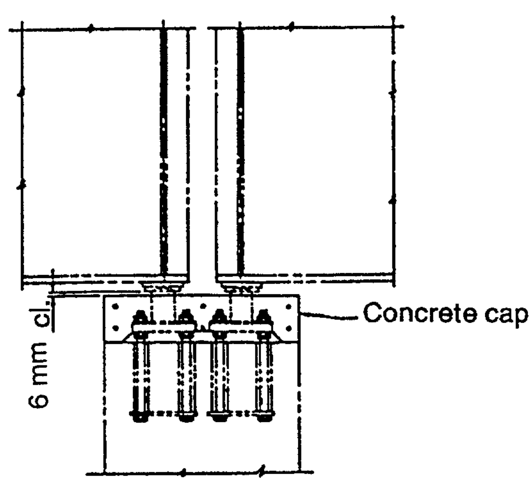


Figure 46. Replacement of rocker bearings using a concrete cap.



MODIFIED FIXED ROCKER BEARING
PART ELEVATION



MODIFIED FIXED ROCKER
BEARING AT PIER

Figure 47. "Fixed" bearing retrofit by embedment in concrete.

and toppling of the bearings. In addition, if spans become displaced from the bearings, the concrete cap will prevent collapse. Again, the concrete cap can double as a shear key and anchorage for vertical motion restrainers. It should be noted that, wherever possible, the replacement bearings at "expansion" and "fixed" ends of a girder should be of the same type so that the girder end rotations are similar and symmetry is preserved.

Replacement or strengthened bearings and their accompanying restraining components 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 A.2, may be used as the design forces.

Note that the capacity of the pedestal or seat supporting the replaced bearing should be checked against the forces that the new bearing can transmit. It may be necessary to provide an alternate bearing seat or support at the same time that the bearing is replaced.

CHAPTER 6

RETROFIT MEASURES FOR COLUMNS, CAP BEAMS, AND JOINTS

6.1 GENERAL

Retrofit measures for improving the seismic resistance of bridge substructures (columns, cap beams, and foundations) have been the subject of intensive research and development since the Loma Prieta earthquake in 1989. In particular, a large number of tests on "as-built" and retrofitted columns have been carried out over the past 4 years at the University of California, San Diego. This program has provided insight into the effectiveness of different retrofit measures to improve both the flexural and shear strength, and the flexural ductility of columns. Additional research is also in progress at the University of California on the Irvine, Davis, and Berkeley campuses, and in Washington State and elsewhere. Most of the research in California has been funded by Caltrans, although significant programs have also been funded by other States and the Federal Highway Administration.

As this research is still in comparative infancy, many of the conclusions and recommendations must be tentative in some areas. Although adequate test data exist to enable rather precise design methods to be developed for retrofitting columns for flexural and shear actions, data for retrofitted joints, footings, and anchorage problems are not currently available. Therefore, design recommendations must be extrapolated from rational analyses and the results of tests on similar building components (where these exist).

This chapter summarizes the available information (principally from reference 18) and indicates where the recommendations should be considered tentative at this stage.

It is to be remembered that the use of an earthquake protective system, such as seismic isolation, may be an acceptable alternative to any of the approaches described below. An overview of this option is given in section 4.3.3. It should also be kept in mind that the optimum retrofit solution may well be a combination of both strategies. This will be particularly true whenever isolation cannot reasonably reduce the seismic demand to a level below the existing column capacity and some strengthening is required to satisfy the demand. In these cases, the amount of strengthening required should be significantly less than if isolation had not been used.

6.2 RETROFIT MEASURES FOR CONCRETE COLUMNS

6.2.1 GENERAL

Concrete columns are commonly deficient in flexural ductility and shear strength. Deficiencies in flexural strength may also exist due to inadequate lap splices in critical regions or due to premature termination of longitudinal reinforcement. Where lap splices are not used, the flexural strength of columns in regular bridge structures is generally adequate as a result of conservative design assumptions inherent in the elastic design approach used before the 1970's.

A number of retrofit techniques have been successfully tested, and a rather smaller number have been implemented in the field. Column retrofit techniques include steel jacketing, active confinement by wire prestressing, containment by a composite fiberglass/epoxy semi-active confinement jacket, jacketing by reinforced concrete, and tensioning of individual hoops with turnbuckle arrangements. Of these, the steel jacket and composite fiberglass/epoxy jacket approaches have been used in retrofitting bridges in California and Nevada. Each technique is briefly described in the following section.

6.2.2 COLUMN RETROFITTING TECHNIQUES

6.2.2.1 Steel Jacketing

This technique was originally developed for circular columns. Two half-shells of steel plate rolled to the column radius with a 13 to 25 mm (0.5 to 1 in) clearance are positioned over the area to be retrofitted, and are site-welded up the vertical seams. The gap between the jacket and the column is grouted with a pure cement grout, after flushing with water. Typically, a space of about 50 mm (2 in) is provided between the jacket and any supporting member (footing or cap beam) to avoid the possibility of the jacket acting as compression reinforcing by bearing against the supporting member at large drift angles. Tests on steel-encased concrete piles, on which this concept is based, had indicated that significant flexural strength increased from this source, which could contribute to subsequent incapacity of the supporting member.

The jacket is effective in passive confinement. That is, lateral confining stress is induced in the concrete as it expands laterally in the compression zone as a function of high axial strain or, in the tension zone, as a function of dilating splice-induced vertical cracks, as a consequence of the hoop strength and stiffness of the jacket. A similar action occurs in resisting the lateral column dilation associated with the development of diagonal shear cracks. Thus, the jacket can be considered as an equivalent to continuous hoop reinforcement.

For rectangular columns, the recommended procedure is to use an oval jacket, which provides a continuous confining action similar to that for a circular column. The space between the jacket and column is filled with concrete. Rectangular columns so retrofitted have also performed exceptionally well in flexure and shear. Attempts to retrofit rectangular columns using rectangular jackets have been less successful, even when the jackets have been extensively stiffened. This is because the confining action of the rectangular jackets can only be developed as a consequence of lateral bending of the jacket sides, which is a very flexible action in comparison to the membrane tension action developed in an oval jacket.

It would appear that a rectangular steel jacket might be effective in enhancing the shear strength of a column. However, a column that has been retrofitted for shear must then also have adequate flexural ductility, which will not be provided by the rectangular steel jacket unless ductility demand is low.

Thin rectangular steel jackets bonded to concrete columns over regions of premature termination of reinforcement have been used in Japan to locally augment the flexural and shear capacity of the columns, ensuring that inelastic action occurs only at the plastic hinge at the column base.

6.2.2.2 Prestressed Wire Wrapping

An enhanced form of confinement may be achieved by wrapping prestressing wire under tension onto a column. The lateral confining stresses needed to increase flexural ductility are thus primarily provided by active pressure, rather than passive pressure resulting from column lateral expansion, though this latter influence will add to the confinement. This procedure has been shown to be successful in enhancing flexural ductility of columns with lapped splices at the critical section. Although it has not yet been tested for its ability to enhance shear strength, it is expected to perform well in this retrofit mode. Note that reliable anchorage of the wire ends and redundancy features are essential to a field application. Also, techniques for cost-effective wrapping of columns in the field are still unproven.

6.2.2.3 Composite Fiberglass/Epoxy Wrapping

A form of confinement consisting of a composite fiberglass/epoxy jacket has also been developed and tested. A degree of active confinement is achieved by pressure grouting between the jacket and the column, using grouting pressures as high as 1.7 MPa (250 psi). Because of the comparatively low modulus of the composite jacket material, little of this active pressure is expected to be lost by creep relaxation. However, there is no consensus on this point at this time, and this is one of several issues delaying widespread field application. Additional passive confinement is provided in critical regions, such as the bottom of columns, by unstressed fiberglass/epoxy wrapped over the active jacket. This system has been successful in enhancing the flexural ductility and shear strength of circular columns in the laboratory. However, there are several installation and durability issues which remain to be addressed before satisfactory field performance can be ensured.

6.2.2.4 Concrete Jacketing

The addition of a comparatively thick layer of reinforced concrete, in the form of a jacket around an existing deficient circular and rectangular column, has been developed recently in New Zealand. The application is comparatively straight-forward, though the adequate confinement of rectangular columns by a rectangular jacket requires extensive dowelling to connect the jacket to the existing column.

6.2.2.5 Other Techniques

Other potential techniques include the use of external hoops which are tensioned around the column using turnbuckles. Limited testing at the University of Washington indicates improved performance of lap-spliced starter bars. However, the lap splice ($35d_b$) was rather long in comparison to many splice lengths found in practice.

6.2.3 RETROFIT DESIGN CRITERIA FOR CIRCULAR COLUMNS

Extensive testing at the University of California at San Diego has enabled the following design criteria to be developed for three of the preceding techniques. These criteria are presented below according to the deficiency being addressed.

6.2.3.1 Flexural Integrity of Column-Base Lap Splices

It is indicated in appendix B that splice failure can be predicted by an assessment of the tensile stress capacity across a potential splitting failure surface (see figure 87 in appendix B). After cracking develops on this interface, splice failure can be inhibited, providing that adequate confining pressure is ensured without excessive dilation. Tests have indicated that the critical radial dilation strain, ε_d, is on the order of 0.001. From equation B-4, the requirement to ensure that splice failure will not occur is:

$$A_b f_s \leq 1.4\, f_\ell \left(\frac{\pi D'}{2n} + 2\,(d_b + c) \right) \ell_s \tag{6-1}$$

where d_b, A_b, and f_s are the diameter, area, and stress required to be developed of a typical spliced longitudinal bar; D' is the pitch circle diameter of the longitudinal reinforcement; n is the number of longitudinal bars; c is the cover to the longitudinal bar; ℓ_s is the splice length; and f_ℓ is the confining pressure that can be developed by the retrofit measure at a radial dilation strain of $\varepsilon_d = 0.001$. Equation 6-1 can be inverted to solve for the minimum effective required confining pressure f_ℓ. Since a steel stress of $f_s = 1.4f_y$ implies development of the bar ultimate strength, it is reasonable to assume that at ultimate:

$$f_\ell \geq \frac{A_b\, f_y}{\left(\frac{\pi D'}{2n} + 2\,(d_b + c) \right) \ell_s} \tag{6-2}$$

However, the splice length should be checked to ensure that it satisfies the minimum requirement of:

$$\ell_s \min \geq \frac{0.25\ d_b\ f_y}{\sqrt{f_c'}}\ (\text{MPa units}) = \frac{0.021\ d_b\ f_y}{\sqrt{f_c'}}\ (\text{psi units}) \tag{6-3}$$

as developed in appendix B.

The required confining pressure, f_ℓ, can be related to the characteristics of the retrofit concept by reference to figure 48, which shows free bodies of half-column sections.

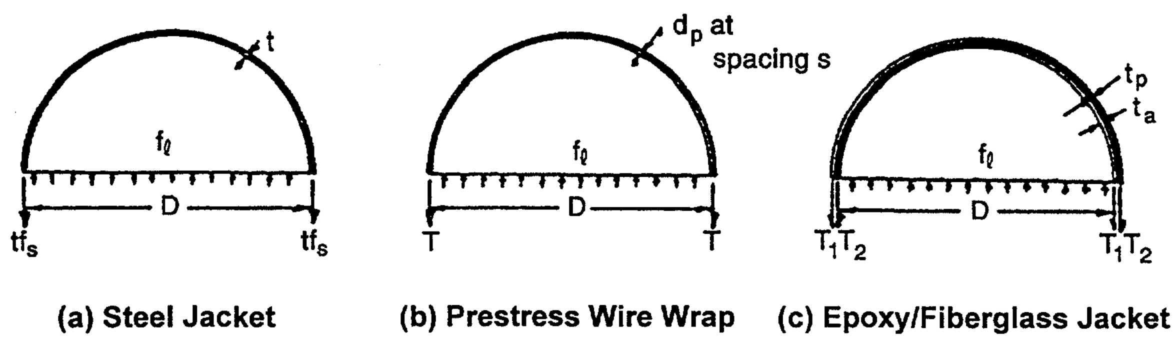


Figure 48. Confinement of circular columns (from ref. 18).

6.2.3.1(a) Steel Jackets

In figure 48(a), equilibrium requires that:

$$2t\ f_s = f_\ell\ D \tag{6-4}$$

where t is the steel jacket thickness; f_s is the stress induced in the jacket; and D is the diameter of the column (mm or in). Assuming the steel modulus of elasticity is E_s = 200 GPa (29,000 ksi), then at $\varepsilon_d = 0.001$, f_s = 200 MPa (29 ksi). Substituting into equation 6-4 and rearranging:

$$t \geq \frac{f_\ell D}{400}\ (\text{mm}) = \frac{f_\ell D}{58}\ (\text{in}) \tag{6-5}$$

where the confining pressure f_ℓ is expressed in MPa or ksi, as appropriate.

6.2.3.1(b) Prestressed Wire Wraps

Consider a column wrapped with wire of diameter d_p at spacing s, stressed to f_i MPa (ksi) after losses. Again assuming E_s = 200 GPa (29,000 ksi), equilibrium of figure 48(b) requires that $2T = f_\ell\ D$; i.e.,

$$\frac{2A_p}{s}(f_i + 200) = f_\ell D \text{ (MPa units)} \tag{6-6}$$

or

$$\frac{2A_p}{s}(f_i + 29) = f_\ell D \text{ (ksi units)}$$

Solving equation 6-6 for the required spacing gives:

$$s \leq \frac{2\,A_p\,(f_i + 200)}{f_\ell\,D} \text{ (MPa units)} = \frac{2\,A_p\,(f_i + 29)}{f_\ell\,D} \text{ (ksi units)} \tag{6-7}$$

where A_p is the wire cross-sectional area ($A_p = 0.785\ d_p^2$), and f_i and f_ℓ are in MPa (ksi) units.

6.2.3.1(c) Fiberglass/Epoxy Jackets[1]

The system in figure 48(c) consists of an active wrap of thickness t_a and modulus of elasticity E_a, stressed to produce an active confining stress of f_a in the column, and an additional passive wrap of thickness t_p and modulus of elasticity E_p. Both layers develop (additional) hoop stress as the jacket expands to $\varepsilon_d = 0.001$ strain. For equilibrium:

$$2(t_a\,E_a + t_p\,E_p)\,(0.001) = D\,(f_\ell - f_a)$$

i.e.,

$$t_a\,E_a + t_p\,E_p \geq 500\,D\,(f_\ell - f_a) \tag{6-8}$$

Equations 6-5, 6-7, or 6-8 can thus be used to design the minimum steel or fiberglass/ epoxy thicknesses or maximum wire spacing required to ensure that no splice failure occurs. A relaxation of the requirements could be allowed if slip is permitted at moderate ductilities. In this case, the maximum tension stress in the rebar could be set to 1.0 f_y, assuming no strain hardening of the longitudinal rebar. Bond slip will occur at moderate ductilities (typically about 3 to 5), but the constant confining stress will provide a rather ductile response with only gradual degradation of performance as a result of dependable friction across the displacing surfaces of the fracture plane. The required confining stress would then be 28.5 percent less than that given by equation 6-1.

6.2.3.2 Flexural Confinement of Column Plastic Hinges

In addition to inhibiting splice failure, the retrofit measure must impart adequate ductility to the column plastic hinge region. Where column bars are not

[1]The equations presented in this section for the design of fiberglass/epoxy jackets were developed by researchers at the University of California, San Diego. The reader should be aware that these expressions have not yet been endorsed or adopted by Caltrans.

spliced in the plastic hinge region, this will be the only design requirement for flexural action unless flexural capacity is inadequate. The procedure adopted is similar to that developed for ductility assessment of existing columns in section B.2. That is, displacement ductility is related to section curvature ductility by geometric considerations and expressions for ultimate compression strain of the concrete, e_{cu}, and an effective plastic hinge length, ℓ_p.

Tests on columns with plastic-hinge retrofits indicate that the plastic hinge length is condensed because of the clamping action of the retrofit measure, particularly if the plastic hinge contains lap-spliced longitudinal reinforcement. The plastic hinge length given by equation B-21 should be replaced by:

$$\ell_p = g + 2\chi\, d_b \tag{6-9}$$

where g is the gap between the retrofit measure and critical section (typically about 50 mm (2 in); d_b is the diameter of longitudinal reinforcement; and $\chi = 6$ for grade 40 rebar (f_y = 275 MPa or 40 ksi) and $\chi = 9$ for grade 60 rebar (f_y = 414 MPa or 60 ksi), as before.

Equation 6-9 has been calibrated to tests on steel-jacketed columns, both with and without lap splices, but is expected to be conservatively small for wire wrap or epoxy retrofits where there is no lap-spliced rebar in the plastic hinge region.

6.2.3.2(a) Steel Jackets

The ultimate compression strain of the concrete may be found using the energy balance approach of equation B-20. The effective volumetric ratio of confining steel for a steel jacket is:

$$\rho_s = 4t/D$$

Hence, equation B-20 can be rewritten as:

$$\varepsilon_{cu} \approx 0.004 + \frac{5.6t\ f_{yh}\ \varepsilon_{sm}}{D\ f'_{cc}} \tag{6-10}$$

where f'_{cc} can be related to the lateral confinement stress, f_ℓ, by the expression:

$$f'_{cc} = f'_c \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94\ f_\ell}{f'_c} - \frac{2\ f_\ell}{f'_c}} \right) \tag{6-11}$$

6.2.3.2(b) Wire Wraps

Taking account of differences in the shape of the stress–strain curves for prestressing steel and mild steel, equation B-20 can be rewritten as:

$$\varepsilon_{cu} = 0.004 + \frac{\rho_s\, f_{pu}\, \varepsilon_{su}}{f'_{cc}} \tag{6-12}$$

where:

$$\rho_s = \frac{4\, A_p}{Ds} \tag{6-13}$$

f_{pu} = ultimate stress for prestressing wire
ε_{su} = fracture strain for prestressing wire

and f'_{cc} is given by equation 6-11.

6.2.3.2(c) Fiberglass/Epoxy Jackets[2]

The stress–strain curve for the fiberglass material is essentially linear up to failure. Equation B-20 should thus be rewritten as:

$$\varepsilon_{cu} = 0.004 + \frac{0.75\, \rho_s\, f_u\, \varepsilon_u}{f'_{cc}} \tag{6-14}$$

where f_u is the ultimate strength of the jacket material and ε_u is the ultimate strain. The effective volumetric ratio is:

$$\rho_s = \frac{4(t_a + t_p)}{D} \tag{6-15}$$

If properties of active and passive wraps are very different, a weighted average for f_u and ε_u should be adopted.

6.2.3.3 Approximate Design Criteria for Flexural Retrofit

Where volumetric ratios of longitudinal reinforcement do not exceed 2.5 percent and axial load ratios are less than $P/f'_c A_g = 0.15$, tests indicate that the following approximate approach will provide satisfactory performance:

(a) lap splices

If $f_\ell \geq 2$ MPa (300 psi) at $\varepsilon_d = 0.001$, equations 6-5, 6-7, and 6-8 may be rewritten as

---

[2]The equations presented in this section for the design of fiberglass/epoxy jackets were developed by researchers at the University of California, San Diego. The reader should be aware that these expressions have not yet been endorsed or adopted by Caltrans.

for steel jackets: $t \geq D/200$ (mm or in) (6-16)

for wire wraps:

$$s \leq \frac{A_p\,(f_i + 200)}{D} \text{ (MPa units)} = \frac{6.7\,A_p\,(f_i + 29)}{D} \text{ (ksi units)} \quad (6\text{-}17)$$

for fiberglass/epoxy jackets[3]:

$$t_a\,E_a + t_p\,E_p \geq 500D(2 - f_a) \text{ (MPa units)} = 500D(0.3 - f_a) \text{ (ksi units)} \quad (6\text{-}18)$$

(b) flexural confinement (no lap splices)

If $f_\ell \geq 2$ MPa (300 psi) at $\varepsilon_d = 0.004$ (which for a steel jacket exceeds the yield strain), then the following may be derived from equation 6-4:

$$\text{for steel jackets: } t \geq \frac{D}{f_{yi}} \text{ (MPa units)} = \frac{0.15D}{f_{yi}} \text{ (ksi units)} \quad (6\text{-}19)$$

where f_{yi} = the jacket yield stress in MPa (ksi)

for wire wraps:

$$s \leq \frac{0.77 A_p f_{pu}}{D} \text{ (MPa units)} = \frac{5.3\,A_p\,f_{pu}}{D} \text{ (ksi units)} \quad (6\text{-}20)$$

where f_{pu} is the ultimate stress for the prestressing wire and recognizing that a passive strain of 0.004 should be sufficient to place the wire on the nonlinear portion of the stress-strain curve provided moderate levels of active stress are used.

for fiberglass/epoxy wraps[3]:

$$t_a\,E_a + t_p\,E_p \geq 125D(2 - f_a) \text{ (MPa units)} = 125D(0.3 - f_a) \text{ (ksi units)} \quad (6\text{-}21)$$

Laboratory tests indicate that columns satisfying these requirements, and the reinforcement and axial load limitations defined above, will be capable of sustaining drift angles of 4 percent with an adequate reserve of displacement capacity.

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[3]The equations presented in this section for the design of fiberglass/epoxy jackets were developed by researchers at the University of California, San Diego. The reader should be aware that these expressions have not yet been endorsed or adopted by Caltrans.

6.2.3.4 Shear Strength

The shear resistance of a column retrofitted by one of the three methods described above can be found from the approach described in section B.4, with suitable enhancement of the truss mechanism strength.

6.2.3.4(a) Steel Jackets

The shear resistance of a passive steel jacket may be found by analogy to a hoop or spiral reinforcement. The jacket may be considered equivalent to a spiral of bar area A_v at spacing $s = A_v/t$. The correct formulation for additional shear capacity V_{sj} is thus, by comparison with equation B-24:

$$V_{sj} = \frac{\pi}{2} t f_{yj} D \cot\theta \tag{6-22}$$

where f_{yj} is the jacket yield stress. Again, tests indicate that a value of $\theta = 30°$ can be taken for columns, provided longitudinal reinforcement is not terminated in the length of column encompassed by the 30° failure plane.

6.2.3.4(b) Wire Wraps

In similar fashion to the above, the expected increase in shear strength may be written:

$$V_{sp} = \frac{\pi}{2} \frac{A_{ps} f_{ps} D}{s} \cot\theta$$

The value of f_{ps} could be conservatively taken as prestress after losses, but the passive stress increase should also be included. Again, conservatively putting $f_{ps} = 0.8 f_{pu}$:

$$V_{sp} = \frac{\pi}{2} \frac{A_{ps} (0.8 f_{pu}) D}{s} \cot\theta \tag{6-23}$$

It should be noted that columns wrapped with prestressed wire have not yet been tested for shear strength enhancement.

6.2.3.4(c) Fiberglass/Epoxy Jackets[4]

Following the same approach as above, the enhancement in column shear strength will be:

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[4]The equations presented in this section for the design of fiberglass/epoxy jackets were developed by researchers at the University of California, San Diego. The reader should be aware that these expressions have not yet been endorsed or adopted by Caltrans.

$$V_{sp} = \frac{\pi}{2}\left((t_a E_a + t_p E_p)\varepsilon_p + T_a\right) D \cot\theta \qquad (6\text{-}24)$$

where

$$T_a = f_a D/2 \qquad (6\text{-}25)$$

is the jacket active tension force per unit height due to active pressure f_a. In tests, the maximum allowable passive strain from shear force was set at $\varepsilon_p = 0.006$, and $\theta = 30^\circ$ was assumed. Columns so designed performed very well, developing ductile flexural response to the limits of travel of the actuator applying the lateral displacement.

6.2.3.5 Extent of Column Retrofit

Flexural retrofit measures should extend from the critical section to that where the moment has decreased to 75 percent of the maximum moment, but not less than an extent equal to the column diameter. The higher level of confinement required for lap splices needs to be provided only over the length of the lap splice.

When retrofit to enhance shear is required, other than in the plastic hinge region, it will generally be necessary to retrofit the full column height.

6.2.3.6 Effect of Retrofit Measures on Column Stiffness

Column elastic (cracked-section) stiffness may be affected by the type and extent of retrofit adopted. For a steel jacket retrofit, tests have indicated the following average increases in stiffness

flexural retrofit (partial height): 10 to 15 percent increase

shear retrofit (full height): 30 percent increase

Prestress wire wrap and fiberglass/epoxy jacket retrofits have a negligible influence on the column stiffness.

6.2.4 RETROFIT DESIGN CRITERIA FOR RECTANGULAR COLUMNS

The principles developed above can be readily extended to rectangular columns. However, at this time, only the oval-steel jacket method of retrofit has been successfully tested. As mentioned above, steel jackets have proven very satisfactory in inhibiting splice failures, providing flexural ductility, and enhancing shear strength. Until test results are available, it is recommended that alternative procedures not be used.

Rectangular columns generally have greater stiffness, and thus greater ductility demand, in the transverse direction. It is normal to provide a jacket with elliptical shape. As a consequence, the curvature is continuously variable.

The equation of the jacket circumference may be expressed as:

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1 \tag{6-26}$$

The extreme radii in the two principal directions are:

$$R_1 = \frac{b^2}{a} \quad , \quad R_2 = \frac{a^2}{b} \tag{6-27}$$

The design equations for flexural integrity and ductility capacity may be adapted from section 6.2.3 for circular columns using an average radius over the extent of the compression zone. A reasonable approximation to this could be obtained by taking the average of the jacket radius at the column section corner and at the principal axis under consideration. With reference to figure 49, for the x direction use:

$$R = \frac{R_1 + R_2}{2}$$

and for the y direction use:

$$R = \frac{R_3 + R_2}{2}$$

The appropriate equations from section 6.2.3 can now be used, substituting D = 2R.

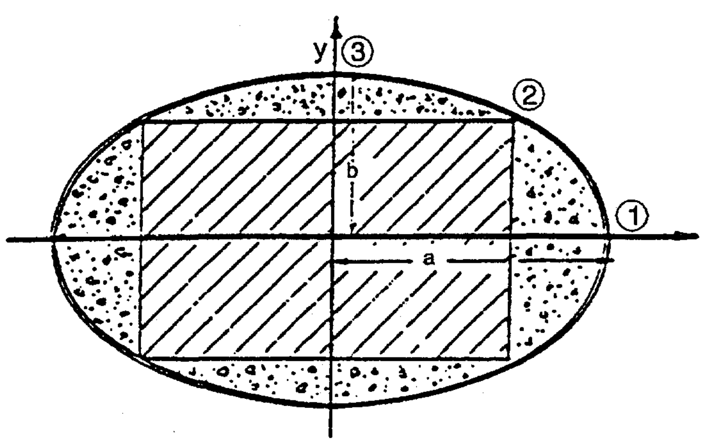


Figure 49. Elliptical jacket column retrofit (from ref. 18).

Note that the effective confining pressure will thus be less in the "weak" direction of the column, and may thus govern design. In piers with high plan aspect ratios it may be almost impossible to obtain adequate confinement parallel to the short direction. However, it will frequently be found that realistic assessment of displacement capacity in the longitudinal direction will indicate that no retrofit is necessary. In this case, design of the jacket can be dictated by the requirements of the strong (transverse) direction.

Shear strength enhancement will be similar to that provided for circular columns. In the strong direction, the shear enhancement is given by

$$V_{sj} = 4ta\ f_{yj}\left(1 - \frac{b}{a} + \frac{\pi b}{4a}\right) \tag{6-28a}$$

and in the weak direction,

$$V_{sj} = 4tb\ f_{yj}\left(1 - \frac{a}{b} + \frac{\pi a}{4b}\right) \tag{6-28b}$$

Equations 6-28a and 6-28b are approximate only, and become conservative in the weak direction for $a/b \geq 1.5$.

6.3 RETROFIT MEASURES FOR CAP BEAMS

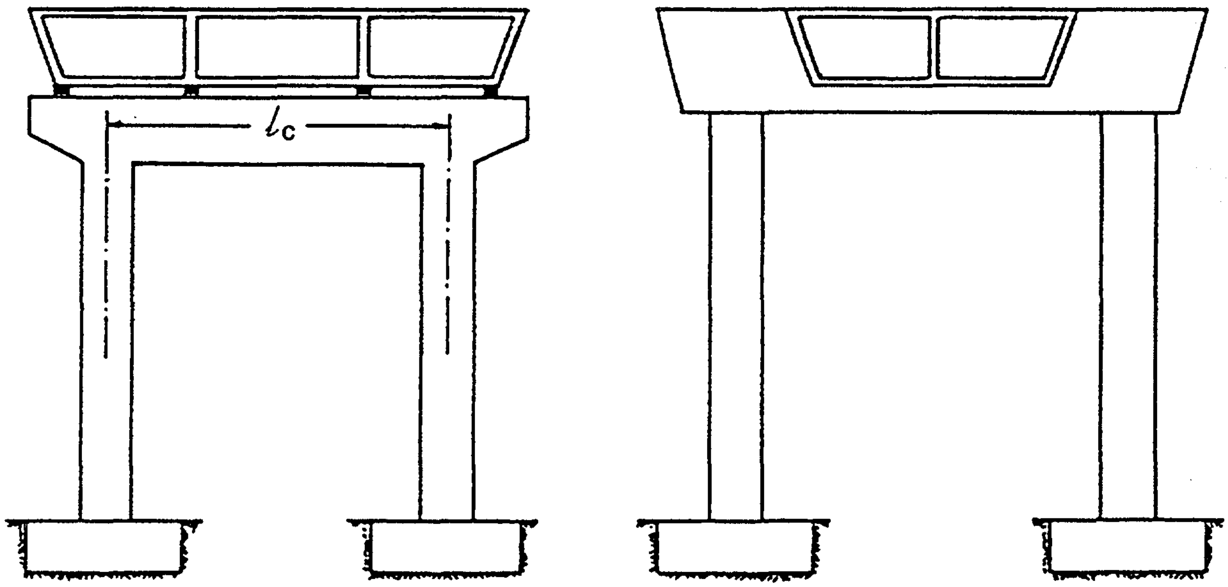
6.3.1 GENERAL

Cap beams provide the link in force transfer between the superstructure and columns. Under transverse seismic response, cap beams of multi-column bents will be subjected to flexure, shear, and joint shear. Deficiencies are common in all three areas. Under longitudinal response, cap beams supporting superstructures via bearings (figure 50(a)) are unlikely to have problems, but monolithic superstructure/cap beam/column designs (figure 50(b)) may exhibit cap beam torsional problems. Following the Loma Prieta earthquake in 1989, cap beam deficiencies were consistently the most serious and the most difficult to retrofit of all the damaged components.

6.3.2 FLEXURAL STRENGTH AND DUCTILITY

Typically, the flexural strength of a cap beam will be found to be less than that of columns framing into the cap beam. This is particularly the case for positive cap beam moment (tension on the soffit), as a result of little and inadequately anchored bottom reinforcement passing into the joint region. Negative moment capacity may also be inadequate to force plastic hinging into columns, particularly when the top reinforcement is prematurely terminated, as is often the case. Both cases are a consequence of the working stress design approach typically used for older bridges, which were designed for full dead load, but a reduced seismic load.

Generally the retrofit philosophy should be to increase the cap beam flexural strength sufficiently to force plastic hinging into the columns. With a separate cap beam supporting the superstructure via bearings, flexural enhancement can be achieved by adding reinforced bolsters to the sides after roughening the interface.



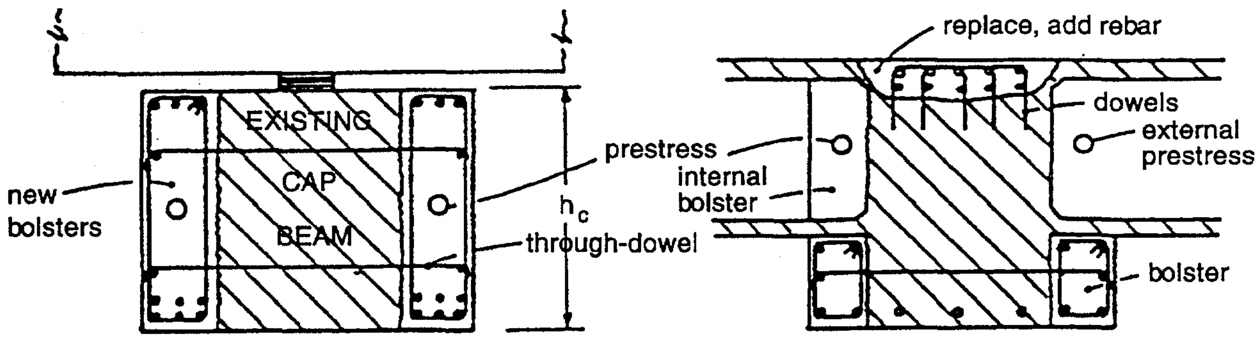
(a) Bearing-supported Superstructure **(b) Integral Cap Beam/Superstructure**

Figure 50. Typical cap beam-to-superstructure connections (from ref. 18).

The new and old concrete should be connected by dowels, preferably passing right through the existing cap beam. Assuming the amount of tension reinforcement in a bolster is A_{sb}, and is stressed to yield at the face of the supporting columns, the amount of dowel reinforcement required to transfer the force back into the existing cap beam, and thus ensure composite action, will also be A_{sb}. This assumes a coefficient of shear friction of $\mu = 1.0$. Thus, for example, the six lower rebar in each bolster of figure 51(a), total area A_b, will require n dowels of area A_d each where $n = A_{sb}/A_d$. These dowels should be distributed over an area $h_c\ell_c/2$ (see figure 50(a) and 51(a)) consisting of the lower half of the cap beam, from column face to cap beam centerline. Flexural strength may also be enhanced by prestressing. This may be inside the bolsters, as shown in figure 51(b), or may be by external prestressing.

Enhancing flexural capacity of integral cap beams is more difficult because of the constraints placed by the existing superstructure on the sides. Bolsters may be added at the bottom to enhance positive moment capacity, and negative moment capacity can be increased by removing top concrete and adding additional reinforcement (see figure 51(b)). External prestressing placed in grouted galvanized ducts will generally be the most economical means for enhancing both positive and negative moment capacity. Prestressing steel may be fully bonded if sufficient steel is provided to ensure elastic behavior in the cap. Prestressing steel should be left unbonded (but environmentally protected) in zones where plastic hinging in the cap is expected.

Although the philosophy of forcing plastic hinging into well-confined columns is generally to be recommended, it is clear that cap beams will have some ductility, even



(a) Bearing-supported Superstructure **(b) Integral Cap Beam**

Figure 51. Flexural and shear retrofit of cap beams (from ref. 18).

if not specifically detailed for plastic hinging by the provision of close-spaced stirrups in the plastic hinge region. The available plastic rotation capacity can be determined using the approach described in section B.3. Note that with tall piers, the ductility imparted to the structure as a whole from cap beam plastic rotation will be minimal.

6.3.3 MEMBER SHEAR STRENGTH

Full-depth bolsters, as shown in figure 51, can be reinforced to enhance shear strength. Strength may be assessed using the approach described in section B.4. Generally, a truss angle of $\theta = 45°$ should be adopted for new works. Prestressing will also enhance the shear strength by increasing the depth of the flexural compression zone and flattening the angle of the critical diagonal compression strut. Again, section B.4 applies.

6.3.4 JOINT SHEAR STRENGTH

Deficiencies in joint shear strength will be common in integral column/cap beam bents. Design concepts developed for the retrofit of the San Francisco double-deck viaducts after the 1989 Loma Prieta earthquake may be applied to the design of retrofit measures for bents of the type shown in figure 51. Generally, both vertical and horizontal special joint shear reinforcement will be needed. The most satisfactory solution will be the complete replacement of the joint, involving removal of existing concrete and temporary propping of the cap beam. This will also enable deficiencies of anchorage of column and beam reinforcement to be rectified. However, it would appear that adequate performance could be ensured by proper reinforcement of additional concrete added to the sides of the joint region, as shown in figure 52. Since the shear resistance at the joint will be provided by reinforcement external to the original joint, the new concrete jacket must be dowelled into the existing concrete to transfer the shear force by shear friction. This is because shear actions develop within the original joint as a consequence of column flexural actions and these must be transferred across the joint by the external reinforcement. Referring to figure 53, the

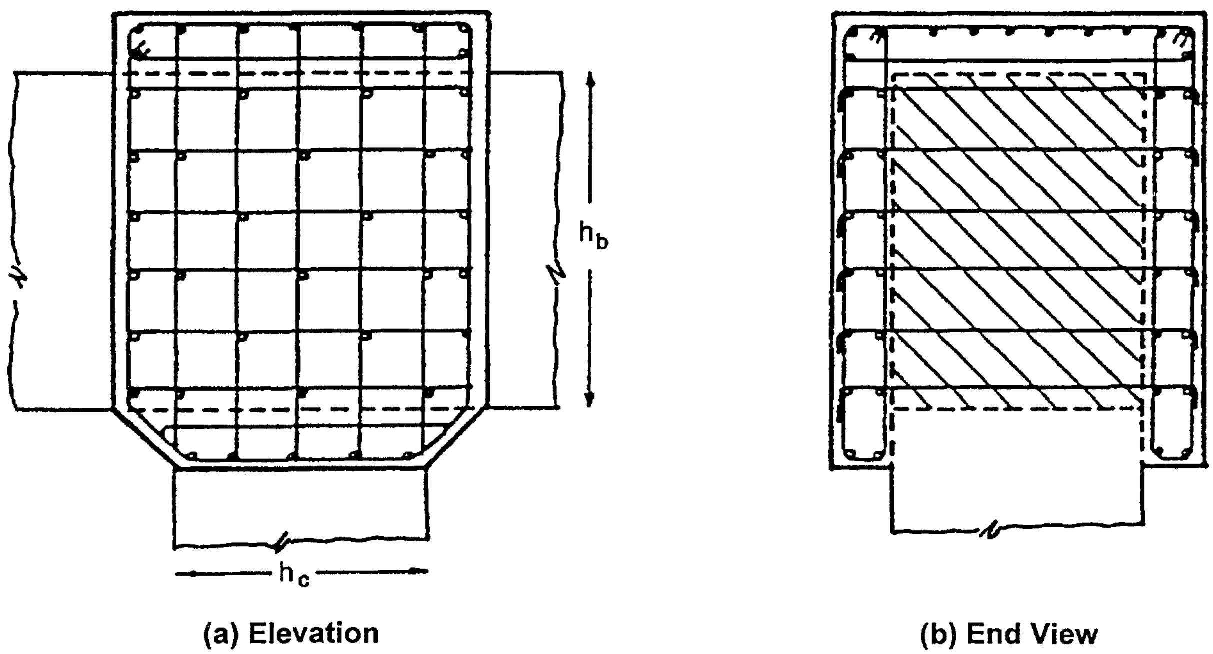


Figure 52. Joint retrofit with external concrete jacket (from ref. 18).

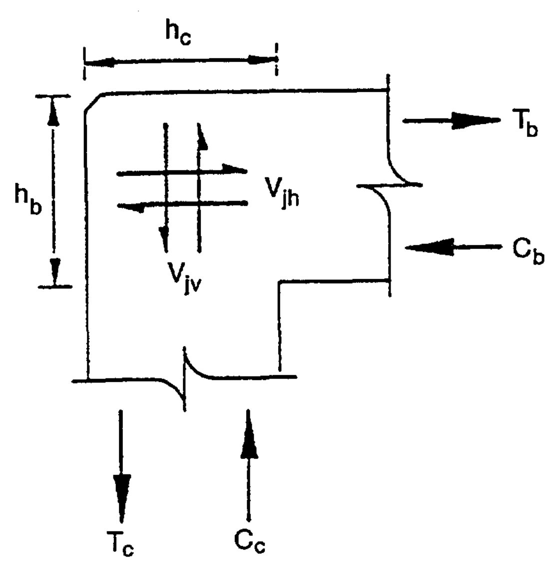


Figure 53. Shear forces on a knee-joint (from ref. 18).

interface between new and old concrete is subjected to shear friction forces simultaneously in vertical and horizontal directions. The shear forces are found from equilibrium with flexural actions in the beam and column, as described in section B.5. Thus in figure 53:

$$V_{jh} = T_b \tag{6-29}$$

and

$$V_{jv} \approx V_{jh} \cdot \frac{h_b}{h_c} \tag{6-30}$$

Assuming all joint shear resistance is provided by reinforcement in the new side concrete, the horizontal and vertical interface shear stresses will be:

$$v_{ih} \approx \frac{V_{jh}}{2h_b\, h_c} \tag{6-31a}$$

$$v_{iv} \approx \frac{V_{jv}}{2h_b\, h_c} \tag{6-31b}$$

where the 2 in the denominator is required because 50 percent of the shear is transmitted across each of the two interfaces between new and old concrete, one on each side of the joint. The maximum interface shear is the vectorial combination of v_{ih} and v_{iv}. That is:

$$v_i = \sqrt{v_{ih}^2 + v_{iv}^2} \tag{6-32}$$

Substituting for V_{jv} from equation 6-30 into equation 6-31b, and further substituting into equation 6-32 and simplifying yields:

$$v_i = \frac{V_{jh}}{2h_b\, h_c} \sqrt{1 + \left(\frac{h_b}{h_c}\right)^2} \tag{6-33}$$

A shear friction clamping stress equal to v_i must be provided across the interface. If dowels of bar area A_d and yield stress f_y are placed on a square grid at spacing s, then the dowels must satisfy:

$$\frac{A_d\, f_y}{s^2} \geq v_i \tag{6-34}$$

It is emphasized that this approach has yet to be confirmed by test results and should be conservatively applied. Wherever possible, the jacket should extend over the top of, and underneath (at the corners), the cap beam, as shown in figure 52, to improve connection to the cap beam. It is also recommended that until test data are available, the interface shear friction stress given by equation 6-33 should be limited to:

$$v_i \leq 0.2f_c' \leq 6.9 \text{ MPa } (1000 \text{ psi}) \tag{6-35}$$

where f_c' is based on the weaker of the existing and new concrete strengths.

It is noted that transverse prestressing reduces the need for horizontal joint shear reinforcement and should also create advantageous conditions for transferring a portion of the vertical joint shear. However, it is felt that bonding steel plates to the sides of a joint is not likely to be as effective in enhancing the shear strength as a concrete jacket. Bolting the plates through the joint, with an equivalent shear friction force to that suggested in equation 6-34, is not expected to improve the situation. This is because the flexibility of the steel plate will localize the shear friction stress in the immediate vicinity of the dowel or prestressing bar with little or no stress midway between dowels. As a consequence, the resulting shear transfer would appear to be less efficient.

6.3.5 TRANSVERSE LINK BEAMS - TRANSVERSE RESPONSE

An alternative method to alleviate cap beam problems is to cast a new link beam below the existing cap beam as shown in figure 54.

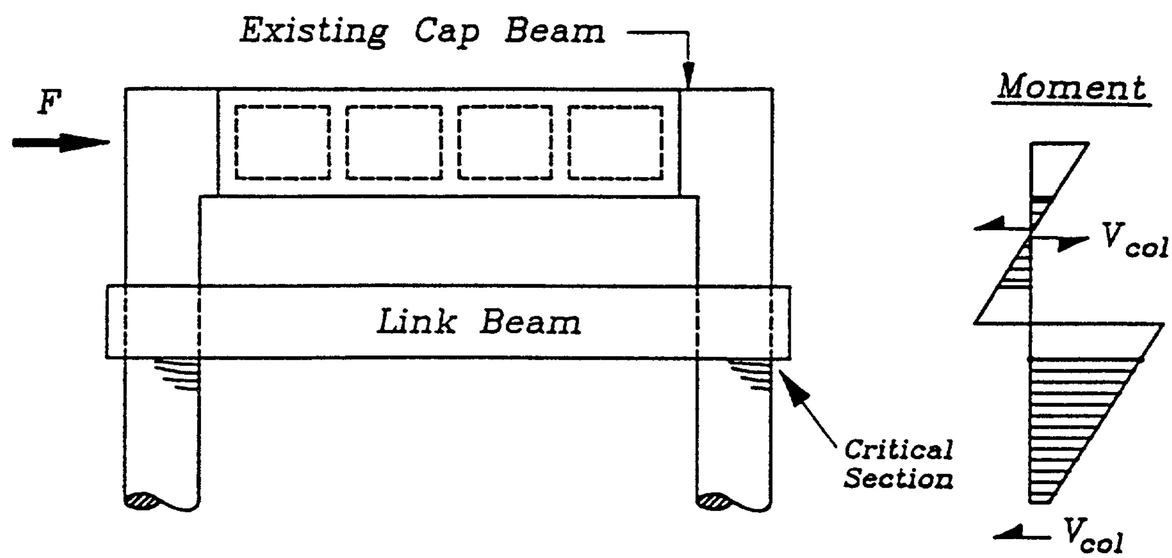


Figure 54. Link beam concept for transverse retrofit (from ref. 18).

The link beam is cast around the existing column and creates a new critical section below the link beam. Provided the height between the link beam and existing cap beam is sufficiently small, column moments between the two beams will be small since column shear forces in the column will be dictated by the flexural strength of the column sections below the link beam, where plastic hinges are expected to form. Moment equilibrium at the cap beam joints then indicates that cap beam forces due to seismic actions will be restored to very small values, and no further retrofit will be needed. The cap beam must be designed according to capacity design principles to ensure that plastic hinges form in the column, not in the link beam.

The use of link beams can be very useful in retrofitting tall piers. Judicious choice of the position of the link beam can result in protection for the existing cap beams, coupled with a substantial increase in lateral strength and stiffness of the bent. The concept can also be used to advantage at ground level, linking columns transversely or longitudinally to alleviate footing problems.

The new location for column hinge formation will need to be checked for ductility capacity using the principles developed in appendix B. Shear forces in the columns will also be increased due to the shorter column length below the link beam. Column retrofit measures will need to be considered.

6.3.6 CAP BEAM TORSION - LONGITUDINAL STRENGTH

If the cap beam and superstructure are integral, the load path from deck inertia force to ground under longitudinal response would appear to depend on superstructure flexural action and cap beam torsion in transmitting forces to the column, particularly when an outrigger cap beam exists, as shown, for example, in figure 50(a). In this case, it is clear that a cap beam torsional moment equal to the column longitudinal plastic moment capacity could be developed. As shown elsewhere, the distribution of this torsional moment to the superstructure girders (or vice versa) is not uniform, as a consequence of cap beam torsional flexibility, and the moment demand on superstructure girders closest to the columns will be highest, while girders close to the bridge axis will be subjected to very little moment. This effect is accentuated when torsional cracking of the cap beam occurs, since the torsional stiffness of a beam, after cracking, typically reduces to about 10 percent of the uncracked value.

For the retrofit designs of the San Francisco double-deck viaducts where the columns are outside the superstructure, the solution generally has involved the addition of an edge beam (or supergirder) in the plane of the columns from bent to bent to reduce the torsional effect. Provided the edge beam is stiff enough and strong enough, inadequacies of cap beam torsional strength and superstructure flexural capacity become largely irrelevant, since satisfactory longitudinal response can be ensured even when the cap beam has zero torsional capacity. Under these conditions, the design approach should be to force plastic hinges into the column rather than the supergirder, since plastic hinges in the supergirder would allow large rotations of the cap beam, with potential for degradation of gravity load-carrying capacity. If the supergirder is protected against plastic rotation by a capacity design approach, torsional rotations of the cap beam will remain small, and will be dictated by rotational compatibility with the supergirder. Torsional cracking of the cap beam, if it occurs, will reduce the torsional moment rather than increasing the torsional rotation.

Under this concept, the critical role of the cap beam will be to support gravity loads by beam action, and to participate in the transverse seismic response in accordance with the design assumptions. It must also maintain its capacity to transmit shear force resulting from gravity loads and longitudinal and transverse seismic response to the columns.

It should be noted, however, that there is room for considerable doubt about the true significance of longitudinal inadequacies. It appears that resonant response of multispan bridges is unlikely because of the improbability of coherent ground motion at the foundations of piers separated by distances which may greatly exceed typical seismic wave lengths.

In the meantime, it is prudent to assume that resonance can occur and provisions should be made for the longitudinal displacements determined by analysis. These displacements, however, need not exceed those that can physically occur on either side of a joint, should all the joints close and the abutments translate. If necessary, a displacement correction could be made for differential foundation displacement due to ground surface waves. Such a correction factor should be provided by the geotechnical engineer.

CHAPTER 7

RETROFIT MEASURES FOR FOUNDATIONS

7.1 GENERAL

Cost-effective retrofit measures for foundations are still being developed and tested. Nevertheless, some field experience is now available, particularly with footings. This experience, along with current research findings, is summarized in this chapter. Foundation components discussed herein include footings, piles, abutments, and settlement slabs. When retrofitting these components, one should not ignore the properties of the underlying soils. Foundation analysis and design should take into account soil strengths and consider all possible failure modes. This chapter summarizes available information (principally from reference 18) and indicates where the recommendations should be considered tentative at this time.

It should be remembered that the use of an earthquake protective system, such as seismic isolation, may be an acceptable alternative to any of the approaches described below. An overview of this option is given in section 4.3.3 and is more fully described in chapter 9. It should also be kept in mind that the optimum retrofit solution may well be a combination of both strategies. This will be particularly true whenever isolation cannot reasonably reduce the seismic demand to a level below the existing foundation capacity and some strengthening is required to satisfy the demand. In these cases, the amount of strengthening required should be significantly less than if isolation had not been used.

7.2 FOOTINGS

7.2.1 GENERAL

In many cases, column footings fail before the supported column or pier develops its full plastic capacity. This is most often due to the absence of a top layer of reinforcement and vertical ties in the footing which have the capability to resist uplift forces. During an earthquake, this can result in flexural cracking of the footing concrete or delamination and in a resulting loss of anchorage for the column longitudinal reinforcement. This condition is usually most critical in single-column bents supported on piled footings.

Retrofitting of footings is probably the most expensive aspect of bridge seismic upgrading. Deficiencies will frequently be found in footing flexural strength, shear strength, footing/column shear strength, anchorage of column rebar, pile capacity, and overturning resistance. To date, no experimental results are available for assessment of footing retrofit techniques, and design is based on theoretical considerations. A test program designed to provide information in this area has recently been initiated at the University of California and elsewhere.

7.2.2 FLEXURAL STRENGTH

Footings with tensile connections to piles will frequently be deficient in flexural strength as a result of a lack of top steel. Bottom steel may also be inadequate, particularly in wide footings where the bars furthest from the column are unlikely to be effective.

Retrofitting to enhance flexural strength may involve an overlay of reinforced concrete dowelled to the existing footing, as shown in figures 55(a) and 55(b). The top reinforcement should be located so that the bulk of it is within a distance of h_f from the column sides, where h_f is the depth of the retrofitted footing. Dowels between the new and old concrete should be capable of transferring the shear stress on the interface, using a shear friction approach and a coefficient of friction of $\mu = 1.0$. This assumes that the surface of the existing footing has been roughened prior to casting the new concrete. If these dowels are also to be used to enhance shear strength, they will need to pass through the full depth of the existing footing and be properly anchored.

Increasing the depth of the footing will also increase the positive moment capacity as a result of increased section depth. If this is insufficient, footing widening will be needed, with additional bottom steel. However, as noted earlier, rebar placed farther than a distance equal to the footing depth from the column sides is unlikely to be effective in resisting column flexure unless high ductilities are accepted in the footing.

The reliability of the dowels in tension have been questioned by some designers. Caltrans usually modifies the above overlay scheme shown in figure 55(a) by extending the footing and adding full-depth perimeter ties as shown in figure 55(c).

When overlaying a footing is not possible because of grade constraints, flexural capacity, both positive and negative, may be increased by prestressing, either in ducts drilled through the length of the footing or in new concrete on the sides, anchored in a new end block. This is likely to be less effective than prestressing in ducts close to the column.

7.2.3 SHEAR STRENGTH

Shear-strength deficiencies in footings are more complicated to retrofit. However, in many existing footings, the shear between the column compression stress resultant and the foundation or pile reaction may be carried by a diagonal compression strut. This, however, requires a dependable bottom steel tie force, which can generally only be relied upon if the bottom steel is anchored by a 90° hook, as shown in figure 56.

If the angle, θ, of the potential compression strut is less than about 30°, or if the bottom tensile reinforcement is inadequately anchored, the shear strength can be increased by:

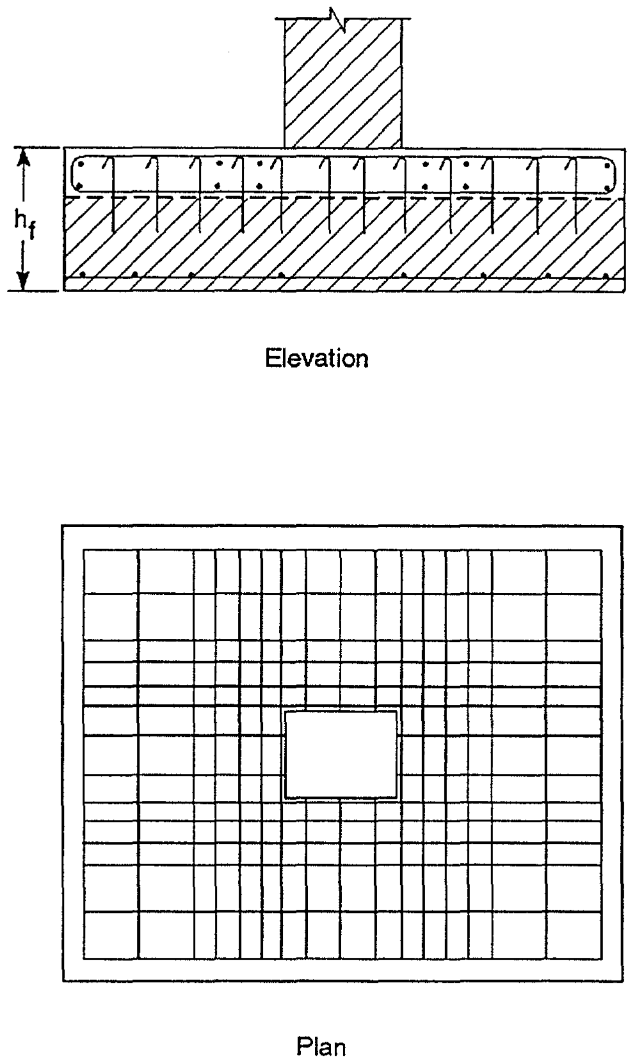


Figure 55(a). Footing flexural strength retrofit (from ref. 18).

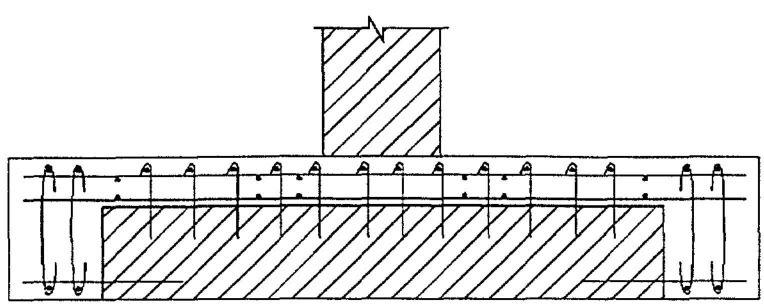


Figure 55(b). Footing flexural strength retrofit with Caltrans perimeter ties.

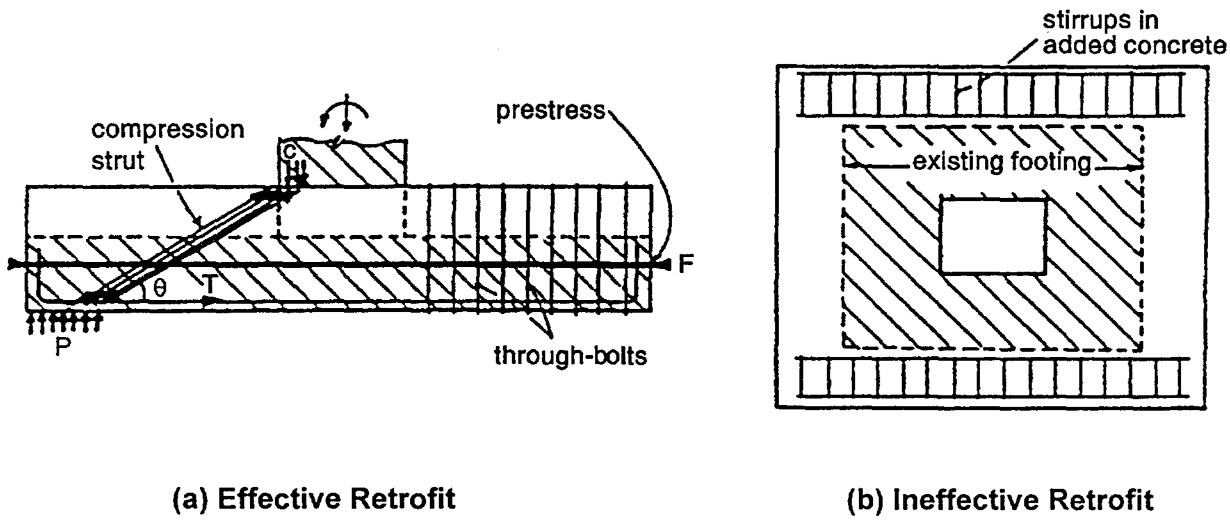


Figure 56. Footing shear strength retrofit alternatives (from ref. 18).

- Increasing the footing depth. This will increase the shear capacity of the concrete shear-resisting mechanisms.
- Drilling vertically through the footing and anchoring vertical rebar, preferably prestressed. This will act as additional shear reinforcement.
- Drilling longitudinally through the footing and prestressing, as recommended for flexural strength enhancement. The shear strength will increase, as with prestressed beams.

Note that the addition of shear reinforcement in a new widened portion of a footing is unlikely to be effective because of its distance from the shear-inducing force, which is primarily the column compression resultant.

7.2.4 JOINT SHEAR STRENGTH

Theoretical considerations and limited test data indicate that many footing/column joints will be deficient in shear strength. Appendix B discusses methods for assessing joint shear strength. Inadequate shear strength can be improved by the same measures suggested for footing shear strength enhancement. However, to be effective, any joint shear reinforcement added in the form of vertical rebar must be placed close to the sides of the column and between the column tensile and compressive stress resultants. Testing is required to check the severity of the problem of column-base joint shear strength and the effectiveness of various retrofit measures.

7.2.5 ANCHORAGE OF COLUMN REINFORCEMENT

It is not uncommon for column rebar to be inadequately anchored in the footing. Methods for assessing required anchorage length are given in appendix B.

However, even if adequate anchorage length is provided, the connection capacity may be suspect if the column reinforcement does not extend down to the bottom rebar layer of the footing, particularly if a spread footing or pile-supported footing without tension capacity is provided, as shown in figure 57. In this case, there is no continuity between the column rebar tensile force T_c and the corresponding footing force T_f. A splitting crack combined with joint shear failure appears probable here.

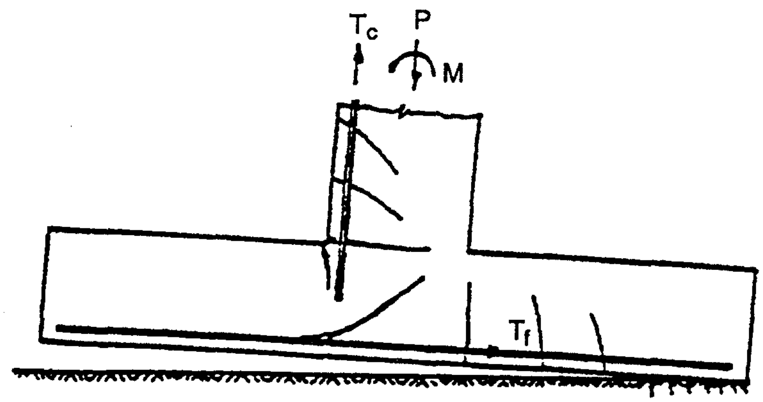


Figure 57. Rocking of spread footing (from ref. 18).

Retrofit for this detail appears difficult. Increasing the footing depth with a reinforced overlay will improve the capacity, but will obviously not solve the problem of lack of continuity of tension force. However, if soil anchors or additional tension piles are provided to cause a moment reversal in the footing over the width of the column, and a reinforced overlay is provided, satisfactory performance should be achieved. Again, confirmation by testing is required.

7.2.6 OVERTURNING RESISTANCE

As mentioned in appendix B, uplift of a spread footing or pile-supported footing does not necessarily represent undesirable response. The action of rocking at an overturning moment less than the column capacity provides a form of seismic isolation that may protect both column and footing from inelastic action, which neither may be detailed to support. However, if rocking displacements are judged to be too large or if failure of the footing is predicted, retrofit measures, including increased resistance to overturning, may be necessary.

Overturning resistance may be improved by increasing the footing plan dimensions, by the addition of tension piles (generally in conjunction with an increase in plan size) or by the use of soil or rock anchors. Existing pile tensile capacity is usually weak because of poor connections to the footings and/or lack of continuous tensile reinforcement in the piles. It should be noted that it may be difficult to achieve much positive or active reaction from the soil anchors and, as a consequence, significant rocking displacement may be needed before adequate overturning restraint is provided to develop a column-base plastic hinge. This situation is not expected to be a problem for relatively short columns, but could pose a stability problem for taller structures.

7.2.7 PILES

Calculations may also indicate that pile flexural or shear failures are probable, or that base shear capacity provided by transfer of forces from the footing to ground is inadequate. Retrofit to improve these defects may be very costly, and the designer should consider whether pile failure or footing sliding is likely to cause the structure to collapse. Since the solution may effectively be a footing replacement, and the probability of collapse from failure in these cases appears small, the designer may elect to take the risk and replace damaged footings after an earthquake. Similar arguments may be applied to other modes of footing failure. However, the designer should be satisfied that under all circumstances the footing will continue to be able to support the column axial forces during and after the earthquake. In the case of joint shear failure, for example, this would mean reliance for support on piles in the area under and immediately adjacent to the column.

7.3 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 accessibility to the bridge, which may be unacceptable 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 sections discuss two possible retrofit measures that will mitigate the effects of abutment failure.

7.3.1 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 Seismic Performance Category 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 3 m (10 ft). 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 dead load multiplied by the sum of the coefficient of friction between the slab and the abutment fill plus the Acceleration Coefficient. That is:

$$F_D = (\mu + A)\ DL_s \qquad (7\text{-}1)$$

where F_D = design force
μ = coefficient of friction
A = acceleration coefficient
DL_s = slab dead load

Note that this connection should be free to rotate so that moment will not be transferred to the abutment backwall when the approach fill settles. Figures 58 and 59 show two different types of settlement slabs that have been used in the past.

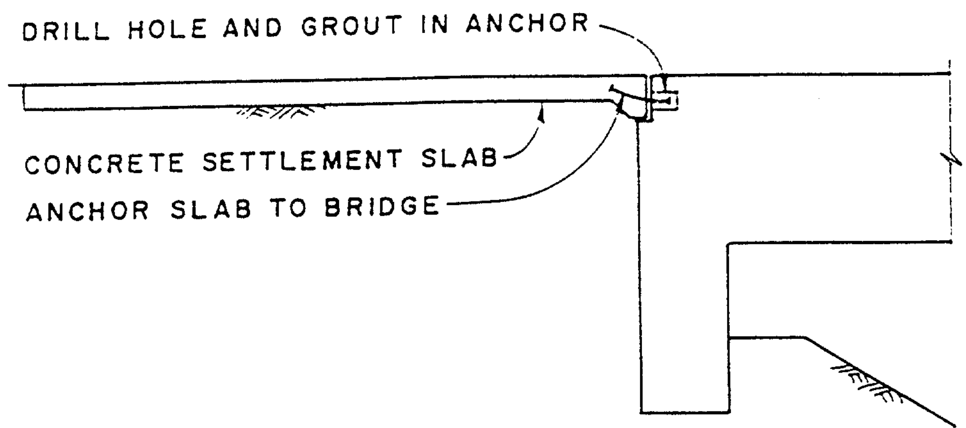


Figure 58. California-style settlement slab.

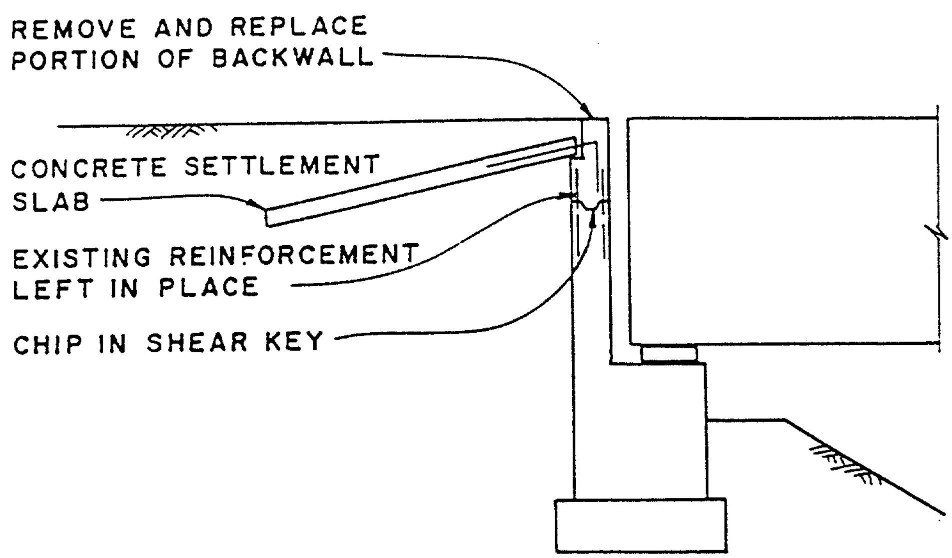


Figure 59. New Zealand-style settlement slab.

7.3.2 SOIL AND GRAVITY ANCHORS

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

Soil anchors similar to those shown in figure 60 have been used as a retrofit measure. Because the backfill may be subject to movement during an earthquake, the anchors should extend a sufficient distance into the backfill so as not to be affected by any such movement.

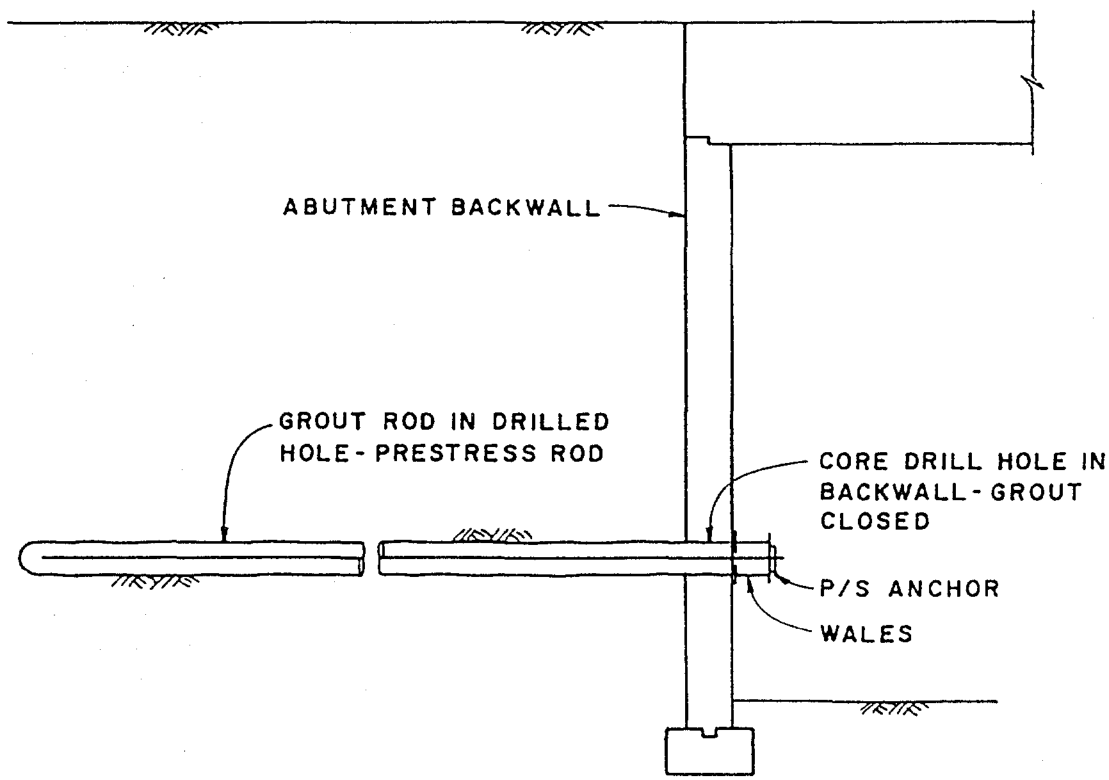


Figure 60. Retrofit of abutment with soil anchors.

Gravity anchors consist of tie rods running between the superstructure or abutment and a dead man, which may be a cast-in-drilled-hole shaft or a large gravity beam cast in a trench some distance behind the abutment. The tie rods are usually embedded in a concrete trench.

The ultimate capacity of both anchor types should be greater than or equal to the 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.

CHAPTER 8

RETROFIT MEASURES FOR BRIDGES ON HAZARDOUS SITES

8.1 GENERAL

Hazardous site conditions for any bridge include those that give rise to extreme forces and/or relative displacements during an earthquake. Such conditions include sites which cross or are immediately adjacent to an active fault with steep unstable slopes and those with liquefiable sands or silty sands. Retrofit measures for these conditions are very limited and few have been proven in the field. This chapter discusses, in general terms, some of the issues that might be considered when retrofitting a bridge on such a site.

8.2 BRIDGES ACROSS OR NEAR ACTIVE FAULTS

Bridges crossing or immediately adjacent to active faults may be subjected to large relative displacements of adjacent piers or supports as a result of surface faulting. Although the probability of such an occurrence at a given location during the remaining life of a bridge will be very low, the possibility should be considered in assessing suitable retrofit measures for the bridge. A conservative retrofit design, particularly in terms of displacement capabilities, should be adopted. Strengthening of substructures should aim at providing the maximum capacity possible by use of extra confinement in the plastic hinge zones.

If the bridge is a multispan crossing with a series of simply supported spans, the relative merits of making the structure continuous should be carefully evaluated. Although simple spans have the advantage of additional flexibility and therefore the capacity to tolerate large relative movements, difficulty will be experienced in ensuring that the spans do not drop from the supports. To minimize this risk, very generous support lengths should be provided. The additional redundancy of continuous superstructures that are monolithic with their supporting substructures will tend to reduce the probability of total collapse. There is, however, a practical limit to the amount of relative displacement across a fault that can be accommodated in a monolithic structure. One alternative is to support a continuous superstructure on elastomeric bearings over each pier and at each abutment. These bearings can be designed to accommodate relatively large displacements and still provide an elastic restoring force to the superstructure. Additional restrainers may also be provided in parallel with the bearings if gross movements are expected. Note that accelerographs of recent earthquakes indicate that vertical ground accelerations close to a fault can substantially exceed 1.0g. In these situations, monolithic construction is to be preferred, but if elastomeric bearings are used, vertical restrainers should be provided to limit the effects of uplift.

It should be recognized that the purpose of retrofitting for such an extreme event will be to avoid, or at least minimize, loss of life by reducing the probability of total collapse. After such an earthquake, it is probable that the bridge will have to be demolished and replaced.

8.3 BRIDGES ON OR NEAR UNSTABLE SLOPES

Many bridges in mountainous regions are sited across steep-sided valleys. Detailed geotechnical investigations should be made to assess the potential for slope instability under seismic conditions. For major structures, these investigations should include geological and geomorphic studies, including expert study of aerial photographs for evidence of bank movement under recent earthquakes, as well as material testing and extensive bore-hole and trenching investigations to check for unstable layers and vertical fissures. Particular attention should be paid to drainage so as to prevent infiltration of surface water and increased porewater pressures in potential failure regions. Special studies should be made to investigate the practicality of improving factors of safety against slope failure using such means as unloading the banks by removal of overburden. For some important structures, it may be advisable to relocate each abutment well back from the top of the slope and reconstruct the two end spans. It may also be prudent to tie back any intermediate pile caps located on the bank using rock anchors or other techniques.

8.4 BRIDGES ON LIQUEFIABLE SOILS

Liquefaction and excessive movement due to lateral spreading have been major causes of bridge failure during past earthquakes. When severe liquefaction is expected, modification and strengthening of the bridge alone will not be effective in preserving the functionality of the bridge. In such cases, soil stabilization will also be necessary to reduce the probability and extent of liquefaction failure. There are two aspects to the retrofitting of a bridge on a liquefiable site. The first is to eliminate or improve the soil conditions that tend to be responsible for liquefaction. Site remediation has been used for dams, power plants, and other structures, but to date has not been widely used as a retrofit measure for bridge sites. The second aspect is to increase the ability of the structure to withstand large relative displacements similar to those caused by liquefaction or large soil movement. Increasing structural capacity uses many of the retrofitting techniques discussed in the previous chapters.

8.4.1 SITE STABILIZATION

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

- Lowering of groundwater table.
- Densification of soil by vibro-compaction, vibro-replacement, or compaction grouting.
- Vertical network of drains (stone columns).

- Placement of permeable overburden.

- Particulate or chemical grouting.

Lowering the groundwater table eliminates or reduces the presence of water, which is one of the three items required before liquefaction can occur. The feasibility of this approach, and the associated costs, will depend on the site. Some type of gravity drainage is preferred to mechanical methods, although mechanical methods such as well points are not out of the question for a major bridge 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 precompaction can reduce the risk of liquefaction. Soil densification through the use of vibro-replacement improves drainage if a porous material is used and is therefore the preferred method. However, precompaction can result in significant settlements, and care should be taken to protect the existing structure from damage. Excessive settlements during construction will often make soil densification an impractical retrofit method. Settlement may be controlled if the compaction grouting technique is used, but this method is likely to be more expensive.

One method which will improve drainage without disrupting the existing structure is to install a network of gravel drains as shown in figure 61. These drains will allow water to escape during an earthquake and thus prevent the buildup 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 permeability of the overburden will not aggravate the buildup of pore pressure. However, the settlements that will accompany this preconsolidation should be considered when using this approach.

The use of chemicals or particulate grouts to increase the shear strength of soil is also a possible solution. However, if not properly designed, these methods may reduce soil permeability and aggravate the buildup of pore pressure. Therefore, the design and use of these methods should be performed by qualified individuals.

Some of these methods may not be suitable or environmentally acceptable and may 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 principles of soil mechanics

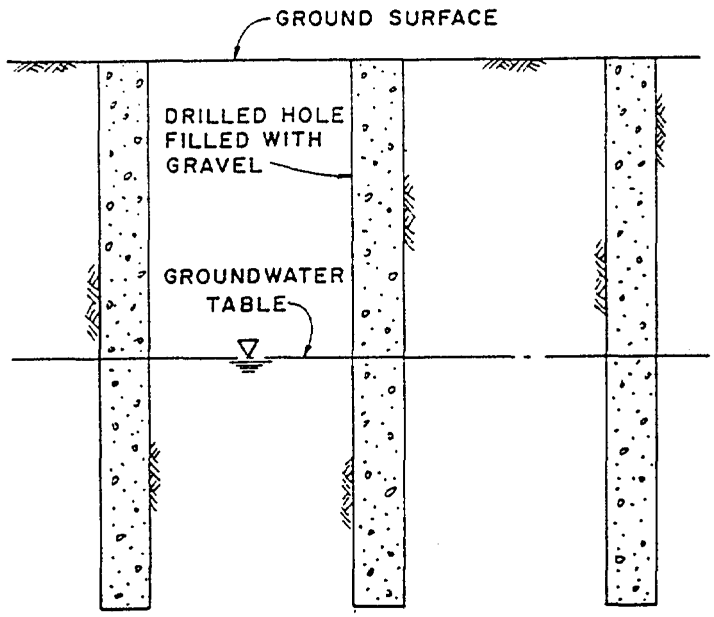


Figure 61. Gravel drain system.

to ensure that the design is effective and that construction procedures will not damage the existing bridge.

Note that a rough estimate of liquefaction potential at a bridge site can be obtained by following the procedure for preliminary screening discussed in section 2.3.1.1(d). However, when retrofitting 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 rigorously evaluating liquefaction potential.

An overview of site stabilization techniques is given by Ledbetter.[32]

8.4.2 STRUCTURAL IMPROVEMENTS

In addition to site stabilization, upgrading a structure will often be necessary so as to improve its ability to tolerate differential displacements. Strengthening methods to be used will depend on the configuration of the structure and the 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 immediate access to the bridge is important.

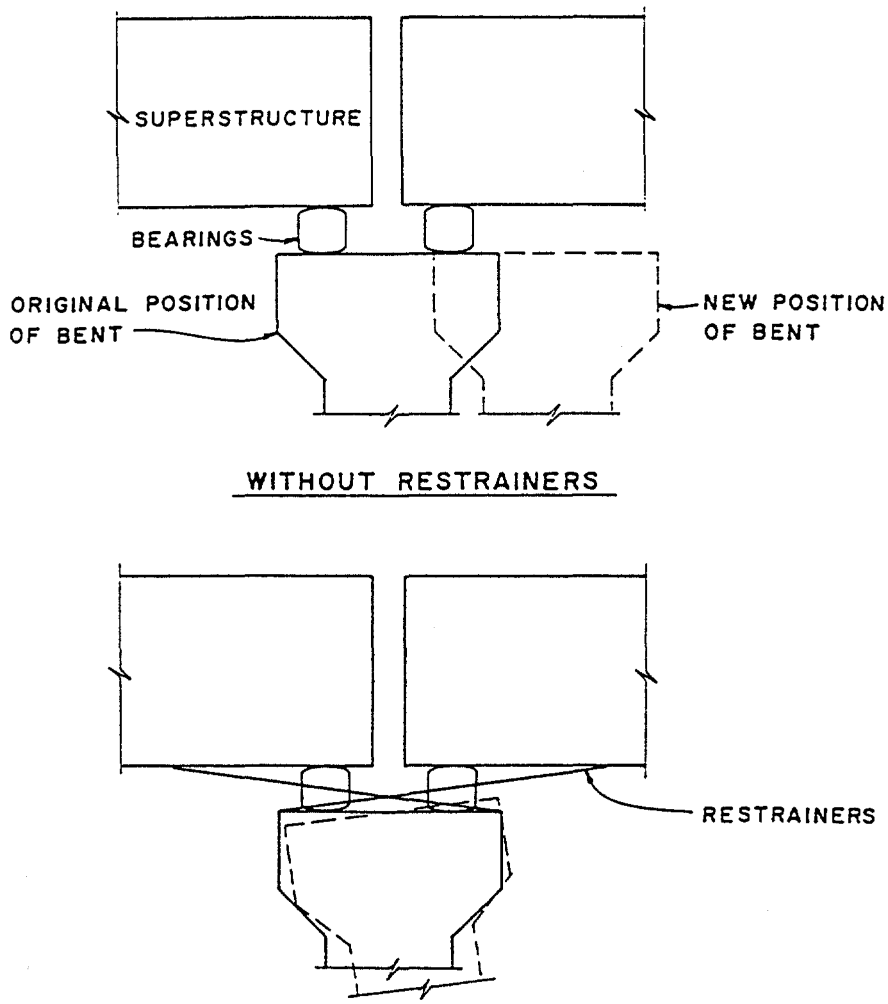


Figure 62. Effect of restrainers at bent during liquefaction failure.

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 62. 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 ensure that a brittle failure does not occur. Suitable methods for retrofitting columns, joints, and foundations are described in chapters 6 and 7.

An alternative to adding ductility is to increase the stiffness of the structure so as to uncouple it from the long period motions associated with liquefiable layers. Stiffening may be accomplished by improving the lateral resistance of the foundations, adding infill walls between columns, or even increasing the number of columns and foundations. These measures may be costly, but may be justified for critical structures.

CHAPTER 9

RETROFIT MEASURES USING EARTHQUAKE PROTECTIVE SYSTEMS

9.1 GENERAL

As noted in section 4.3.1, there are two alternative strategies that a designer can adopt when faced with retrofitting a bridge. The first approach, as discussed in chapters 5 through 8, is to increase the capacity of the structure's deficient components. An alternate approach is to reduce the demand on these components by using earthquake protective systems. Of the family of protective systems, seismic isolation is now being widely used as a cost-effective retrofit measure. This chapter explains the principles of seismic isolation and provides several options for the design and implementation of this protective system.

9.2 PRINCIPLES OF SEISMIC ISOLATION

9.2.1 GENERAL

Isolation of structures from the damaging effects of earthquakes is not a new idea. The first patents for seismic isolation schemes were filed at the turn of the century, but until very recently, few structures had been built using this concept. Early concerns were focused on the possibility of uncontrolled displacements at the isolation interface, but these have since been largely overcome with the successful development of mechanical energy dissipators as discussed in section 9.4. When used in combination with a flexible device such as an elastomeric bearing or a sliding plate, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in seismic isolation as an effective means of protecting bridges from earthquakes has, therefore, been revived in recent years. To date, there are several hundred bridges in New Zealand, Japan, Italy, and the United States that use isolation principles and technology in their seismic design.

The basic intent of seismic isolation is to increase the fundamental period of vibration such that the bridge is subject to lower earthquake forces. However, the reduction in force is accompanied by an increase in displacement demand that must be accommodated within the flexible mount. Furthermore, longer period bridges can be lively under service loads. For these reasons, additional damping is often introduced at the same time that the period is lengthened. In this way, the increases in displacement can be controlled, while at the same time, stiffness against service loads can be provided. Studies have shown that the cost of the isolation hardware can be offset against the savings in the substructures and foundations (because of the reduced forces) and the long-term reduction in repair costs for expected seismic damage.

Therefore, there are three basic elements in a bridge isolation system:

1. A flexible mounting so that the period of vibration of the bridge is lengthened sufficiently to reduce the force response.

2. A damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level.

3. A means of providing rigidity under low (service) load levels, such as wind and braking forces.

9.2.2 FLEXIBILITY

An elastomeric bearing is not the only means of introducing flexibility into a structure, but it certainly appears to be the most practical and the one with the widest range of application. The idealized force response with increasing period (flexibility) is shown schematically in the acceleration response curve of figure 63. Reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced is primarily dependent on the nature of the earthquake ground motion and the period of the fixed base structure. However, as noted above, the additional flexibility needed to lengthen the period of the structure will give rise to large relative displacements across the flexible mount. Figure 64 shows an idealized displacement response curve from which displacements are seen to increase with increasing period (flexibility).

9.2.3 ENERGY DISSIPATION

Large relative displacements can be controlled if substantial additional damping is introduced into the structure at the isolation level. This is shown schematically in figure 65. Also shown schematically in this figure is the smoothing effect of higher damping.

One of the most effective means of providing a substantial level of damping is through hysteretic energy dissipation. The term hysteretic refers to the offset between the loading and unloading curves under cyclic loading. Energy not recovered during unloading is lost from the system and dissipated as heat in most cases. Figure 66 shows an idealized force-displacement loop where the enclosed area is a measure of the energy dissipated during one cycle of motion. Mechanical devices have been developed that use the plastic deformation of either mild steel or lead to achieve this behavior (section 9.4). Mild steel bars in torsion, cantilevers in flexure, and lead extrusion devices have been tested, refined, and are now included in several bridges. Lead-rubber (elastomeric) bearings have also been developed and used in New Zealand, Japan, Italy, and the United States.

9.2.4 RIGIDITY UNDER LOW LATERAL LOADS

While lateral flexibility is desirable for high seismic loads, it is clearly undesirable to have a structural system that will vibrate perceptibly under frequently occurring loads, such as wind loads or braking loads. Mechanical energy dissipators may be used to provide rigidity at these service loads by virtue of their high initial

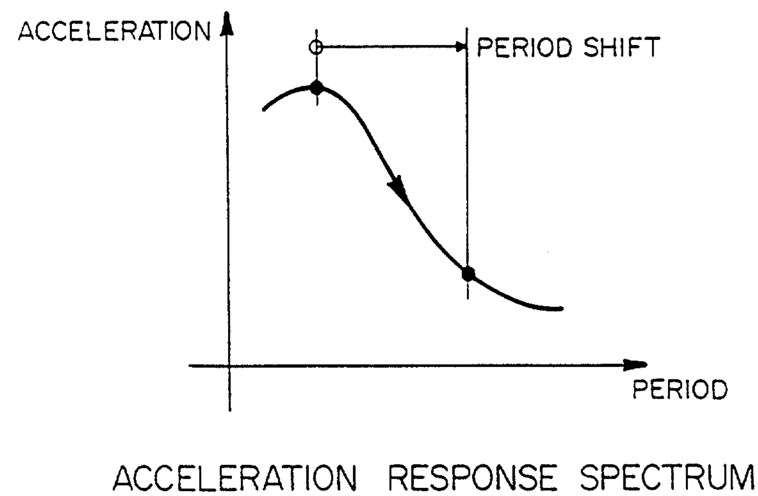


Figure 63. Idealized force response curve.

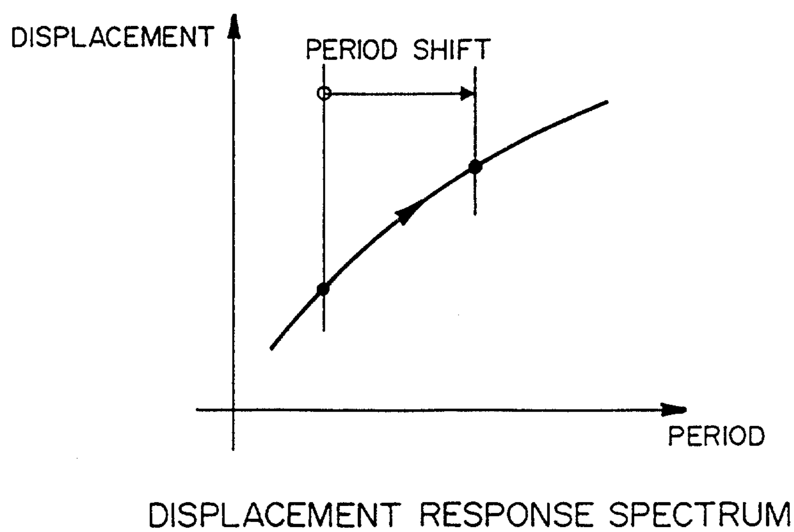


Figure 64. Idealized displacement response curve.

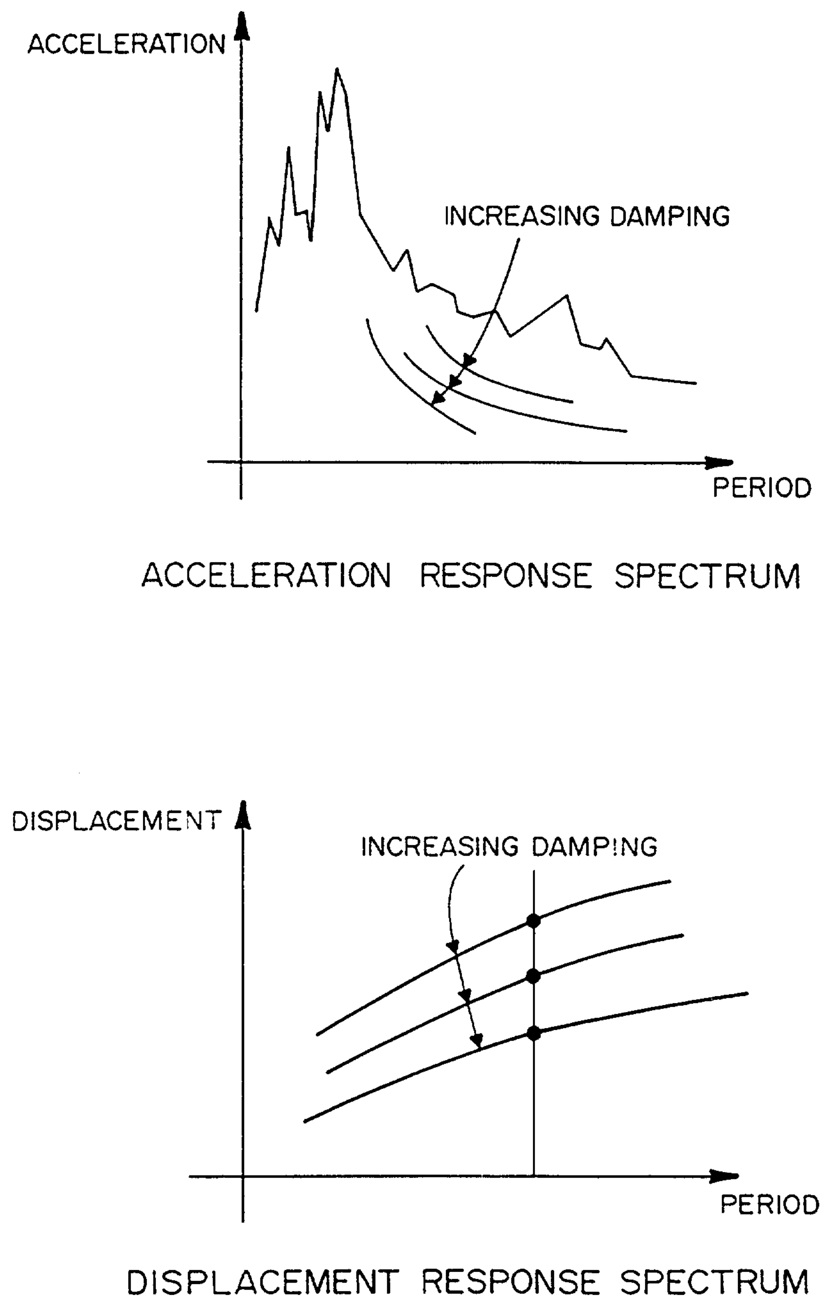


Figure 65. Response curves for increasing damping.

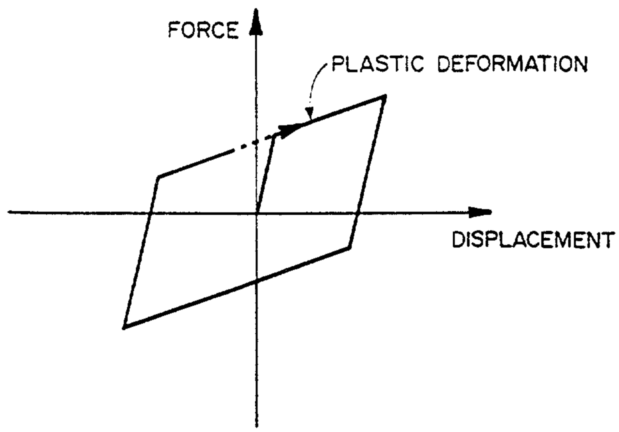


Figure 66. Idealized hysteresis loops.

elastic stiffness. Alternately, some seismic isolation systems use a separate wind restraint device for this purpose—typically a rigid component that is designed to fail at a given level of lateral load.

9.3 DESIGN OBJECTIVES

The design objectives for seismic isolation are best illustrated by figure 67. The solid uppermost line, curve 1, is the realistic (elastic) ground response spectrum as recommended in the AASHTO Specifications when A = 0.4. This is the spectrum that is used to determine actual forces and displacements to which a bridge will be subjected. The lowest solid line, curve 4, is the design curve from an earlier edition of the AASHTO Specifications (the 13th Edition). It is seen to be approximately one-fifth of the realistic forces given by the current specification. This reduction, which is used to obtain the design forces, is consistent with an R-factor of 5 for a multicolumn bent.

Also shown in figure 67 is curve 3, the probable overstrength of a bent designed to the AASHTO Specifications. This has been obtained by assuming an overstrength factor of 1.5. Curve 3, therefore, represents the probable capacity of the bent.

The demand on this bent is represented by curve 1 and the difference between demand and capacity results in damage—possibly in the form of plastic hinging in the columns. This difference is highlighted in figure 67 by the arrow and note just above the legend for curve 1.

If the bridge is isolated, the actual shear forces that the bridge will be subjected to may be represented by curve 2 (the small dashed line). This curve corresponds to the same seismic input as curve 1, but it includes the effect of the substantial level of damping inherent in hysteretic isolation systems. The period of the isolated bridge will be in the 2.0- to 2.5-second range, and it is seen that in this range,

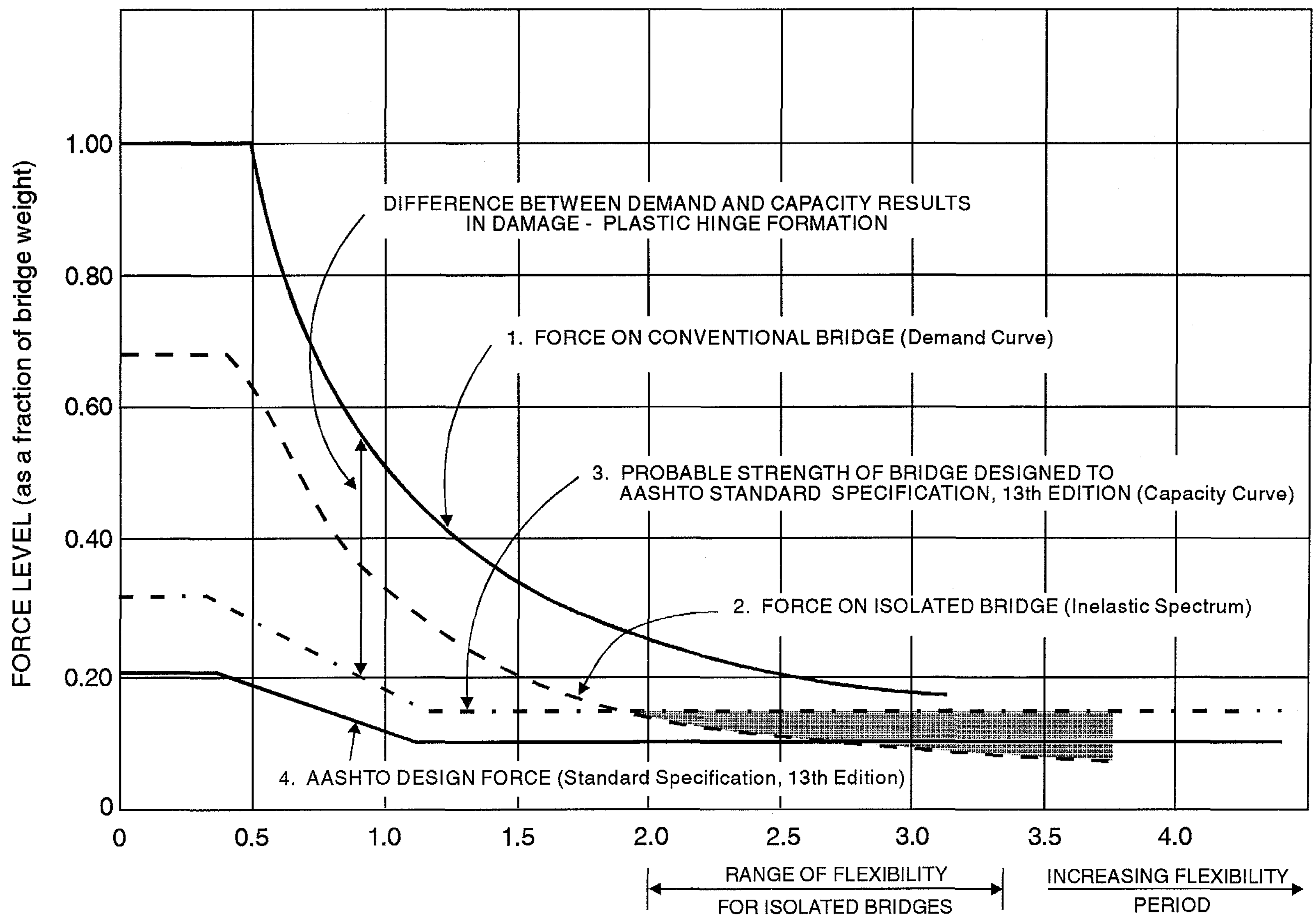


Figure 67. Comparison of earthquake force effects between conventional and isolated bridges.

the overstrength (actual capacity) of the bent exceeds the realistic forces (demand) for the isolated bridge. This region has been shaded in figure 67. In this region, there is minimal inelastic deformation or ductility required of the bent and essentially elastic performance may be expected.

The objective of seismic isolation is therefore to change the natural period of the bridge such that it falls within this shaded region. This means that strengthening is no longer needed because the demand is now less than the existing capacity.

The benefits of seismic isolation for bridges may be summarized as follows:

- Reduction in the realistic forces to which a bridge will be subjected by a factor of between 5 and 10 (based on curves 1 and 2 of figure 67, and a period shift due to isolation of from 0.4 to 2.0 s).

- Reduction and perhaps elimination of the ductility demand and hence damage to the piers.

- Control of the distribution of the seismic forces to the substructure elements with appropriate sizing of the elastomeric bearings.

- Reduction in column design forces by a factor of at least 2, compared to conventional design (based on curves 2 and 4 of figure 67, and a period shift due to isolation of from 0.4 to 2.0 s).

- Reduction in foundation design forces by a factor greater than 2.5, compared to conventional design (based on the fact that conventional design requires higher design forces for the foundations than for columns).

These attributes have significant implications for seismic retrofitting, particularly for those bridges with girders already supported on bearings at piers and abutments. Replacement of these bearings with isolation bearings can reduce the seismic demand on columns, footings, cap beams, and all joints and connections. On the other hand, larger clearances may be required at the abutments than would normally be provided (for thermal and creep movements) and special abutment details may then be necessary to accommodate the expected movements.

It follows that isolation is attractive when it is not practical or desirable to retrofit deficient columns and foundations by conventional means. Since with isolation it is usually feasible to keep these members in their elastic range, strengthening may be avoided altogether. This is particularly advantageous in the following situations: (a) traffic lanes close to columns that cannot be closed; (b) critical utility or communication lines that are buried alongside existing footings which cannot be disturbed; or (c) when piers are in deep water or located in an environmentally sensitive area, such as a wetland or national park. Isolation should also be considered when serviceability immediately following an earthquake is a priority.

Guide specifications for the design of new isolated highway bridges have been published by AASHTO and these requirements should be satisfied, as far as they are applicable, when retrofitting existing construction using these systems.[19]

9.4 SEISMIC ISOLATION BEARINGS

As noted in section 9.2, isolation bearing systems should provide rigidity under service load, and flexibility and damping under seismic loads. This usually means that the force-deflection characteristics of these systems are nonlinear. This is because, at small amplitudes, the isolators are stiff so that service loads can be resisted without the bridge being "lively"; while at higher amplitudes, they soften to give the required flexibility to isolate the bridge during an earthquake.

In some instances, these nonlinear characteristics are enhanced to provide a source of hysteretic energy dissipation. In other cases, a separate dissipator is provided to dampen and control the displacements of the superstructure.

Isolation systems currently being used for bridges and buildings include both rubber-based and friction-based systems. However, the majority of bridge applications are rubber-based and the principal type being used in the United States, Japan, and New Zealand is the lead-filled elastomeric bearing. Friction-based systems are commonly used in Italy. Examples of some of the isolation systems currently in use for bridge applications are given in table 7 and illustrated in figures 68 through 71.

The selection of isolation hardware is an important decision since both short-term and long-term performance characteristics are of interest. In the short term, resistance to wind and braking loads without excessive deflection implies rigidity at small deformations. However, the same devices must also permit thermal expansion to occur in the superstructure without overstressing the substructures. These two requirements may be in conflict with one another in some isolation systems. In the long term, reliability of performance is essential. It may be many decades before the design earthquake occurs, and over this period of time, the isolator properties must remain stable. The best hardware in this regard is that which is maintenance-free, does not require precise field tolerances in order to operate, and is constructed from materials that are chemically inert (resistant to atmospheric pollutants, deicing salts, and other roadside hazards).

All isolation systems should satisfy rigorous testing requirements and quality control standards. Guidance on suitable requirements in this regard is given in the AASHTO guide specification.[19]

9.5 BRIDGE SUITABILITY

Not all bridges are suitable for retrofitting by seismic isolation. Those most suitable are bridges founded on rock or competent soil and those with stiff substructures and, therefore, short periods. On the other hand, bridges on very soft sites and those with tall flexible columns (long-period bridges) will be difficult to isolate

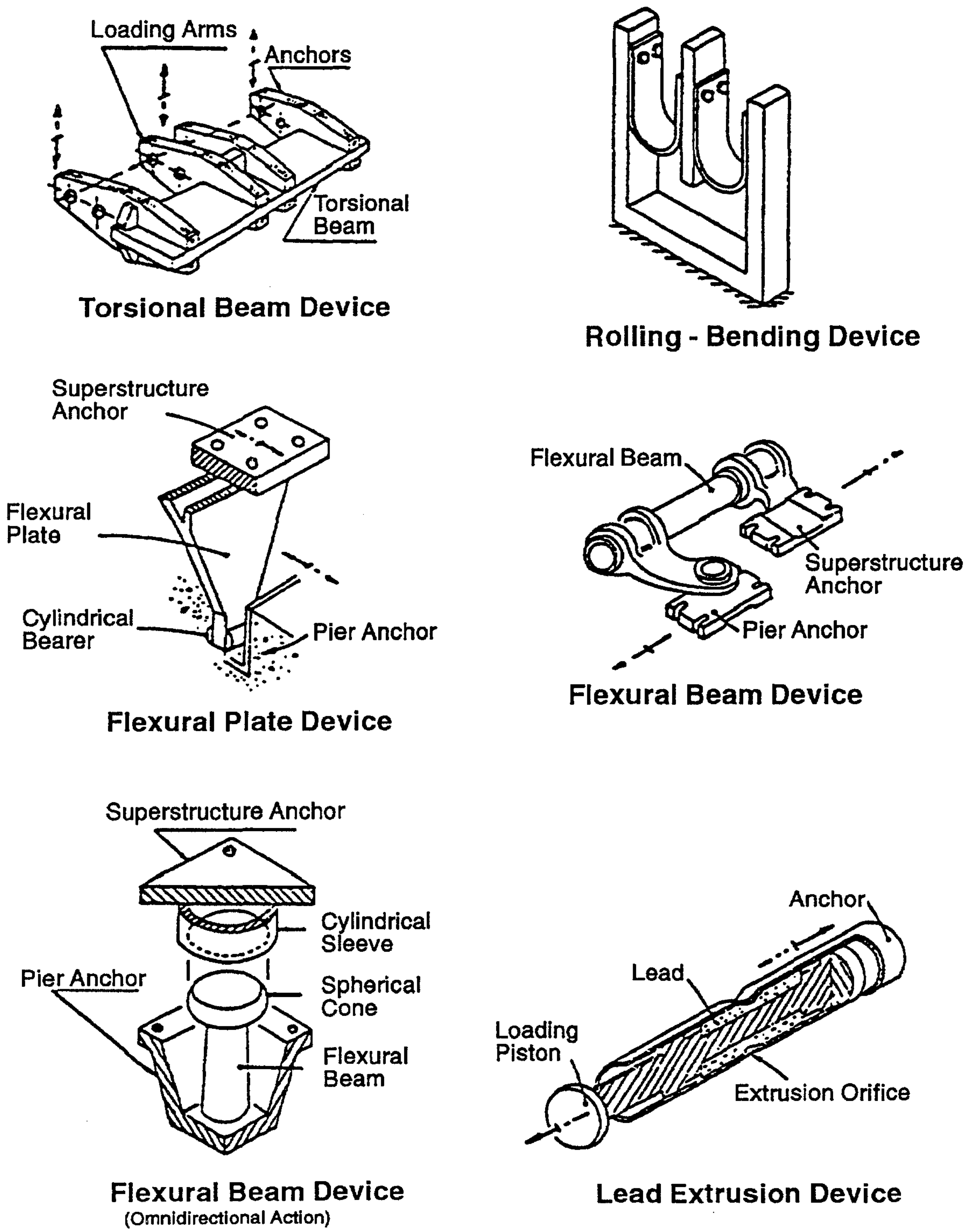
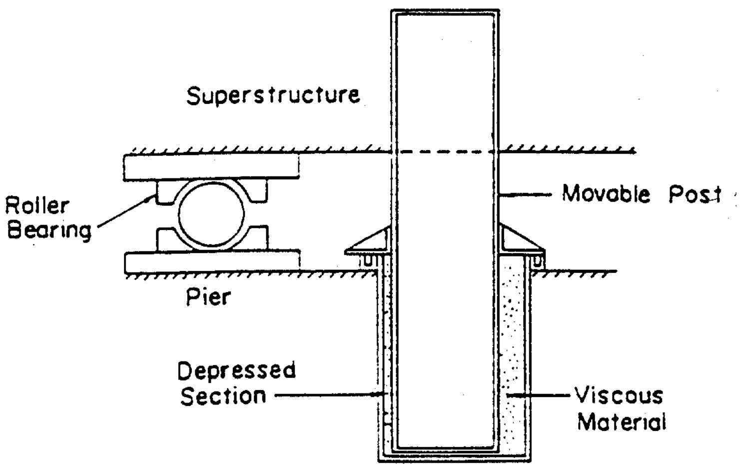


Figure 68. Energy dissipation devices developed in New Zealand.



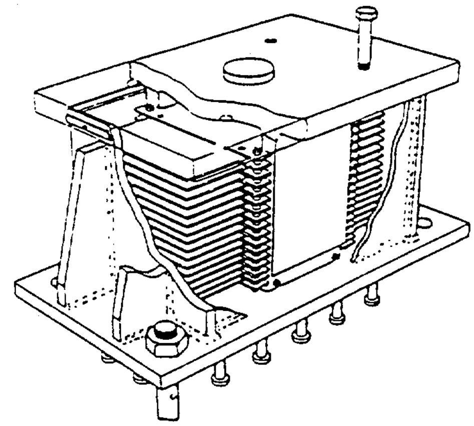


Figure 69. Energy dissipation devices developed in Japan.

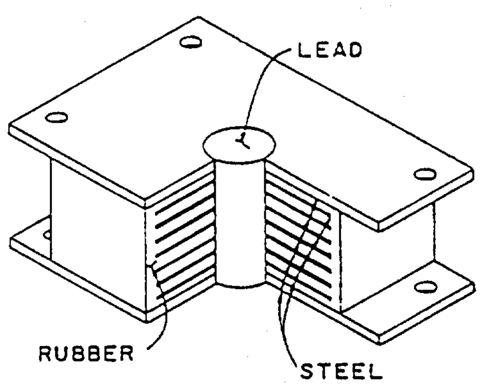


Figure 70. Lead-filled elastomeric isolation bearing.

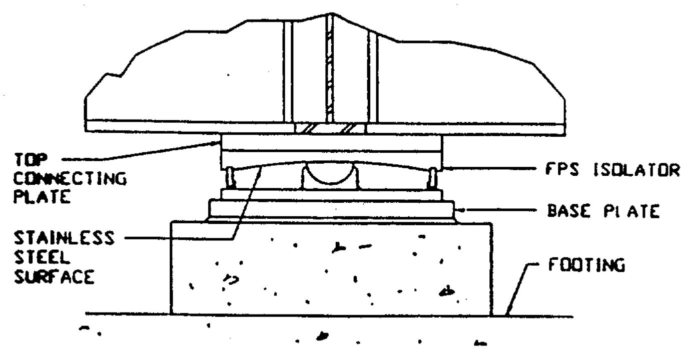


Figure 71. Friction-pendulum isolation bearing.

Table 7. Isolation hardware.

	Flexible Element	Energy Dissipation	Rigidity for Service Loads
PARTIAL SYSTEMS			
(a) Flexible Mounts	Standard elastomeric bearing Flat plate slider with low friction coefficient		
(b) Dampers		Plastic deformation of steel cantilevers and beams Lead-extrusion devices Viscous devices Friction devices	
FULL SYSTEMS			
(a) Lead-filled elastomeric bearings	Standard elastomeric bearing	Plastically deformed lead core	Elastic stiffness of lead core
(b) High-damping rubber bearings	Elastomeric bearing	Special hysteretic rubber compound	High rubber modulus at small shear strains
(c) Friction-pendulum bearings	Spherical slider with low friction coefficient	Friction	No slip until friction coefficient exceeded

Note: Some of the above hardware is patented or proprietary and is only available through licensed suppliers.

satisfactorily and should be carefully analyzed and checked for adverse response if retrofitted in this way.

Bridges that already have expansion bearings supporting the superstructure at every pier and abutment are also good candidates because bearing installation is relatively straightforward. However, the lack of these bearings does not necessarily preclude isolation, since columns may be cut and space created for the isolators to be installed.

9.6 DETAILING FOR BRIDGE MOVEMENTS

When a bridge superstructure is isolated, it must be free to move in any horizontal direction in order for the isolation to be effective. This is not usually a problem in the transverse direction, but longitudinally it means that special care is necessary at the abutments. This is because the clearance at existing expansion joints at most abutments will be insufficient to accommodate the expected movements under seismic loads. If this clearance is not increased by, for example, reconstructing the backwall of the abutment, impact will most likely occur during an earthquake. The resulting damage to the backwall is unlikely to require closure since temporary access should be easily provided.

This leads to the strategy to *not* provide the required clearance at the time of retrofitting, but to wait until the wall is damaged in an earthquake. When the wall is repaired, adequate clearances can be provided for future earthquakes. Whereas this approach is not recommended for new bridges, it has merit in retrofit situations and has been used by Caltrans in several instances.

Alternatively, the backwall can be modified at the time of retrofit and one way to do this is shown in figure 72. Here a knock-off element is incorporated into the top of the wall, which is considered to be sacrificial and easily replaced once disturbed. Impact is still to be expected in a longitudinal earthquake, but the consequences are minor. Another alternative is to provide for a much larger clearance such that impact does not occur. In this case, the gap must be bridged with an expansion joint. This may be the ideal solution from a structural response point-of-view, but may not be the most economical one since road joints that can accommodate large opening and closing movements are expensive and difficult to maintain.

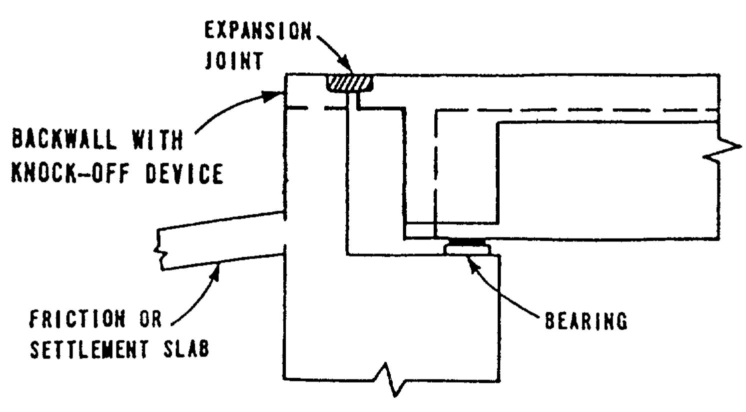


Figure 72. Knock-off device in backwall of seat-type abutment.

APPENDIX A

DETERMINATION OF SEISMIC CAPACITY/DEMAND RATIOS FOR BRIDGE COMPONENTS

A.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.

This appendix presents a methodology for calculating C/D ratios for various component failure modes. Eleven ratios are defined based on a combination of analysis, testing, and engineering judgment. The relative magnitudes of these ratios may be used to sequentially upgrade a deficient bridge. Section A.8 provides a summary of all of the component C/D ratios discussed in this appendix.

A.2 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 20 percent of the dead load of the superstructure should be assumed.

Bearing or restrainer force demands are generally obtained from an analysis of the structure. However, bearing or restrainer 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 identify bearings that have unreasonably low force capacities. These minimum forces (20 percent of the dead load) are intended for evaluation and should not be confused with minimum design forces (35 percent of the dead load) for bearing restrainers. Different minimum forces for evaluation and design are consistent with other requirements of this manual 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 engineer 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-resisting 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 adjacent bearings or restrainers.

A.3 MINIMUM SUPPORT LENGTHS

The supports at the abutments, columns, and expansion joints must be of sufficient length to accommodate anticipated relative displacements. Minimum support lengths are specified 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, N(d), for bearing seats supporting the unrestrained expansion ends of girders, as shown by the dimension N in figure 73, are used to calculate bearing displacement C/D ratios, r_{bd}, by Method 1, as described in section A.4. 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:

$$N(d) = 200 + 1.67L + 6.67H \quad \text{(mm)} \qquad \text{(A-1a)}$$
$$N(d) = 8 + 0.02L + 0.08H \quad \text{(in)} \qquad \text{(A-1b)}$$

Seismic Performance Categories C and D:

$$N(d) = 300 + 2.5L + 10H \quad \text{(mm)} \qquad \text{(A-2a)}$$
$$N(d) = 12 + 0.03L + 0.12H \quad \text{(in)} \qquad \text{(A-2b)}$$

where

L = length, in m (equations A-1a and A-2a) or ft (equations A-1b and A-2b), of the bridge deck from the support under consideration to the adjacent expansion joint or to the end of the bridge deck. For hinge seats within a span, L is the sum of L_1 and L_2, the distances on either side of the hinge. For single-span bridges, L equals the length of the bridge deck. These lengths are shown in figure 73.

For abutments:

H = average height, in m, (equations A-1a and A-2a) or ft (equations A-1b and A-2b), of columns supporting the bridge deck to the next expansion joint. H = 0 for single-span bridges.

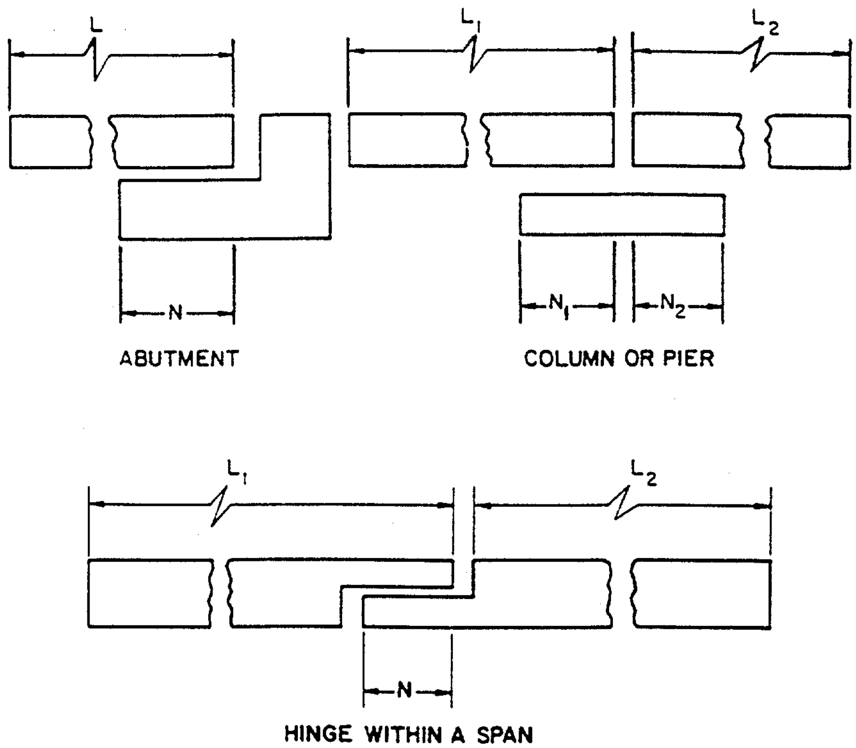


Figure 73. Minimum support length requirements.

For columns and/or piers:

H = average height, in m, (equations A-1a and A-2a) or ft (equations A-1b and A-2b), of column or pier and the adjacent two columns or piers.

For hinges within a span:

H = average height, in m, (equations A-1a and A-2a) or ft (equations A-1b and A-2b), of adjacent two columns or piers.

A.4 CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS AND BEARINGS

A.4.1 GENERAL

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. Joints 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 vulnerable 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. Some recent examples of earthquakes in which bridge collapse resulted from bearing failure include the San Fernando, California, earthquake of 1971; the Eureka, California, earthquake of 1980; the Loma Prieta, California, earthquake of 1989; and the Scotts Mills earthquake in Oregon of 1993.[5,20,21,22] 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 in 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 has been observed in the results from nonlinear analytical case studies of several bridge structures. 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 traveling surface-wave motions also tend to result in larger displacements. As noted in the previous section, minimum support lengths are required in the AASHTO specifications to allow for this possiblity. These support lengths are useful in evaluating the girder seats 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 pounding 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 74.

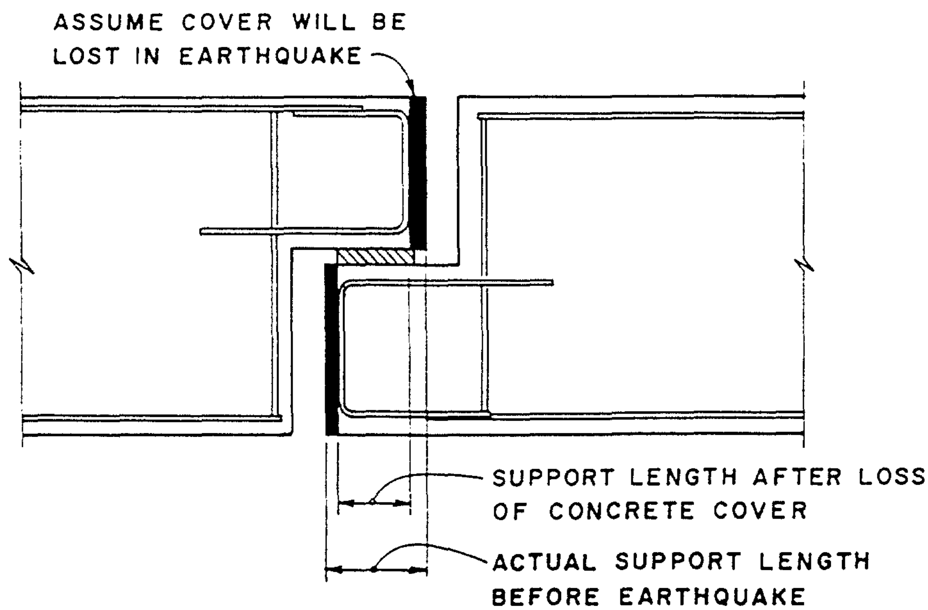


Figure 74. Effective seat width.

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 other causes, 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, 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 installed with threads that extend below the top surface of the pier or abutment seat. This gives a reduced area for shear and may reduce the flexural capacity of the bolt due to notch sensitivity at the root of the threads leading to brittle fracture.

- Anchor bolts may have insufficient uplift capacity unless provided with an embedded anchor plate.

- Anchor bolts may be too close to the edge of the bearing seat and may spall the concrete when subjected to horizontal loads.

- All of the bearings supporting one end of a span do not resist horizontal forces equally or even simultaneously. Because keeper bars or other devices are not set with exactly the same clearances, the bearings will not be equally effective in resisting load. It is quite common for bearings on the same support line to be damaged to different degrees during an earthquake.

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

A.4.2 DISPLACEMENT CAPACITY/DEMAND RATIOS

The displacement C/D ratios, r_{bd}, 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 that case, Method 2 should be used.

Method 1:

$$r_{bd} = \frac{N(c)}{N(d)} \qquad \text{(A-3)}$$

where

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

N(d) = the minimum support length defined in section A.3.

Method 2:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)} \qquad \text{(A-4)}$$

where

$\Delta_s(c)$ = the available capacity of the expansion joint or bearing for movement. For structures in SPC D, 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 of available capacity in older bridges are used for $\Delta_s(c)$, then only temperature effects need to be considered here.

$\Delta_{eq}(d)$ = the maximum calculated relative displacement due to earthquake loading for the load cases described in section 3.3.2.4.

A.4.3 FORCE CAPACITY/DEMAND RATIOS

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

$$r_{bf} = \frac{V_b(c)}{V_b(d)} \tag{A-5}$$

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 3.3.2.4, multiplied by 1.25. The minimum bearing force demand, as specified in section A.2, is used when an analysis is not performed, or when it exceeds the force demand obtained from an analysis.

A.5 CAPACITY/DEMAND RATIOS FOR REINFORCED CONCRETE COLUMNS, WALLS, AND FOOTINGS

It is not uncommon for reinforced concrete columns, walls, 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, walls, 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. Walls, 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, axes.

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, walls, and footings as illustrated in the flow chart in figure 75. 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 A.5.1 through A.5.5 describe detailed procedures for investigating different column and/or footing failure modes sometimes associated with plastic hinging.

Step 1: Determine the elastic moment demands at both ends of the column or wall for the seismic load cases described in section 3.3.2.4. Moment demands for both the columns and footings should be determined. The elastic moment demand may be taken as the sum of the absolute values of the earthquake and dead load moments.

Step 2: Calculate nominal ultimate moment capacities for both the column and the footing at axial loads equal to the dead load plus or minus the seismic axial load resulting from plastic hinging in the columns, walls, or footings as discussed in section 7.2.2 of Division I-A of the AASHTO Specifications.

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 A.5.1) and splices in longitudinal reinforcement (section A.5.2) 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 percent, 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

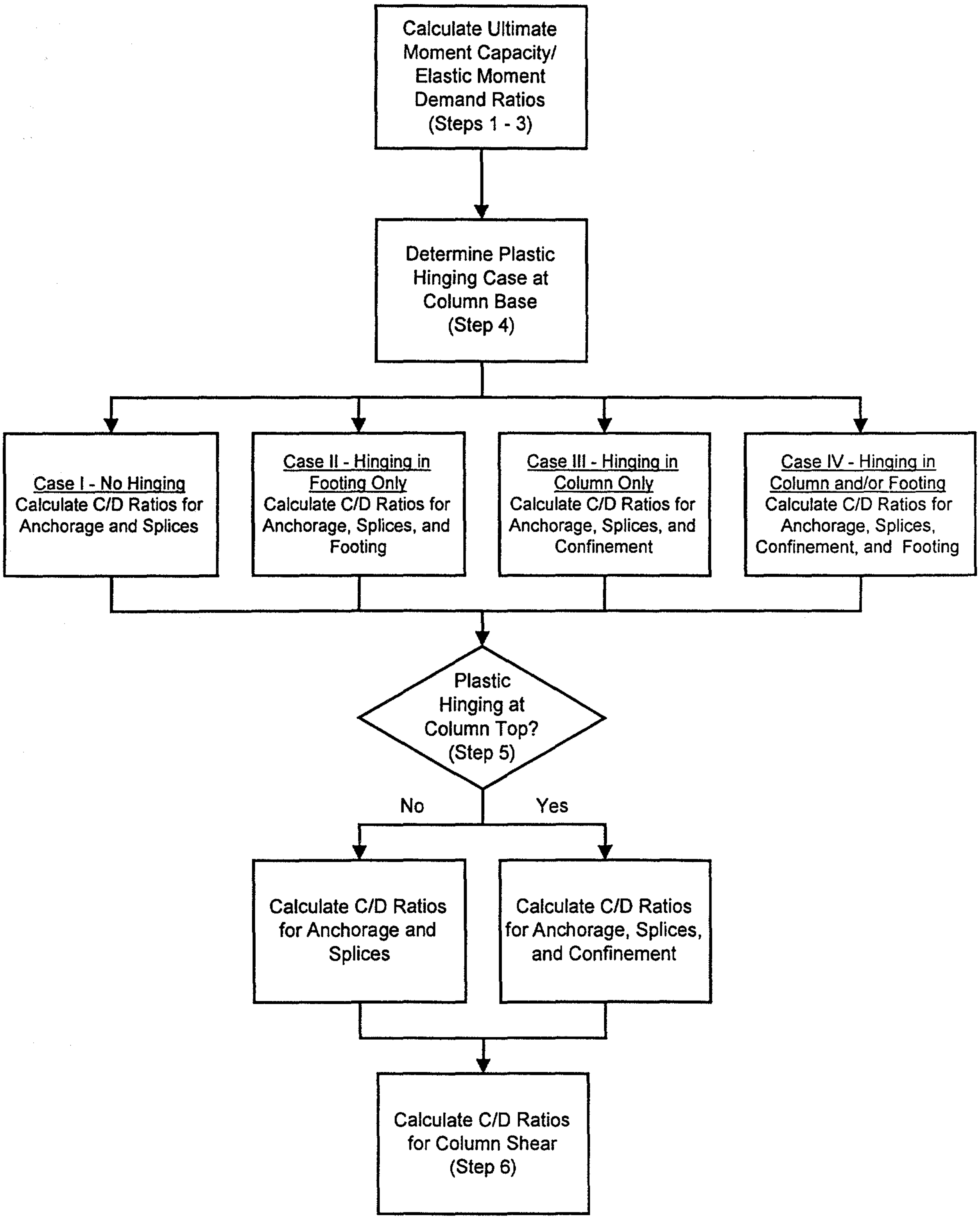


Figure 75. Procedures for determining capacity/demand ratios for columns, walls, and footings.

failures may be assumed when either the C/D ratio for anchorage of longitudinal reinforcement (section A.5.1) or for splices in longitudinal reinforcement (section A.5.2) is less than 80 percent 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 percent, 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 A.5.1), splices in longitudinal reinforcement (section A.5.2), and column transverse confinement (section A.5.4).

Case IV: When r_{ec} and r_{ef} are both less than 0.8 and within 25 percent 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 A.5.1), splices of longitudinal reinforcement (section A.5.2), and column transverse confinement (section A.5.4) should be calculated first. When all of these C/D ratios exceed 80 percent of r_{ef}, then the C/D ratio for rotation and/or yielding of the footing (section A.5.5) should also be calculated.

Step 5: Calculate the column C/D ratios for anchorage of longitudinal reinforcement (section A.5.1) and splices in longitudinal reinforcement (section A.5.2) 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 A.5.4) should also be calculated.

Step 6: Calculate the column C/D ratios for column shear (section A.5.3).

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 that 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 76 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.

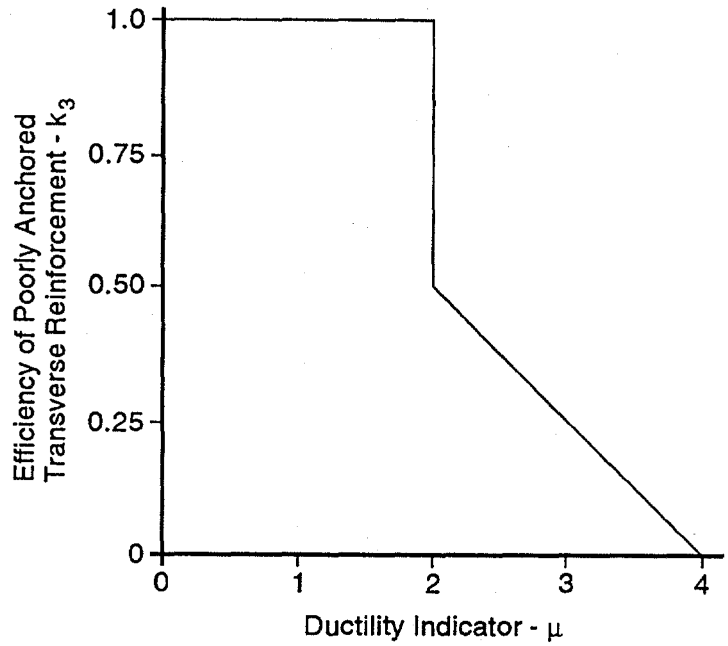


Figure 76. Effectiveness of poorly anchored transverse reinforcement as a function of ductility indicator.

Commentary

Reinforced concrete columns or walls 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, walls, 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 walls, 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 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, the state-of-the-art is such that column evaluation must rely heavily on engineering judgment, especially in the case of existing columns which may have vulnerable details.(23,24) The methods proposed for evaluating the C/D ratios are based on the latest research related to the behavior of reinforced-concrete columns, but still reflect considerable judgment 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 percent. 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 judgment, although observed column behavior during past earthquakes lends support to the conclusions drawn.

When a column failure occurs due to insufficient transverse reinforcement in any of the four potential failure modes, 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.

A.5.1 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 77.
$\ell_a(d)$ = required effective anchorage length of longitudinal reinforcement.

For straight anchorage, the effective anchorage length, in mm or in, is given by:

$$\ell_a(d) = \frac{(2.626)k_s\, d_b}{(1 + 2.5c/d_b + k_{tr})\sqrt{f_c'}} \geq 30\ d_b \quad \text{(kPa units)} \tag{A-6}$$

$$= \frac{k_s\, d_b}{(1 + 2.5c/d_b + k_{tr})\sqrt{f_c'}} \geq 30\ d_b \quad \text{(psi units)}$$

where

k_s = a constant for reinforcing steel with a yield stress of f_y (kPa or psi) $(f_y - 75{,}845)/33.1$ kPa or $(f_y - 11{,}000)/4.8$ psi
d_b = nominal bar diameter (mm or in).

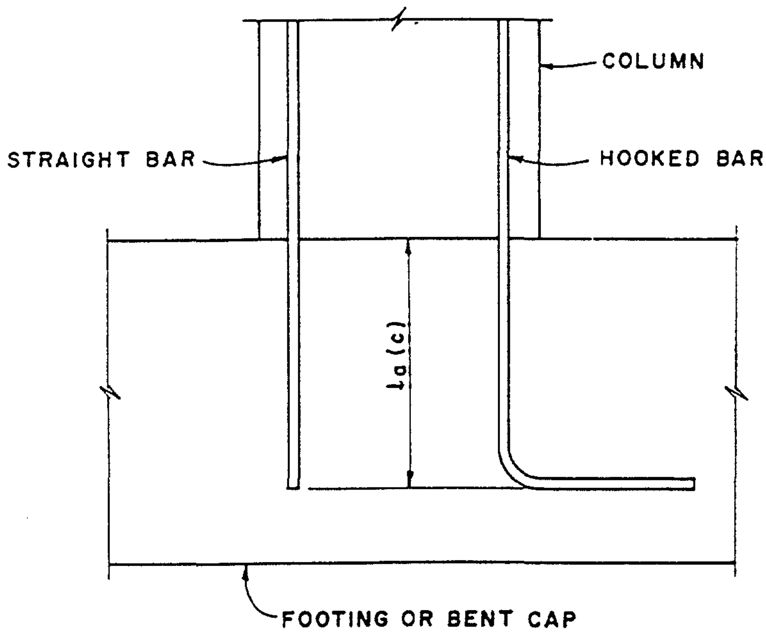


Figure 77. Effective anchorage length of longitudinal reinforcement.

f'_c	=	concrete compression strength (kPa or psi).	
c	=	the lesser of the clear cover over the bar, or half the clear spacing between adjacent bars.	
k_{tr}	=	$(A_{tr}(c)f_{yt})/(4137\ sd_b) \leq 2.5$ (kPa units)	(A-7)
	=	$(A_{tr}(c)f_{yt})/(600\ sd_b) \leq 2.5$ (psi units)	

where variables used to calculate k_{tr} are:

$A_{tr}(c)$	=	area of transverse reinforcing normal to potential splitting cracks. 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 (kPa or psi).
s	=	spacing of transverse reinforcement (mm or in).

Note that the value for c/d_b should not be taken as more than 2.5.

For anchorage with 90° standard hooks, the effective anchorage length, in mm, is:

$$\ell_a(d) = 1200 k_m\ d_b \left(\frac{(2.626) f_y}{60000 \sqrt{f'_c}} \right) > 15\ d_b \quad \text{(kPa units)} \qquad \text{(A-8)}$$

$$\ell_a(d) = 1200 k_m\, d_b \left(\frac{f_y}{60000 \sqrt{f_c'}} \right) > 15\ d_b \quad \text{(psi units)}$$

where

k_m	=	0.7	for #11 bars or smaller, when side cover (normal to plane of the hook) is not less than 63 mm (2.5 in), and cover on the bar extension beyond the hook is not less than 50 mm (2 in).
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 78. 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.

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

$$r_{ca} = \frac{\ell_a(c)}{\ell_a(d)} r_{ec} \tag{A-9}$$

Case B: 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:

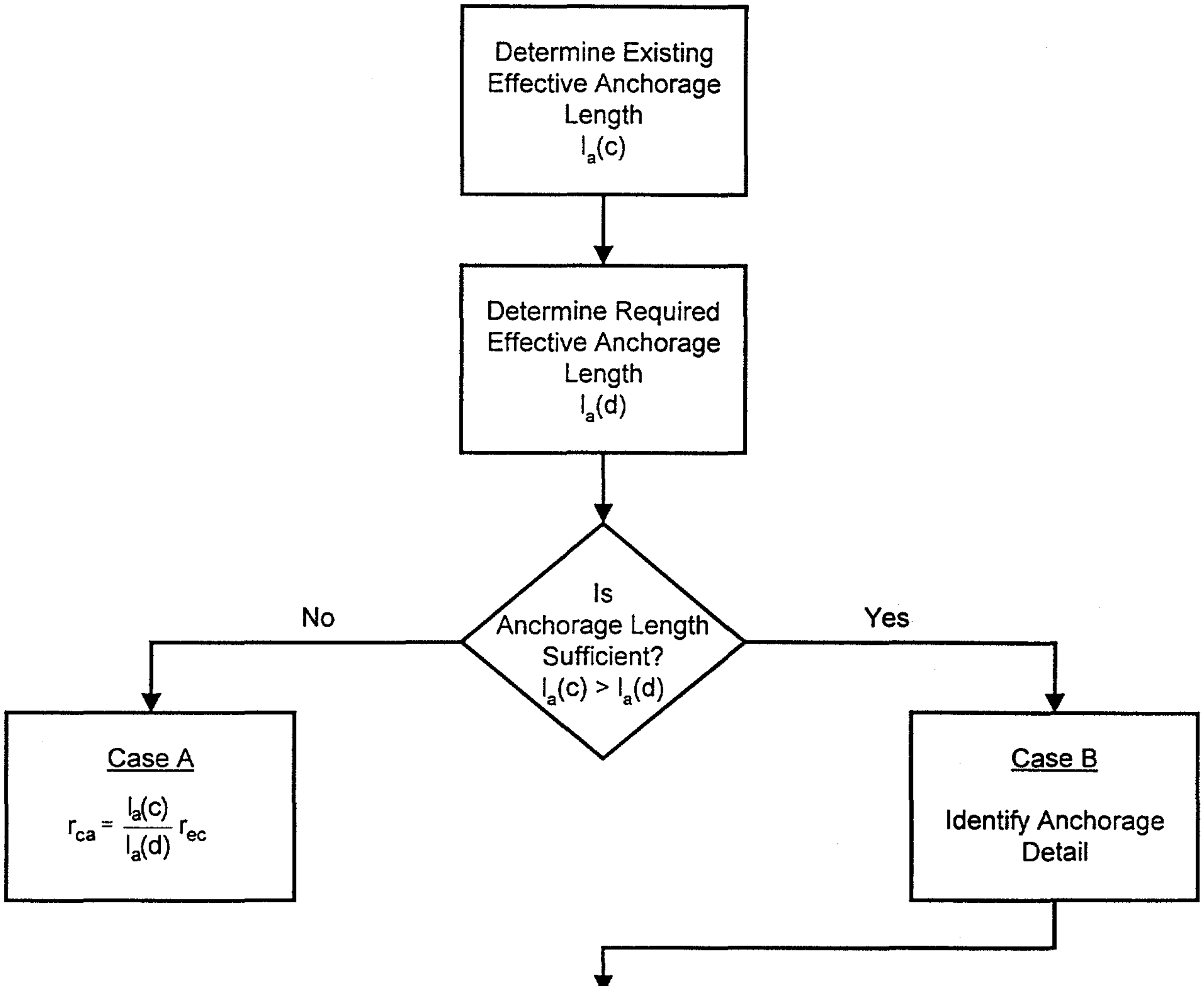
Detail 1: When no flexural tensile reinforcement is present in the top of the footing and column bar development is by straight anchorage, i.e., no hooks are present at the bottom of the footings,

$$r_{ca} = r_{ef} \tag{A-10}$$

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 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, turned away from the column towards the edges of the footing,

$$r_{ca} = 1.3\ r_{ef} \leq 1.0 \tag{A-11}$$



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 centerline	No	$r_{ca} = 1.3\ r_{ef}$
3	Footing	90° hook toward centerline	No	$r_{ca} = 2.0\ r_{ef}$
4	Footing	Straight	Yes	$r_{ca} = 1.5\ r_{ef}$
5	Footing	90° hook	Yes	1.0
6	Bent Cap	---	---	1.0

Figure 78. Procedure for determining capacity/demand ratios for anchorage of longitudinal reinforcement.

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,

$$r_{ca} = 2\ r_{ef} \leq 1.0 \tag{A-12}$$

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,

$$r_{ca} = 1.5\ r_{ef} \leq 1.0 \tag{A-13}$$

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 adequately anchored 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 anchorage is in a bent cap, the C/D ratios for anchorage should also be taken as 1.0.

Commentary

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 yield in the footing. This is accounted for by multiplying the moment C/D 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 C/D 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 A-6 and A-8, based on the ratio 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 progress 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 judgment.

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.[25] 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 79. 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 splitting cracks may occur between adjacent bars, resulting in all bars failing in anchorage as a group and pulling out of the footings as

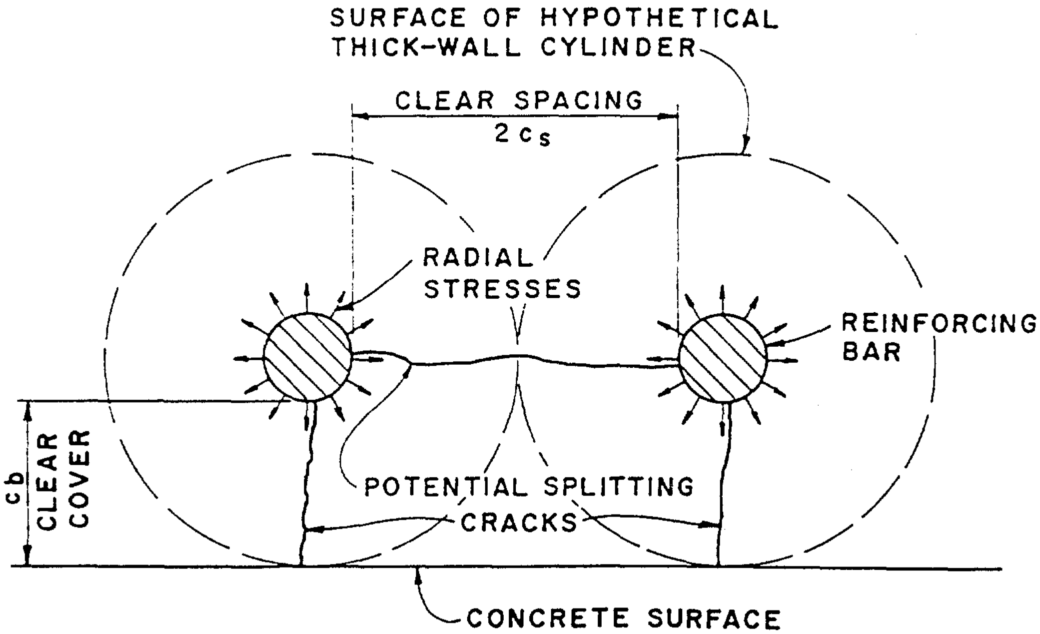


Figure 79. Radial stresses developed due to bar anchorage.

a plug. In this case, the amount of transverse steel, $A_{tc}(c)$, can be assumed to be twice the cross-sectional area of a single hoop divided by half the number of anchored bars. A similar group anchorage failure can occur with columns having cross sections of different shapes.

A.5.2. 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 yield strength of the spliced bars is given by:

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

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 $4d_b$, then $A_{tr}(c)$ 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 $4885\ d_b/\sqrt{f'_c}$ mm ($1860\ d_b/\sqrt{f'_c}$ in).

The procedure for calculating the seismic C/D ratio for splices in longitudinal reinforcement, r_{cs}, is shown in figure 80. 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 splice length, transverse reinforcement amount, or transverse reinforcement spacing is inadequate [$\ell_s < 4885\ d_b/\sqrt{f'_c}$ mm ($1860\ d_b/\sqrt{f'_c}$ in); $A_{tr}(c) < A_{tr}(d)$; or $s > 150$ mm (6 in)], then the C/D ratio for splices in longitudinal reinforcement, r_{cs}, is given by:

$$r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)}\left(\frac{\left(\frac{150}{s}\right)\ell_s}{\left(\frac{4885}{\sqrt{f'_c}}\right)d_b}\right)r_{ec} \leq \frac{A_{tr}(c)}{A_{tr}(d)}\, r_{ec} \text{ (mm and kPa)} \qquad \text{(A-15)}$$

$$r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)}\left(\frac{\left(\frac{6}{s}\right)\ell_s}{\left(\frac{1860}{\sqrt{f'_c}}\right)d_b}\right)r_{ec} \leq \frac{A_{tr}(c)}{A_{tr}(d)}\, r_{ec} \text{ (in and kips)}$$

where the ratio 150/s (6/s) should not be taken larger than 1 and $4885/\sqrt{f'_c}$ ($1860/\sqrt{f'_c}$) should not be taken less than 30. The C/D ratio for splices, r_{cs}, need not be taken as less than $0.75r_{ec}$ when the minimum splice length is provided.

Case B: When the splice is sufficient [$\ell_s > 4885\ d_b/\sqrt{f'_c}$ mm ($1860\ d_b/\sqrt{f'_c}$ in); $A_{tr}(c) \geq A_{tr}(d)$; and $s \leq 150$ mm (6 in)], then the C/D ratio for splices in longitudinal reinforcement, r_{cs}, is given by:

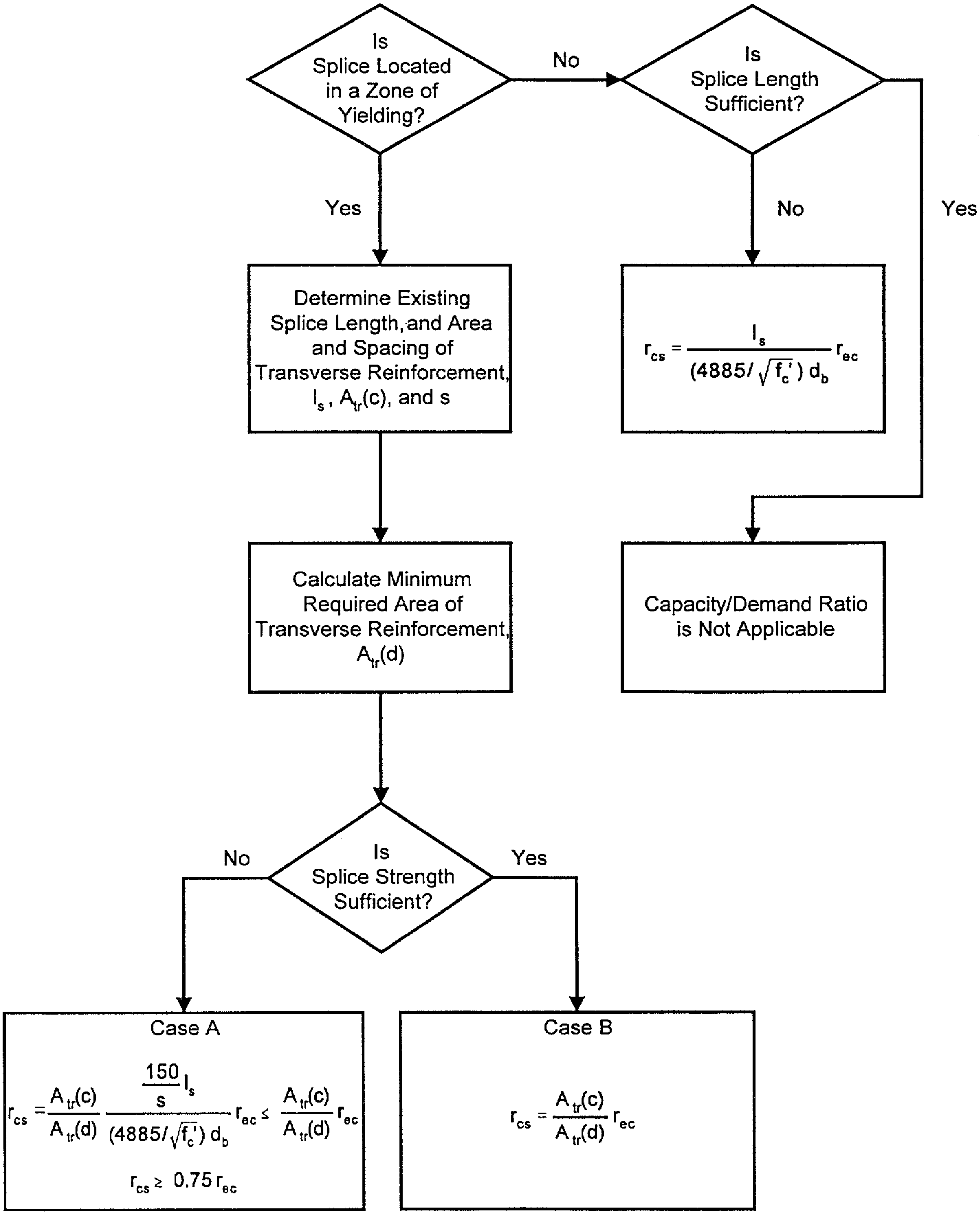


Figure 80. Procedure for determining capacity/demand ratios for splices in longitudinal reinforcement (mm and kPa).

$$r_{cs} = \frac{A_{tr}(c)}{A_{tr}(d)} r_{ec} \leq 2r_{ec} \tag{A-16}$$

Commentary

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 propagate 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 and the University of Canterbury in New Zealand.(26,27,28) 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 identifying 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 unyielding 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 unyielding case was used, splices were shown to be capable of withstanding reversed loading with displacement ductilities as high as 6 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 judgment, 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 C/D ratio for splices should not be less than 0.75 r_{ec} when sufficient splice length is provided.

A.5.3 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 (if both ends are fixed or at one end if the other end is pinned) 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 the resistance of the concrete core of the column and only that transverse steel which is effectively anchored. When the axial stress is greater than or equal to 0.10 f'_c, an allowable shear stress of $5.2\sqrt{f'_c}$ kPa ($2\sqrt{f'_c}$ psi) may be assumed for the core of the concrete column. Otherwise, a value of zero will be assumed.

Note 1: Procedures for calculating shear forces resulting from column hinging are given in section 7.2.2 of Division I-A of the AASHTO Specifications. 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. The shear force associated with a pinned end should also be included. Few pin connections are frictionless, which will cause a shear demand during member rotation.

Note 2: The shear resistance of concrete columns is calculated using the provisions of section 8.16.6 of Division I of the AASHTO Specifications, except that capacity reduction factors are not used, i.e., $\phi = 1.0$.

The procedure for calculating the C/D ratio for column shear is shown in figure 81.

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

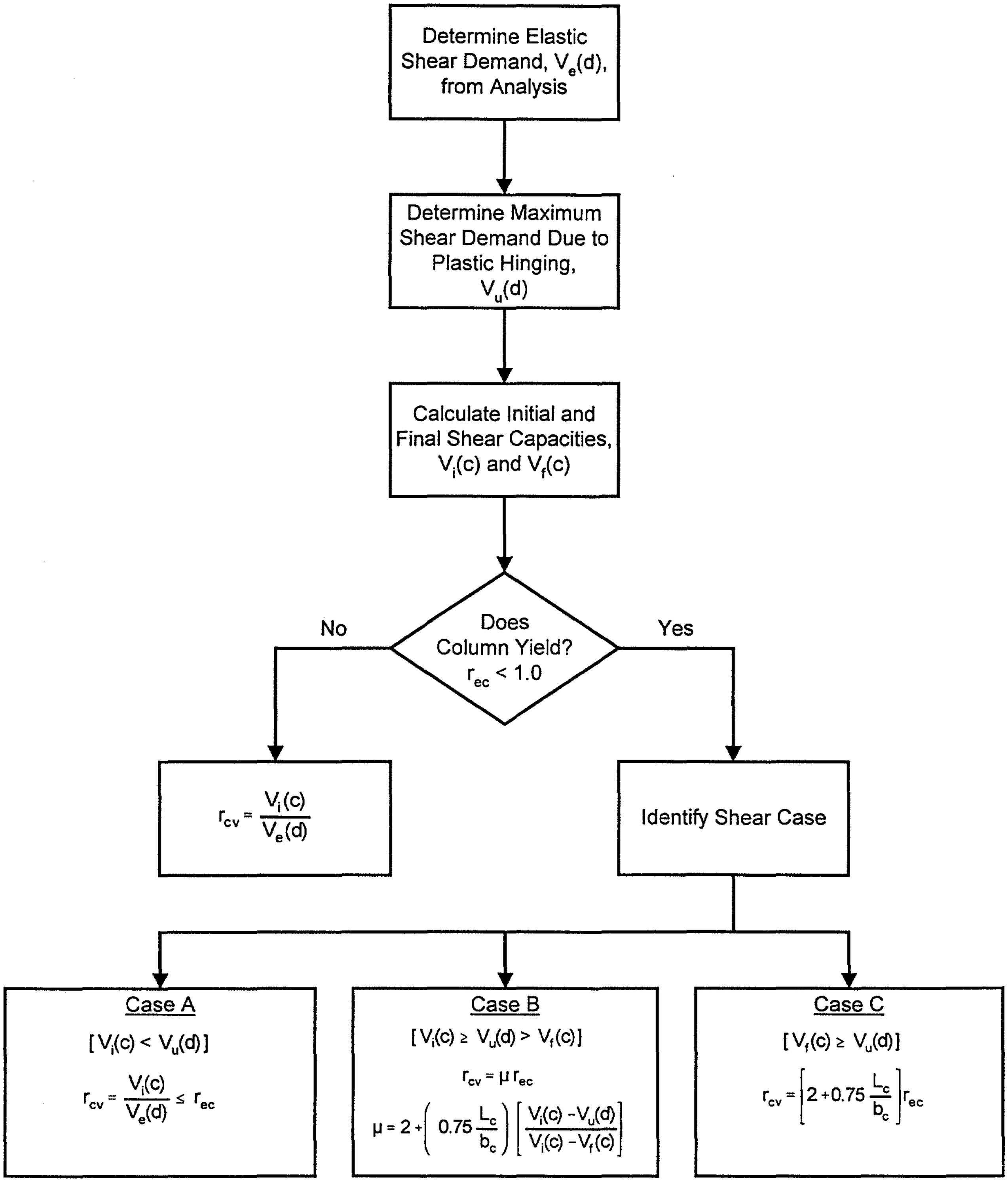


Figure 81. Procedure for determining capacity/demand ratios for column shear.

for column shear, r_{cv}, is calculated according to the procedure outlined in figure 81. Each of three possible cases are described below.

Case A: If the initial 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, i.e.,

$$r_{cv} = \frac{V_i(c)}{V_e(d)} \leq r_{ec} \tag{A-17}$$

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:

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

where

$$\mu = 2 + \left(0.75\frac{L_c}{b_c}\right)\frac{V_i(c) - V_u(d)}{V_i(c) - V_f(c)} \tag{A-19}$$

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 A-19.

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:

$$r_{cv} = \left(2+0.75\ \frac{L_c}{b_c}\right) r_{ec} \tag{A-20}$$

where the terms are defined above. As with Case B, the column height-to-width ratio should not be taken to be greater than 4.

Commentary

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 judgment 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 82. 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 initial 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 1.0 or less. At a ductility indicator above 1.0, the shear demand is

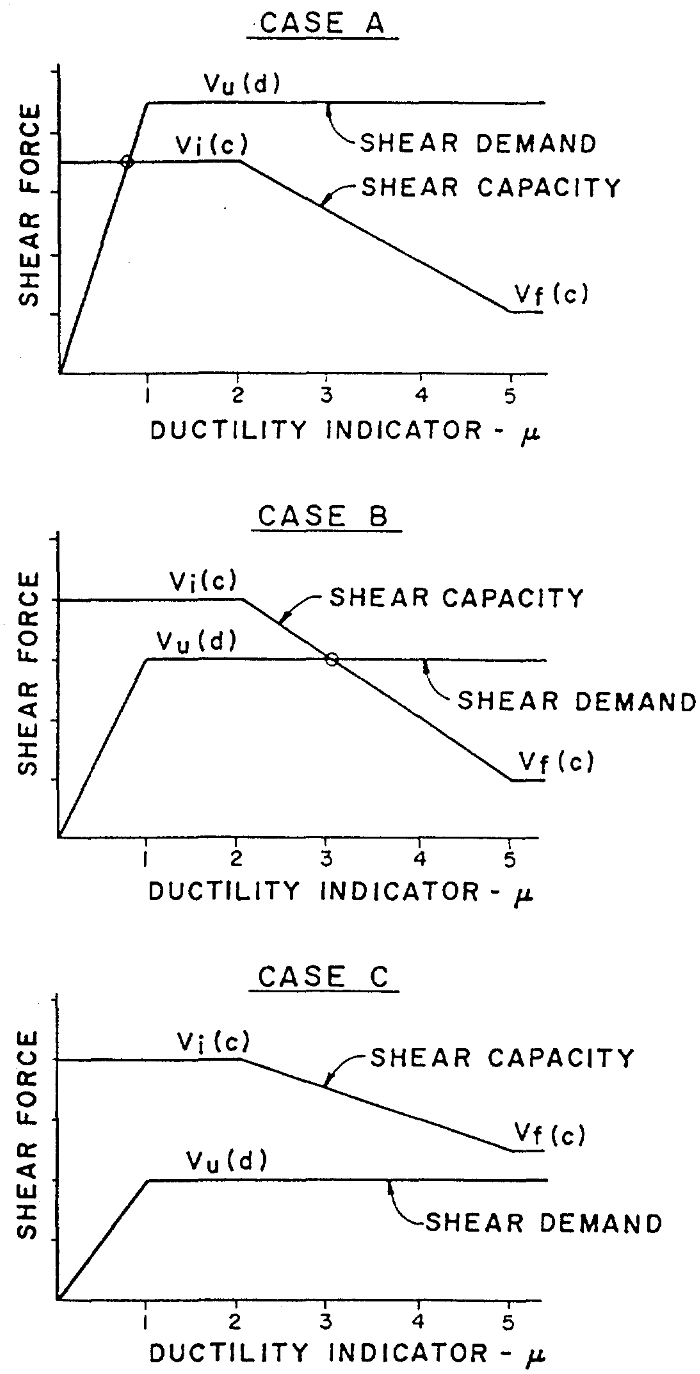


Figure 82. Resolution of shear demand and shear capacity.

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 1.0 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 judgment 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 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 this manual. 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 2 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 2, 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, this manual considers height-to-width ratios when evaluating columns for a Case B or Case 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 existing columns were built prior to 1973, when transverse reinforcement anchorage requirements were first included in the AASHTO 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 that is 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 researchers have suggested that the concrete shear resistance be ignored in columns with an average axial stress below 0.10 f_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 this manual 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 this manual, 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.

A.5.4 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}:

$$r_{cc} = \mu r_{ec} \tag{A-21}$$

where

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

where

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

k_2 = $6d_b/s \leq 1$ or $0.2\ b_{min}/s \leq 1$, 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 76 shall be used to determine k_3. Note that when this is the case, an itertative solution of equation A-22 will be required.
ρ (c) = volumetric ratio of existing transverse reinforcement.
ρ (d) = required volumetric ratio of transverse reinforcement determined in accordance with the provisions of section 7.6 of Division I-A of the AASHTO Specifications.
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.

Commentary

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 AASHTO Specifications 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.[23] 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 percent) 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).[29] The confinement provisions of NZS 3101 are based on the confinement requirements used in the AASHTO Specifications 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 AASHTO Specifications. The New Zealand code requires that maximum spacing of transverse steel for adequate concrete confinement be 20 percent 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.[30]

Despite the work mentioned in the previous paragraphs, the evaluation of transverse confinement in existing columns must be tempered with judgment 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 uses 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 judgment, it will allow for a reasonably accurate evaluation of the C/D ratio for transverse confinement.

A.5.5 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:

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

where μ, the ductility indicator, is taken from table 8 and depends on the type of footing and mode of failure. See the commentary below for a discussion on the method for calculating the nominal ultimate capacity of the footing.

Table 8. 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 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

Commentary

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 A.7.

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.

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, as shown in figure 83:

- 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 that, in some cases, 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 A.5.1. 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 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

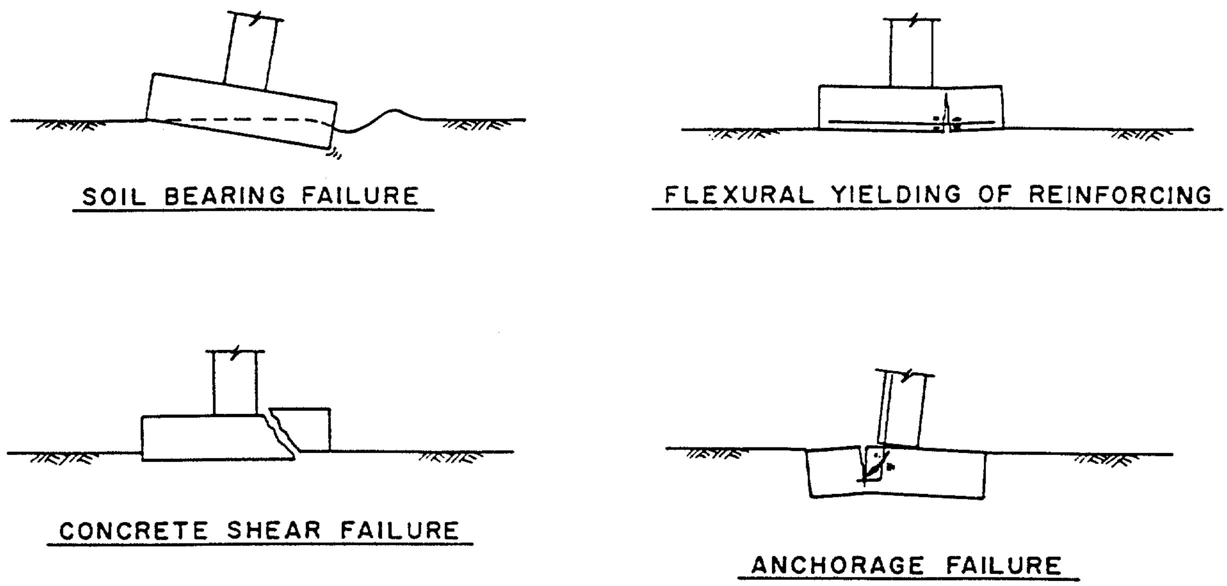


Figure 83. Modes of failure for spread footings.

should be checked at the critical section according to the AASHTO Specifications. This capacity should be sufficient to resist uniform footing pressures 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 that will indicate the possibility of bearing failure only at the locations where this surface falls within the column interaction surface factored for overstrength. Ultimate soil bearing pressures can generally be taken at three times the design allowable value. The actual ultimate capacity should be provided by the geotechnical engineer.

This mode of "failure" is considered acceptable by Caltrans because it generally does not lead to structure collapse. Retrofitting would only be considered if the structure was required to perform to a higher level, i.e., to meet certain functionality criteria immediately following an earthquake.

Pile Footings

Possible failure modes for pile footings are shown in figure 84 and may be classified as follows:

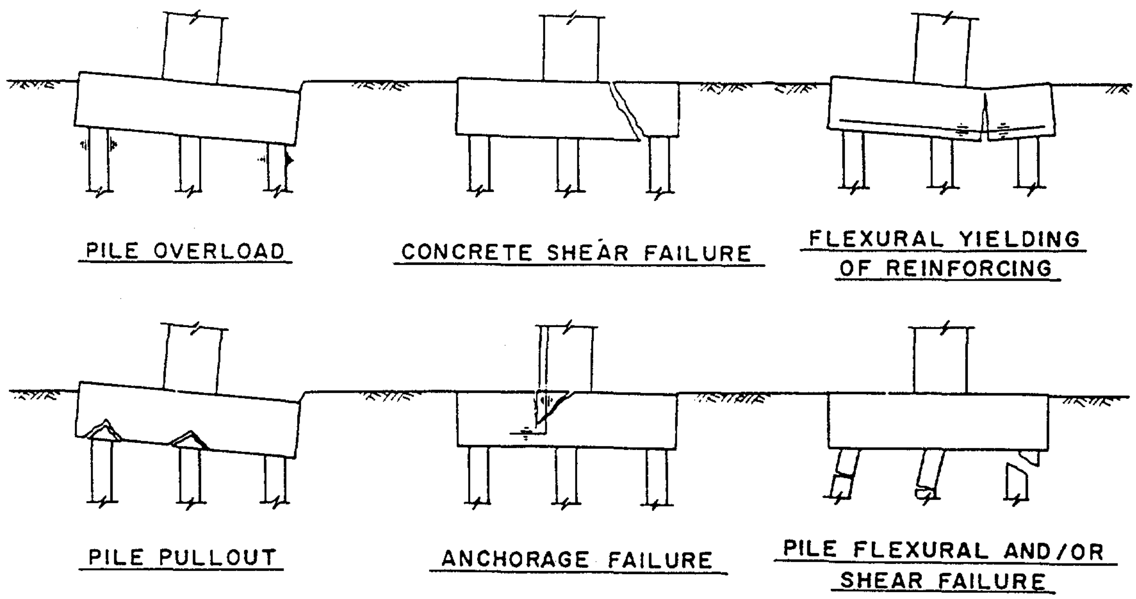


Figure 84. Modes of failure for pile footings.

- 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 A.5.1 of this manual.

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 the AASHTO Specifications. This capacity should be sufficient to resist the moment produced by the piles acting at 1.3 times their ultimate 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 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. Ultimate pile compression capacity for friction piles can generally be taken at four times the design allowable capacity. The actual capacity should be provided by the geotechnical engineers.

This mode of "failure" is considered acceptable by Caltrans because it generally does not lead to structure collapse. Retrofitting would only be considered if the structure was required to perform to a higher level, i.e., to meet certain functionality criteria immediately following an earthquake.

Geotechnical specialists should be consulted for the ultimate capacity of the soils, especially with respect to the uplift capacity. At poor soil sites, the potential degradation of soil strength should be evaluated. The soil capacities can then be compared with the capacities of the connection details and the pile members based on the pile type and the expected failure mode, i.e., pile overload or pile pullout at the footings.

A.6 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 to seismically induced 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 modelling the abutment stiffness (see section 3.3.2.2(a)). The displacement capacity, D(c), is taken as 75 mm (3 in) in the transverse direction and 150 mm (6 in) in the longitudinal direction, unless determined otherwise by a more detailed evaluation. Therefore:

$$r_{ad} = \frac{D(c)}{D(d)} \tag{A-24}$$

Commentary

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 75 mm (3 in) in the transverse direction and 150 mm (6 in) in the longitudinal direction were chosen. These values are based largely on engineering judgment and are likely to be modified as more experience is gained.

A.7 CAPACITY/DEMAND RATIOS FOR LIQUEFACTION INDUCED FOUNDATION FAILURE

Many foundation failures that occur during earthquakes are the result of loss of foundation support due to liquefaction. A C/D ratio should be calculated when the preliminary screening indicates the potential exists for 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. Second, 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:

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

where

$A_L(c)$ = effective peak ground acceleration at which liquefaction failures are likely to occur.
$A_L(d)$ = A, the 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 3 m (10 ft) depth of liquified soil to occur near the pile head followed by 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 may be of assistance 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.

Commentary

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 liquefaction is provided in chapter 3. Bridge failures in recent Alaskan and Japanese earthquakes are probably the best documented examples.[6,7] 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 Alaskan 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 liquefaction is a combination of earthquake intensity and duration. In the 1964 Alaskan earthquake, it is estimated that maximum ground accelerations as low as 0.1g to 0.2g were responsible for the extensive and widespread bridge foundation failures.[6] 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 AASHTO Specifications. 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 AASHTO Specifications.

A.8 SUMMARY

This appendix presented a methodology for determining the capacity/demand ratios for displacements and forces for a number of bridge components, including expansion joints and bearings, reinforced concrete columns and walls, and foundations. A concise summary of these C/D ratios is provided in table 9.

Table 9. List of capacity/demand ratios.

Symbol	Definition	Equation	Page
r_{ad}	Displacement ratio for abutment	A-24	196
r_{bd}	Displacement ratio for bearing seat or expansion joint	A-3, A-4	166
r_{bf}	Force ratio for bearing or expansion joint restrainer	A-5	167
r_{ca}	Anchorage length ratio for column longitudinal reinforcement	A-9 through A-13	174-176
r_{cc}	Confinement ratio for column transverse reinforcement	A-21	188
r_{cs}	Splice length ratio for column longitudinal reinforcement	A-15, A-16	179-181
r_{cv}	Shear ratio for column	A-17 through A-20	184-185
r_{ec}	Moment ratio for column (Steps 1-3)	–	168
r_{ef}	Moment ratio for footing (Steps 1-3)	–	168
r_{fr}	Rotation ratio for footing	A-23	190
r_{sl}	Acceleration ratio for liquefaction potential	A-25	196

APPENDIX B

ASSESSMENT OF MEMBER STRENGTH AND DEFORMATION CAPACITY

The structure lateral strength method of bridge evaluation requires the assessment of member strength and deformation capacity. This appendix addresses this issue; it is extracted from reference 18.

B.1 ELASTIC DISPLACEMENTS

The approach outlined in section 3.5 requires the estimation of ductility capacity by comparing ultimate and yield displacements. Thus, in addition to providing a best estimate of plastic displacement, equal emphasis must be placed on providing a best estimate of yield displacement. Clearly, this means consideration must be given to the stiffness of cracked sections, rather than the use of gross member stiffnesses. Natural periods should also be based on the stiffness of members at first yield. Some attempt should therefore be made to assess which members will be essentially uncracked and which will be cracked, before computing elastic displacements. For example, in the assessment of the longitudinal response of bridge structures with prestressed superstructures, it may be appropriate to use gross section properties for the superstructure and cracked-section properties for the columns.

The stiffness of columns at first yield of the longitudinal reinforcement is a function of the material properties, the longitudinal reinforcement ratio, and the axial load level. Figure 85 shows the moment of intertia ratio, I_{eff}/I_{gross}, for circular and rectangular columns calculated for typical variations of the critical parameters. The value I_{eff} in figure 85 takes into account the distribution of cracking up the height of a column, with some sections cracked and others uncracked, and can therefore be used as an average value applicable to the full column height.

The value of I_{gross} in figure 85 is based on the concrete section alone. This is,

for circular sections:

$$I_{gross} = \frac{\pi D^4}{64} \tag{B-1a}$$

for rectangular sections:

$$I_{gross} = \frac{bh^3}{12} \tag{B-1b}$$

where D is the diameter of a circular column, and b and h are the gross dimensions of a rectangular section. As discussed in section B.2, the concrete compression strength, f'_c should be a probable, rather than minimum, level and the modulus of elasticity estimated from:

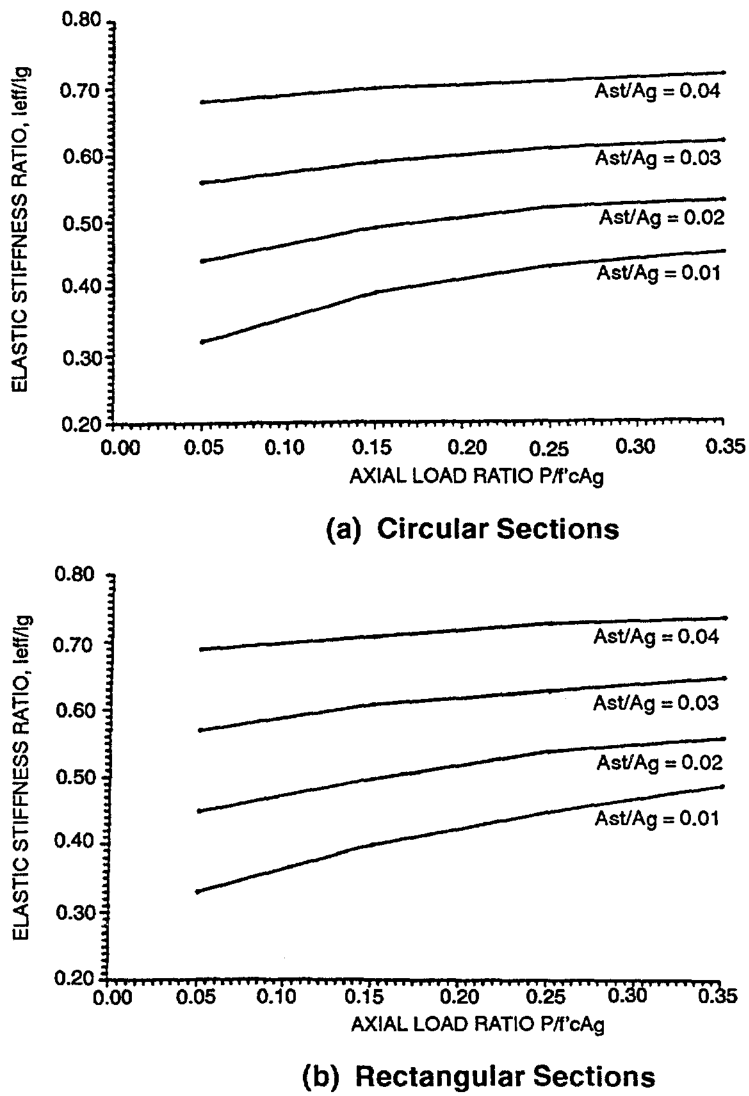


Figure 85. Effective stiffness of bridge columns (from ref. 18).

$$E = 4980\sqrt{f_c'} \quad (\text{MPa}) = 60{,}000\sqrt{f_c'} \quad (\text{psi})$$

Elastic displacements and natural periods should include the influence of foundation flexibility effects modelled by appropriate translational and rotational springs, or by the use of elastic or inelastic Winkler foundation springs.

Yield displacements should correspond to the elastic displacement at the lateral force required to develop the full plastic mechanism strength, without considering strain-hardening effects, as implied by figures 21 and 23. Displacements computed at first yield of the extreme tension reinforcement do not correspond to the bilinear approximation of inelastic response implied in the approach outlined in section 3.5 for estimating equivalent elastic lateral force.

B.2 FLEXURAL STRENGTH

B2.1 GENERAL

Critical sections to be assessed for flexural strength will include columns, cap beams, footings, and superstructure (for longitudinal response). Actual, rather than the minimum specification, material strengths should be used. For concrete compression strength, this should preferably be based on compression tests of cores, but carefully interpreted impact hammer or sonic tests may also be used. Failing these direct methods, a conservatively low value of 1.5 times the specified strength may be used. The 50 percent increase over specified strength recognizes the typically conservative mix designs of bridges during the period 1940 through 1970, and makes some allowance for natural strength gain with age. Cores of concrete from Californian bridges built in the 1950's and 1960's have produced results consistently in excess of 1.5 times the specified strength. However, if the concrete appears to be in poor condition in critical regions, strength testing is essential.

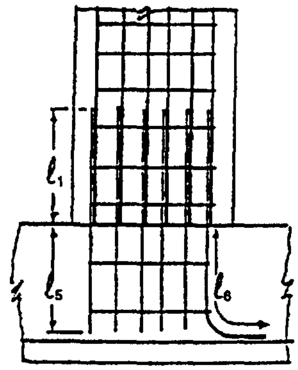
Yield strength of reinforcement should be based on mill certificate or tensile test results if these are available in the bridge archives. If not, a nominal strength of 1.1 times specified minimum strength should be assumed, resulting in 300 MPa (44 ksi) and 450 MPa (66 ksi) for grade 40 and grade 60 reinforcement, respectively. These are slightly on the low side of actual average strengths.

An ultimate compression strain of 0.005 may be used for assessing flexural strength, since tests on beams and columns subjected to moment gradients invariably result in strains corresponding to first crushing in excess of 0.005. An advanced section analysis technique using a compression stress-strain model, capable of representing the influence of confinement should be used.

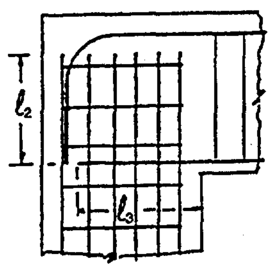
Flexural strength of sections should be based on a realistic assessment of member axial force, including seismic effects. Since these will depend on the capacity of the lateral mechanism, and hence on member strengths in plastic hinge regions, some iteration may be needed.

B.2.2 SECTIONS WITH INADEQUATELY ANCHORED OR SPLICED BARS

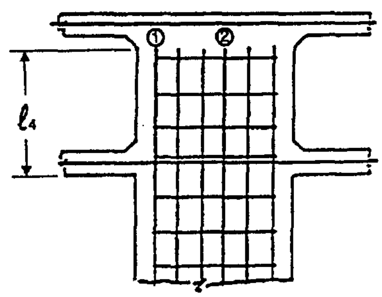
Inadequately anchored or spliced bars are common in older bridges, examples of which are shown in figure 86. The assessment of strength and ductility of such details is difficult, since existing design methods do not produce good representation of actual performance. Two separate conditions can be identified: confined and unconfined splices or anchorages. Confinement may result from a large amount of well-anchored transverse reinforcement around a lap splice, or by the influence of transverse reinforcement intended for flexural reinforcement in an adjacent member. The lap, ℓ_1, at the base of a column in figure 86(a) is unlikely to be adequately confined if designed before 1970, since transverse reinforcement is likely to consist of #4 hoops at 300 mm (12 in) centers. This produces a maximum confinement stress of only 172 MPa (25 psi) in a circular column of 1500 mm (60 in) in diameter, and less in an equivalent rectangular column.



(a) Column Base



(b) Knee Joint



(c) Column Top

Figure 86. Anchorage of reinforcement (from ref. 18).

A similar situation occurs with the length ℓ_2 at the top of the column in figure 86(b). The situation at the outside will be similar to the base of the column as a result of inclined shear cracking in the joint, but at the inside of the joint the vertical bars will be better anchored because of the clamping effect of the horizontal beam reinforcement and the restraint provided by adjacent concrete, compared with the cover concrete at the outside of the joint which can be expected to spall off.

Anchorage of the horizontal bottom beam steel in figure 86(b) will be assisted by clamping pressure provided by the vertical reinforcement. However, if the column reinforcement is expected to yield with high ductility demand, this clamping pressure may become ineffective.

At the top of the column shown in figure 86(c), where the column frames into a transverse cap beam, the anchorage of bars close to the longitudinal axis (identified as point 1 in figure 86(c)) is unlikely to benefit from any clamping pressure other than that provided by transverse hoops. Bars close to the transverse axis (i.e., bars at point 2) may, however, have improved anchorage as a result of clamping action provided by the main cap beam flexural reinforcement or transverse prestressing.

Lap splices in column bars of bridges designed prior to 1970 were typically in the range of 20 to 35 longitudinal bar diameters. Anchorage lengths are often much less where the reinforcement is anchored in footings (length ℓ_5 in figure 86(a)), though when bent out to provide support for the column cage, as with ℓ_6 in figure 86(a), anchorage will normally be adequate. It should, however, be noted that the practice of bending the column bars out creates an undesirable situation for the transfer of moment between column and footing, and may also reduce shear strength in the critical joint region under the column.

Little research has been directed towards understanding the behavior of lap splices at the critical sections of columns. However, recent research on the behavior of "as-built" circular and rectangular bridge columns, modelling typical pre-1971 designs, supports the mechanism suggested in figure 87.

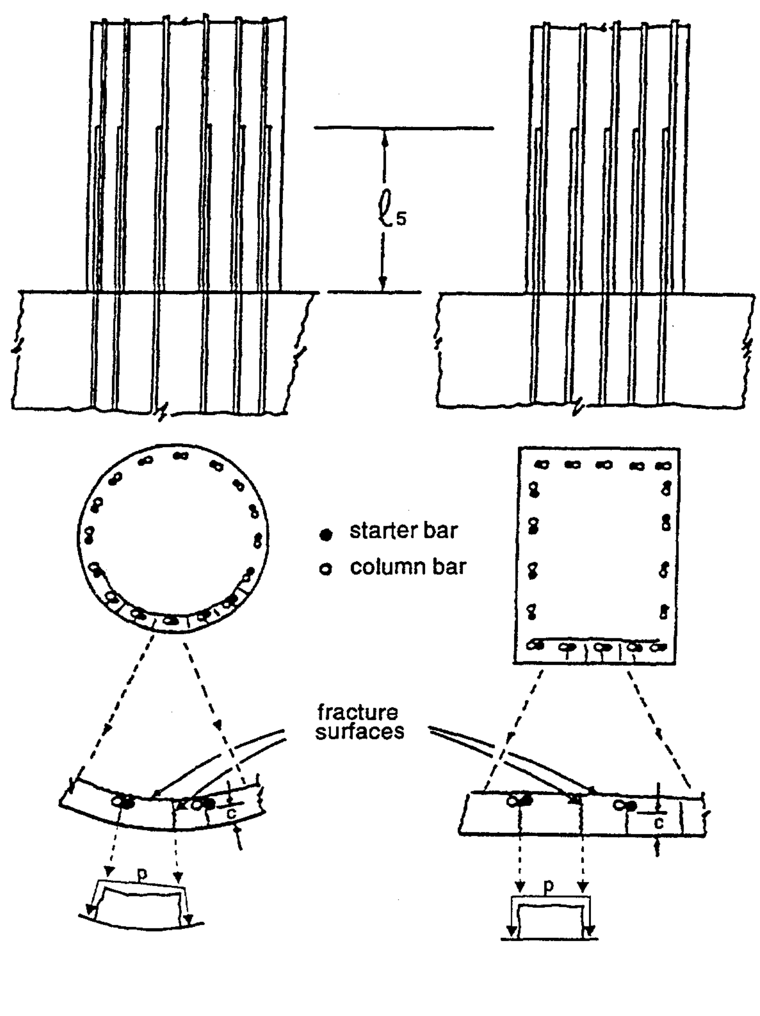


Figure 87. Lap splices in columns (from ref. 18).

The failure mechanism involves the development of vertical cracks parallel to the column bars as is evidenced by test results. For these cracks to develop, it may readily be shown that a second crack surface inside the column bars and parallel to the plane of reinforcement must develop to facilitate the lateral dilation implied by the vertical cracks.

For failure to occur, it may be hypothesized that the area associated with a column bar must separate from the concrete attached to the starter bar. The tensile stress necessary to fracture this surface may be assumed equal to the direct tension strength, i.e., $f_t = 10.4\sqrt{f'_c}$ kPa ($4\sqrt{f'_c}$ psi).

For a circular column, the total tensile force on the rupture surface at failure will be:

$$T_b = f_t \left[\frac{\pi D'}{2n} + 2(d_b + c) \right] \ell_s = f_t \, p \ell_s \qquad \text{(B-2)}$$

where

p	=	perimeter of the crack surface
n	=	number of verticle bars
c	=	bar cover
d_b	=	bar diameter
D'	=	bar pitch circle diameter
ℓ_s	=	splice length

The extreme right expression in equation B-2 also applies for rectangular columns, where p is based on the local bar spacing. The parameter T_b is the maximum force that can be developed in the column bar unless significant transverse confinement is present. If $T_b > A_b f_y$, the bar can develop its yield force and the initial ideal flexural strength may be developed. If $T_b < A_b f_y$, bond failure will occur at less than the flexural strength, with rapid strength degradation under cyclic loading.

Even if equation B-2 indicates that the bar strength can be fully developed, available ductility will be small. As compression strains adjacent to the compression reinforcement approach 0.002, extensive longitudinal microcracking will develop. This will clearly degrade the effective tension strength, f_t, in equation B-2 on reversal of the direction of seismic response, reducing T_b, and causing bond failure. It would appear that bond failure will be inevitable in lap-spliced bars if they are cyclically loaded and subjected to compressive strains greater than 0.002, unless the length ℓ_s is such that a sufficient length of the lap will not develop these high compressive strains.

However, if adequate confinement is provided, the cracks shown in figure 87 can be contained, and the frictional resistance across the crack provided by the clamping pressure may be sufficient to develop the bar strength. Tests at the University of California, San Diego, on various confinement systems indicate that there is a limit to the dilation strain that can be permitted before bond slip occurs. This strain is approximately $\varepsilon_d = 0.001$. The tests also support a coefficient of friction of $\mu = 1.4$ across the fractured surface. To ensure that bond failure does not develop regardless of ductility level, the confining stress corresponding to a radial dilation of $\varepsilon_d = 0.001$ should be sufficient to develop the tensile strength of the bar. For a circular column with transverse hoops or spirals of cross-sectional area A_b at spacing s, the effective confining stress at $\varepsilon_d = 0.001$, assuming E_s = 200 GPa (29,000 ksi), will be:

$$f_\ell = \frac{400A_b}{D's}\ (\text{MPa}) \quad = \quad \frac{58A_b}{D's}\ (\text{ksi}) \tag{B-3}$$

where D' is the diameter of the hoop or spiral.

Allowing a coefficient of $\mu = 1.4$ on the crack surface, the tensile force capable of being developed in the bar is, by analogy with equation B-2:

$$T_b = 1.4\, f_\ell \left[\frac{\pi D'}{2n} + 2(d_b + c)\right] \ell_s \tag{B-4}$$

To prevent bond failure at high ductilities:

$$T_b \geq A_b\, f_u \approx 1.5\, A_b\, f_y \tag{B-5}$$

It is also clear that there is a lower limit to the lap length ℓ_s below which bond failure will develop regardless of the confining stress provided by transverse reinforcement. In this case, the bond fails not by developing a fracture surface as shown in figure 87, but by shearing off a cylinder of concrete of a diameter slightly larger than that of the deformation of the reinforcing steel. This type of failure has been observed in the beam steel passing through beam-column joints, where high transverse clamping action is provided by the column axial compression and the column flexural reinforcement. Results from such tests indicate that the effective bond stress corresponding to this failure may be taken as:

$$\sigma_u = 1.5\sqrt{f_c'}\ (\text{MPa}) \quad = \quad 18\sqrt{f_c'}\ (\text{psi}) \tag{B-6}$$

Thus, the minimum splice length for which it should be possible to develop the ultimate bar strength is:

$$\ell_{s_{min}} \approx \frac{1.5\ A_b\ f_y}{\pi d\ \sigma_u} \approx \frac{0.25d\ f_y}{\sqrt{f_c'}}\ \text{(MPa units)} \approx \frac{0.021d\ f_y}{\sqrt{f_c'}}\ \text{(psi units)} \qquad \text{(B-7)}$$

To develop just the bar yield strength, the coefficient should be 0.167 (MPa units) (0.014 for ksi units), rather than 0.25 (MPa units) (0.021 for ksi units).

Equation B-7 may also be used to determine the sufficiency of anchorage of bars in different characteristic locations noted in figure 86. At the knee joint in figure 86(b), the adequacy of the length ℓ_2 at the outside of the joint may be checked as for a column-base splice. If a column plastic hinge is expected at the top, equations B-2, B-4, and B-5 will apply. If the column will not form a plastic hinge at the top (as a consequence of a weaker cap beam), the yield strength of the bar, rather than the ultimate strength, may be used in equation B-5. At the inside of the joint, the development length may be based on equation B-7, provided the column forms the plastic hinge and adequate clamping is provided by the beam reinforcement. If the beam is weaker than the column under positive (opening) moment, a wide crack will develop along the outside of the plane of the inner bars. Anchorage should then be checked with the assumption of a fracture plane on the inside of the bars, thus completing the bond failure. Without additional confinement from the transverse reinforcement, the maximum force that could be developed in the column bars would be:

$$T_s = f_t\ \ell_2\ s \qquad \text{(B-8)}$$

where

$$f_t = 0.33\sqrt{f_c'}\ \text{(MPa)} = 4\sqrt{f_c'}\ \text{(psi)}$$

s = center-to-center spacing of the bars.

Similarly, the bottom beam steel development length, ℓ_3, should be checked using equation B-7 if the plastic hinge forms in the beam, and with equation B-8 (substituting ℓ_3 for ℓ_2) if the hinge forms in the column under opening moment.

The situation at the column top/cap beam intersection of figure 86(c) needs special consideration. As noted earlier, the effectiveness of the embedment length ℓ_4 is expected to depend on the location of the bar around the diameter, with bar 2 being potentially less critical than bar 1.

This is discussed further in the plan views of cap beams with intersecting circular or rectangular columns as shown in figure 88. Under longitudinal response, a number of potential failure surfaces are possible with a circular column, as illustrated by the lines 1-1 and 2-2 etc., where 1-1 corresponds to a fracture surface involving just the extreme tension bar with a 90° wedge of concrete split off as shown. By analogy with the approach developed in equation B-2 for development length, it would appear that the maximum tension force that could be developed in this bar would be:

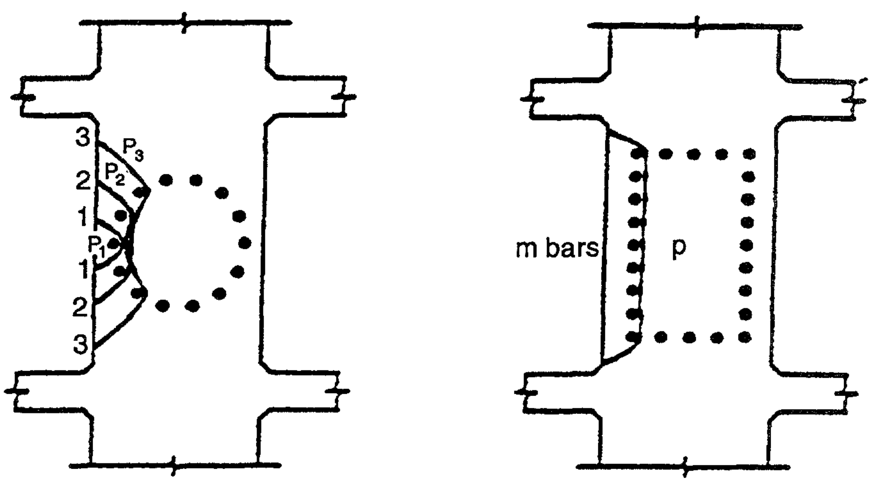
$$T_{b1} = f_t p_1 \ell_e \tag{B-9}$$

where

p_1	=	horizontal perimeter length of the fracture wedge
ℓ_e	=	embedment length

However, an alternative fracture surface running inside the three extreme tension bars and then out at 45° to the edge of the cap beam is possible with a horizontal perimeter of p_2. Since this surface must be able to develop three bars, the maximum tension force per bar will be:

$$T_{b2} = \frac{1}{3} f_t p_2 \ell_e \tag{B-10}$$



(a) Circular Column

(b) Rectangular Column

Figure 88. Possible fracture patterns associated with anchorage failure in cap beam/column connections (from ref. 18).

A third alternative involves five bars and a larger perimeter p_3, as shown in figure 88(a). The actual maximum force that can be developed in the bar will be the lowest associated with the different failure paths. Typically the second or third path is critical, depending on the spacing of vertical bars around the circumference.

For the rectangular case of figure 88(b), the probable failure surface involves all of the m bars of one side, resulting in:

$$T_b = \frac{1}{m} f_t p \ell_e \tag{B-11}$$

However, if the bar spacing is larger than the cover plus bar diameter, a value based on equation B-9 may govern.

The approach outlined above is very different from that proposed by existing code approaches. It has not yet been confirmed by a substantive body of test data and, until then, the approach should be viewed with caution. On the other hand, little confidence can be felt in the existing codified design equations because they show such scatter.

When the bridge is subjected to transverse response, the critical column bars will be at the center of the cap beam rather than close to the side. If the cap beam remains elastic under transverse response, the anchorage should be rather well confined, as a fracture surface would seem to need 45° surfaces in elevation. Bottom reinforcement in the cap beam will provide a clamping force, and vertical stirrups in the cap beam close to the column face will also assist. In this case, the strength may be calculated assuming a bond strength as given by equation B-5.

If, however, the cap beam flexural strength is less than the column strength, wide cracks in the cap beam adjacent to the column bars associated with ductile response may destroy the benefits of anchorage confinement, as shown in figure 89. Failure will then occur at much lower bar forces than implied by equation B-5.

Again, more test results are needed to further quantify and assess this approach.

B.3 FLEXURAL DUCTILITY

B.3.1 GENERAL

The flexural ductility capacity of existing members can be expressed either in terms of section curvature ductility factors or displacement ductility factors. Although the latter is more convenient in terms of structural assessment in that it frequently compares rather closely to the force reduction factor relating elastic response level to design or provided strength, there are problems and differences which must be appreciated. The structural ductility factor relates to the structure as a whole. Individual member ductility factors can differ widely from the global factor and

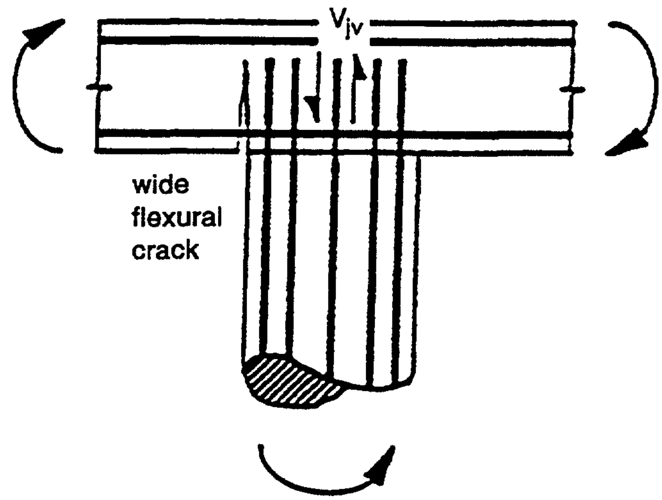


Figure 89. Joint shear mechanism in cap beam/column connections (from ref. 18).

may bear little resemblance to the force reduction factor resulting from an elastic C/D ratio.

This may be illustrated by reference to the two models of figure 90. In figure 90(a), the columns of a double-decker viaduct are sized such that the elastic displacements at first yield are proportional to height, and of such strength that a plastic hinge develops at the tops of the lower columns at 50 percent of the elastic response level, implying a force reduction factor of 2 related to this section. If other sections of the bent are stronger, a soft-story sway mechanism will develop with all plastic deformation occurring in the lower column hinges. Overall structure ductility should be assessed at the center of seismic force which, assuming equal mass at the two deck levels, is about 2.5h from the ground in the example. At this level, the equal displacement approach would imply that plastic displacement equals elastic displacement, since $\mu = R = 2$. In other words, $\Delta_y = \Delta_p$. For the lower columns, however, the displacement at first yield is $\Delta_y/2.5$, while all plastic displacement, $\Delta_p = \Delta_y$, occurs at this level. Hence the member displacement ductility factor for the lower column is:

$$\mu_\Delta = \frac{0.4\ \Delta_y + \Delta_p}{0.4\ \Delta_y} = 3.5$$

or nearly twice the force reduction or structural ductility factor.

In figure 90(b), lateral resistance is provided by a single column/pile bent. The critical section is at a distance z below ground. At first yield, the total yield displacement, Δ_y, at the center of mass consists of a structural deformation, Δ_s, relative to the

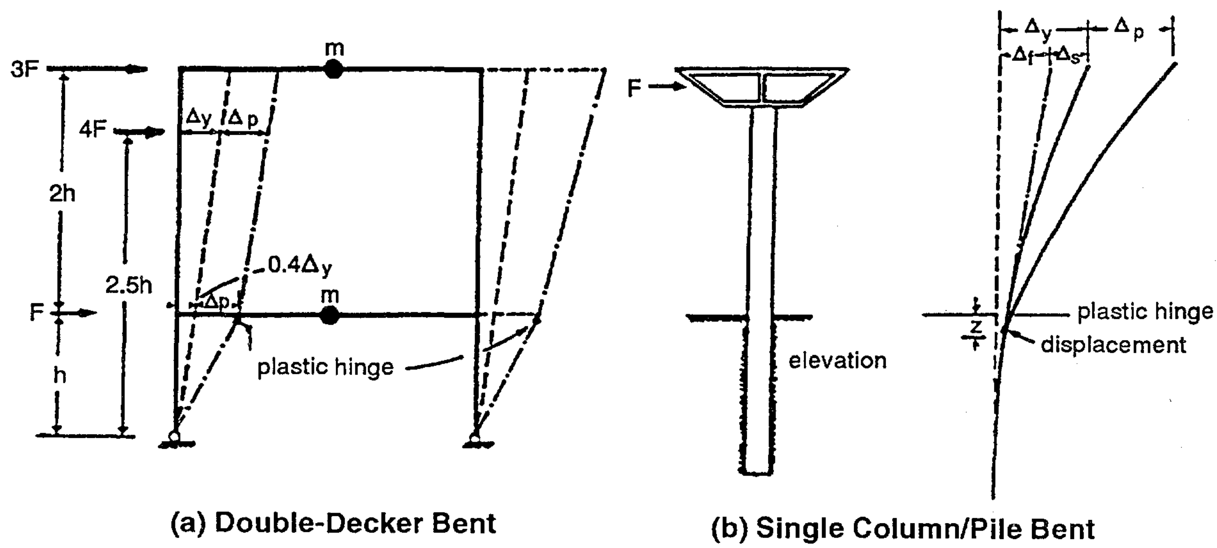


Figure 90. Relationship between member and structure ductility (from ref. 18).

tangent to the deflection curve at the critical section, plus a component, Δ_f, due to flexibility occurring below the critical section.

Again, the plastic rotation concentrates at the critical section, inducing a plastic displacement, Δ_p, at the height of the center of mass. The structural displacement ductility factor, which is related to the force reduction factor in accordance with equation 3-17, is:

$$\mu_c = \frac{\Delta_s + \Delta_p}{\Delta_s}$$

If we consider the member ductility as though it were a fixed-base vertical cantilever, the required column displacement ductility factor is:

$$\mu_c = \frac{\Delta_s + \Delta_p}{\Delta_s}$$

It is not uncommon for Δ_f to be as high as $3\Delta_s$, with the result that for $\mu_\Delta = 2$, $\mu_c = 5$.

Figure 90(a) also yields another example of how computed force reduction factors can give misleading indicators of ductility demand. Suppose that analyses showed that the lower columns would reach their flexural strengths at 20 percent of the elastic response and the upper column at 30 percent of the elastic response.

The initial indication is that the upper columns will have a ductility demand of about $\mu_\Delta = 3.3$. However, since a sway mechanism involving the lower columns develops at a much lower lateral force level, the upper columns are essentially isolated against inelastic response, since the lower columns could not support the lateral force levels necessary to develop plastic hinges in the upper columns. In fact, it is possible that some ductility demand may result from a second mode response, but this will be much less than predicted from the elastic force reduction factor. Thus, in this case, the elastic analysis and force reduction factor may lead to the erroneous decision that retrofit of the upper column is needed. The relationship between member and structure displacement ductility is thus a matter of geometry, coupled with the correct identification of the inelastic deformation mode.

It is now relevant to examine the relationship between curvature and displacement ductility for a simple cantilever element. It will be appreciated that the element could be a column or part of a cap beam. It will be assumed for convenience that the ductility is assessed in terms of transverse displacements measured at the position of a point of contraflexure.

The simple vertical cantilever of figure 91 is considered fixed at the base. Elastic curvatures at first yield are assumed linearly distributed with height. Although this is a significant approximation, the resulting yield displacement is quite accurate. The yield displacement is thus:

$$\Delta_y = \frac{\phi_y \ell^2}{3} \tag{B-12}$$

The plastic rotation, θ_p, occurs within a plastic hinge of length ℓ_p. The length ℓ_p is chosen such that the plastic curvature $\phi_p = \phi_{max} - \phi_y$, assumed constant over ℓ_p, produces the correct plastic displacement, Δ_p, and is calibrated by analytical and experimental results. Hence:

$$\phi_p = (\phi_{max} - \phi_y)\ \ell_p \tag{B-13}$$

and:

$$\Delta_p = \theta_p \left(\ell - \frac{\ell_p}{2} \right) \tag{B-14}$$

Equations B-12 through B-14 may be combined to form a relationship between the member displacement ductility factor, $\mu_\Delta = (\Delta_y + \Delta_p)/\Delta_y$, and the critical-section curvature ductility factor, $\mu_\phi = \phi_{max}/\phi_y$, as given by:

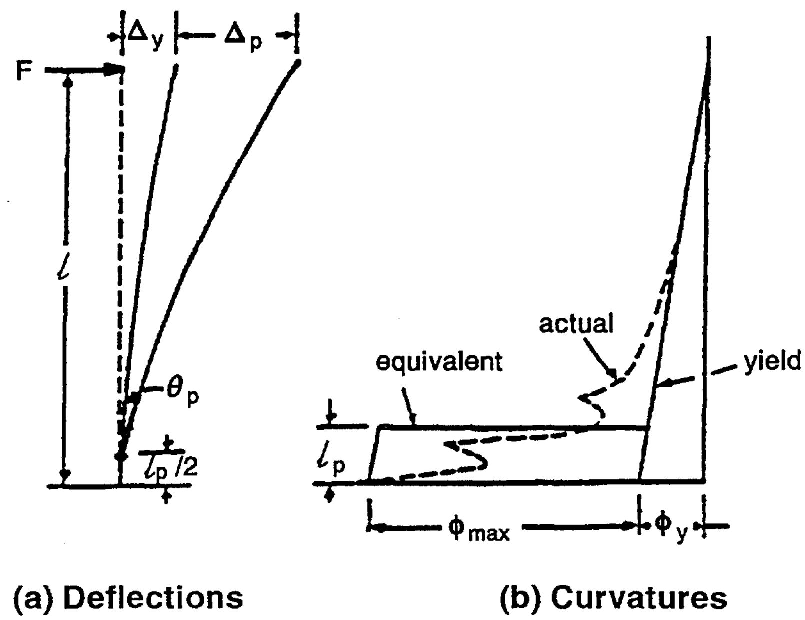


Figure 91. Deflections of a simple cantilever (from ref. 18).

$$\mu_\Delta = 1 + 3\,(\mu_\phi - 1)\,\frac{\ell_p}{\ell}\,(1 - 0.5\frac{\ell_p}{\ell}) \tag{B-15}$$

or, conversely:

$$\mu_\phi = 1 + \frac{(\mu_\Delta - 1)}{3\,\frac{\ell_p}{\ell}\left(1 - 0.5\,\frac{\ell_p}{\ell}\right)} \tag{B-16}$$

In the above approach, both μ_ϕ and μ_Δ are based on an elasto-plastic approximation of the force/deformation relationship, as shown in figure 92.

To obtain the equivalent elasto-plastic yield curvature, ϕ_y, extrapolation from conditions at first yield $\left(\phi_y^{\prime}, M_n\right)$ is needed. Hence:

$$\phi_y = \frac{M_n}{M_y} \cdot \phi_y^{\prime} \tag{B-17}$$

where M_n is the nominal moment capacity and M_y = first yield moment. The curvatures $\phi_y^{\prime}$ and ϕ_u, which are the ultimate curvature the critical section can sustain, are found from the strain profiles at the critical section, as illustrated in figure 93.

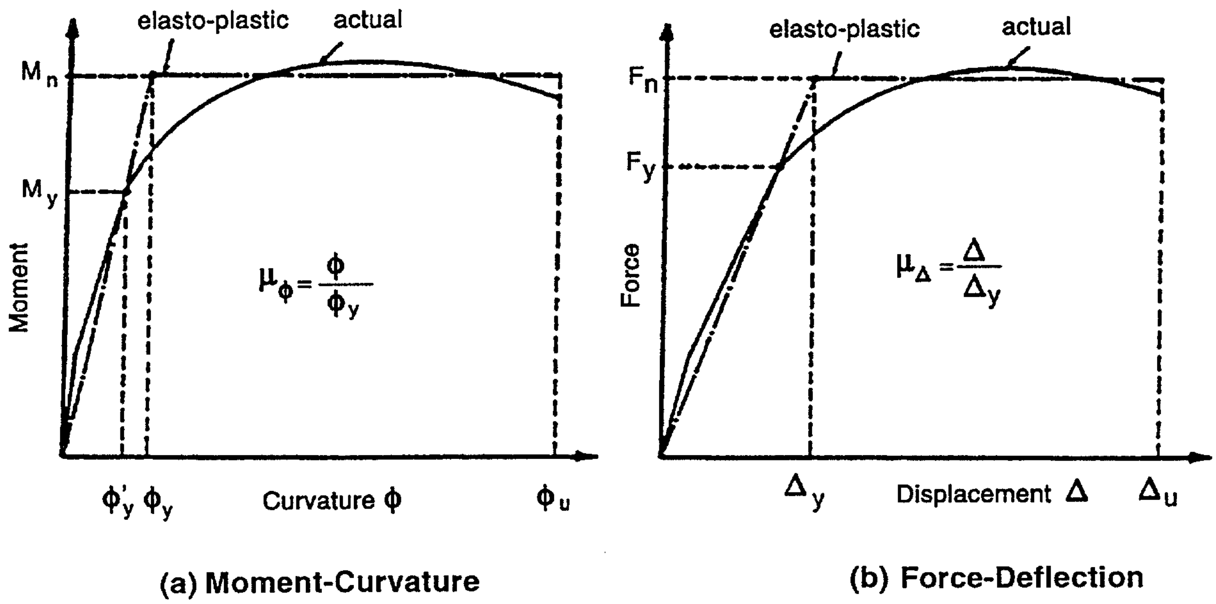


Figure 92. Elasto-plastic approximation of lateral response (from ref. 18).

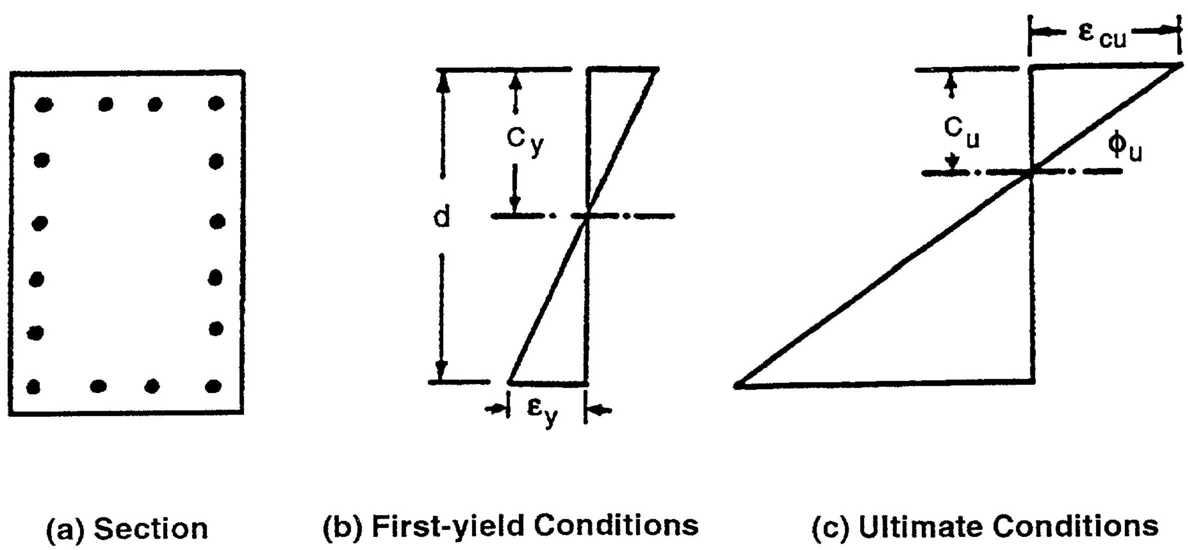


Figure 93. Yield and ultimate curvatures (from ref. 18).

From figure 93:

$$\phi_y' = \frac{\varepsilon_y}{(d-C_y)} = \frac{(f_y / E_s)}{(d - C_y)} \tag{B-18}$$

and:

$$\theta_\mu = \frac{\varepsilon_{CU}}{C_\mu} \tag{B-19}$$

Two further pieces of information are needed: ultimate compression strain, ε_{cu}, and plastic hinge length, ℓ_p.

Ultimate Compression Strain: For unconfined concrete, an ultimate compression strain of 0.005 may be used, as in calculating flexural strength. For confined concrete, the following equation, based on an energy balance, may be used:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\ \rho_s\ f_{yh}\ \varepsilon_{sm}}{f_{cc}'} \tag{B-20}$$

where

ρ_s	=	effective volumetric ratio of confining steel
f_{yh}	=	jacket yield stress
ε_{sm}	=	strain at peak stress of confining reinforcement taken as 0.15 for grade 40 reinforcement and 0.12 for grade 60 reinforcement.
f_{cc}'	=	confined strength of concrete. This can be assessed by an energy-balance method or could be approximated by $f_{cc}' = 1.5 f_c'$ for low-to-moderate confinement ratios.

Plastic Hinge Length: The plastic hinge length for ductility calculations may be taken as:

$$\ell_p = 0.08\ell + \chi\ d_b \tag{B-21}$$

where

ℓ	=	length from the point of contraflexure to the section with maximum moment
d_b	=	diameter of longitudinal reinforcement
χ	=	6 for grade 40 rebar
	=	9 for grade 60 rebar

The first term in equation B-21 is calibrated to experimental and theoretical results for typical curvature distributions for an element with linear moment variation, and the second term provides for the increase in plastic rotation resulting from strain penetration of the longitudinal reinforcement into the concrete beyond the critical section (i.e., the footing or cap beam). Equation B-21 will not apply for a plastic hinge forming in a column/pile system as shown in figure 90(b), where nearly constant moment exists in the hinge region. For such a case, it is recommended that $\ell_p = D$, where D is the section depth or diameter to be used, unless a detailed analysis involving integration of the column curvature distribution is carried out.

For squat columns, where shear inclination to flexural cracks is expected, the spread of plasticity predicted by equation B-21 may be conservative.

B.3.2 FLEXURAL DUCTILITY OF SECTIONS WITH LAP SPLICES

Experimental evidence indicates that the flexural strength of columns with lap splices in the potential plastic hinge region degrades rapidly to a value equal to that which can be sustained by the axial compression force on the column, with no contribution from reinforcement, using a reduced section size taken to the inside of the layer of longitudinal reinforcement. The reduced effective sections after bond failure are shown for rectangular and circular sections in figure 94. For the rectangular section, the residual moment capacity based on axial load alone will be seen to be:

$$M_r = P\left(\frac{h'}{2} - \frac{a}{2}\right) \tag{B-22}$$

where: $a = P / 0.85\, f_c'\, b'$

is the depth of the equivalent rectangular stress block. For a circular column, the corresponding strength is:

$$M_r = P(D'/2 - x) \tag{B-23}$$

where x is the centroid of the curved compression zone (see figure 94(b)).

Figure 95 shows typical hysteresis loops for rectangular and circular columns, which indicate that the residual strength is achieved at a displacement ductility of about $\mu_\Delta = 3$. From this data, and the discussion above, a model describing the flexural strength and ductility of column sections is proposed in the form shown in figure 96.

Four different conditions are depicted. All have the same initial elastic (cracked-section) stiffness. Line 1 is a bi-linear representation of response of a comparatively well confined section. The nominal moment capacity is reached at $\mu_\Delta = 1$, and an overstrength moment capacity, M_o, is attained at μ_1, found from equations B-15, B-19, and B-20. The overstrength moment, M_o, exceeds M_n as a result of strain-hardening of flexural reinforcement and confinement effects.

Line 2 represents a poorly confined column without lap splices in the plastic hinge region. The maximum strength does not exceed M_n, and the maximum displacement ductility factor, $\mu_2 \approx 3$, will be found. When the ductility limit for lines 1

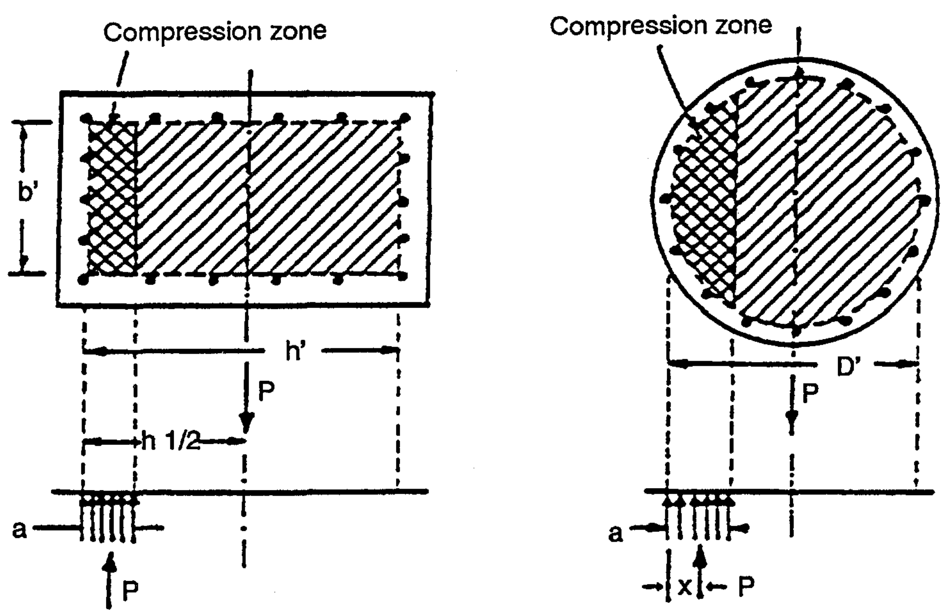


Figure 94. Residual moment capacity of columns after lap splice failure (from ref. 18).

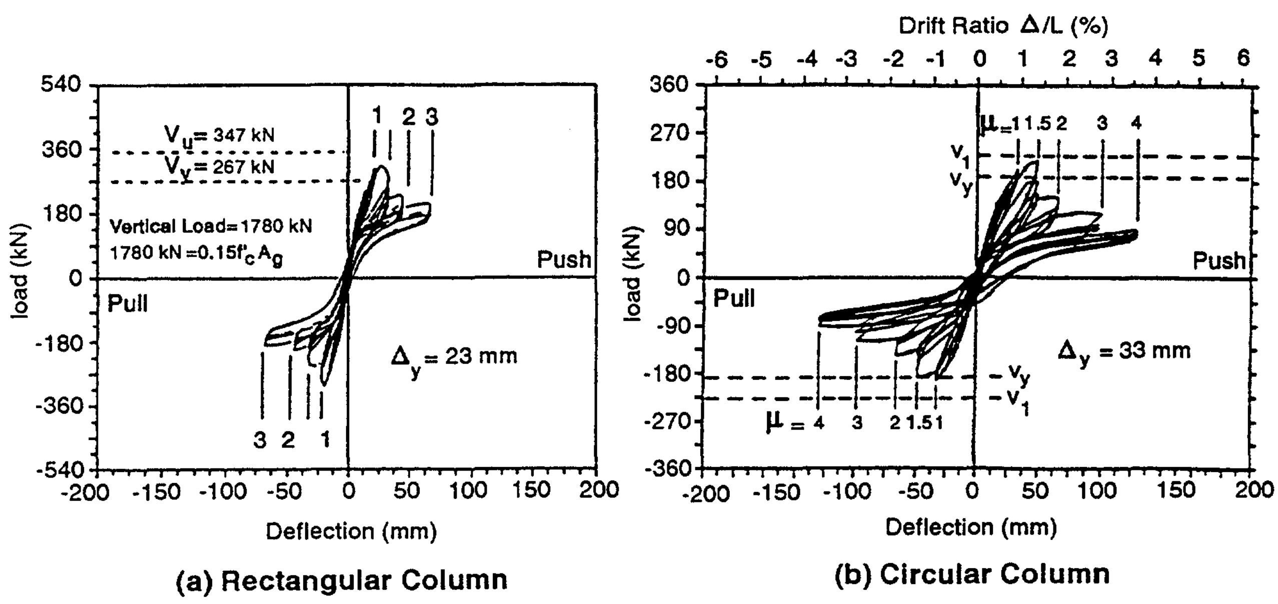


Figure 95. Strength degradation of columns with lap splices of length $20d_b$ (from ref. 18).

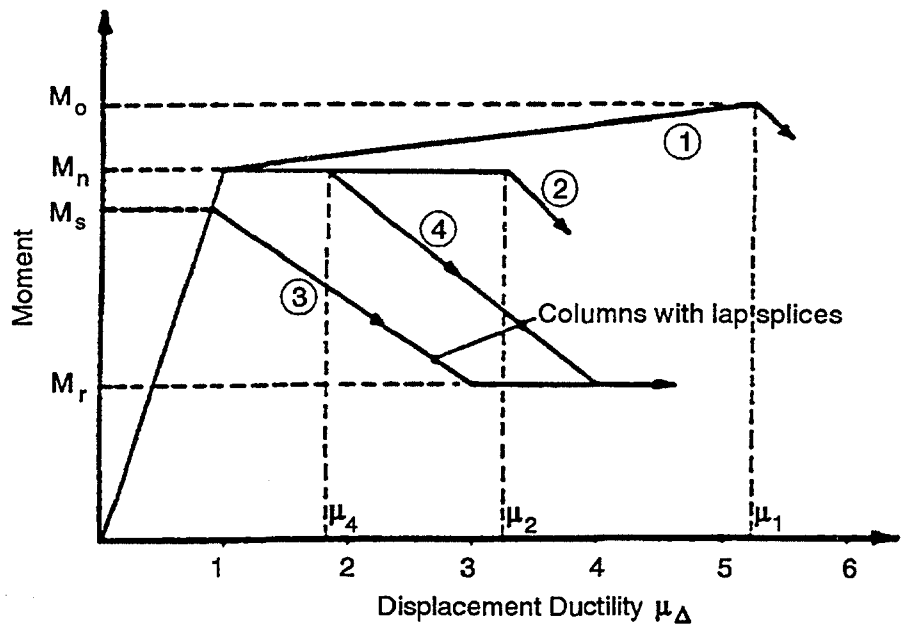


Figure 96. Flexural strength and ductility of sections (from ref. 18).

or 2 is reached, strength degrades rapidly due to crushing of core concrete and buckling of longitudinal reinforcement.

Line 3 represents degradation of a column with lap splices, where equation B-2 indicates that nominal moment capacity will not be achieved. Strength starts degrading at less than $\mu_\Delta = 1$ from a maximum strength M_s (based on T_b in equation B-2) to the residual strength M_r (from equation B-22 or B-23) at $\mu_\Delta = 3$.

Line 4 represents degradation of a column with lap splices where equation B-2 indicates that $T_b = A_b f_y$ can be developed. The nominal moment capacity, M_n, is reached and degradation occurs when a ductility μ_4 corresponding to an extreme fiber compression strain of $\varepsilon_c = 0.002$ is developed. The line degrades to M_4 with and parallel to curve 3.

Although experimental evidence indicates that higher ductilities than $\mu = 3$ could be sustained from line 3 behavior, it is felt that this is a reasonable upper limit for dependable performance. Line 4 behavior should have an upper limit of ductility when the residual strength develops.

B.4 MEMBER SHEAR STRENGTH

Shear strength equations in existing codes such as ACI 318-89 tend to give poor estimates of actual strength. They tend to be conservative for columns at low flexural ductility levels, and unconservative for beams and columns at high flexural ductilities. An exception is in plastic hinge regions with low axial load when concrete shear resistance is set equal to zero and all shear resistance is provided by transverse reinforcement and diagonal concrete compression struts.

Shear strength may be based on the following relationship for rectangular sections:

$$V_n = v_c A_e + A_v f_y \frac{d}{s} \cot\theta + 0.2P \tag{B-24}$$

and for circular sections:

$$V_n = v_c A_e + \frac{\pi}{2} \frac{A_s f_y D'}{s} \cot\theta + 0.2P \tag{B-25}$$

Each equation is of the form:

$$V_n = V_c + V_t + V_p$$

where V_c is the shear carried by concrete shear-resisting mechanisms (aggregate interlock, compression zone shear transfer, dowel action); V_t is the shear carried by truss mechanisms; and V_p is the shear carried by axial compression provided by gravity loads, seismic loads, and prestress, primarily as an increase in compression-zone shear transfer.

In equations B-24 and B-25, v_c is a nominal shear stress assumed constant over the effective shear area, which may be taken as $b_w d$ for rectangular beams, and 0.8 A_g for rectangular or circular column sections (A_g = gross section area). The truss mechanism components in equation B-24 are the traditional values, and θ is the angle between the column axis and the diagonal concrete compression strut, taken as $\theta = 45^\circ$ in the ACI approach. The truss mechanism terms for a circular section, in equation B-25, consider the relative effectiveness of different hoops intersected progressively further from the central axis by a potential inclined shear crack. As shown in figure 97, the forces $A_s f_y$ exposed by the cut must be resolved parallel to the applied shear force for equilibrium. Equation B-25 results from integrating the relative effectiveness down the full length of the inclined crack.

For beam sections with $\mu_\Delta \leq 2$, ASCE/ACI Committee 426 recommends the use of equation B-26 for estimating concrete shear mechanism:

$$v_c = (0.7 + 10\ \rho_\omega) \sqrt{f_c'} \leq 0.2 \sqrt{f_c'} \quad \text{(MPa)} \tag{B-26}$$

$$= (0.85 + 120\ \rho_\omega) \sqrt{f_c'} \leq 2.4 \sqrt{f_c'} \quad \text{(psi)}$$

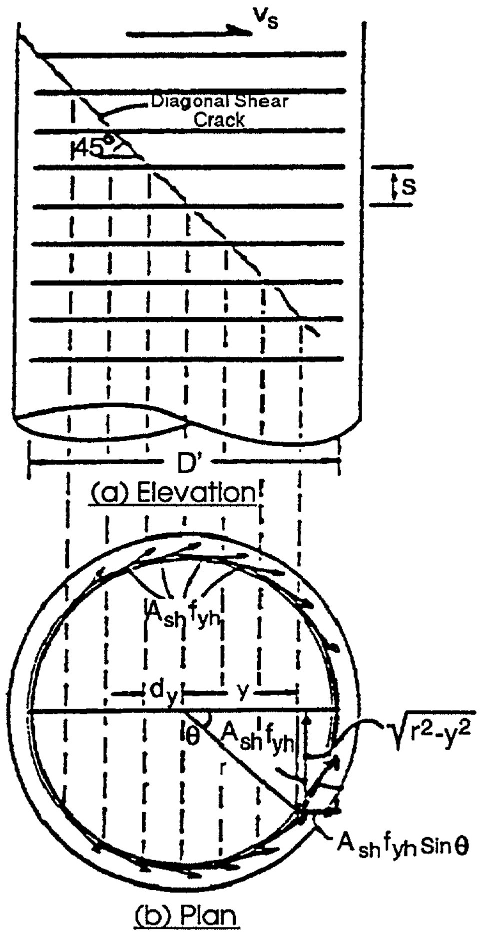


Figure 97. Truss mechanisms for circular columns (from ref. 18).

where ρ_ω is the longitudinal tension steel ratio. Equation B-26 is more conservative than ACE 318-89 for low ρ_ω values, but less conservative for high ρ_ω values. Since many cap beam sections have low steel ratios at sections critical for shear, the ACI equations should not be used.

Where transverse reinforcement in the form of stirrups is light, or where considerable axial compression (say from prestress) is present, the angle of the diagonal compression struts (and hence diagonal tension cracks) to the member axis will be less than 45°, and a value of 30° could be used in assessing strength. The increasing slope of the critical crack means that more stirrups are crossed by the potential crack, increasing the efficiency of shear resistance. However, it must also be recognized that this is achieved at the expense of a greater tension shift in the longitudinal reinforcement. Since tension reinforcement stress is related to the moment at the other end of the crack, a flatter crack will result in higher flexural tension stresses further away from the critical section. If the flexural reinforcement in the cap beam is prematurely terminated, as is frequently the case, shear failure may therefore result.

For displacement ductility factors of $\mu_\phi \geq 4$, $v_c = 0$ should be adopted in the plastic hinge region because of the loss of aggregate interlock when wide flexural cracks develop, reducing the efficiency of compression zone shear transfer under cyclic loading in the absence of axial compression force. Between $\mu_\phi = 2$ and $\mu_\phi = 4$, a linear interpolation may be assumed as in figure 98.

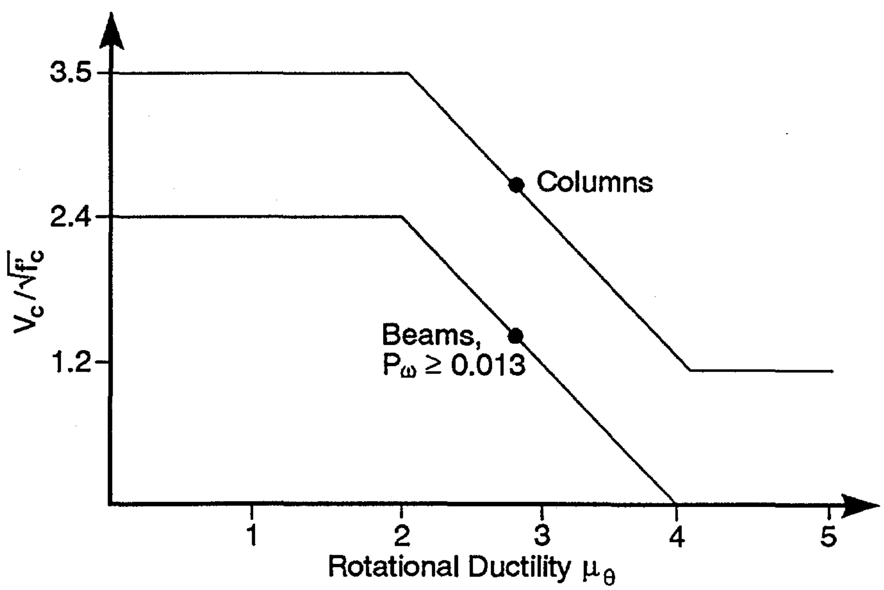


Figure 98. Concrete shear strength/ductility relationships for plastic hinges (from ref. 18).

For columns with well-distributed flexural reinforcement, equation B-26 is unduly conservative. Recent testing of circular and rectangular columns indicates that the following values may be used.

For non-ductile regions and for plastic hinges with $\mu_\Delta \leq 2$:

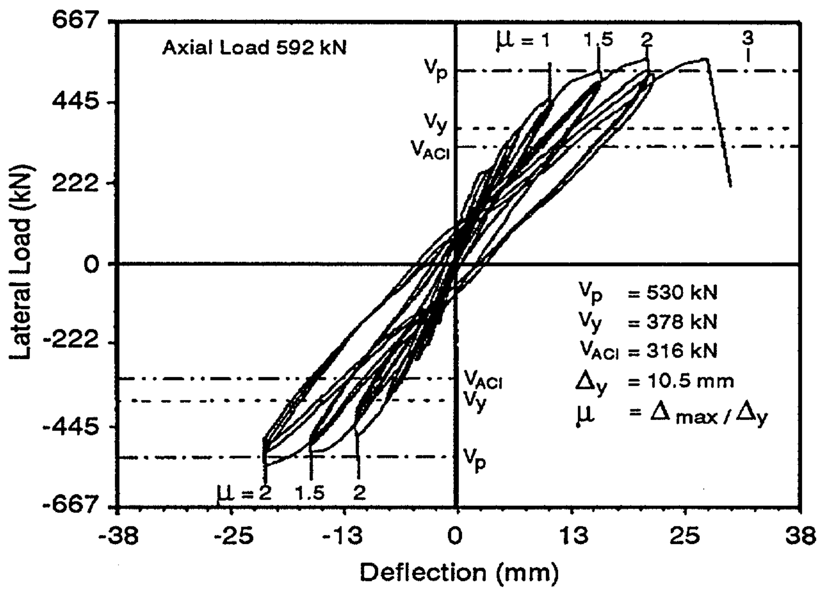
$$v_c = 0.29\sqrt{f_c'}\ \text{(MPa)} = 3.5\sqrt{f_c'}\quad \text{(psi)} \tag{B-27a}$$

For plastic hinges with $\mu_\phi \geq 4$:

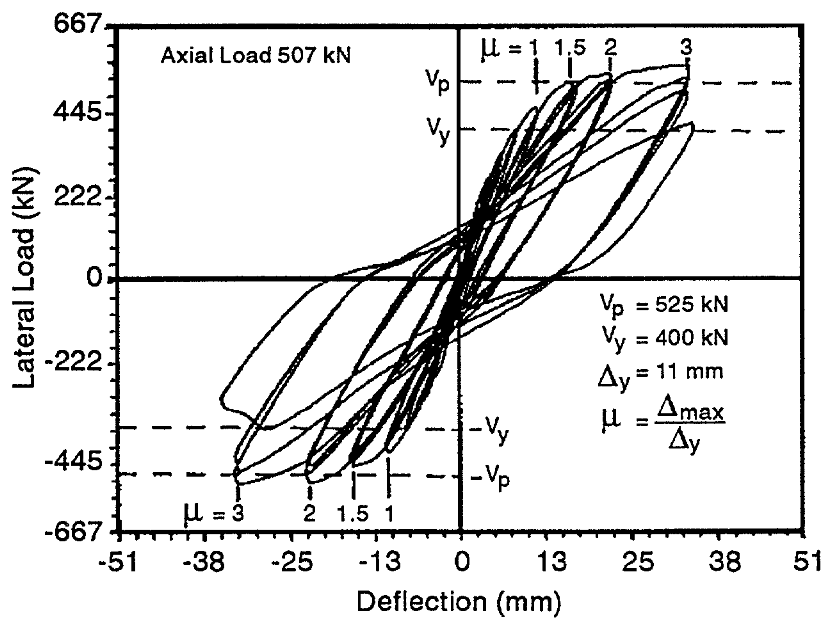
$$v_c = 0.1\sqrt{f_c'}\ \text{(MPa)} = 1.2\sqrt{f_c'}\quad \text{(psi)} \tag{B-27b}$$

A linear interpolation between $\mu_\phi = 2$ and $\mu_\phi = 4$, as shown in figure 99, is proposed.

The degradation of shear strength with increasing flexural ductility may reduce the flexural ductility capacity below that calculated in section B.3. When shear strength is reached, the structural response degrades rapidly and the structure should be assumed to have reached its survival limit state. This behavior is represented by



(a) "As-built" Circular Column



(b) "As- built" Rectangular Column (Strong Directon)

Figure 99. Shear failures of initially ductile columns (from ref. 18).

the hysteresis loops of circular and rectangular columns in figure 99 that initially developed ductile flexural response, then failed in shear at no higher lateral load as the ductility level was being increased. Degradation was rapid once shear failure developed, particularly for the circular column, which was unable to continue supporting its comparatively light axial load after the shear failure developed.

Shear capacity provided by steel truss mechanisms for columns may also be based on a 30° truss angle (equations B-24 or B-25), as with beams, provided the effects of greater tension shift associated with the flatter diagonal shear cracks are considered.

B.5 JOINT SHEAR STRENGTH

Although shear stress levels in beam/column joints of building frames are checked as a routine part of seismic design, shear stress levels in joints in bridges have generally been ignored. Joint damage to several bridges in the 1989 Loma Prieta earthquake indicated the real potential for damage and possible collapse from this source. Consequently, joints should be assessed to determine their capacity to sustain expected actions from columns and beams framing into the joint. This presents some problems, since available research data relate more to the design of joints specifically reinforced for shear than to the performance of existing inadequately designed joints.

B.5.1 BEAM/COLUMN JOINTS

A rational analysis is required to determine the joint shear stress v_{cj}. In conjunction with any effective axial stress on the joint, f_a, the principal tension stress should be determined from equation B-28:

$$f_t = \left(\frac{f_a}{2}\right) - \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{cj}^2} \tag{B-28}$$

where f_t is negative for tension and f_a is positive for axial compression.

B.5.1.1 Knee Joints

Figure 100 represents total forces due to seismic and gravity loads acting on a knee joint. Figure 100(a) shows the beam shear acting upwards (i.e., seismic dominated), resulting in column tensions; but gravity shears will frequently reverse the direction. Column and beam forces act at the joint boundaries. From equilibrium considerations for figure 100(a), the following equations can be deduced:

$$M_B + V_B \frac{h_c}{2} = M_c + V_c \frac{h_b}{2} \tag{B-29}$$

$$V_B = P \tag{B-30}$$

$$V_C = T \tag{B-31}$$

$$C_B = T_B - T \tag{B-32}$$

$$C_C = T_C - P \tag{B-33}$$

$$V_{jh} = C_B \tag{B-34}$$

$$V_{jv} = C_C \approx V_{jh}\, h_b/h_c \tag{B-35}$$

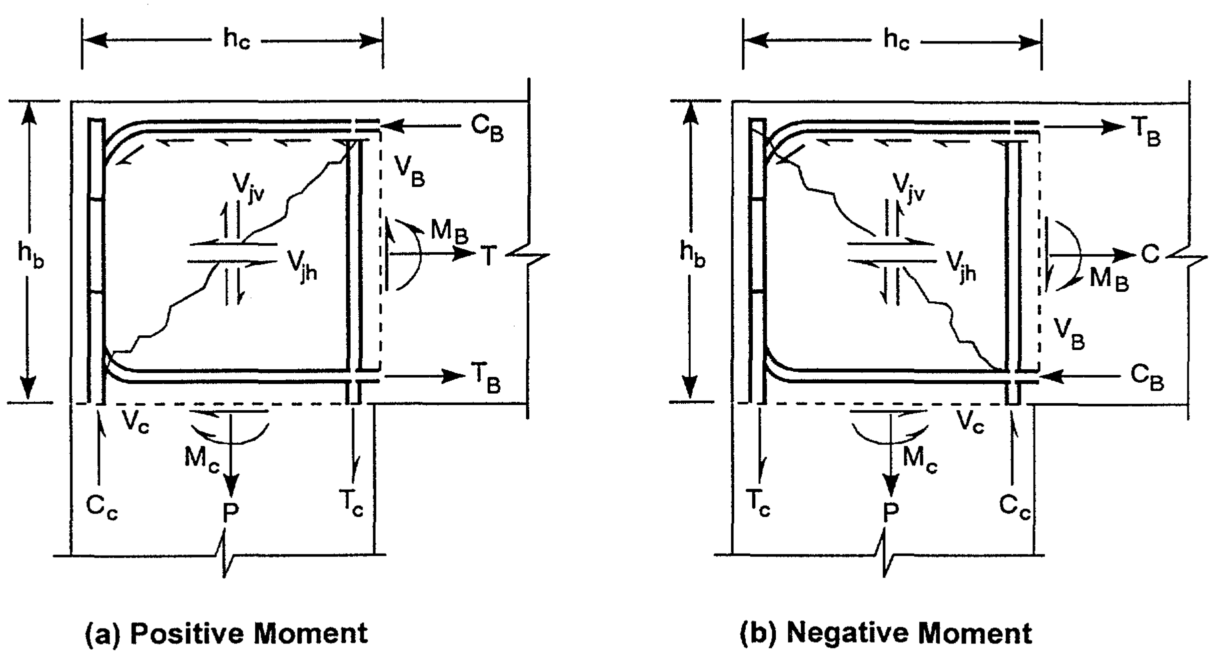


Figure 100. Joint shear forces in knee joints (from ref. 18).

The approximate form of equation B-35 is more convenient to use in analysis because of the influence of distributed column reinforcement, making computation of C_c and T_c tedious.

For the case represented by figure 100(b), equations B-29 and B-30 still hold, and:

$$V_C = C \tag{B-36}$$

$$V_{jh} = T_B = C_B - C \tag{B-37}$$

$$V_{jv} = T_C = C_C - P \approx V_{jh}\, h_b/h_c \tag{B-38}$$

For both cases represented in figure 100, the joint shear stress v_{jh} is found from:

$$v_{jh} = \frac{V_{jh}}{h_c\, b} \tag{B-39}$$

where b is the joint width.

In figure 100(a) for positive moment, the beam shear force will be introduced to the joint near the top of the beam section, developing axial stress in the joint. As a result:

$$f_a = \frac{V_B}{h_c\, b} \tag{B-40}$$

should be used in equation B-28 to determine the principal tension stress. In figure 100(b) for negative moment, the beam shear will be introduced to the joint near the bottom of the beam section, and therefore is unlikely to induce significant axial (vertical) stress in the body of the joint. For this case, $f_a = 0$ should be assumed in equation B-28.

It will be apparent that once a diagonal crack develops under negative moment as shown in figure 100(b), it is possible to conceive of joint stability resulting from a diagonal compression strut forming, anchored at the bottom right corner by the flexural compression forces, C_C and C_B, and at the upper left corner by the bend in the top reinforcement. However, to sustain this reaction, the tails of the beam top reinforcement must be anchored well down in the column and tied back into the column core by large amounts of transverse reinforcement. Under the opening moment of figure 100(a), stability for a diagonal compression strut from the upper-right to lower-left corner requires both horizontal and vertical joint shear reinforcement once diagonal cracking develops.

Analysis of joints that developed cracks and those that did not develop cracks indicates that diagonal cracks will begin to develop when the principal tension stress predicted by equation B-28 exceeds:

$$f_t = 0.29\sqrt{f_c'}\ \text{(MPa)} = 3.5\sqrt{f_c'}\ \text{(psi)} \tag{B-41}$$

Recent tests on a ⅓-scale model of a joint of I-980 Connector in Oakland, which exhibited joint shear failure, confirmed this stress level for initiating diagonal cracking. Failure, however, did not occur until, under negative moment, a joint shear stress of $0.62\sqrt{f'_c}$ MPa ($7.5\ \sqrt{f'_c}$ psi) was developed. Despite this, shear cracking at a stress level of $0.41\sqrt{f'_c}$ MPa ($5\ \sqrt{f'_c}$ psi) was widespread with one dominant crack developing, and it is probable that failure would have resulted if the joint had been cycled at this lower stress level. Figure 101 shows the hysteresis loops for this model, indicating the very rapid degradation of strength associated with the shear failure.

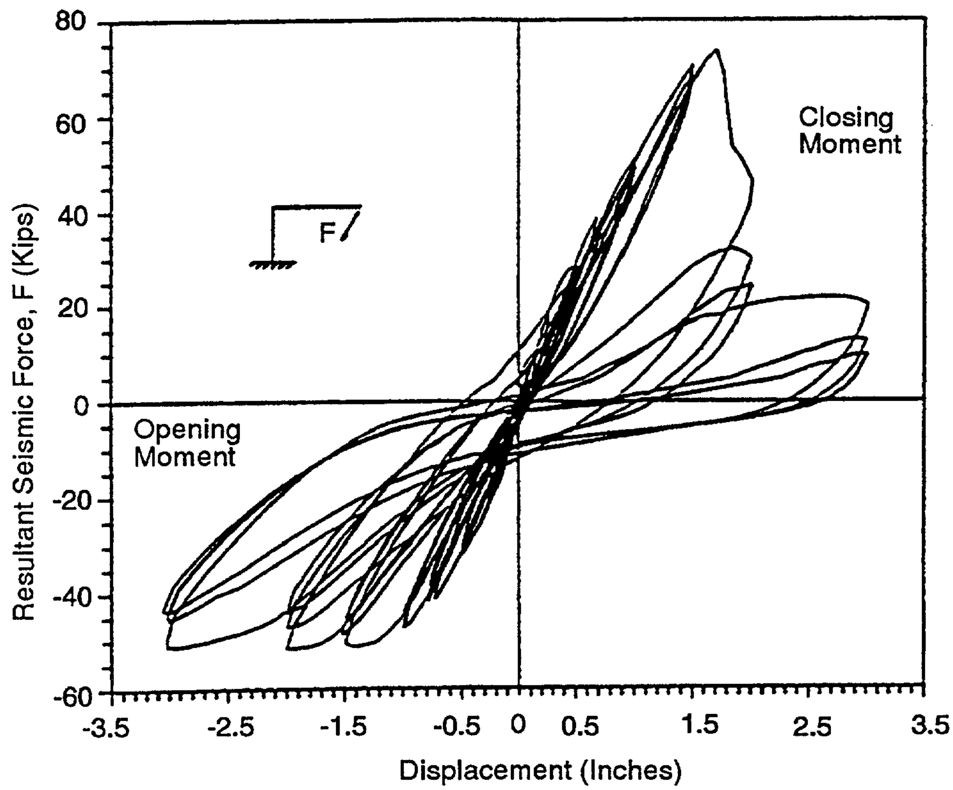


Figure 101. Load-deflection hysteresis loops of a knee joint failing in shear (from ref. 18).

In light of the limited test data currently available, the failure model represented in figure 102 is proposed. Two limiting conditions are identified. If a flexural plastic hinge develops adjacent to the joint at a joint shear stress less than $0.29\sqrt{f'_c}$ MPa ($3.5\sqrt{f'_c}$ psi), joint failure will not affect ductility. This is line 1 in figure 102. In a brittle system where joint shear failure occurs at force levels less than that required to develop a plastic hinge in an adjacent member, a maximum shear stress of $0.41\sqrt{f'_c}$ MPa ($5\sqrt{f'_c}$ psi) may be obtained (line 2), but strength immediately deteriorates and joint tension strength f_t degrades to zero at a member displacement of three times that at the onset of diagonal cracking, which is taken to occur at $f_t = 0.29\sqrt{f'_c}$ MPa ($3.5\sqrt{f'_c}$ psi). If a flexural plastic hinge forms in a member with a corresponding joint shear stress between $0.29\sqrt{f'_c}$ MPa ($3.5\sqrt{f'_c}$ psi) and $0.41\sqrt{f'_c}$ MPa ($5\sqrt{f'_c}$ psi), initially ductile response occurs (line 3), but the same falling branch as in case 2 occurs. This recognizes the degradation of joint performance resulting from strain penetration from yielding bars into the joint region.

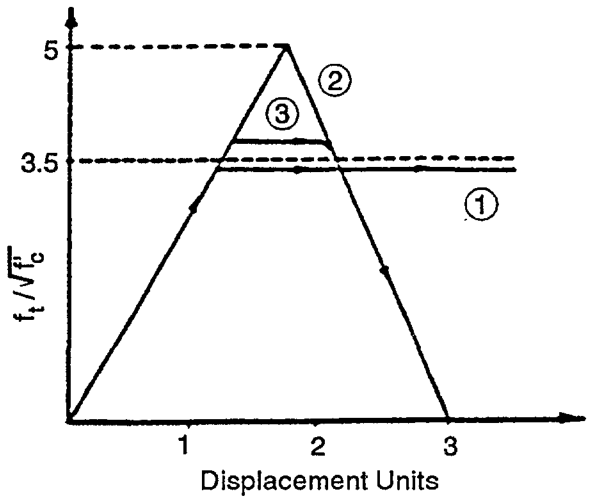


Figure 102. Joint tension strength failure model (from ref. 18).

The total joint strength is the sum of the joint shear force corresponding to the diagonal tension strength model of figure 102, plus the contribution of any joint shear reinforcement. Thus, the horizontal joint shear capacity is:

$$V_{jh} = v_{cj}\, b\, h_c + A_{jh}\, f_y \tag{B-42}$$

where, by rearranging equation B-28:

$$v_{cj} = \sqrt{f_t\,(f_t - f_a)} \tag{B-43}$$

and where A_{jh} is the total area of horizontal joint shear reinforcement between the top and bottom beam reinforcement in the joint region. This should be in the form of closed stirrups, but beam bars in the central region of the joint, properly anchored at the back of the joint, may be considered effective if the beam is not required to form a plastic hinge adjacent to the joint. This would be the case when the beam flexural capacity is significantly higher than the column flexural capacity.

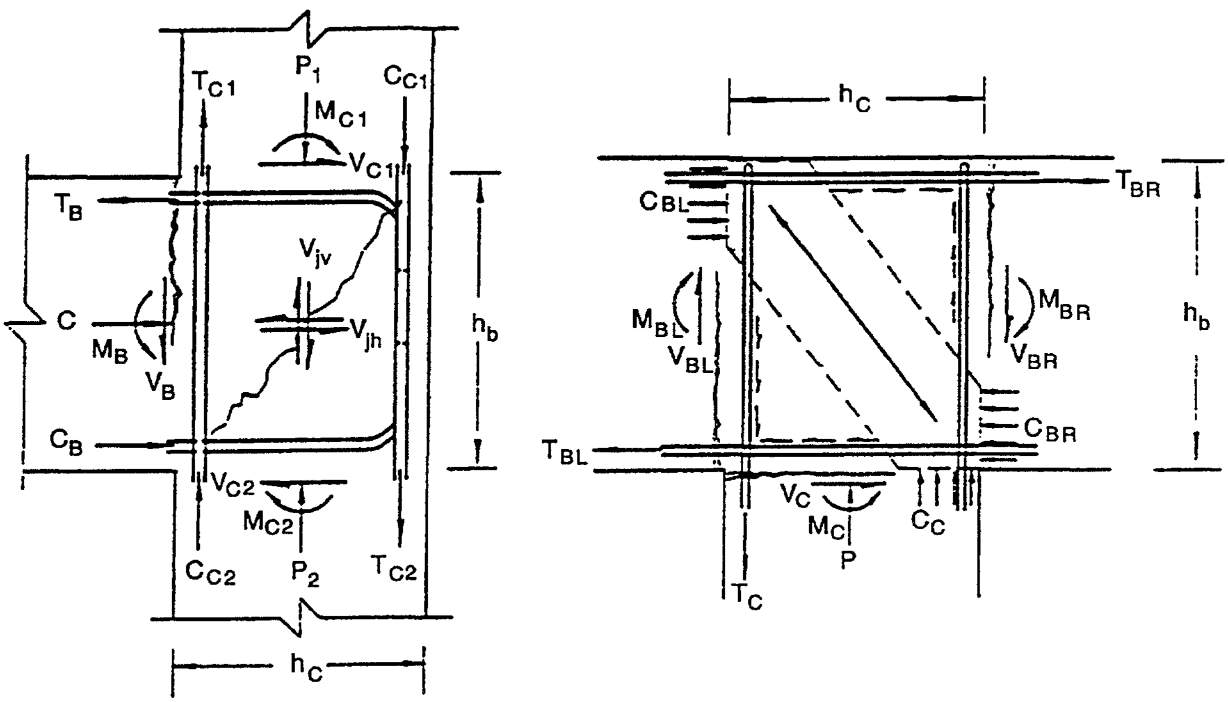
Note that similar expressions must be checked for vertical joint shear:

$$V_{jv} = v_{cj}\, b\, h_b + A_{jv}\, f_y \tag{B-44}$$

where A_{jv} is the area of effective vertical joint shear reinforcement. Normally, this will be zero unless beam flexural capacity exceeds column flexural capacity by, say, at least 30 percent, in which case, column side-face reinforcement can be considered as effective as joint shear reinforcement.

B.5.1.2 T-Joints

Expressions similar to those developed above may be developed for T-joints, such as the joint between a two-story column and a lower cap beam, or a column and a continuous beam, as shown in figures 103(a) and (b), respectively.



(a) Joint Between Column and Cap Beam

(b) Joint Between Column and Superstructure

Figure 103. Typical bridge T-joints (from ref. 18).

For example, the equilibrium relationships for figure 103(b) may be written as:

$$M_{BL} + M_{BR} + (V_{BL} + V_{BR}) \frac{h_C}{2} = M_c + V_c \frac{h_b}{2} \tag{B-45}$$

and:

$$P = P_T + V_{BR} - V_{BL} \tag{B-46}$$

where P_T is the column reaction introduced by the transverse cap beam framing into the joint (not shown), and V_{BL} and V_{BR} are the combined dead load plus seismic shears of girders framing into the joint. The horizontal joint shear force is:

$$V_{jh} = T_{BR} + C_{BL} \cong \frac{1}{jd} (M_{BR} + M_{BL}) \qquad \text{(B-47)}$$

where jd = lever arm between centers of tensile and compressive resultants in the beam. For distributed reinforcement, it is sufficient to approximate jd = 0.75 h_b. The vertical joint shear force is again given by equation B-35. Equations B-39 through B-44 may be used to determine joint shear capacity. Some allowance for reduced axial stress, f_a, in equation B-38 must be made due to the dispersion of stress at higher levels in the cap beam, and an effective value should be estimated at mid-height of the cap beam. For columns framing into transverse cap beams, the effective joint width, b, may be taken as the column width plus a tributary width of cap beam of d/2 on either side of the column, provided concrete exists on both sides for at least this width. This provision is shown in figure 104 and is based on the assumption of a St. Venant 45° spread of influence between the tension and compression resultants, with approximations for reduction in effectiveness of the cap beam concrete to participate in shear resistance as the distance from the column face increases.

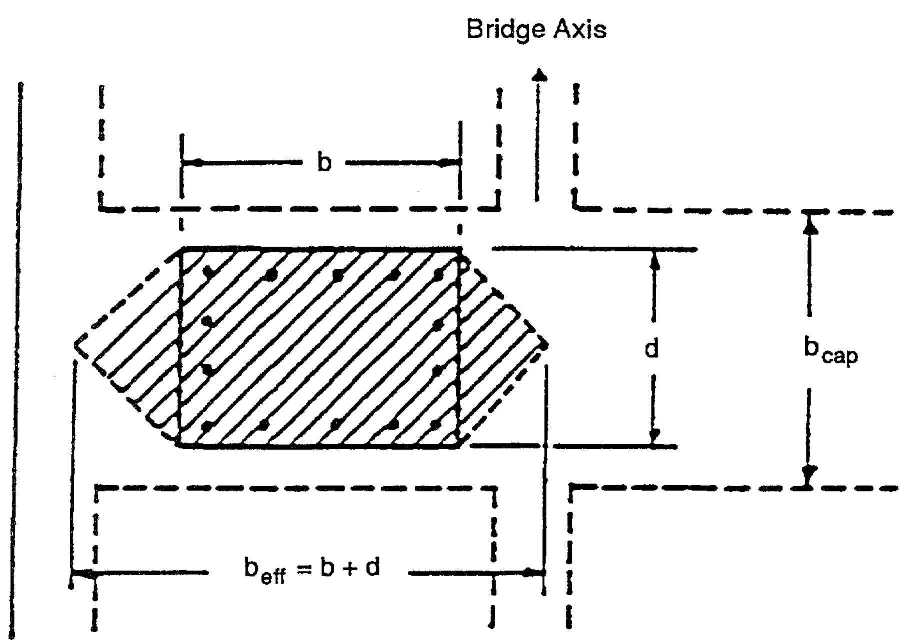


Figure 104. Effective joint width in cap beam (from ref. 18).

Thus, for longitudinal response, the effective joint width is:

$$b_{eff} = b + d \tag{B-48}$$

For transverse response, the effective joint width should be taken as the smaller of the cap beam width b_{cap}, and $b_{eff} = b + d$. For circular columns, the effective width may be taken as 2D, unless limited by cap beam width.

The rather specialized problem of joints in double-decker freeways can be handled in similar fashion to the above.

B.5.2 COLUMN/FOOTING JOINTS

The column/footing joint can be considered to be analogous to an inverted column/cap beam joint. It will be subjected to high joint shear stresses unless specifically reinforced for shear. Since existing designs are likely to be based on a ductile column, it would appear that vertical joint shear reinforcement would be needed if shear stress levels are higher than given by equation B-43. Testing at the University of California, San Diego, of a typical 1960's design of footing without joint shear reinforcement resulted in a joint shear failure under the column at a shear stress level of 3.5 MPa (509 psi). This is only 10 percent more than the predicted value of 3.2 MPa (465 psi) based on the axial stress level of 1.8 MPa (261 psi) at mid-height of the footing and a maximum tensile strength of $0.41\sqrt{f'_c}$ MPa ($5\sqrt{f'_c}$ psi), in accordance with figure 101 and equation B-43.

B.6 FOOTING ASSESSMENT

Footings should be assessed for flexural and shear strength as well as joint shear capacity. In carrying out these assessments, a critical parameter will be the effective section width. Also, the footing should be assessed for overturning capacity.

B.6.1 FLEXURAL AND SHEAR STRENGTH

Because of moment reversal in the footing on opposite sides of the column, as shown in figure 105(a), the effective section width will be less than for gravity loads where the footing has moments of the same sign on opposite sides of the column and shear lag effects are less severe. In assessing flexural and shear strength, it is thus recommended that the effective section width, and hence the effective contributory reinforcement, be taken not larger than the column width plus twice the effective depth of the footing.

In fact, at very large footing deformations, the effective width for flexure will increase. However, the wide flexural cracks in the footing adjacent to the column at this stage will greatly reduce the effective shear strength of the footing. More research is needed in the general area of seismic performance of footings.

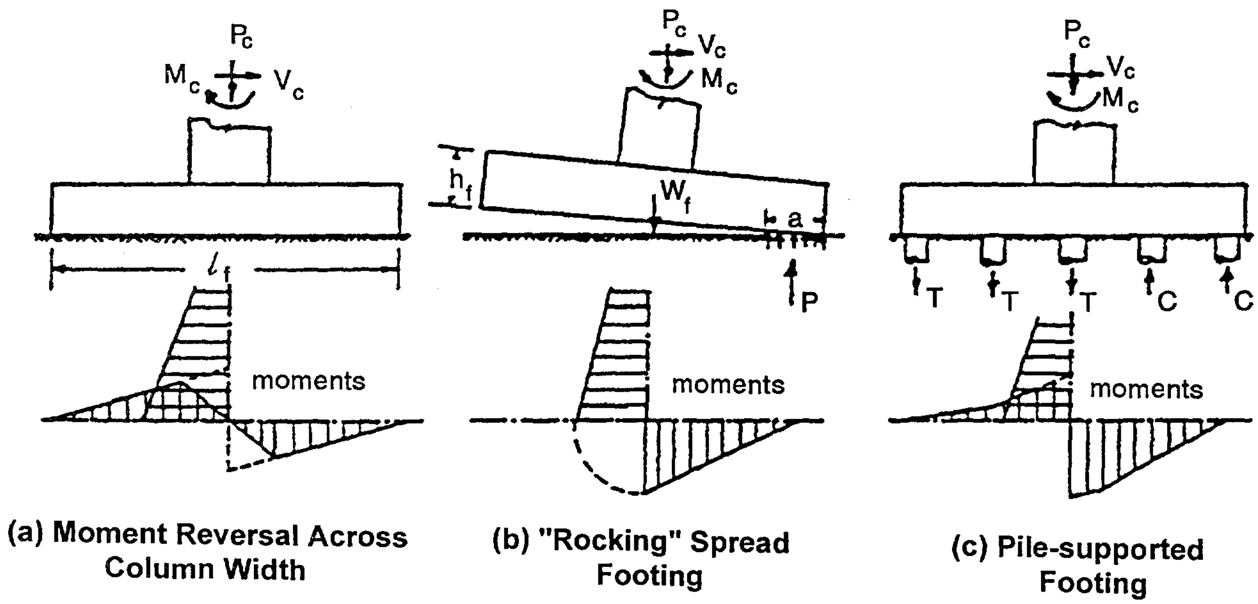


Figure 105. Seismic actions on footings (from ref. 18).

B.6.2 OVERTURNING CAPACITY

It is generally felt that footing overturning capacity should be adequate to develop the flexural overstrength of the column, if this is required to develop inelastic response. For the spread footing of figure 105(b), the overturning capacity will be:

$$M_f = \frac{(P_c + W_f)}{2} (\ell_f - a) \tag{B-49}$$

and for stability:

$$M_f > M_c + V_c h_f \tag{B-50}$$

where

a = depth of the compression zone between soil and footing
$\quad$ = $(P_c + W_f)/b_f f_b$ (B-51)
b_f = footing width
f_b = ultimate bearing capacity of the soil
P_c, M_c, and V_c = column actions at the top of the footing

For the pile-supported footing of figure 105(c), overturning capacity is likely to be limited by the capacity of the tension piles. This should be taken as the lower of the pull-out capacity of the pile in the soil and the tensile strength of the pile/footing connection. In many older bridges, there is no reinforcement connecting the piles to footings and, hence, T = 0 should be assumed for these cases.

Pile footing overturning capacity is often assessed on the basis of "elastic" distribution of pile forces. That is, the pile loads increase linearly with distance from a calculated neutral axis. This is clearly conservative for assessment purposes, and an "ultimate" condition where all tension piles have the same tension force and all compression piles have the same compressive force may be assumed. This implies a certain amount of plastic action of the pile system. The critical case for overturning assessment will often be a diagonal attack, where the principal loading direction places one corner pile in maximum compression and the diagonally opposite pile in extreme tension.

B.6.3 ROCKING RESPONSE

A situation where the overturning capacity of the footing is less than the column flexural strength (i.e., for a spread footing, where equation B-50 is not satisfied) does not necessarily indicate undesirable response, even when the assessment analysis indicates a need for significant column ductility. Uplifting of footings acts as a means of base isolation, limiting the seismic input to the structure. Instability will only result if excessive displacements develop during the rocking response. Structural rocking can be considered a satisfactory mode of response provided the level of displacement occurring during a rocking response does not cause instability nor result in displacement that cannot be tolerated by the complete bridge system.

To estimate the peak rocking response of a bent, the following procedure may be used: (1) develop a relationship between amplitude of displacement and rocking period; (2) develop a displacement response spectrum from the design acceleration response spectrum; and (3) adopt a trial-and-error approach where the final displacement is initially guessed, and steps (1) through (3) are followed until convergence is reached. This procedure is further described below.

<u>Step 1. Displacement: Period Relationship</u> – Housner has developed an analytical procedure to estimate the relationship between period and displacement which is appropriate for a rigid block impacting plastically on a semi-infinite foundation. A simpler, more versatile "structural" approach is as follows:

The load-deflection relationship for the rocking system is developed as shown in figure 106(b). With respect to this figure, the system is essentially elastic from the origin to point 1, with displacements resulting from structural deformation and soil compliance. At point 1, initial uplift occurs at the tension end of the footing and soil compliance effects become greater because of the reduced contact area between footing and soil, or footing and piles, resulting in a non-linear curve between points 1 and 2. At point 2, the soil or pile support resistance has reached its plastic capacity, as calculated for example by equation B-49 for the case of a spread footing. From points 2 to 3, true rocking response occurs, with lateral resistance decreasing as displacements increase. Taking moments about the center of rotation in figure 106(a):

$$F\left(h + \frac{\Delta}{h}\left(\frac{\ell}{2} - \frac{a}{2}\right)\right) = W\left(\frac{\ell}{2} - \frac{a}{2} - \Delta\right)$$

i.e.,

$$F = \frac{Wh\left(\frac{\ell}{2} - \frac{a}{2} - \Delta\right)}{h^2 + \Delta\left(\frac{\ell}{2} - \frac{a}{2}\right)} \qquad \text{(B-52)}$$

where W is the total weight supported by the soil, including footing and pier weight; and h is the height of the centroid of W above the rocking interface.

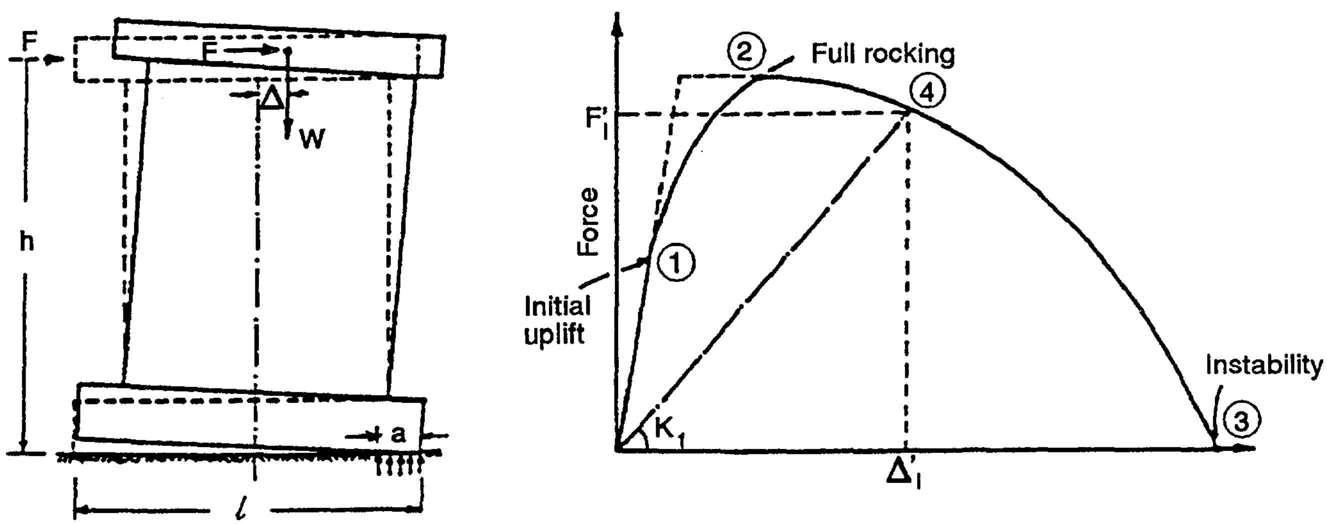


Figure 106. Rocking response of a bridge bent (from ref. 18).

A reasonable approximation to the full force-deflection curve may be found from extrapolating the elastic curve up to the force level indicated by equation B-52, as shown by the dashed line in figure 106(b).

The approximate rocking period is found from the secant slope to the maximum displacement. From figure 106(b), the initial displacement is guessed as Δ_1' (at point 4). The corresponding period is:

$$T_1 = 2\pi\sqrt{\frac{W}{g\,K_1}} \qquad \text{(B-53)}$$

where:

$$K_1 = \frac{F_1'}{\Delta_1'} \qquad \text{(B-54)}$$

and F_1' = force corresponding to the displacement Δ_1' from figure 106(b).

When several bents of different heights are linked together by an essentially rigid diaphragm such as a continuous deck, the lateral displacements of the adjacent bents will be essentially identical. It is suggested that the most realistic response will be obtained by summing the lateral load-deflection relationships for the individual bents of a given frame (that portion of the superstructure between joints) and considering a single-degree-of-freedom model with mass equal to the total mass of the frame. An example is shown in figure 107, where, for a given displacement Δ_1', the equivalent stiffness is:

$$K_1 = \frac{F_1' + F_2' + F_3'}{\Delta_1'} \qquad \text{(B-55a)}$$

and the corresponding period is:

$$T_1 = 2\pi \frac{\sqrt{(W_1 + W_2 + W_3)}}{g\ K_1} \qquad \text{(B-55b)}$$

Step 2. Displacement Response Spectrum – The displacement response spectrum, $S_{D(T)}$, may be found from the design acceleration response spectrum ordinates, $S_{A(T)}$, by the approximate relationship:

$$S_{D(T)} = \frac{T^2}{4\pi^2} S_{A(T)} \qquad \text{(B-56)}$$

Rocking dissipates a considerable amount of energy by plastic impact on the soil and radiation damping. As a consequence, it would be reasonable to use an increased damping of at least 10 percent rather than the 5 percent level commonly adopted for design spectra. This would reduce the acceleration (and hence displacement) spectral ordinates by approximately 30 percent. Consequently, the following modified form of equation B-56 is recommended for design use:

$$\Delta = S_{D(T)} = 0.7\ \frac{T^2}{4\pi^2} S_{A(T)} \qquad \text{(B-56a)}$$

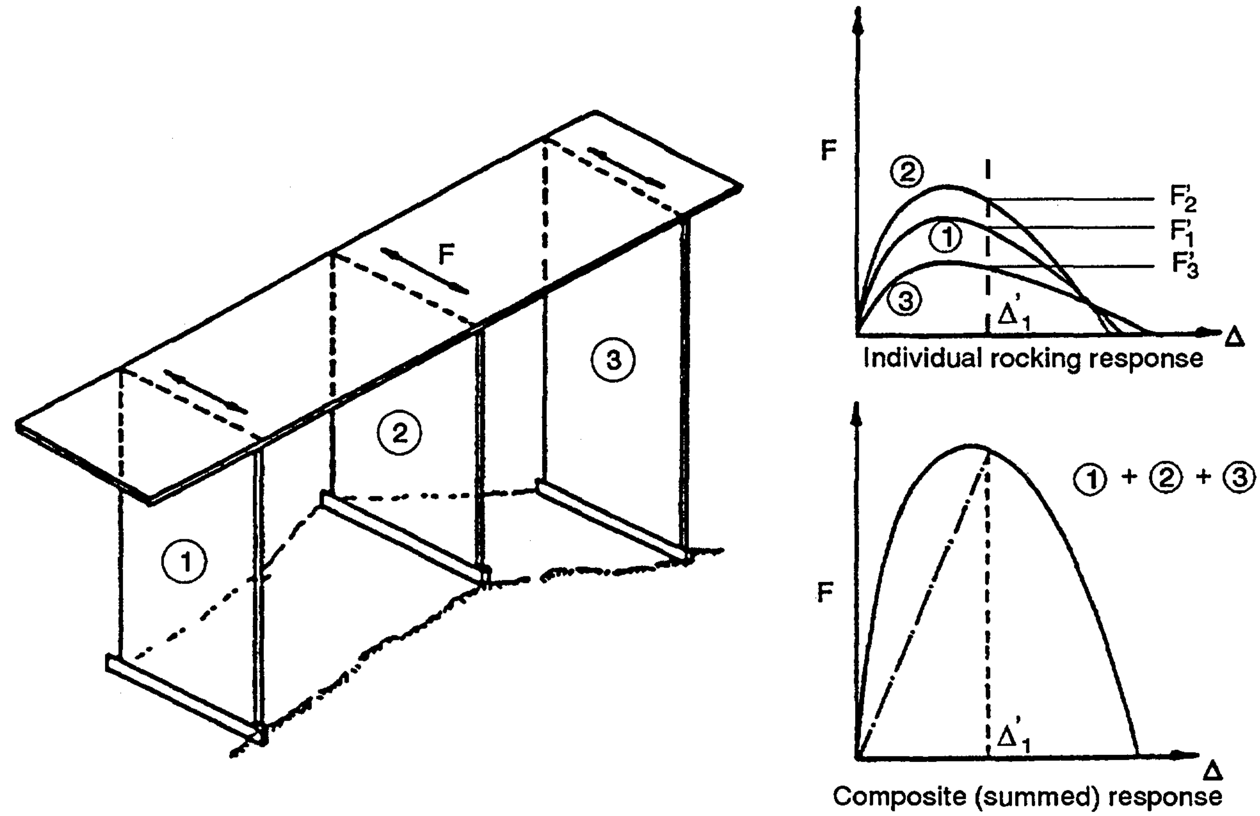


Figure 107. Rocking of a multiple bent system (from ref. 18).

Step 3. Trial-and-Error Procedure – With the above information, the procedure follows the cyclic procedure of guessing Δ_1'; calculating K_1 and, hence, T_1, from equations B-53 and B-54; calculating the corresponding displacement from equation B-56a; and using figures 106(b) or 107(c) to calculate a new stiffness based on the revised displacements and then cycling to convergence.

Note that if the superstructure is torsionally very stiff it may modify the rocking response. The uplifting edge of a squat pier will rise more than that of a slender pier for the same lateral displacement, causing warping of the superstructure. A torsionally stiff superstructure will resist this by transferring gravity load from the slender piers to the squat piers, thus reducing displacement response. Although the approach outlined above should be conservative for computed displacements, the consequence in terms of superstructure forces and loads on the squat pier should be investigated.

In the above analysis, it is assumed that the footing lifts off any tension piles. In the event that the tension capacity of the pile/footing connection exceeds the frictional capacity of the pile/soil interface, a heavily damped response will result, with piles being lifted and thrust down again in opposite halves of a response cycle. This could be incorporated in the above analysis by estimating an equivalent increase in the lateral rocking force, together with further increased equivalent damping. Greatly reduced response could be expected.

In some cases, where the superstructure width is small compared to span length, there may be a small influence of superstructure flexibility on rocking response. This, and the superstructure flexural strength under transverse actions, should be checked. In the limit of superstructure flexibility, the bents will act as independent rocking elements, and this can be used as an upper bound on response. If the calculated superstructure flexural and in-plane shear deformations are less than 20 percent of the predicted rocking displacements, they are not likely to significantly influence response, and could be ignored.

The level of rocking response predicted is very dependent on the shape of the long-period portion of the response spectrum.

B.7 SUPERSTRUCTURE CAPACITY UNDER TRANSVERSE RESPONSE

Seismic inertia forces in bridges result primarily from the superstructure mass. It is thus important to check the strength of the superstructure in relation to these forces, and the load path from the deck inertia forces to the substructure. Normally, the capacity of the deck to function as a diaphragm to distribute forces from the mid-region of the span back to adjacent piers will not be in question. However, the load path from the deck down into the piers requires special consideration, especially for steel bridges, where the bracing system will frequently be found to have inadequate strength.

To ensure satisfactory response, the lateral bracing system of steel bridges should be checked against force levels corresponding to a full lateral plastic collapse mechanism developing in the substructure or the elastic response level for the bridge assuming no ductility, whichever is lower.

B.8 BEARINGS AND SUPPORTS

Bearings and shear keys at supports (piers or abutments) also provide a critical element in the load path between the deck inertia force and the ground when the connections are not monolithic. These should be checked for the same level of force as for the superstructure bracing system.

The consequences of bearing failure will generally not be as disastrous as a pier failure unless girder unseating results. However, failures will generally be brittle, and will cause a change to the relative flexibilities of different load paths (such as through the abutment or through the piers) that may have secondary significance and should be checked.

APPENDIX C

WORKED EXAMPLE PROBLEM 1

C.1 INTRODUCTION

This example problem illustrates how the provisions of this Manual 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. 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.

C.2 DESCRIPTION OF THE EXAMPLE BRIDGE

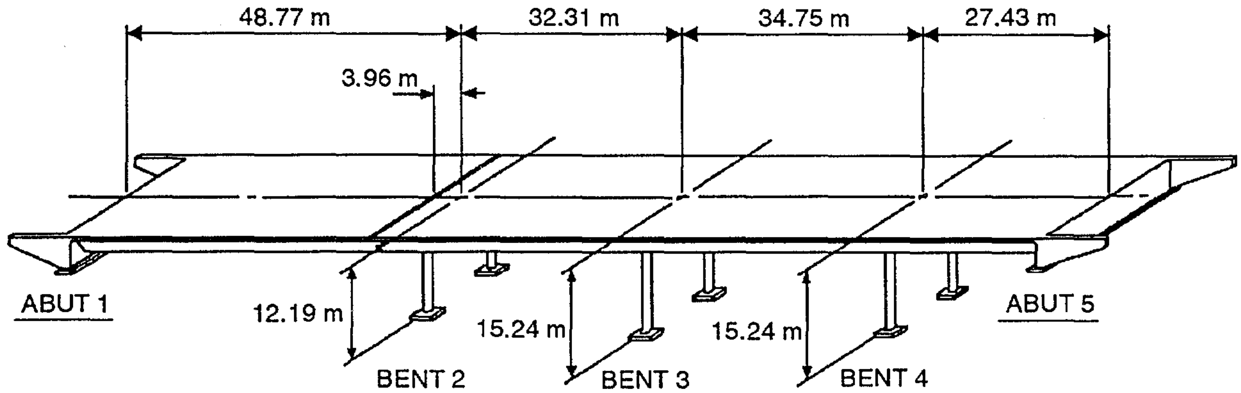
The bridge to be examined is a typical freeway overcrossing of the type that was being constructed in California prior to improved seismic design provisions. It carries a major city street over an urban freeway in a region that falls within the 0.4 contour on the Acceleration Coefficient map shown in figure 1 of the manual. 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. An in-span expansion joint is located between the prestressed and reinforced sections. The 143 m (470 ft) long superstructure is divided into four spans that are continuous over three 2-column bents, as shown in figure 108. The diaphragm-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 109. 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 110.

C.3 SEISMIC RATING

A seismic rating system is used to identify those bridges that are in greatest need of retrofitting. This process is sometimes called preliminary screening because it quickly identifies structures that are at risk and which deserve closer examination. In practice, all bridges in a region should be rated and their ratings compared in order to develop a prioritized list of bridges requiring detailed evaluation.



SUPERSTRUCTURE PROPERTIES

$A_X = 8.44\ m^2$
$I_{11} = 1.45\ m^4$
$I_{22} = 177.80\ m^4$
$I_{33} = 4.63\ m^4$

LOADING

A = 0.4
SOIL TYPE = II
ADDITIONAL DEAD LOAD = 35.90 kN/m
$f'_c = 2.24 \times 10^4$ kPa
$f_y = 400$ MPa

Figure 108. Example bridge.

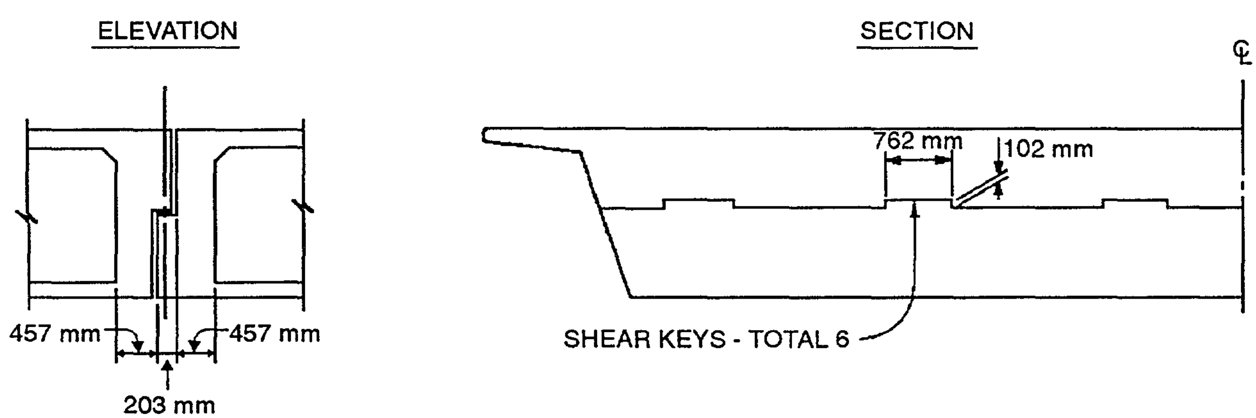


Figure 109. Expansion joint detail.

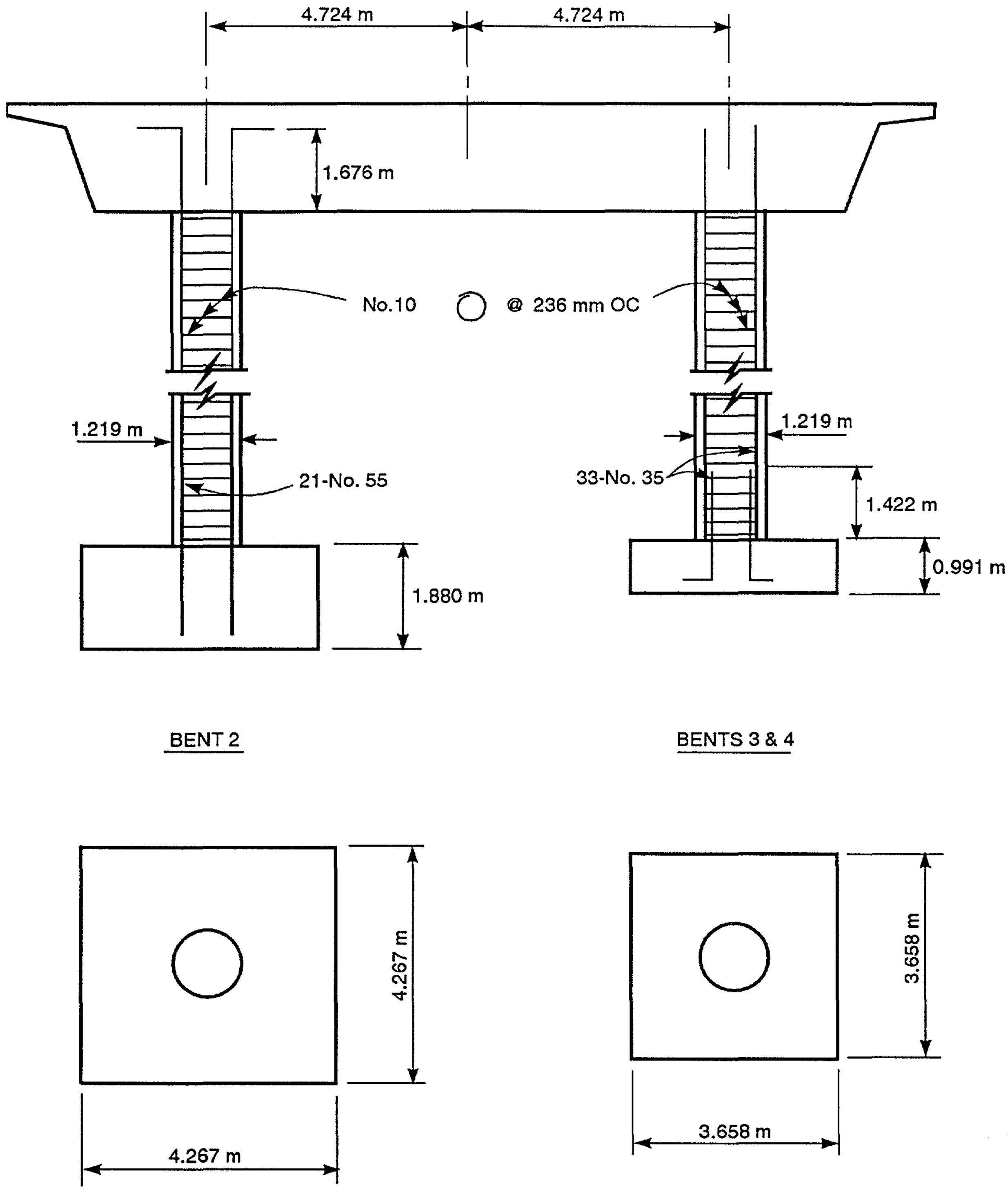


Figure 110. Column details.

As described in section 2.3, this rating system requries the calculation of a priority index for each bridge. This index is based on an assessment of bridge rank; importance; non-seismic deficiencies; and other factors such as network redundancy, political, and economic considerations. One of the critical elements in this calculation is the bridge rank (R), which is obtained from the vulnerability rating of the structure (V) and the seismic hazard rating of the site (E), as follows:

$$R = V \cdot E$$

Both of these ratings are determined for the example bridge in the following sections.

C.3.1 VULNERABILITY RATING, V

The procedure suggested in section 2.3 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 have satisfactory bearing details.

<u>Step 2</u>: 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, i.e., $V_T = 5$.

<u>Step 3</u>: In the longitudinal direction, calculate the minimum required support length at the hinge seat.

$$L = L_1 + L_2 = 143 \text{ m (470 ft)}$$
$$H = (12.2 + 0) \div 2 = 6.1 \text{ m (20 ft)}$$

Therefore:

$$\begin{aligned} N(d) &= 300 + 2.5L + 10H \\ &= 300 + 2.5\,(143) + 10\,(6.10) \\ &= 719 \text{ mm (28.3 in)} \end{aligned}$$

$$\frac{N(c)}{N(d)} = \frac{203}{719} = 0.28 < 0.50$$

Therefore, $V_L = 10$ and the overall rating for connections, bearings, and seatwidths is:

$$V_1 = \text{maximum of } V_L \text{ and } V_T$$
$$= 10$$

B. Columns, Piers, and Footing

Step 1: Does not apply

Step 2: Does not apply

Step 3: Does not apply

Step 4: Calculate the value for Q for the shortest and most heavily reinforced columns, which are the columns in bent 2.

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

$$= 13 - 6 \left(\frac{12.19}{4.6(2)(1.219)} \right)$$

$$= 6.5$$

Because $A \leq 0.4$ and the support skew is less than 20°, the maximum reduction of 4 can be made, i.e.,

$CVR = Q - 4 = 6.5 - 4 = 2.5 \approx 3$

Step 5: Does not apply

Step 6: Does not apply

Therefore, the column vulnerability rating CVR = 3

C. Abutments

Step 1: Does not apply

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

Step 3: Does not apply

Therefore, the abutment vulnerability rating AVR = 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, the liquefaction vulnerability rating LVR = 0

E. Vulnerability Rating for Components Other Than Bearings, V_2

$$\begin{aligned} V_2 &= CVR + AVR + LVR \\ &= 3 + 0 + 0 = 3 \end{aligned}$$

F. Overall Bridge Vulnerability (V)

The overall bridge vulnerability rating is the maximum of V_1 and V_2, i.e.,

$$V = 10.$$

C.3.2 SEISMIC HAZARD RATING, E

The seismic hazard rating is a function of both the Acceleration Coefficient, A, and the Site Coefficient, S. For the given example, A = 0.4 and, in the absence of specific soils data, a default Site Coefficient of 1.2 is assumed based on a Type II soil profile. It therefore follows that:

$$\begin{aligned} E &= 12.5 \cdot A \cdot S \\ &= 12.5\,(0.4)\,(1.2) \\ &= 6.0 \end{aligned}$$

C.3.3 BRIDGE RANK, R

Bridge rank is given by:

$$\begin{aligned} R &= V \cdot E \\ &= 10\,(6) \\ &= 60 \end{aligned}$$

C.3.4 PRIORITY INDEX

A priority index is assigned once all of the bridges are listed in order of their bridge rank, R. This process will require considerable judgment since this prioritized list must also take into account such factors as structure importance, non-seismic deficiences, remaining useful life, network redundancy, and the like.

C.4 DETAILED EVALUATION

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

C.4.1 CAPACITY/DEMAND RATIOS - EXISTING BRIDGE

Capacity/Demand ratios are calculated for the applicable components as shown in table 5 on page 62 of the Manual.

Analysis Procedure (Section 3.3.2)

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 seismic design provisions of Division I-A of the AASHTO Standard Specifications and automatically combines orthogonal elastic forces as described in section 3.3.2.4. Foundation stiffnesses at the abutments were selected using the procedure outlined in section 3.3.2.2. 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 consistent with the input, which in this case are kilonewtons and meters. Output forces and moments should be interpreted according to the convention shown in figures 111 and 112.

Minimum Bearing Force Demands (Appendix A.2)

The minimum force demand for the transverse shear key at the expansion joint is calculated by considering the equivalent static load to be acting only on the suspended portion of the first span.

Superstructure Weight = 8.44 (23.56) + 35.90 = 234.7 kN/m

Minimum Force Demand = 0.2(234.7)(44.81 ÷ 2) = 1052 kN

Minimum Support Length (Appendix A.3)

$$N(d) = 300 + 2.5(143) + 10(6.10) = 719 \text{ mm}$$

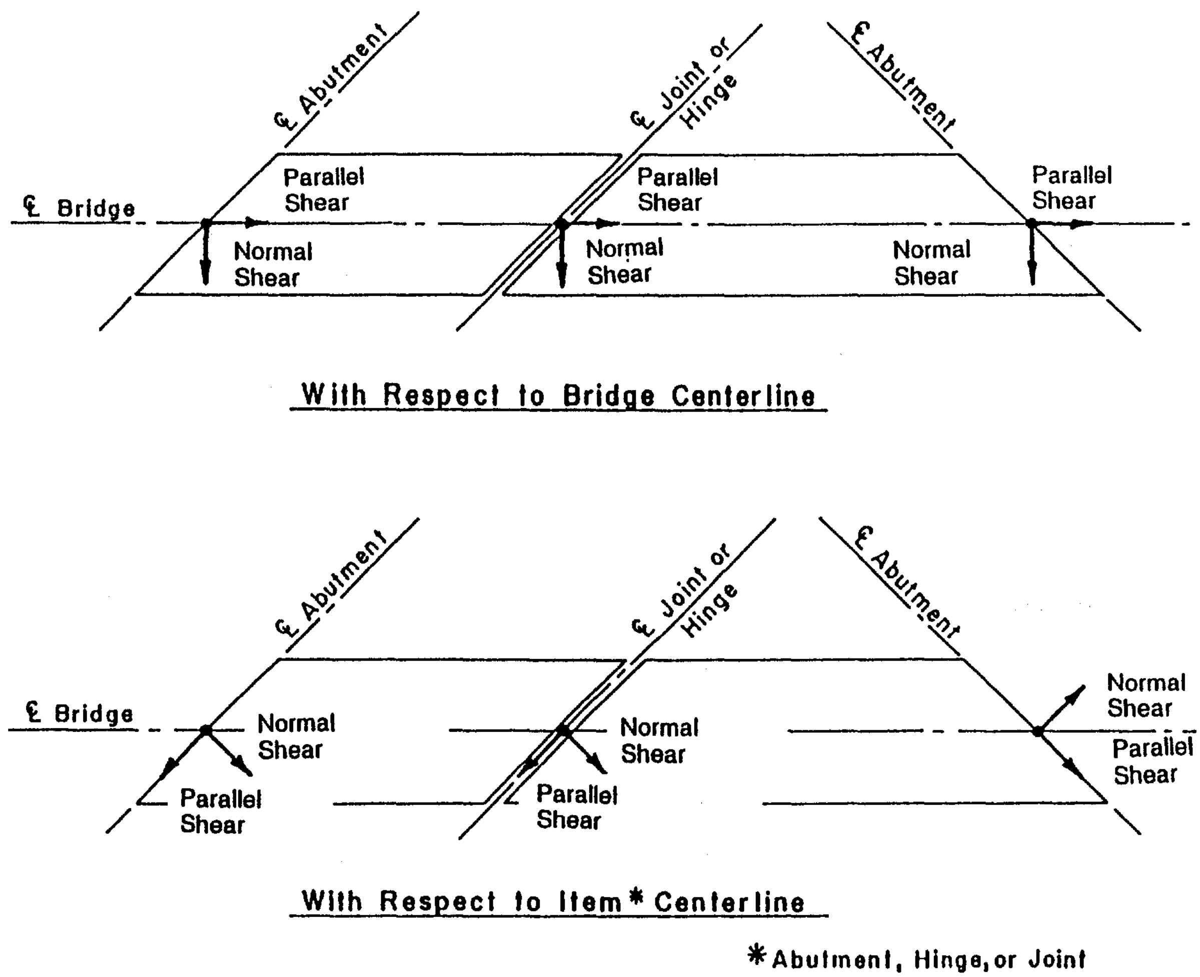


Figure 111. Positive sign convention for abutment, hinge, and joint forces.

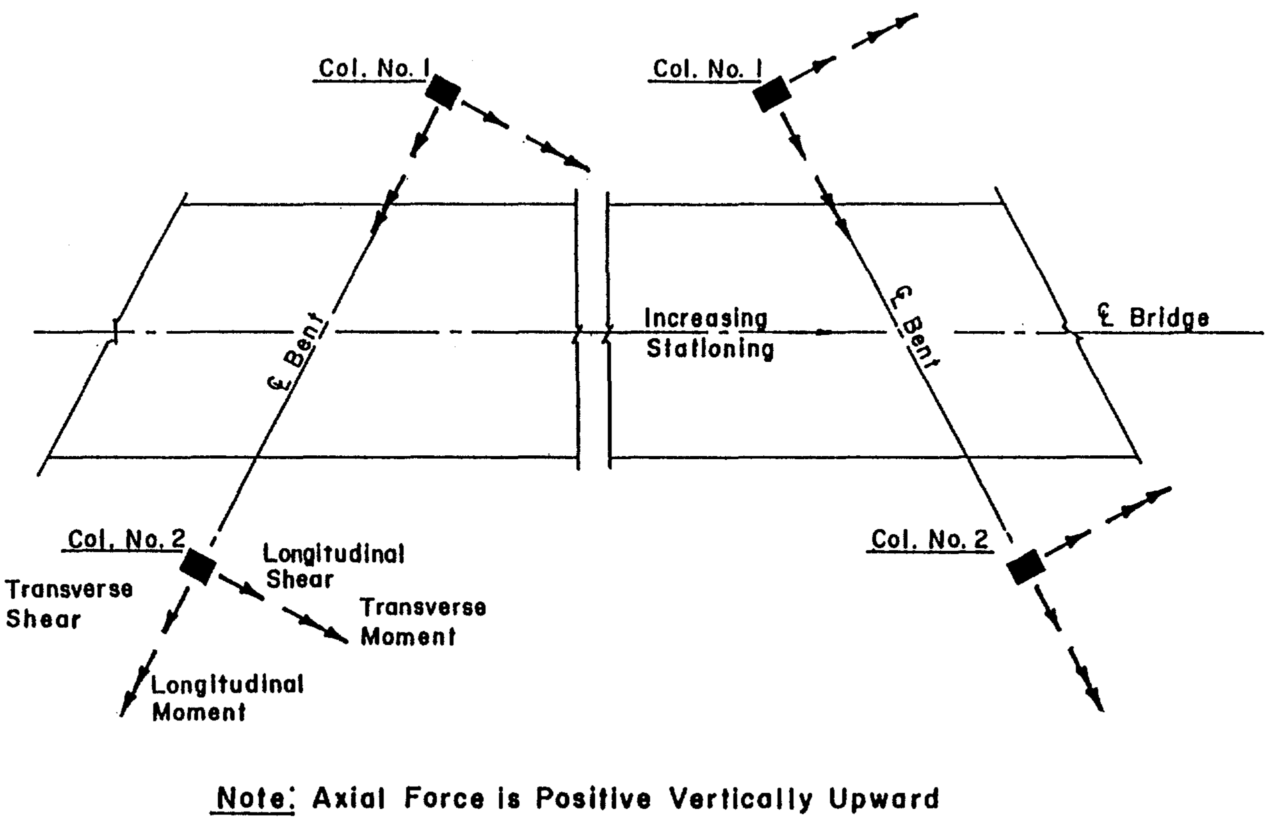


Figure 112. Positive sign convention for column forces and moments.

```
C ****************************************************************************
C *                                                                          *
C *                             WORKED EXAMPLE                               *
C *   RESPONSE SPECTRUM ANALYSIS OF THE EXISTING BRIDGE - F'c = 2.2408E+04 KPa  *
C *                                                                          *
C ****************************************************************************
C
SEISAB 'WORKED EXAMPLE - NO RETROFITTING'
      RESPONSE SPECTRUM ANALYSIS
C
C --- ALIGNMENT DATA ---
   ALIGNMENT
      STATION 0.00
      COORDINATES N 0.00 E 0.00
      BEARING N 0 35 27 E
C
C --- SPAN DATA ---
   SPANS
      LENGTHS /48.590/ /32.310/ /34.750/ /27.010/
      AREAS /8.4400/ /8.4400/ /8.4400/ /8.4400/
      I11 /1.4500/ /1.4500/ /1.4500/ /1.4500/
      I22 /177.80/ /177.80/ /177.80/ /177.80/
      I33 /4.6300/ /4.6300/ /4.6300/ /4.6300/
      DENSITY /23.563/ /23.563/ /23.563/ /23.563/
      E /2.2408E+07/ /2.2408E+07/ /2.2408E+07/ /2.2408E+07/
      WEIGHT /35.900/ /35.900/ /35.900/ /35.900/
C
C --- DESCRIBE DATA BLOCK ---
C
   DESCRIBE
C
C ** INDIVIDUAL COLUMN PROPERTIES **
      COLUMN 'Type 1' "4 FT. ROUND COLUMN"
         AREA 1.1710
         I11 0.21700
         I22 0.10800
         I33 0.10800
         DENSITY 23.563
         E 2.2408E+07
C
C ** WALL PROPERTIES ***
      WALL 'Type 1' " ABUTMENT 1 BACKWALL"
         AREA 13.006
         I11 2.5200
         I22 0.63000
         I33 315.89
         DENSITY 23.563
         E 2.2408E+07
      WALL 'Type 2' "ABUTMENT 5 BACKWALL"
         AREA 13.285
         I11 2.5600
         I22 0.64000
         I33 333.16
         DENSITY 23.563
         E 2.2408E+07
C
C ** BENT CAP PROPERTIES **
      CAP 'Type 1'
         AREA 2.7220
```

```
        I11 889.00
        I22 889.00
        I33 0.88900
        DENSITY 23.563
        E 2.2408E+07
C
C --- ABUTMENT DATA ---
   ABUTMENT STATION 0.00
      BEARING -
        S 85 38 0 W -
        N 78 30 0 W
        ELEV TOP 418.11 AT ABUT 1
        ELEV WALL BOTTOM 415.12 AT ABUT 1
        ELEV TOP 427.62 AT ABUT 5
        ELEV WALL BOTTOM 424.60 AT ABUT 5
        CONNECTION PIN AT ABUT 1 5
        WALL 'Type 1' AT ABUTMENT 1
        WALL 'Type 2' AT ABUTMENT 5
C
C --- BENT DATA ---
   BENT
      BEARING N 89 36 58 W, N 89 36 58 W, N 79 52 0 W
      ELEVATION TOP    421.53, 423.66, 425.96
      ELEVATION BOTTOM 408.01, 408.01, 410.14
      CAP 'Type 1' AT BENT 2 3 4
      COLUMN SKEWED LAYOUT 'Type 1' 9.4500 'Type 1' AT BENT 2 3
      COLUMN SKEWED LAYOUT 'Type 1' 9.5700 'Type 1' AT BENT 4
C
C --- EXPANSION JOINT DATA ---
   HINGE
      AT 1 44.620              $ Data for Hinge 1
      BEARING N 89 36 58 W
C
C --- FOUNDATION STIFFNESSES ---
   FOUNDATION
      AT ABUTMENT 1
        SPRING CONSTANTS
           KF1F1 980710.0
           KF2F2 291878.0
           KM1M1 1.356E+12
           KM2M2 1.356E+12
           KM3M3 1.356E+12
      AT ABUTMENT 5
        SPRING CONSTANTS
           KF1F1 77348.0
           KF2F2 291878.0
           KM1M1 1.356E+12
           KM2M2 1.356E+12
           KM3M3 1.356E+12
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C --- LOAD DATA ---
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      RESPONSE SPECTRUM
        ATC6 CURVE
           SOIL TYPE  II
           ACCELERATION COEFFICIENT 0.40000
        GRAVITY 9.8070
        DAMPING COEFFICIENT 0.050000
FINISH
```

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS	PARTICIPATION FACTORS Long	Vert	Tran	% OF TOTAL MASS Long	Vert	Tran
1	1.359	0.47	-0.494	-0.001	-53.394	0.006	0.000	71.101
2	0.937	0.60	52.104	-2.628	2.259	67.711	0.172	71.228
3	0.687	0.74	-12.650	27.806	0.111	71.702	19.454	71.228
4	0.375	1.00	-3.621	0.736	15.299	72.030	19.468	77.065
5	0.324	1.00	-12.841	-1.988	0.778	76.142	19.556	77.080
6	0.315	1.00	28.320	11.245	-1.069	96.144	22.720	77.109
7	0.268	1.00	-4.011	-18.588	0.061	96.546	31.337	77.109
8	0.242	1.00	-2.957	-42.741	0.055	96.764	76.895	77.109
9	0.210	1.00	1.934	6.908	-0.155	96.857	78.085	77.110
10	0.153	1.00	4.505	-22.219	-0.373	97.363	90.398	77.113
11	0.147	1.00	0.468	0.411	-0.278	97.369	90.402	77.115
12	0.124	1.00	0.890	-2.352	-0.949	97.388	90.540	77.137

ABUTMENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.009	0.001	0.009	0.001	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.009	0.001	0.009	0.001	0.000	0.000
	4	0.003	0.000	0.003	0.000	0.000	0.000
ABU 5	1	0.117	0.023	0.117	0.023	0.000	0.000
	2	0.006	0.001	0.006	0.001	0.000	0.000
	3	0.119	0.023	0.119	0.023	0.000	0.000
	4	0.041	0.008	0.041	0.008	0.000	0.000

SPAN HINGE CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
S 1 H1	1	0.024	0.012	0.130	0.012	0.133	0.000
	2	0.001	0.317	0.006	0.317	0.007	0.000
	3	0.025	0.107	0.132	0.107	0.135	0.000
	4	0.008	0.320	0.045	0.320	0.047	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

WORKED EXAMPLE - NO RETROFITTING

CQC COLUMN FORCES

			....LNGITUDNL....		....TRANSVRSE....			
CL	LOC	LC	SHEAR	MOMENT	SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	1468.1	10097.	112.1	749.	1179.3	68.7
		2	168.7	1132.	3427.3	23469.	4751.2	461.0
		3	1518.7	10436.	1140.3	7790.	2604.7	207.0
		4	609.1	4161.	3460.9	23693.	5105.0	481.6
1	TOP	1	1394.3	9335.	100.8	710.	1176.4	68.7
		2	162.9	1116.	3345.8	22406.	4750.5	461.0
		3	1443.1	9670.	1104.6	7432.	2601.6	207.0
		4	581.2	3916.	3376.0	22619.	5103.4	481.6
2	BOT	1	1420.0	9776.	138.5	876.	1273.4	68.9
		2	199.9	1349.	3426.6	23466.	4732.7	461.0
		3	1480.0	10180.	1166.5	7915.	2693.2	207.2
		4	625.9	4281.	3468.2	23728.	5114.7	481.7
2	TOP	1	1348.5	9020.	129.6	950.	1270.7	68.9
		2	191.6	1305.	3345.1	22400.	4732.0	461.0
		3	1406.0	9412.	1133.1	7670.	2690.3	207.2
		4	596.2	4011.	3384.0	22685.	5113.2	481.7
BNT 3								
1	BOT	1	1027.9	7938.	33.1	256.	618.5	53.6
		2	108.7	845.	1512.0	11914.	2393.9	408.9
		3	1060.6	8191.	486.7	3830.	1336.6	176.3
		4	417.1	3227.	1522.0	11991.	2579.5	425.0
1	TOP	1	940.1	7573.	23.6	204.	617.4	53.6
		2	103.3	821.	1446.9	11319.	2393.4	408.9
		3	971.1	7819.	457.7	3600.	1335.4	176.3
		4	385.4	3092.	1454.0	11380.	2578.6	425.0
2	BOT	1	998.3	7710.	53.3	370.	693.3	53.7
		2	138.3	1068.	1511.1	11910.	2364.3	408.9
		3	1039.8	8030.	506.6	3943.	1402.6	176.4
		4	437.8	3381.	1527.1	12021.	2572.3	425.0
2	TOP	1	913.0	7353.	46.8	419.	692.3	53.7
		2	130.6	1045.	1446.0	11310.	2363.8	408.9
		3	952.1	7667.	480.6	3813.	1401.4	176.4
		4	404.5	3251.	1460.1	11436.	2571.5	425.0
BNT 4								
1	BOT	1	1004.2	7808.	33.2	281.	822.4	42.8
		2	44.5	348.	653.3	5191.	1073.1	431.0
		3	1017.6	7913.	229.2	1838.	1144.4	172.1
		4	345.7	2690.	663.3	5275.	1319.9	443.9
1	TOP	1	915.2	7487.	22.5	174.	821.9	42.8
		2	40.5	329.	619.4	4914.	1072.9	431.0
		3	927.4	7586.	208.3	1648.	1143.8	172.1
		4	315.0	2575.	626.1	4966.	1319.4	443.9
2	BOT	1	980.6	7624.	61.5	436.	433.4	42.9
		2	252.8	1992.	652.4	5186.	1042.1	431.0
		3	1056.4	8222.	257.3	1992.	746.0	172.2
		4	547.0	4280.	670.9	5317.	1172.1	443.9
2	TOP	1	893.7	7311.	52.9	478.	432.8	42.9
		2	240.8	1927.	618.5	4905.	1041.8	431.0
		3	965.9	7889.	238.5	1949.	745.3	172.2
		4	508.9	4120.	634.4	5048.	1171.7	443.9

WORKED EXAMPLE - NO RETROFITTING

ABUTMENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	TRANSVERSE	W/R TO ITEM C.L. NORMAL	PARALLEL
ABU 1	1	3527.3	9082.2	268.9	9066.4	599.2
	2	135.9	370.4	1286.7	387.3	1281.8
	3	3568.1	9193.3	654.9	9182.5	983.7
	4	1194.1	3095.1	1367.4	3107.2	1461.5
ABU 5	1	1045.1	9289.9	1001.8	9253.2	1298.0
	2	102.5	709.5	2901.8	445.8	2953.9
	3	1075.9	9502.8	1872.4	9386.9	2184.1
	4	416.1	3496.4	3202.4	3221.8	3343.2

SPAN HINGE CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	TRANSVERSE	W/R TO ITEM C.L. NORMAL	PARALLEL
S 1 H1	1	1789.9	0.0	366.6	1.3	366.6
	2	49.8	0.0	2816.0	10.2	2816.0
	3	1804.9	0.0	1211.4	4.4	1211.4
	4	586.7	0.0	2926.0	10.6	2926.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Capacity/Demand Ratio at the Expansion Joints and Bearings (Appendix A.4)

Displacement C/D Ratio (appendix A.4.2) - Expansion joint

Method 1:

$$N(c) = 203 \text{ mm}$$

$$r_{bd} = \frac{N(c)}{N(d)} = \frac{203}{719} = 0.28$$

Method 2:

Assume that of the 203 mm of total seat length, 76 mm may be considered ineffective because it is the cover on the expansion joint reinforcement.

$$\begin{aligned} \Delta_s(c) &= 203 - 76 = 127 \text{ mm} \\ \Delta_i(d) &= 84 \text{ mm (temperature, etc.)} \\ \Delta_{eq}(d) &= 135 \text{ mm (from computer output)} \\ r_{bd} &= (127 - 84) \div 135 = 0.32 \end{aligned}$$

Shear Force C/D Ratio (appendix A.4.3)

$$V_b(d) = 2926 \times 1.25 = 3658 \text{ kN} > 1052 \text{ kN}$$

$$V_b(c) = 6 \times 12 \times \frac{100}{10^6} \times 400 \times 10^3 = 2880 \text{ kN}$$

The shear resistance is provided by the six, 762 mm long shear keys with twelve No. 4 bars in each key.

$$r_{bf} = \frac{2880}{3658} = 0.79$$

Capacity/Demand Ratios at Columns, Piers, and Footings (Appendix A.5)

Step 1: Elastic Moment Demands

The elastic moment demands are calculated by combining the moments about the principal axes of the columns to obtain the maximum moments. In most cases, Load Case 2 has the highest demands. Dead-load 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 moments of highest demands are summarized in table 10.

Table 10. Maximum elastic moment demands (kN-m).

Location	Component	Trans. Moment		Long. Moment		Elastic Moment Demand
		EQ	DL	EQ	DL	
B-2 (C-2) Top	Column	22685	1786	4011	107	24815
B-2 (C-2) Bottom	Column	23728	895	4281	14	24995
B-2 (C-2) Bottom	Footing	30248	1268	5458	27	31990
B-3 (C-2) Top	Column	11436	1243	3251	228	13148
B-3 (C-2) Bottom	Column	12021	619	3381	168	13129
B-3 (C-2) Bottom	Footing	13534	737	3815	193	14823
B-4 (C-2) Top	Column	1949	1315	7889	33	8568
B-4 (C-2) Bottom	Column	1992	660	8222	68	8704
B-4 (C-2) Bottom	Footing	2247	784	9269	74	9822

Step 2: Ultimate Moment Capacities

Ultimate moment capacities for the columns are obtained from the computer-generated column interaction diagrams shown in figures 113 and 114. Ultimate moment capacities for the footing are obtained from interaction diagrams for the footings, which are also shown in figures 113 and 114. The development of coordinates on the footing interaction diagram for bent 2 is illustrated in figure 115. The development of these diagrams for bents 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 overturning as outlined in the iterative procedure presented in section 4.8.2 of the AASHTO seismic design specifications. The steps of this procedure are as follows:

Step 2.1 Overstrength Moment Capacities at Axial Load Corresponding to Dead Load

Table 11 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 115. Bents 3 and 4 have identical capacities.

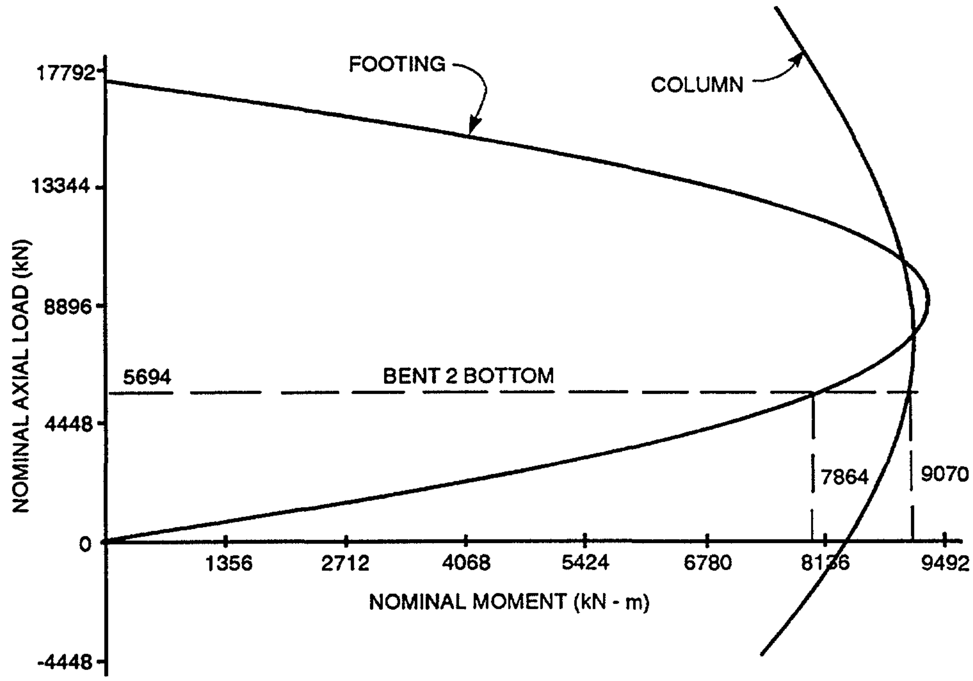


Figure 113. Bent 2 column and footing interaction diagrams.

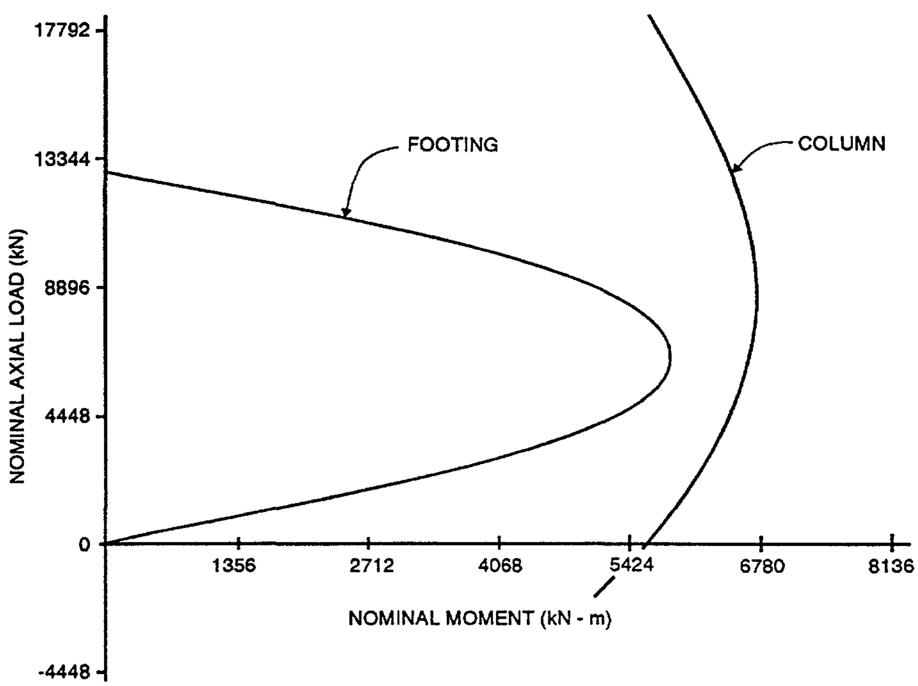
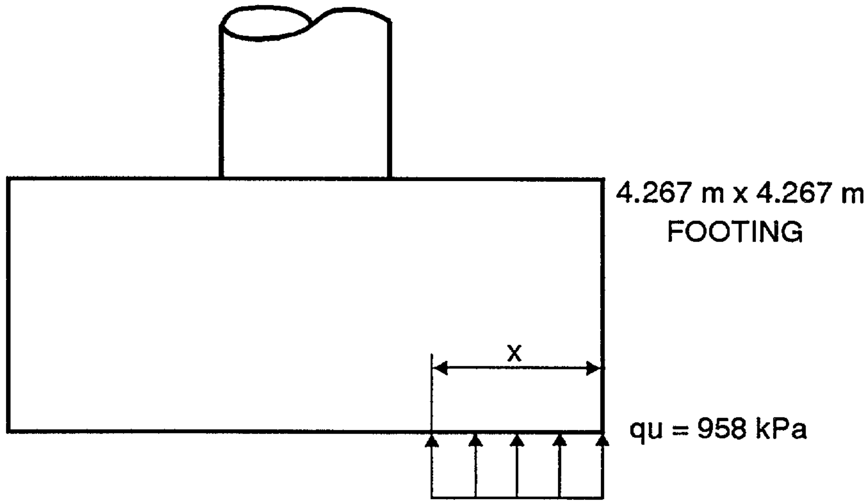


Figure 114. Bents 3 and 4 column and footing interaction diagrams.



ASSUMED ULTIMATE LOAD AT
BENT 2 FOOTING

DEVELOPMENT OF EQUATIONS OF NOMINAL CAPACITY:

AXIAL LOAD CAPACITY = (958)(4.267)x = 4088x

MOMENT CAPACITY = (4088x)(2.134 - x/2) = 8724x - 2044x^2

x	AXIAL FORCE	MOMENT
0.61 m	2494	4561
1.22 m	4987	7601
1.83 m	7481	9120
2.44 m	9975	9117
3.05 m	12 468	7594
3.66 m	14 962	4549
4.27 m	17 456	0

Figure 115. Development of footing interaction diagram at bent 2.

Table 11. Column and footing overstrength moments.

Bent	End	Axial Force Due to Dead Load	1.3 M_u	
			Column	Footing
2	Top	5293	11796	--
2	Bottom	5694	11796	10223
3 & 4	Top	4270	8379	--
3 & 4	Bottom	4715	8528	7213

*Step 2.2 Column Shear Forces**

Bent 2: V_u = (11769 + 10223) ÷ 14.08 = 1562 kN

Bents 3 & 4: V_u = (8379 + 7213) ÷ 16.25 = 960 kN

* 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 2.3 Axial Forces Due to Overturning in the Transverse Direction

Bent 2: Axial Force = 2(1562)(14.08) ÷ 9.45 = +4655

Bents 3 & 4: Axial Force = 2(960)(16.25) ÷ 9.45 = +3302

Step 2.4 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 12 summarizes these revised moment capacities.

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

Bent 2: Shear = (11036 + 2088) ÷ 14.08 + (11796 + 11674) ÷ 14.08
= 2599 kN

Bents 3 & 4: Shear = (6996 + 2996) ÷ 16.25 + (8786 + 7132) ÷ 16.25
= 1594 kN

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.

Table 12. Revised column and footing overstrength moments (iteration 1).

Bent	End	Axial Force Due to Dead Load + Overturning	1.3 M_u	
			Column	Footing
2	Top	638	11036	-
2	Top	9948	11796	-
2	Bottom	1039	11145	2088
2	Bottom	10349	11741	11674
3 & 4	Top	968	6996	-
3 & 4	Top	7572	8786	-
3 & 4	Bottom	1413	7769	2996
3 & 4	Bottom	8017	8813	7132

Bent 2: Axial Force = 2599(14.08) ÷ 9.45 = 3872 kN

Bents 3 & 4: Axial Force = 1594(16.25) ÷ 9.45 = 2741 kN

These axial loads are used to recalculate the overstrength moments that are summarized in table 13.

Table 13. Revised column and footing overstrength moments (iteration 2).

Bent	End	Axial Force Due to Dead Load + Overturning	1.3 M_u	
			Column	Footing
2	Top	1421	11240	-
2	Top	9165	11836	-
2	Bottom	1822	11321	3715
2	Bottom	9566	11823	11972
3 & 4	Top	1529	7728	-
3 & 4	Top	7011	8759	-
3 & 4	Bottom	1974	7850	3864
3 & 4	Bottom	7456	8786	7389

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

Bent 2: Shear $= (11240 + 3715) \div 14.08 + (11836 + 11972) \div 14.08$
$= 2753$ kN

Bents 3 & 4: Shear $= (7728 + 3864) \div 16.25 + (8759 + 7389) \div 16.25$
$= 1707$ kN

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 14.

Table 14. Ultimate moment capacity/elastic moment demand ratios.

Bent	End	Axial Load	Column			Footing		
			Demand	Capacity	r_{ec}	Demand	Capacity	r_{ef}
2	Top	Min.	24815	8650	0.35	–	–	–
2	Top	Max.	24815	9111	0.37	–	–	–
2	Bottom	Min.	24995	8704	0.35	31990	2861	0.09
2	Bottom	Max.	24995	9098	0.36	31990	9206	0.29
3	Top	Min.	13148	5938	0.45	–	–	–
3	Top	Max.	13148	6738	0.51	–	–	–
3	Bottom	Min.	13129	6033	0.46	14823	2969	0.20
3	Bottom	Max.	13129	6752	0.51	14823	5681	0.38
4	Top	Min.	8568	5938	0.69	–	–	–
4	Top	Max.	8568	6738	0.79	–	–	–
4	Bottom	Min.	8704	6033	0.69	9822	2969	0.30
4	Bottom	Max.	8704	6752	0.76	9822	5681	0.58

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

Bent 2 - Case II ($r_{ec} = 0.35$ and $r_{ef} = 0.09$):

1. Anchorage (appendix A.5.1) - Straight anchorage

$$\ell_a(c) = 1880 - 76 = 1804 \text{ mm}$$

For anchorage in the footing, assume the large cover (1575 mm) 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 = $19.69\sqrt{f_c'} = 2947$ kPa

$$k_{tr} = 2947(1575)(2)/[(21 \div 2)(4137)(55)] = 3.89 > 2.5$$

$$\ell_a(d) = \frac{((400 \times 10^3 - 75842) \div 33.1)\ 55}{\left(\sqrt{2.24 \times 10^4}\right)/2.63\ (1 + 2.5(55 \div 55) + 2.5)}$$

$$= 1578 \text{ mm}$$

Therefore, Case B applies. Calculate the negative moment capacity of the footing using a concrete tensile strength of 2947 kPa ($19.69\sqrt{f_c'}$).

$$\text{Negative Moment Capacity} = 2947(4.267(1.88)^2 \div 6) = 7407 \text{ kN-m}$$

This capacity is sufficient to resist the weight of the overburden. Therefore:

$$r_{ca} = 1.0$$

2. Splices (appendix A.5.2) - Does not apply

3. Footing Rotation (appendix A.5.5)

Because anchorage or splice failures will not prevent footing rotation:

$$r_{fr} = \mu r_{ef} = 4(0.09) = 0.36$$

Bent 2 - Case II ($r_{ec} = 0.36$ and $r_{ef} = 0.29$):

1. Anchorage - Same as before

$$r_{ca} = 1.0$$

2. Splices - Does not apply

3. Footing Rotation

$$r_{fr} = \mu r_{ef} = 4(0.29) = 1.16$$

Bent 3 - Case II (Two possible combinations of r_{ec} and r_{ef} must be investigated: $r_{ec} = 0.46$ and $r_{ef} = 0.20$, plus $r_{ec} = 0.51$ and $r_{ef} = 0.38$).

1. Anchorage - Hooked Anchorage

$$\ell_a(c) = 838 \text{ mm}$$

$$\ell_a(d) = 0.7(1200)(35) \div (\sqrt{2.24 \times 10^4}/2.63) = 517 \text{ mm}$$

Case B applies.

$$\text{Negative Moment Capacity} = 2947(3.658(0.991)^2 \div 6) = 1764 \text{ kN-m}$$

Because this capacity is sufficient to resist the weight of the overburden,

$$r_{ca} = 1.0$$

2. Splices (appendix A.5.2)

Because the clear spacing between splices = 38 mm < 4(35)

$$A_{tr}(c) = 2(100)/(33 \div 2) = 12.1$$

$$A_{tr}(d) = \frac{236}{1422} 1000 = 166$$

$$\ell_s = 1422 > \left(\frac{4885}{\sqrt{f_c'}}\right) d_b = 1144$$

Therefore, Case A applies.

$$r_{cs} = 0.75\,(0.46) = 0.35$$
$$r_{cs} = 0.75\,(0.51) = 0.38$$

Notice that the minimum value for r_{cs} controls.

3. Footing

$$0.8\ r_{ef} = 0.8(0.38) = 0.30 < 0.38$$
$$0.8\ r_{ef} = 0.8(0.20) = 0.16 < 0.35$$

Therefore, a splice failure cannot be assumed to prevent footing rotation. The minimum C/D ratio for the footing is given by

$$r_{fr} = 4(0.20) = 0.80$$

Bent 4 - Case II ($r_{ec} = 0.69$ and $r_{ef} = 0.30$):

1. Anchorage - Same as bent 3

$$r_{ca} = 1.0$$

2. Splices

$$r_{cs} = 0.75\ (0.69) = 0.52$$

3. Footing

$$0.8\ r_{ef} = 0.8(0.30) = 0.24 < 0.52$$

Bent 4 - Case II ($r_{ec} = 0.76$ and $r_{ef} = 0.58$):

1. Anchorage

$$r_{ca} = 1.0$$

2. Splices

$$r_{cs} = 0.75\ (0.76) = 0.57$$

3. Footing

$$0.8\ r_{ef} = 0.8(0.58) = 0.46 < 0.57$$

Therefore, a splice failure cannot be assumed to prevent footing rotation. The minimum C/D ratio for the footing is given by:

$$r_{fr} = 4(0.30) = 1.20$$

Step 5: Calculate C/D Ratio at the Top of the Column

Bent 2

1. Anchorage

$$\ell_a(c) = 1676 \text{ mm}$$

$$\ell_a(d) = 1200\,(55) \div \sqrt{2.24\text{x}10^4/2.63} = 1160 \text{ mm}$$

Therefore, Case B applies:

$$r_{ca} = 1.0$$

2. Splices - Does not apply

3. Confinement (appendix A.5.4)

$$\rho(c) = 100(\pi)(1118) \div [\pi(610)^2(236)] = 0.0013$$

$$\rho(d) = 0.45\left(\frac{\pi(610)^2}{\pi(559)^2} - 1\right)\frac{2.24\text{x}10^4}{4\text{x}10^5} = 0.0048$$

$$\frac{P_c}{f_c' A_g} = \frac{9165}{(2.24\text{x}10^4)\pi(610)^2/(1000)^2} = 0.35$$

$$k_1 = \frac{0.0013}{0.0048\ (0.5 + 1.25(0.35))} = 0.29$$

$$k_2 = \frac{0.2}{(305 \div 1219)} = 0.8$$

Because transverse steel is poorly anchored, an iterative solution for μ is required.

Try k_3 = 0.35 (corresponds to $\mu = 2.7$)

$$\mu = 2 + 4\left(\frac{0.29 + 0.80}{2}\right)0.35 = 2.8 \qquad \text{ok}$$

$$r_{cc} = \mu r_{ec} = 2.8(0.35) = 0.98$$

Bent 3

1. Anchorage

$$\ell_a(c) = 1676 \text{ mm}$$

$$\ell_a(d) = \frac{((400 \times 10^3 - 75842) \div 33.1)\ 35}{(\sqrt{2.24 \times 10^4/2.63})\ (1 + 2.5\ \frac{36}{35})} = 1687 \text{ mm}$$

Case A applies

$$r_{ca} = \frac{1676}{1687}(0.45) = 0.45$$

2. Splices - Does not apply

3. Confinement

$$k_1 = \frac{0.0013}{0.0048\ (0.5 + 1.25(0.270))} = 0.32$$

$$k_2 = \frac{6}{305/35} = 0.69$$

Try k_3 = 0.35 (corresponds to μ = 2.7)

$$\mu = 2 + 4\left(\frac{0.32 + 0.69}{2}\right) 0.35 = 2.7 \qquad \text{ok}$$

$$r_{cc} = 2.7\ (0.45) = 1.22$$

Bent 4

1. Anchorage

$$r_{ca} = \frac{1676}{1687}(0.69) = 0.69$$

2. Splices - Does not apply

3. Confinement

$$r_{cc} = 2.7(0.69) = 1.86$$

Step 6: Calculate C/D Ratios for Column Shear (appendix A.5.3)

Bent 2 - Transverse bending - 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.

$$V_u(d) = \frac{11836 + 11972}{14.08} = 1691 \text{ kN}$$

$$V_c(d) = 3468 + 198 = 3666 \text{ kN}$$

$$V_i(c) = v_c db + \frac{A_{tr} f_{yt} d}{s}$$

$$= \frac{787(1059)(1219)}{(1000)^2} + \frac{(2 \times 100)(4 \times 10^5)(1059)}{236(1000)^2}$$

$$= 1375 \text{ kN}$$

Because column axial stress may fall below $0.10f_c'$ and transverse steel is ineffective:

$$V_f(c) = 0$$

Therefore, Case A applies, i.e.,

$$r_{cv} = \frac{1375}{3666} = 0.38 > 0.36 \text{ (the value of } r_{ec}\text{)}$$

$$r_{cv} = 0.36$$

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.

$$V_u(d) = \frac{(1676/1687)\ 8759 + 7389}{16.25} = 990 \text{ kN}$$

$$V_e(d) = 1527 + 119 = 1646 \text{ kN}$$

$$V_i(c) = 1375 \text{ kN}$$

$$V_f(c) = 0$$

Therefore, Case B applies, i.e.,

$$\mu = 2 + (0.75)(4)\left(\frac{1375 - 990}{1375 - 0}\right) = 2.8$$

$$r_{cv} = 2.8(0.45) = 1.26$$

Bent 4

$$r_{ec} = 0.69$$

$$V_e(d) = 671 + 125 = 796 \text{ kN}$$

$$\mu = 2.8$$

$$r_{cv} = 2.8\ (0.69) = 1.93$$

Capacity/Demand Ratio for Abutments (Appendix A.6)

Abutment C/D ratios are based on the displacements from an analysis.

<u>Transverse Displacement</u>

$$d(c) = 75 \text{ mm}$$

Abutment 1:

$$d(d) = 1 \text{ mm}$$

$$r_{ad} = \frac{75}{1} = 75$$

Abutment 5:

$$d(d) = 23$$

$$r_{ad} = \frac{75}{23} = 3.3$$

<u>Longitudinal Displacement</u>

$$d(c) = 150 \text{ mm}$$

Abutment 1:

$$d(d) = 9 \text{ mm}$$
$$r_{ad} = \frac{150}{9} = 16.7$$

Abutment 5:

$$d(d) = 119 \text{ mm}$$
$$r_{ad} = \frac{150}{119} = 1.26$$

Capacity/Demand Ratio for Liquefaction (Appendix A.7)

Because the preliminary screening (Seismic Rating System) indicated that low liquefaction-related damage was likely, a C/D ratio is not determined.

C.4.2 IDENTIFICATION AND ASSESSMENT OF POTENTIAL RETROFIT MEASURES

Table 15 summarizes the C/D ratios that are less than 1.0 for the existing bridge.

Table 15. Capacity demand ratios for the existing bridge.

Component	Notation	As-Built Bridge
Expansion Joint	r_{bd}	0.28
	r_{bf}	0.79
Bent 2 (Overall)	r_{cv}	0.36
Bent 2 (Bottom)	r_{fr}	0.36
Bent 3 (Bottom)	r_{fr}	0.80
Bent 3 (Top)	r_{ca}	0.45
Bent 4 (Top)	r_{ca}	0.69

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 disintegration 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 warrant 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 retrofit would also eliminate the potential for 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.

The top column steel anchorage of bent 4 has a C/D ratio (r_{ca}) of 0.69. Since the C/D ratio of footing rotation (r_{fr} = 1.20) is larger than 1.0, failure of steel anchorage would not cause a collapse mechanism to occur, and is considered an acceptable failure. Therefore, retrofitting of bent 4 is not proposed either.

Because the infill shear wall at bent 2 would significantly affect 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:

$$\Delta_s(c) = 127 \text{ mm}$$
$$\Delta_i(d) = 84 \text{ mm}$$
$$\Delta_{eq}(d) = 24 \text{ mm}$$
$$r_{bd} = \frac{(127 - 84)}{24} = 1.79 > 1 \quad \text{ok}$$

```
C ******************************************************************************
C *                                                                            *
C *                            WORKED EXAMPLE                                  *
C *   RESPONSE SPECTRUM ANALYSIS OF THE EXISTING BRIDGE - F'c = 2.2408E+04 KPa *
C *                                                                            *
C ******************************************************************************
C
SEISAB 'WORKED EXAMPLE - RETROFITTED'
      RESPONSE SPECTRUM ANALYSIS
C
C --- ALIGNMENT DATA ---
   ALIGNMENT
      STATION 0.00
      COORDINATES N 0.00 E 0.00
      BEARING N 0 35 27 E
C
C --- SPAN DATA ---
C
   SPANS
      LENGTHS /48.590/ /32.310/ /34.750/ /27.010/
      AREAS /8.4400/ /8.4400/ /8.4400/ /8.4400/
      I11 /1.4500/ /1.4500/ /1.4500/ /1.4500/
      I22 /177.80/ /177.80/ /177.80/ /177.80/
      I33 /4.6300/ /4.6300/ /4.6300/ /4.6300/
      DENSITY /23.563/ /23.563/ /23.563/ /23.563/
      E /2.2408E+07/ /2.2408E+07/ /2.2408E+07/ /2.2408E+07/
      WEIGHT /35.900/ /35.900/ /35.900/ /35.900/
C
C --- DESCRIBE DATA BLOCK ---
C
   DESCRIBE
C
C ** INDIVIDUAL COLUMN PROPERTIES **
C
      COLUMN 'Type 1' "4FT. ROUND COLUMN"
         AREA 1.1710
         I11 0.21700
         I22 0.10800
         I33 0.10800
         DENSITY 23.563
         E 2.2408E+07
      COLUMN 'INFILL'
         AREA 4.757
         I11  0.518
         I22  0.236
         I33 61.582
         DENSITY 23.563
         E 2.2408E+07
C
C ** RESTRAINER PROPERTIES **
C
      RESTRAINER 'Type 1' "CALIFORNIA CABLE RESTRAINER"
         LENGTH 2.438
         AREA   0.002
         E 1.24E+08
C
C ** WALL PROPERTIES **
C
      WALL 'Type 1' "ABUTMENT 1 BACKWALL"
```

```
         AREA 13.006
         I11 2.5200
         I22 0.63000
         I33 315.89
         DENSITY 23.563
         E 2.2408E+07
      WALL 'Type 2' "ABUTMENT 5 BACKWALL"
         AREA 13.285
         I11 2.5600
         I22 0.64000
         I33 333.16
         DENSITY 23.563
         E 2.2408E+07
C
C ** BENT CAP PROPERTIES **
C
      CAP 'Type 1'
         AREA 2.7220
         I11 889.00
         I22 889.00
         I33 0.88900
         DENSITY 23.563
         E 2.2408E+07
C
C --- ABUTMENT DATA ---
C
   ABUTMENT STATION 0.00
      BEARING -
         S 85 38 0 W -
         N 78 30 0 W
         ELEV TOP 418.11 AT ABUT 1
         ELEV WALL BOTTOM 415.12 AT ABUT 1
         ELEV TOP 427.62 AT ABUT 5
         ELEV WALL BOTTOM 424.60 AT ABUT 5
         CONNECTION PIN AT ABUT 1 5
         WALL 'Type 1' AT ABUTMENT 1
         WALL 'Type 2' AT ABUTMENT 5
C
C ---- BENT DATA ---
C
   BENT
      BEARING N 89 36 58 W, N 89 36 58 W, N 79 52 0 W
      ELEVATION TOP    421.53, 423.66, 425.96
      ELEVATION BOTTOM 408.01, 408.01, 410.14
      CAP 'Type 1' AT BENT 2 3 4
      COLUMN 'INFILL'                               AT BENT 2
      COLUMN SKEWED LAYOUT 'Type 1' 9.4500 'Type 1' AT BENT 3
      COLUMN SKEWED LAYOUT 'Type 1' 9.5700 'Type 1' AT BENT 4
C
C --- EXPANSION JOINT DATA ---
C
   HINGE
      AT 1 44.620               $ Data for Hinge 1
         BEARING N 89 36 58 W
         RESTRAINER NORMAL LAYOUT 'Type 1' 2.286, 7.62, 2.286 'Type 1' AT 1
C
C --- FOUNDATION STIFFNESSES ---
   FOUNDATION
      AT ABUTMENT 1
         SPRING CONSTANTS
```

```
            KF1F1 980710.0
            KF2F2 291878.0
            KM1M1 1.356E+12
            KM2M2 1.356E+12
            KM3M3 1.356E+12
      AT ABUTMENT 5
         SPRING CONSTANTS
            KF1F1 77348.0
            KF2F2 291878.0
            KM1M1 1.356E+12
            KM2M2 1.356E+12
            KM3M3 1.356E+12
C
C --- LOAD DATA ---
C
   LOADS
      RESPONSE SPECTRUM
         ATC6 CURVE
            SOIL TYPE  II
            ACCELERATION COEFFICIENT 0.40000
         GRAVITY 9.8070
         DAMPING COEFFICIENT 0.050000
FINISH
```

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS	PARTICIPATION FACTORS Long	Vert	Tran	% OF TOTAL MASS Long	Vert	Tran
1	0.727	0.71	-44.508	22.031	-3.565	49.360	12.094	0.317
2	0.539	0.87	39.793	17.654	7.838	88.816	19.859	1.848
3	0.444	0.99	10.037	1.221	-44.675	91.326	19.897	51.578
4	0.320	1.00	1.213	1.431	0.270	91.362	19.948	51.580
5	0.267	1.00	3.269	16.783	-0.185	91.628	26.966	51.581
6	0.242	1.00	4.808	43.297	-0.192	92.204	73.676	51.582
7	0.227	1.00	12.317	2.492	-1.797	95.984	73.831	51.662
8	0.197	1.00	-4.682	-1.764	0.881	96.531	73.909	51.681
9	0.159	1.00	2.036	0.160	30.945	96.634	73.909	75.542
10	0.147	1.00	-0.275	-0.995	0.923	96.636	73.934	75.564
11	0.131	1.00	-5.077	17.580	0.819	97.278	81.635	75.580
12	0.121	1.00	2.717	-5.041	0.418	97.462	82.268	75.585

ABUTMENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.013	0.001	0.013	0.001	0.000	0.000
	2	0.002	0.000	0.002	0.000	0.000	0.000
	3	0.014	0.001	0.014	0.001	0.000	0.000
	4	0.006	0.001	0.006	0.001	0.000	0.000
ABU 5	1	0.059	0.011	0.059	0.011	0.000	0.000
	2	0.008	0.002	0.008	0.002	0.000	0.000
	3	0.061	0.012	0.061	0.012	0.000	0.000
	4	0.026	0.005	0.026	0.005	0.000	0.000

SPAN HINGE CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
S 1 H1	1	0.045	0.001	0.063	0.001	0.023	0.000
	2	0.005	0.006	0.009	0.006	0.004	0.000
	3	0.047	0.003	0.065	0.003	0.024	0.000
	4	0.019	0.006	0.028	0.006	0.011	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

WORKED EXAMPLE - RETROFITTED

CQC COLUMN FORCES

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	1695.9	11124.	2294.7	30816.	4708.4	167.4
		2	212.3	1448.	13045.0	174563.	680.9	628.8
		3	1759.6	11559.	6208.2	83185.	4912.7	356.0
		4	721.1	4786.	13733.5	183808.	2093.4	679.0
1	TOP	1	1419.5	10278.	2282.3	308.	4704.1	167.4
		2	151.7	1084.	12838.0	664.	680.2	628.8
		3	1465.0	10603.	6133.7	507.	4908.2	356.0
		4	577.6	4167.	13522.7	756.	2091.4	679.0
BNT 3								
1	BOT	1	518.1	3936.	114.7	861.	651.8	36.1
		2	70.1	514.	458.8	3371.	744.4	111.7
		3	539.1	4090.	252.3	1873.	875.1	69.7
		4	225.5	1695.	493.2	3629.	939.9	122.6
1	TOP	1	427.3	3573.	79.8	701.	649.7	36.1
		2	54.4	471.	297.6	2740.	743.0	111.7
		3	443.6	3714.	169.1	1523.	872.6	69.7
		4	182.6	1543.	321.6	2950.	937.9	122.6
2	BOT	1	502.5	3818.	127.6	930.	759.1	36.2
		2	106.2	772.	450.8	3330.	497.9	111.7
		3	534.3	4049.	262.9	1929.	908.4	69.7
		4	256.9	1918.	489.1	3609.	725.6	122.6
2	TOP	1	414.2	3463.	92.7	833.	757.1	36.2
		2	74.0	668.	291.2	2667.	497.0	111.7
		3	436.4	3663.	180.1	1633.	906.2	69.7
		4	198.3	1707.	319.0	2917.	724.1	122.6
BNT 4								
1	BOT	1	518.5	3955.	86.3	638.	396.8	35.8
		2	71.8	535.	382.9	2837.	570.9	161.6
		3	540.0	4116.	201.1	1489.	568.1	84.3
		4	227.4	1722.	408.8	3028.	690.0	172.4
1	TOP	1	427.3	3638.	56.4	527.	396.0	35.8
		2	53.7	477.	242.8	2267.	569.9	161.6
		3	443.4	3781.	129.2	1207.	567.0	84.3
		4	181.9	1568.	259.7	2425.	688.7	172.4
2	BOT	1	515.9	3936.	83.8	624.	198.6	35.8
		2	95.4	719.	383.2	2839.	580.5	161.6
		3	544.5	4152.	198.8	1476.	372.8	84.3
		4	250.1	1900.	408.4	3026.	640.1	172.4
2	TOP	1	425.2	3620.	53.6	499.	197.6	35.8
		2	67.8	604.	243.1	2271.	579.4	161.6
		3	445.5	3801.	126.5	1180.	371.4	84.3
		4	195.4	1689.	259.1	2420.	638.7	172.4

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

WORKED EXAMPLE - RETROFITTED

ABUTMENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	5298.8	12691.7	292.5	12661.0	928.9
	2	1108.9	2448.9	2509.3	2407.5	2549.0
	3	5631.5	13426.3	1045.2	13383.3	1693.6
	4	2698.5	6256.4	2597.0	6205.8	2827.6
ABU 5	1	580.6	4499.1	2213.0	4612.6	1965.4
	2	152.3	2073.8	8645.8	658.5	8866.6
	3	626.3	5121.2	4806.7	4810.2	4625.4
	4	326.5	3423.5	9309.7	2042.3	9456.2

SPAN HINGE CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
S 1 H1	1	4409.9	9404.6	283.8	9404.5	287.1
	2	541.4	1829.6	4216.7	1829.2	4216.9
	3	4572.3	9953.5	1548.8	9953.2	1552.2
	4	1864.3	4650.9	4301.9	4650.6	4303.0

SPAN HINGE RESTRAINER CQC FORCES

RES	LC	AXIAL
SPN 1 - JNT 1		
1	1	2489.5
	2	993.6
	3	2787.5
	4	1740.5
2	1	2435.0
	2	649.9
	3	2630.0
	4	1380.4
3	1	2276.7
	2	952.1
	3	2562.3
	4	1635.1
4	1	2236.9
	2	1327.4
	3	2635.1
	4	1998.5

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Capacity/Demand Ratios at the Columns, Walls, and Footings

Bent 3 (Bottom)

$$r_{ef} \approx 2969 \div 6052 = 0.49$$
$$r_{fr} = 4(0.49) = 1.96$$

Bent 3 (Top)

$$r_{ec} \approx 5938 \div 5058 = 1.17$$

$$r_{ca} = \frac{1676}{1687}(1.17) = 1.16$$

C.5 DESIGN OF RETROFIT DETAILS

The retrofitting details are designed for the force level specified in the seismic design guidelines of Division I-A of the AASHTO Specifications. The results of the computer analysis of the retrofitted structure are used.

C.5.1 LONGITUDINAL EXPANSION JOINT RESTRAINERS (Section 5.2.1)

The design of longitudinal restrainers at hinges is based on Caltran's method, which uses the Equivalent Static Analysis method and approximates the nonlinear behavior of expansion joints. The number of required longitudinal restrainers determined is 28. Four units of 7-cable restrainers (see figure 33) were used in the analysis of retrofitted structures. Their layout along the hinge centerline is 2.29 m, 7.62 m, and 2.29 m apart. For detailed procedures of Caltran's method for the design of cable restrainers, see Worked Example Problem 3 of appendix E.

C.5.2 TRANSVERSE BEARING RESTRAINERS (Section 5.2.2)

The transverse design force at the expansion joint is given by

$$V_{eq} = 4302 \times 1.25 = 5378 \text{ kN}$$

The design capacity of the existing concrete shear keys is

$$V_c = 0.85 \times 2880 = 2448 \text{ kN}$$

Therefore, it is proposed that transverse pipe restrainers (see figure 30) be added to provide the additional capacity to carry the design load.

Required Pipe Restrainer Capacity = 5378 - 2448 = 2930 kN

A 100 mm double-extra-strong pipe restrainer has a design capacity of 778 kN 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.

C.5.3 INFILL SHEAR WALL

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 specifications of AASHTO. A 300 mm (12 in) thick infill wall is assumed.

Design Shear (Load Case 2) = 13734 ÷ 2 = 6867 kN

Design Moment (Load Case 2) = 183 808 ÷ 2 = 91 904 kN-m

The ultimate moment capacity of the pier, ignoring axial load, is given by

$$M_u = A_s f_y \left(d - \frac{a}{2} \right)$$

$$\approx \frac{54\,193}{10^6} (400 \times 10^3)(10.12)(0.9)$$

$$= 197\,436 \text{ kN-m}$$

Therefore, the moment capacity is sufficient.

The ultimate shear capacity of the pier is calculated according to the following formula:

$$v_u = 5.25\sqrt{f_c'} + \rho_h f_y$$

Design Shear Stress = 6867 ÷ (10.12)(0.305) = 2225 kPa < $21\sqrt{f_c'}$

Therefore:

$$\rho_h = \frac{2225 - 786}{400 \times 10^3} = 0.0036 > 0.0025$$

Two curtains of vertical and horizontal reinforcing consisting of No. 15 bars at 360 mm 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

$$Q \approx \pi(0.610)^2(9.60 \div 2) = 5.61 \text{ m}^3$$
$$I \approx \pi(0.610)^2(9.60 \div 2) + (0.305)(8.382)^3 \div 12$$
$$= 26.93 + 14.97 = 41.90 \text{ m}^4$$

Therefore:

$$\text{Shear Force} = \frac{6867(5.61)}{41.90} = 919 \text{ kN/m}$$

This force will be resisted by shear friction if No. 30 dowels at 300 mm on center are used to anchor the new construction to the existing structure.

APPENDIX D

WORKED EXAMPLE PROBLEM 2

D.1 INTRODUCTION

This example problem contains a detailed evaluation of a multiple simple span steel girder bridge based on the Capacity/Demand ratio method and includes two cases. Case I assumes a Seismic Performance Category B, and Case II assumes a Seismic Performance Category C with a bridge Importance Classification of "Essential."

D.2 DESCRIPTION OF THE EXAMPLE BRIDGE

The example bridge is part of the L.R. 767-5 bridge over L.R. 22019 in Dauphin County, Pennsylvania. The bridge contains several slab-on-steel-girders simple spans and several continuous spans. A plan and elevation view of the portion of the bridge used in this example is shown in figure 116. Pier 2 is evaluated in detail, and is shown in figure 117. The bridge was built in 1968. The bridge bearings are of the steel rocker type. Two 32 mm anchor bolts per bearing are used to connect the bearing shoes to the bent cap. Both piers 1 and 2 are founded on rock.

D.3 BRIDGE EVALUATION

D.3.1 ACCELERATION COEFFICIENT

Based on the Acceleration Coefficient map in Division I-A of AASHTO Specifications, the acceleration coefficient, A, for Dauphin County, Pennsylvania, is 0.1. Note that the Case II example has been run assuming an acceleration coefficient of 0.2. If the bridge had been located in an area with an actual acceleration coefficient of 0.2 or greater, the steps outlined below would still apply. For an acceleration coefficient of 0.2, the Seismic Performance Category is C, regardless of whether the bridge is "essential" or not (see table 1 of the Manual).

D.3.2 COMPONENTS FOR SEISMIC CAPACITY/DEMAND RATIO EVALUATION

Case I Example

Based on table 5 of the Manual, the components that must be evaluated are primarily the bearings (support length and forces).

Case II Example

Based on table 5, the components that must be evaluated include the bearings (support length and forces), the reinforced concrete column bents and their footings (anchorage, splices, shear, and confinement), and the potential for liquefaction.

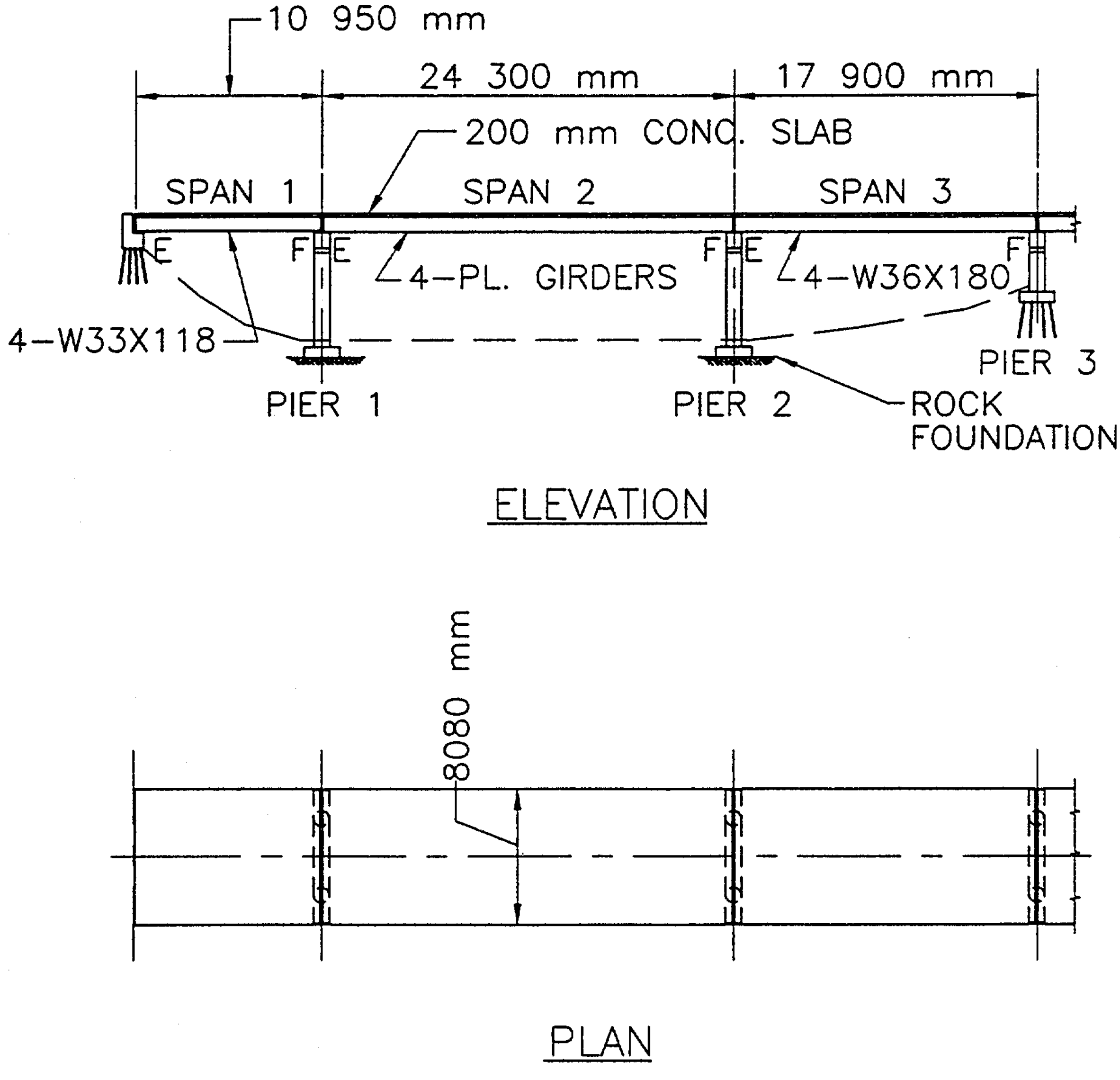


Figure 116. Elevation and plan of example bridge.

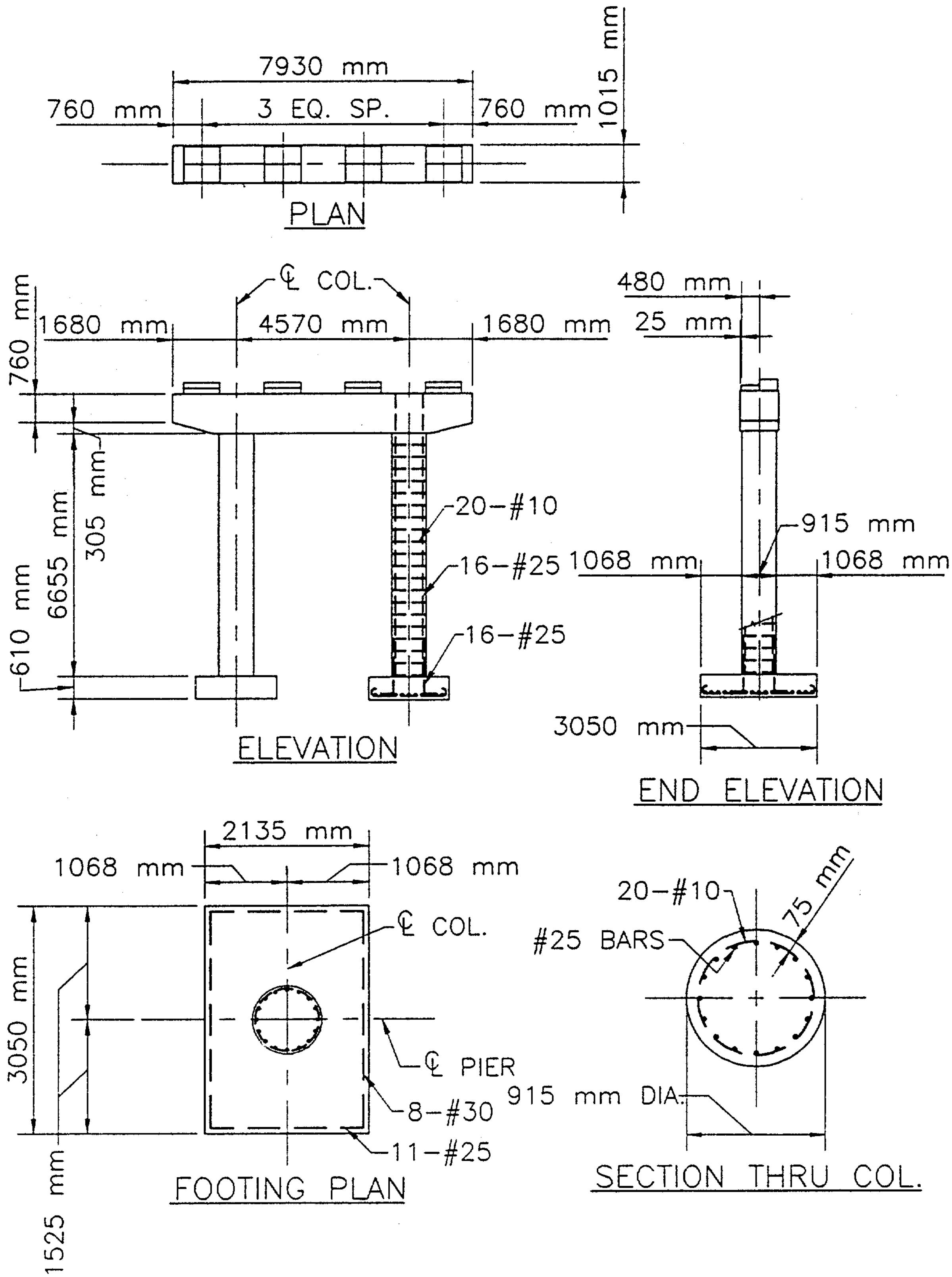


Figure 117. Pier 2 bent details.

D.3.3 ANALYSIS PROCEDURE

Case I Example

Based on tables 4(a) and 4(b) of the Manual, since the example bridge is a "regular" bridge, analysis is not required for this case.

Case II Example

Based on tables 4(a) and 4(b), since the example bridge is a "regular" bridge with more than two spans, Procedure 1, the single-mode spectral method, is required for this case.

D.3.4 SINGLE-MODE SPECTRAL ANALYSIS FOR CASE II EXAMPLE

D.3.4.1 Structural Data - Pier 2

Column diameter	D	=	915 mm
Column area	A	=	660×10^3 mm^2
Column moment of inertia	I	=	34.5×10^9 mm^4
Bent cap area	A_c	=	1100×10^3 mm^2
Bent cap moment of inertia	I_c	=	103×10^9 mm^4
Modulus of elasticity	E	=	26×10^3 MPa
Weight of span 2	W_{sp2}	=	1920 kN
Average weight of spans 2+3	$W_{sp2,3}$	=	1530 kN

D.3.4.2 Longitudinal Earthquake Loading

The expansion rocker bearings provide very little resistance to longitudinal movements and most of the longitudinal load will be transferred to the fixed end of the span. The simple span 2 will most likely move as a rigid body and its movement will be resisted by the longitudinal stiffness of pier 2. The weight distribution along the span is uniform and, if the response of each simple span is assumed to be independent of the other spans, the natural period in the longitudinal direction may be computed as follows:

<u>Longitudinal Stiffness of Bent 2 Acting as a Cantilever</u>:

$$K_L = (2)\,\frac{3EI}{H_L^3} = (2)\,\frac{(3)(26\times10^3\,\text{MPa})(34.5\times10^9\,\text{mm}^4)}{(7700\,\text{mm})^3} = 11.8\,\text{kN/mm}$$

Note: uncracked section properties were used due to an anticipated overdesign.

Natural Period of Pier 2 in the Longitudinal Direction:

$$T_L = 2\pi\sqrt{\frac{W_L}{K_L g}} = 2\pi\sqrt{\frac{1920\,\text{kN}}{(11.8\,\text{kN/mm})(9810\,\text{mm/s}^2)}} = 0.81\,\text{s}$$

Elastic Seismic Response Coefficient:

$$C_S = \frac{1.2SA}{T_L^{2/3}} = \frac{(1.2)(1.0)\ A}{(0.81)^{2/3}} = 1.4\ A\ (\leq 2.5A)$$

S = 1.0 (Rock Foundation)

Thus,
$$C_S = 0.14 \text{ for } A = 0.1$$
$$C_S = 0.28 \text{ for } A = 0.2$$

D.3.4.3 Transverse Earthquake Loading

Since the superstructure is made of several simple spans sitting on rocker-type bearings, the in-plane continuity effect of the spans is neglected (note: including the in-plane stiffening effect of the spans would increase C_S over 2.5A without affecting the results shown below). Thus, the natural period of pier 2 in the transverse direction is computed independently, based on the stiffness of the bent and the tributary mass of the superstructure from spans 2 and 3.

Transverse Stiffness of Bent 2 Acting Along:

The transverse stiffness of bent 2 was obtained from a plane frame model of analysis by computing the horizontal displacement of the cap due to a unit horizontal load and then taking its inverse.

$$K_T = 54\ \text{kN/mm}$$

The transverse stiffness of the bent may also be approximated by assuming a rigid cap and fixed end conditions:

$$K_T' = (2)\frac{12EI}{H_T^3} = (2)\frac{(12)(26\text{x}10^3\,\text{MPa})(34.5\text{x}10^9\,\text{mm}^4)}{(7200\,\text{mm})^3} = 58\ \text{kN/mm}$$

which yields a value about 9 percent higher.

Natural Period of Pier 2 in the Transverse Direction:

$$T_T = 2\pi\sqrt{\frac{W_T}{K_T g}} = 2\pi\sqrt{\frac{1.53\,\text{kN}}{(54\,\text{kN/mm})(9810\,\text{mm/s}^2)}} = 0.34\,\text{s}$$

Elastic Seismic Response Coefficient:

$$C_S = \frac{1.2SA}{T_T^{2/3}} = \frac{(1.2)(1.0)A}{(0.34)^{2/3}} = 2.5A$$

Thus, $C_S = 0.25$ for $A = 0.1$
$C_S = 0.50$ for $A = 0.2$

D.4. CAPACITY/DEMAND RATIO FOR BEARINGS FOR CASE I EXAMPLE

The rocker-type bearings are very vulnerable to earthquakes.

D.4.1 DISPLACEMENT CAPACITY/DEMAND RATIO

The available support length is:

$$N(c) = 430\text{ mm}$$

The minimum support length for SPC B is:

$$N(d) = 200 + (1.67)(24.3\text{ m}) + (6.66)(7.6\text{ m}) = 297\text{ mm}$$

The displacement capacity/demand ratio is:

$$r_{bd} = \frac{N(c)}{N(d)} = \frac{430}{297} = 1.45$$

D.4.2 FORCE CAPACITY/DEMAND RATIO

Bearing failure may be caused by shearing of the anchor bolts, sliding along the rotation pin in the transverse direction, or shifting over the rotation pin in the longitudinal direction.

Shearing of Anchor Bolts

The shear capacity of the anchor bolts is:

$$V_b(c) = (2)(4)(800\text{ mm}^2)(186\text{ MPa}) = 1200\text{ kN}$$

The minimum bearing force demand is 20 percent of the dead load:

$V_b(d) = (0.20)(1{,}920 \text{ kN}) = 384 \text{ kN}$

The anchor bolts C/D ratio is:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} = \frac{1200}{384} = 3.1$$

<u>Sliding of Bearing Along the Rotation Pin</u>

If a coefficient of friction of 0.2 is assumed, a minimum bearing C/D ratio of 1.0 is obtained.

Thus, the governing C/D ratio for the Case I example is 1.0.

D.5 CAPACITY/DEMAND RATIO FOR BEARINGS FOR CASE II EXAMPLE

The rocker-type bearings are vulnerable to earthquakes and they may suffer damage even during a moderate earthquake.

D.5.1 DISPLACEMENT CAPACITY/DEMAND RATIO

<u>Method 1</u>

The available support length is:

$N(c) = 430 \text{ mm}$

(This is the length from the end of the girder to the edge of the cap.)

The minimum support length for SPC C is:

$N(d) = 300 + (2.5)(24.3 \text{ m}) + (10)(7.6 \text{ m}) = 435 \text{ mm}$

The displacement C/D ratio is:

$$r_{bd} = \frac{N(c)}{N(d)} = \frac{430}{435} \approx 1.0$$

<u>Method 2</u>

The available capacity of the expansion bearing for movement is:

$\Delta_s(c) = 430 \text{ mm}$

The maximum possible movement resulting from temperature changes is:

$$\Delta_i(d) = (11.7 \times 10^{-6})(42°C)(24\ 300 \text{ mm}) = 1 \text{ mm}$$

The maximum calculated relative displacement in the longitudinal direction for A = 0.2 is:

$$\Delta_{eq}(d) \approx (2)(0.28)(1920 \text{ kN})/(11.8 \text{ kN/mm}) = 100 \text{ mm}$$

(This conservatively assumes full out-of-phase motion at the two adjacent piers, i.e., $\Delta_{eq}(d)$ has been increased by a factor of 2.)

The displacement C/D ratio is:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)} = \frac{430 - 13}{100} = 4.1 > 1.0$$

Thus, the governing displacement C/D ratio is 1.0.

D.5.2 FORCE CAPACITY/DEMAND RATIO

Bearing failure may be caused by shearing of the anchor bolts, sliding along the rotation pin in the transverse direction, or shifting over the rotation pin in the longitudinal direction.

Shearing of Anchor Bolts

$$V_b(c) = (800 \text{ mm}^2)(186 \text{ MPa}) = 150 \text{ kN}$$

The minimum bearing force demand per anchor bolt is:

$$V_b(d)_{min} = (0.20)(1920 \text{ kN})/8 = 48 \text{ kN}$$

The longitudinal seismic force demand per anchor bolt for A = 0.2 is:

$$V_b(d)_\ell = (1.25)(0.28)(1920 \text{ kN})/8 = 84 \text{ kN}$$

The transverse seismic force demand per anchor bolt for A = 0.2 is:

$$V_b(d)_t = (1.25)(0.50)(1530 \text{ kN})/16 = 60 \text{ kN}$$

The combined seismic force demand per anchor bolt for A = 0.2 is:

$$V_b(d)_c = \sqrt{(84)^2 + [(0.3)(60)]^2} = 86 \text{ kN}$$

The anchor bolt C/D ratio for A = 0.1 is:

$$r_{bf} = \frac{V_b(c)}{V_b(d)_{min}} = \frac{150}{48} = 3.1$$

The anchor bolt C/D ratio for A = 0.2 is:

$$r_{bf} = \frac{V_b(c)}{V_b(d)_c} = \frac{150}{86} = 1.7$$

Sliding of Bearing Along the Rotation Pin

If a coefficient of friction of 0.2 is assumed, the C/D ratio of the bearing for A = 0.1 is given by:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} = \frac{0.2}{0.25} = 0.8$$

and the C/D ratio of the bearing for A = 0.2 is:

$$r_{bf} = \frac{V_b(c)}{V_b(d)} = \frac{0.2}{0.5} = 0.4$$

Thus, the governing force C/D ratio is 0.8 for A = 0.1, and 0.4 for A = 0.2.

D.6 CAPACITY/DEMAND RATIO FOR BENT 2

The C/D ratio computations for bent 2 are shown in the following pages. The computations include derivation of the column interaction surface, the footing interaction surface in the transverse and the longitudinal direction (see figures 118, 119, and 120), plastic hinge analysis for the transverse direction, seismic elastic load analysis, ultimate moment capacity calculations based on seismic axial loads, and a C/D ratio evaluation for the bent.

The plastic hinge analysis in the transverse direction shows that the capacity of the bent is limited by uplift of the column footing sitting on rock foundation before any hinge mechanism can form. Low axial compression load in a column can significantly reduce the C/D ratio at the base of the footing since the resistance to rotation provided by the foundation rock is dependent on the existence of compression stresses at the interface.

In the longitudinal direction, the acceleration coefficient of 0.1 yields C/D ratios of more than 1.0. If an acceleration coefficient of 0.2 is assumed, C/D ratios lower than 1.0 are obtained. The limiting factor is also the resistance to rotation provided by the foundation rock. This condition is caused by the relatively large moment arm-to-foundation width ratio. Thus, for an acceleration coefficient of 0.2, the assumption of column base fixity used in the computation of the natural period is not adequate. A partial column base fixity assumption would be more appropriate for both the longitudinal and the transverse directions. This would increase the natural periods, lower the seismic demand, and further increase the column C/D ratios. For all cases, the seismic elastic loads are relatively low. Due to the limited capacity for uplift and rotation provided by the foundation rock, flexural yielding of the columns is not expected. Thus, the assumption of uncracked column section properties used is applicable in this case. In addition, only the C/D ratios for anchorage, splices of longitudinal reinforcement, and column shear need to be evaluated (see figure 75 of the manual). The anchorage of the reinforcing bars is satisfactory. The minimum splice length for the longitudinal reinforcement is provided; however, the C/D ratio for the splices is lower than 0.75 r_{ec} and therefore a value of 0.75 r_{ec} is applicable. Also, since the columns do not yield, the C/D ratio for column shear is given by the ratio of initial shear resistance of the undamaged column to the maximum calculated elastic shear force, which is greater than 1.0.

D.7 POTENTIAL FOR LIQUEFACTION

Since bent 2 is founded on rock, there is no potential for liquefaction.

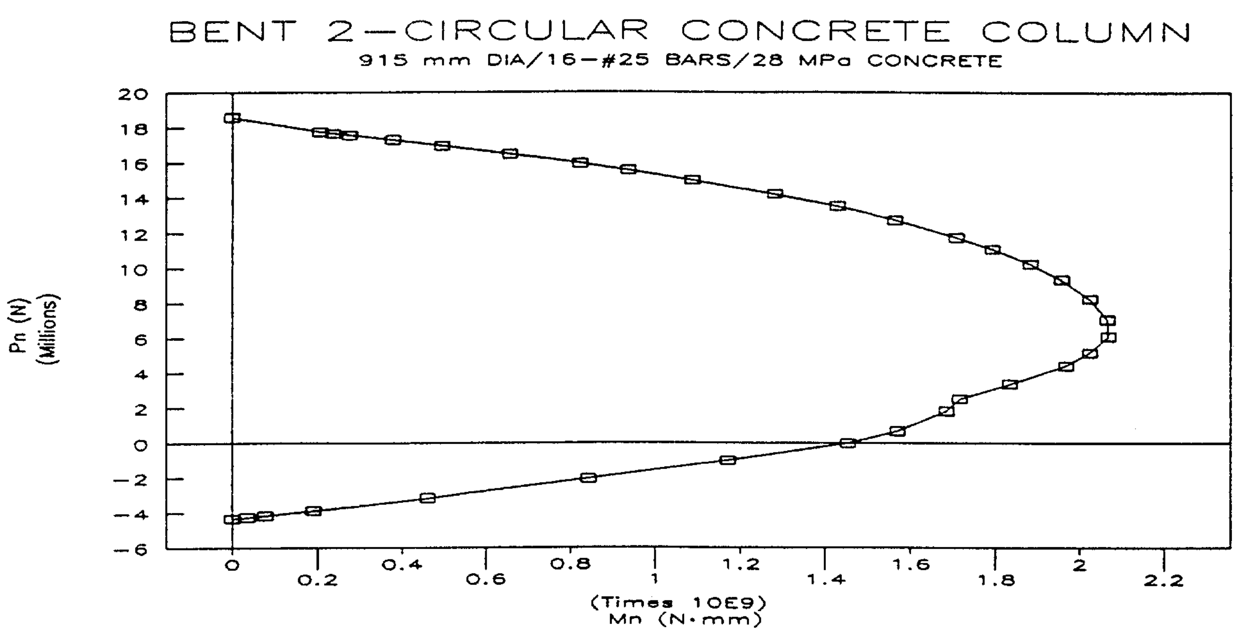


Figure 118. Column interaction diagram.

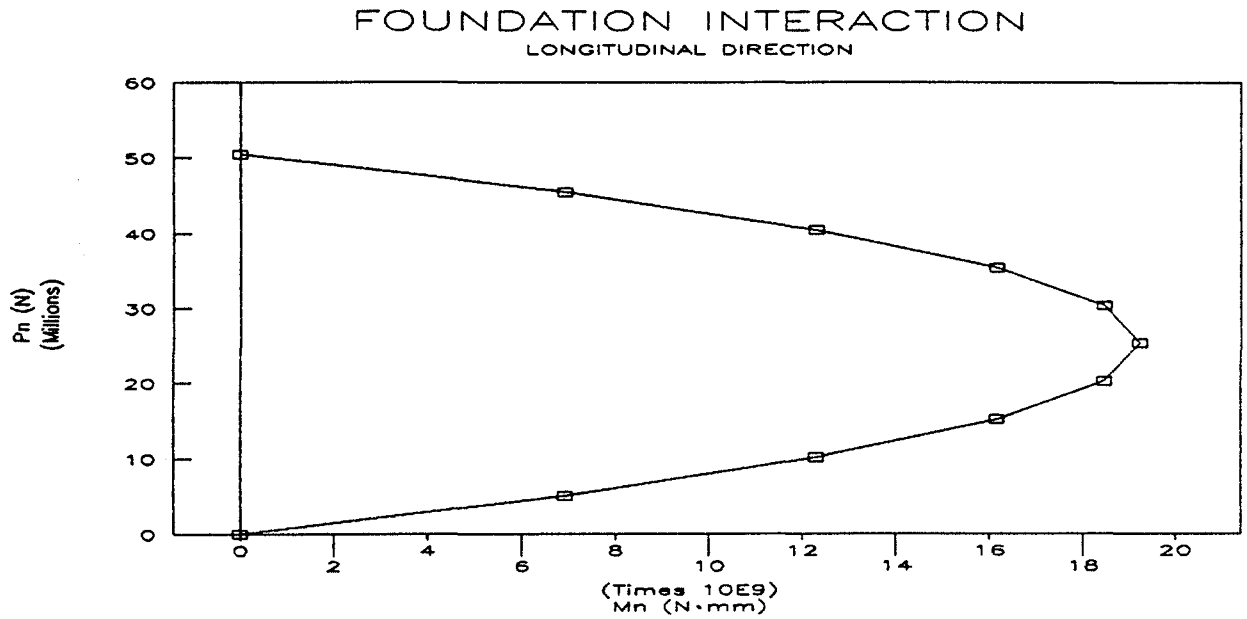


Figure 119. Foundation interaction diagram.

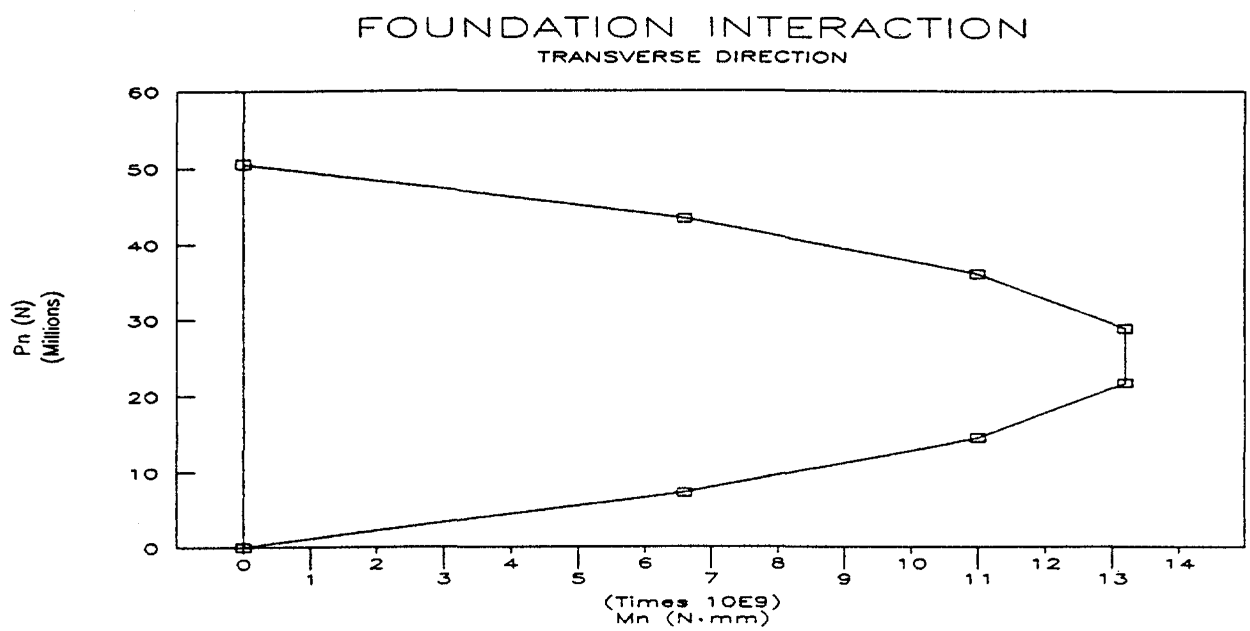


Figure 120. Foundation interaction diagram—transverse direction.

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

DEAD LOAD FORCES AND BENT GEOMETRY

DEAD LOAD AXIAL FORCES

@ TOP OF COLUMN:	866 x 10^3 N
@ BOTTOM OF COLUMN:	978 x 10^3 N
@ BASE OF FOOTING	1085 x 10^3 N

BENT GEOMETRY

COLUMN HEIGHT	6655 mm
COLUMN SPACING	4570 mm
COLUMN TOP TO C.G. OF SUPERSTRUCTURE	3050 mm
FOOTING DEPTH	610 mm

INTERACTION DIAGRAMS FOR THE COLUMN AND THE FOOTING

COLUMN INTERACTION DIAGRAM

Pn (N * 10^3)	Mn (N.mm * 10^3)
18584	0
17755	205531
17669	237959
17568	276372
17302	376154
16973	495056
16499	655684
15968	821891
15580	934981
15010	1088461
14198	1281265
13486	1428294
12730	1563952
11774	1708812
11108	1793233
10256	1883542
9379	1956146
8266	2023423
7088	2065866
6114	2068055
5190	2023481
4437	1967479
3385	1835830
2502	1717142
1802	1685934
681	1571139
0	1454484
-1002	1171526
-2045	842134
-3164	461098
-3892	191738
-4164	77660
-4261	35062
-4339	0

FOOTING INTERACTION DIAGRAM

TRANSVERSE DIRECTION

Pn (N * 10^3)	Mn (N.mm * 10^3)
0	0
7209	6590160
14418	10983600
21627	13180320
28836	13180320
36045	10983600
43254	6590160
50463	0

LONGITUDINAL DIRECTION

Pn (N * 10^3)	Mn (N.mm * 10^3)
0	0
5046	6919668
10093	12301632
15139	16145892
20185	18452448
25232	19221300
30278	18452448
35324	16145892
40370	12301632
45417	6919668
50463	0

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

PLASTIC HINGE ANALYSIS FOR THE TRANSVERSE DIRECTION

OVERSTRENGTH MOMENT CAPACITY

1. OVERSTRENGTH MOMENT CAPACITY AT DEAD LOAD AXIAL LOADS

	DL AXIAL LOAD (N * 10^3)	Mn (N.mm * 10^3)	1.3*Mn (N.mm * 10^3)	
TOP OF COLUMN	866	1590083	2067108	HINGE
BOTTOM OF COLUMN	978	1601524	2081982	
BASE OF FOOTING	1085	1145422	1489049	HINGE

2. TOTAL COLUMN SHEAR FORCE

Vu : 978 x 10^3 N
Vu : 768 x 10^3 N <------ADJUSTED TO AVOID FOOTING UPLIFT, ASSUMING NO TENSION CAPACITY BETWEEN FOOTING AND ROCK.

3. COLUMN AXIAL FORCE DUE TO OVERTURNING

AXIAL FORCE : 1556 x 10^3 N > 1085 N * 10^3 : DEAD LOAD AT BASE OF FOOTING
AXIAL FORCE : 1082 x 10^3 N <------ADJUSTED TO AVOID FOOTING UPLIFT

REVISED OVERSTRENGTH MOMENT CAPACITY

1. OVERSTRENGTH MOMENT CAPACITY AT DEAD LOAD AXIAL LOADS PLUS OVERTURNING

	DL +/- OVERTURN (N * 10^3)	Mn (N.mm * 10^3)	1.3*Mn (N.mm * 10^3)	
TOP OF COLUMN (+)	1948	1692459	2200197	HINGE
TOP OF COLUMN (-)	-216	1393589	1811666	HINGE
BOTTOM OF COLUMN (+)	2060	1697441	2206674	HINGE
BOTTOM OF COLUMN (-)	-104	1425139	1852680	
BASE OF FOOTING (+)	2167	2211811	2875354	
BASE OF FOOTING (-)	0	0	0	HINGE

2. TOTAL COLUMN SHEAR FORCE

Vu : 933 x 10^3 N WITHIN 10% OF PREVIOUS Vu

3. AXIAL FORCE DUE TO OVERTURNING

AXIAL FORCE : 1500 x 10^3 N > 1085 x 10^3 N : DEAD LOAD AT BASE OF FOOTING FOOTING UPLIFT OCCURS FIRST.

NOTE: FOOTING UPLIFT GOVERNS AND THERE IS NO HINGE MECHANISM.

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

CALCULATED SEISMIC AXIAL LOADS

TRANSVERSE DIRECTION

FOR A=0.1

TOP OF COLUMN (+)	500 x 10^3 N
TOP OF COLUMN (-)	-500 x 10^3 N
BOTTOM OF COLUMN (+)	500 x 10^3 N
BOTTOM OF COLUMN (-)	-500 x 10^3 N
BASE OF FOOTING (+)	500 x 10^3 N
BASE OF FOOTING (-)	-500 x 10^3 N

FOR A=0.2

TOP OF COLUMN (+)	1000 x 10^3 N
TOP OF COLUMN (-)	-1000 x 10^3 N
BOTTOM OF COLUMN (+)	1000 x 10^3 N
BOTTOM OF COLUMN (-)	-1000 x 10^3 N
BASE OF FOOTING (+)	1000 x 10^3 N
BASE OF FOOTING (-)	-1000 x 10^3 N

LONGITUDINAL DIRECTION

FOR A=0.1 AND A=0.2

TOP OF COLUMN (+)	0 x 10^3 N
TOP OF COLUMN (-)	0 x 10^3 N
BOTTOM OF COLUMN (+)	0 x 10^3 N
BOTTOM OF COLUMN (-)	0 x 10^3 N
BASE OF FOOTING (+)	0 x 10^3 N
BASE OF FOOTING (-)	0 x 10^3 N

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

ULTIMATE MOMENT CAPACITY AT DEAD LOAD AXIAL FORCES PLUS THE SEISMIC AXIAL LOADS

TRANSVERSE DIRECTION

FOR A=0.1

	DL +/- EQ (N * 10^3)	Mn (N.mm * 10^3)
TOP OF COLUMN (+)	1366	1641297
TOP OF COLUMN (-)	366	1517213
BOTTOM OF COLUMN (+)	1478	1652739
BOTTOM OF COLUMN (-)	478	1536333
BASE OF FOOTING (+)	1585	1637202
BASE OF FOOTING (-)	585	616630

FOR A=0.2

	DL +/- EQ (N * 10^3)	Mn (N.mm * 10^3)
TOP OF COLUMN (+)	1866	1688798
TOP OF COLUMN (-)	-134	1416776
BOTTOM OF COLUMN (+)	1978	1693780
BOTTOM OF COLUMN (-)	-22	1448325
BASE OF FOOTING (+)	2085	2131639
BASE OF FOOTING (-)	85	90496

LONGITUDINAL DIRECTION

FOR A=0.1 AND A=0.2

	DL AXIAL FORCE (N * 10^3)	Mn (N.mm * 10^3)
TOP OF COLUMN (+)	866	1590083
TOP OF COLUMN (-)	866	1590083
BOTTOM OF COLUMN (+)	978	1601524
BOTTOM OF COLUMN (-)	978	1601524
BASE OF FOOTING (+)	1085	1617427
BASE OF FOOTING (-)	1085	1617427

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

SUMMARY OF MAXIMUM ELASTIC MOMENT DEMANDS

A=0.1

LOADING	LOCATION	COMPONENT	LONG. MOMENT (N * 10^3) EQ	DL	TRANSVERSE MOMENT (N.mm * 10^3) EQ	DL	TOTAL MOMENT (N.mm * 10^3) DEMAND
CASE 1	TOP	COLUMN	1110	0	198478	197323	520736
	BOTTOM	COLUMN	4291	0	214343	98485	1344385
	BASE	FOOTING	4560	0	249328	123536	1438678
CASE 2	TOP	COLUMN	333	0	661592	197323	864894
	BOTTOM	COLUMN	1287	0	714476	98485	902642
	BASE	FOOTING	1368	0	831092	123536	1041674

A=0.2

LOADING	LOCATION	COMPONENT	LONG. MOMENT (N * 10^3) EQ	DL	TRANSVERSE MOMENT (N.mm * 10^3) EQ	DL	TOTAL MOMENT (N.mm * 10^3) DEMAND
CASE 1	TOP	COLUMN	2221	0	396955	197323	900665
	BOTTOM	COLUMN	8582	0	428686	98485	2667574
	BASE	FOOTING	9120	0	498655	123536	2847839
CASE 2	TOP	COLUMN	666	0	1323185	197323	1534004
	BOTTOM	COLUMN	2574	0	1428953	98485	1717117
	BASE	FOOTING	2736	0	1662185	123536	1970755

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

ULTIMATE MOMENT CAPACITY/ELASTIC MOMENT DEMAND RATIOS

A=0.1

LONGITUDINAL DIRECTION 100% + TRANSVERSE DIRECTION 30% (N.mm * 10^3)

LOAD CASE	LOCATION	ELASTIC DEMAND	NOMINAL CAPACITY	CAPACITY/DEMAND RATIO
1	TOP OF COLUMN (+)	520736	1590083	3.05 rec
1	TOP OF COLUMN (-)	520736	1590083	3.05 rec
1	BOTTOM OF COLUMN (+)	1344385	1601524	1.19 rec
1	BOTTOM OF COLUMN (-)	1344385	1601524	1.19 rec
1	BASE OF FOOTING (+)	1438678	1617427	1.12 ref
1	BASE OF FOOTING (-)	1438678	1617427	1.12 ref

TRANSVERSE DIRECTION 100% + LONGITUDINAL DIRECTION 30% (N.mm * 10^3)

LOAD CASE	LOCATION	ELASTIC DEMAND	NOMINAL CAPACITY	CAPACITY/DEMAND RATIO
2	TOP OF COLUMN (+)	864894	1641297	1.90 rec
2	TOP OF COLUMN (-)	864894	1517213	1.75 rec
2	BOTTOM OF COLUMN (+)	902642	1652739	1.83 rec
2	BOTTOM OF COLUMN (-)	902642	1536333	1.70 rec
2	BASE OF FOOTING (+)	1041674	1637202	1.57 ref
2	BASE OF FOOTING (-)	1041674	616630	0.59 ref.

A=0.2

LONGITUDINAL DIRECTION 100% + TRANSVERSE DIRECTION 30% (N.mm * 10^3)

LOAD CASE	LOCATION	ELASTIC DEMAND	NOMINAL CAPACITY	CAPACITY/DEMAND RATIO
1	TOP OF COLUMN (+)	900669	1590083	1.77 rec
1	TOP OF COLUMN (-)	900669	1590083	1.77 rec
1	BOTTOM OF COLUMN (+)	2667577	1601524	0.60 rec
1	BOTTOM OF COLUMN (-)	2667577	1601524	0.60 rec
1	BASE OF FOOTING (+)	2847844	1617427	0.57 ref
1	BASE OF FOOTING (-)	2847844	1617427	0.57 ref

TRANSVERSE DIRECTION 100% + LONGITUDINAL DIRECTION 30% (N.mm * 10^3)

LOAD CASE	LOCATION	ELASTIC DEMAND	NOMINAL CAPACITY	CAPACITY/DEMAND RATIO
2	TOP OF COLUMN (+)	1534002	1688798	1.10 rec
2	TOP OF COLUMN (-)	1534002	1416776	0.92 rec
2	BOTTOM OF COLUMN (+)	1717116	1693780	0.99 rec
2	BOTTOM OF COLUMN (-)	1717116	1448325	0.84 rec
2	BASE OF FOOTING (+)	1970756	2131639	1.08 ref
2	BASE OF FOOTING (-)	1970756	90496	0.05 ref

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

CAPACITY/DEMAND RATIO OF ANCHORAGE REINFORCEMENT

AT THE BOTTOM OF THE COLUMN

#25 BAR - 90 DEGREE HOOK

Km:	0.7	
db:	25 mm	
fc':	28 MPa	
fy:	414 MPa	
la(c):	502 mm	
la(d):	337 mm	
la(d):	381 mm	
la(d):	381 mm	USED

502 mm > 381 mm
la(c) > la(d)

CASE B APPLIES

NO TOP REINFORCING BARS AT TOP OF FOOTING
AND HOOK OUTWARDS

DETAIL 2 APPLIES

rca = 1.3 ref <= 1.0

AT THE TOP OF THE COLUMN

#25 BAR - STRAIGHT

Ks:	10208.33	
Ktr:	0	
db:	25 mm	
fc':	28 MPa	
fy:	414 MPa	
C:	76 mm	
la(c):	914 mm	
la(d):	482 mm	
la(d):	762 mm	
la(d):	762 mm	USED

914 mm > 762 mm
la(c) > la(d)

CASE B APPLIES

DETAIL 6 APPLIES

rca = 1.0

WORKED EXAMPLE PROBLEM 2, CASE II
CAPACITY/DEMAND RATIO CALCULATION WORKSHEET FOR BENT 2

CAPACITY/DEMAND RATIO FOR COLUMN SHEAR

SINCE THE COLUMNS DO NOT YIELD, THE CAPACITY/DEMAND RATIO CAN BE CALCULATED AS FOLLOWS:

1. ELASTIC SHEAR DEMAND

	LONG	TRANS	CASE 1	CASE 2	
A=0.1	135	191	146	196	(N * 10^3)
A=0.2	269	383	293	391	(N * 10^3)

Ve(d) : 196 N * 10^3 A=0.1
Ve(d) : 391 N * 10^3 A=0.2

2. INITIAL SHEAR CAPACITY

fc'=	28 MPa
fy=	414 MPa
Ag=	656708 mm^2
d=	737 mm
S=	305 mm
Atr=	258 mm^2

Vi(c) : 831 N * 10^3

3. CAPACITY/DEMAND RATIO FOR COLUMN SHEAR

rcv : 4.25 A=0.1
rcv : 2.12 A=0.2

CAPACITY/DEMAND RATIO FOR SPLICE

s=	305 mm
ls=	838 mm
fc'=	28 MPa
fy=	414 MPa
fyt=	276 MPa
db=	25 mm
Ab=	510 mm^2

Atr(d)= 278 mm^2

SINCE THE CLEAR SPACING BETWEEN SPLICES 114 mm > 4db : 102 mm:
Atr(c)= 129 mm^2

Atr(c) < Atr(d), CASE A APPLIES.

rcs= 0.00 rec <= 0.46 rec

SINCE ls=838 mm IS GREATER THAN MINIMUM SPLICE LENGTH 763 mm:

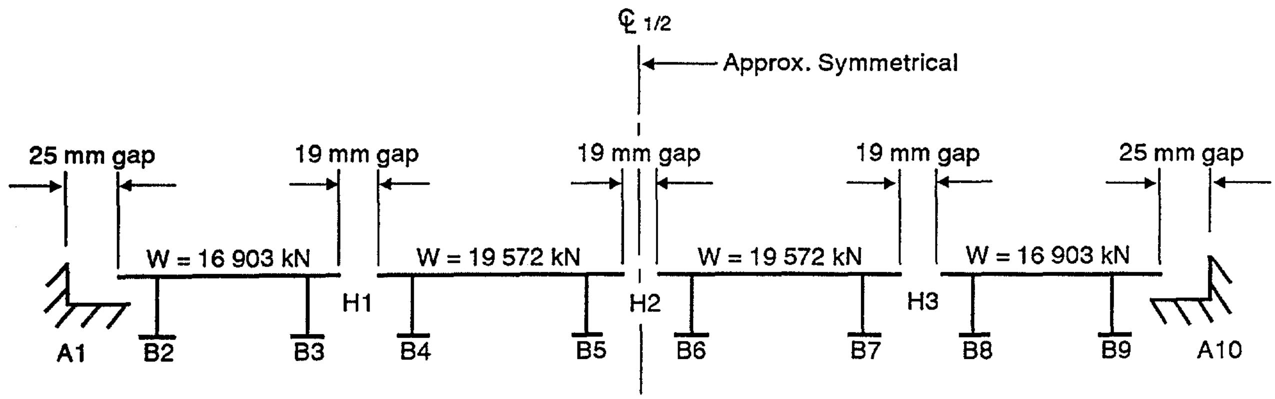
rcs= 0.75 rec CONTROLS

APPENDIX E

CABLE RESTRAINER EXAMPLES

These two examples illustrate the application of the Caltrans method for the design of cable restrainers described in section 5.2.4.1. The source of this material is the Caltrans' Bridge Design Aid Number 14, pages 18 through 25 inclusive, dated October, 1989.

Example 1 — 3 Hinged Retrofit



Seismic data: A = 0.6g, 3 m to 24 m alluvium

Hinge data: Seat width = 152 mm, diaphragms = 172 mm thick
19 mm gap in hinge - no material in hinge
Hinge has steel angles - allow 76 mm minimum seat

Abutment data: 12.19 m wide x 3.05 m high
25 mm gap - no material in joint

Columns: All columns 7.32 m long, longitudinal I = 0.276 m^4. Assume column bottoms to be poorly detailed with inadequate lap splices and lightly reinforced footings. Assume the bottom of all columns have failed (i.e., pinned) and that 50 percent of all column tops have failed.

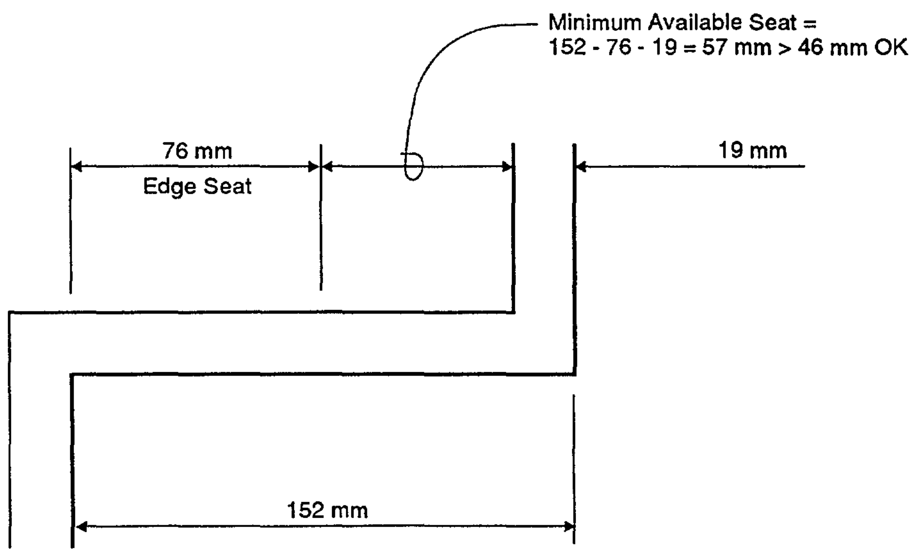
Restrainers: Minimum length = 762 mm thru diaphragm
610 mm thru bolsters
1372 mm minimum

Try 1.524 m cables - Restrainer C-1 - 19 mm cables. Leave 19 mm gap for thermal expansion.

Step 1a - Compute Maximum Restrainer Deflection, D_r.

$$D_r = D_y + D_g = \frac{1.21 \times 1.524 \times 1000}{69.57} + 19$$
$$= 27 + 19 = 46 \text{ mm for } 1.524 \text{ m cables with } 19 \text{ mm gap}$$

Step 1b - Check Seat Width.



Seat will allow up to 57 mm of movement or 57 - 19 = 38 mm cable movement (with 19 mm gap).

$$\text{Maximum cable} = \frac{38(69.57)}{1.21(1000)} = 2.18 \text{ m with } 19 \text{ mm gap}.$$

Restrainer Summary:

Length (m)	D_y (mm)	D_g (mm)	D_r (mm)
1.52	27	19	46
2.18	38	19	57

Step 2 - Compute Unrestrained Longitudinal Earthquake Deflection.

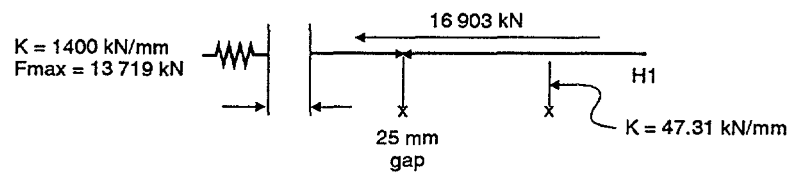
$$\text{Stiffness of one superstructure unit} = 3EI/L^3 = \frac{3(22.41 \times 10^6)(0.276)}{(7.32)^3 \times 1000} = 47.31\,\frac{kN}{mm}/\text{unit}$$

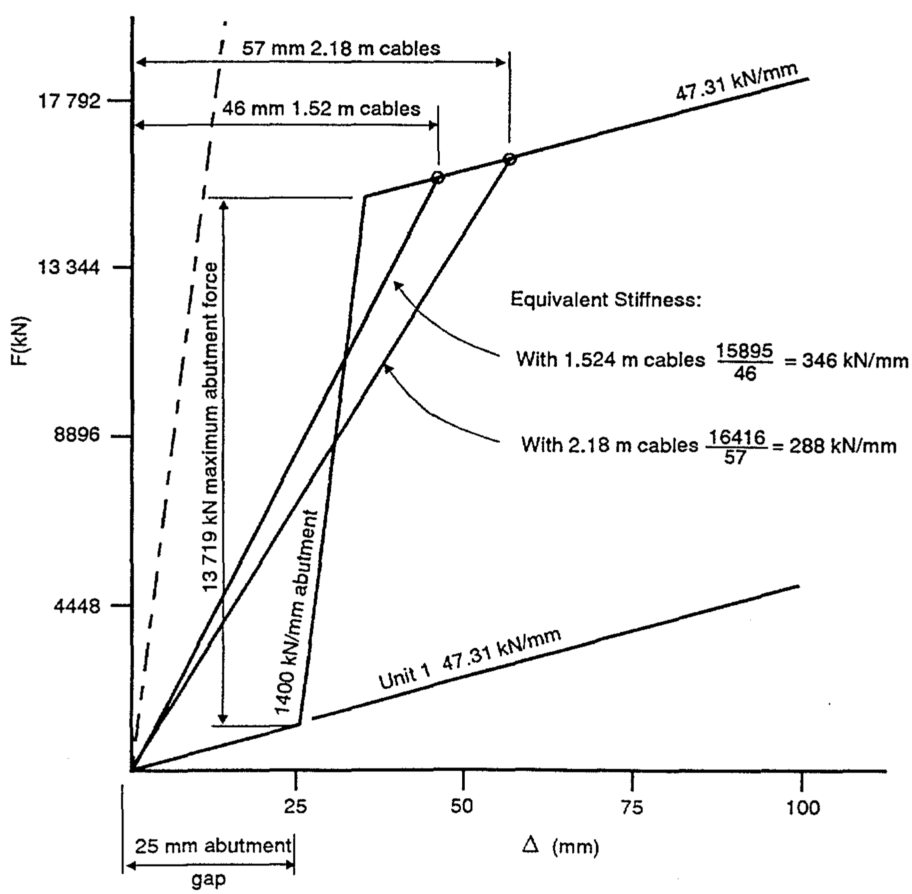
Abutment stiffness = 114.91(12.19) = 1400 kN/mm

Maximum abutment force = 369(12.19 x 3.05) = 13719 kN

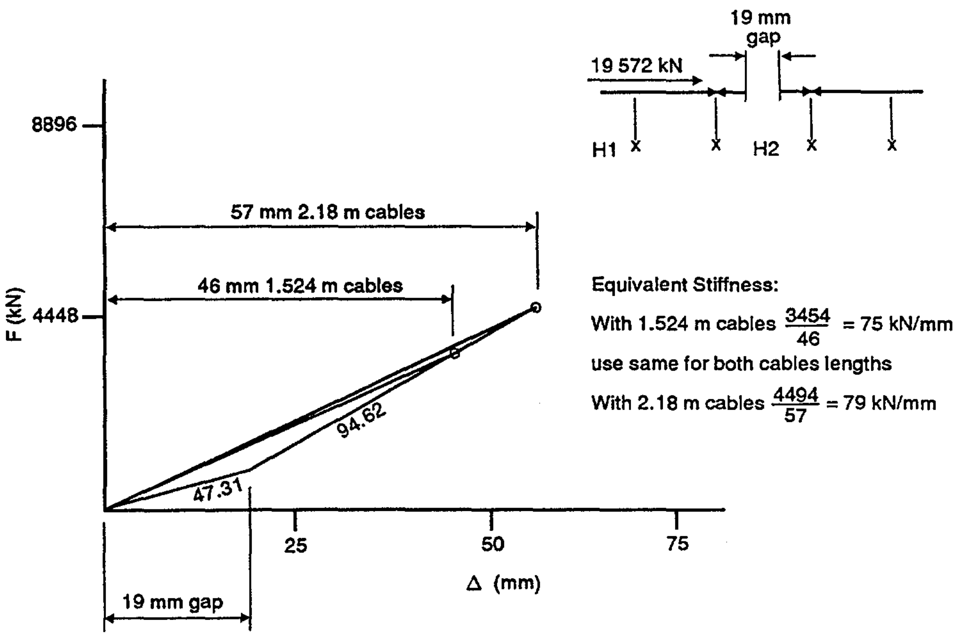
Step 2a - Evaluate System Stiffness K_u.

Unit 1 moving away from hinge 1 - mobilize abutment:

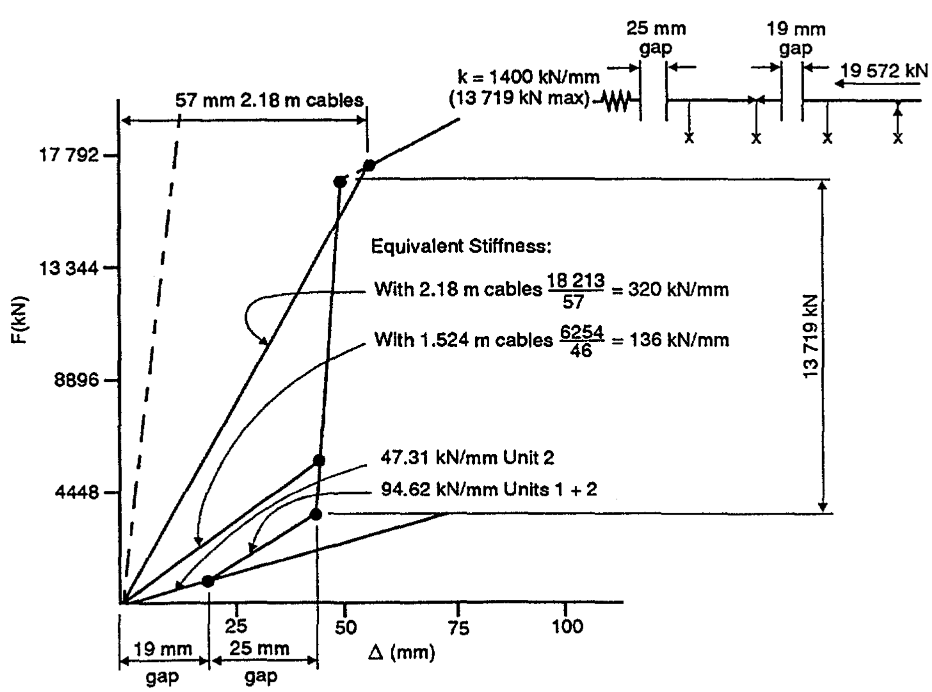




Unit 2 moving away from hinge 1 - mobilize unit 3:



Unit 2 (or 3) moving away from hinge 2 - mobilize unit 1 + abutment:



Step 2b - Compute Maximum Unrestrained Seismic Deflections.

Unit 1 moving away from hinge 1:

$W = 16903$ kN
$K_u = 346$ kN/mm with 1.524 m cables
$= 288$ kN/mm with 2.18 m cables

$$T = 0.063\sqrt{\frac{W}{K_u}} = 0.063\sqrt{\frac{16903}{346}} = 0.45 \text{ sec with } 1.524 \text{ m cables}$$

$$= 0.063\sqrt{\frac{16903}{288}} = 0.49 \text{ sec with } 2.18 \text{ m cables}$$

ARS (from curves) = 1.7g with 1.524 m cables
= 1.65g with 2.18 m cables

$$D_{eq} = \frac{ARS(W)}{K_u} = \frac{1.7(16903)}{346} = 83 \text{ mm } (1.524 \text{ m cables})$$

$$= \frac{1.65(16903)}{288} = 97 \text{ mm } (2.18 \text{ m cables})$$

Unit 2 moving away from hinge 1:

$W = 19572$ kN
$K_u = 75$ kN/mm (for both 1.524 m and 2.18 m cables)

$$T = 0.063\sqrt{\frac{W}{K_u}} = 0.063\sqrt{\frac{19572}{75}} = 1.02 \text{ sec; ARS } = 0.93g$$

$$D_{eq} = \frac{ARS(W)}{K_u} = \frac{0.93(19572)}{75} = 243 \text{ mm}$$

Larger than unit 1 moving away from H1 — does not govern.

Unit 2 moving away from hinge 2:

$W = 19572$ kN
$K_u = 136$ kN/mm with 1.524 m cables
$= 320$ kN/mm with 2.18 m cables

$$T = 0.063\sqrt{\frac{W}{K_u}} = 0.063\sqrt{\frac{19572}{136}} = 0.75\,\text{sec}$$

$$= 0.063\sqrt{\frac{19572}{320}} = 0.50\,\text{sec}$$

ARS = 1.20g (with 1.524 m cables) and ARS = 1.7g (with 2.18 m cables) respectively.

$$D_{eq} = \frac{ARS(W)}{K_u} = \frac{1.20(19572)}{136} = 173\text{ mm (1.524 m cables)}$$

$$= \frac{1.7(19572)}{320} = 104\text{ mm (2.18 m cables)}$$

Step 3 - Compare Deflections.

Hinge	L (m)	D_{eq}	D_r	D_{eq}-D_r
1	1.524	83	46	37
1	2.18	97	57	40
2	1.524	173	46	127
2	2.18	104	57	47

Step 4 - Determine Number of Restrainers.

Hinge	L (m)	D_{eq}-D_r	K_u	$F_y(AR)$	$N_r = \frac{K_u(D_{eq}-D_r)}{F_y(AR)}$	No. of 10-Cable Units
1	1.524	37	346	174	74	8 ←
1	2.18	40	288	174	66	7
2	1.524	127	136	174	99	10
2	2.18	47	320	174	86	9 ←

Try: 8 - 10-cable units per hinge, 1.524 m long at H1 and H3
9 - 10-cable units per hinge, 2.18 m long at H2

Step 5 - Check Deflections.

Compute K_r for restrainers:

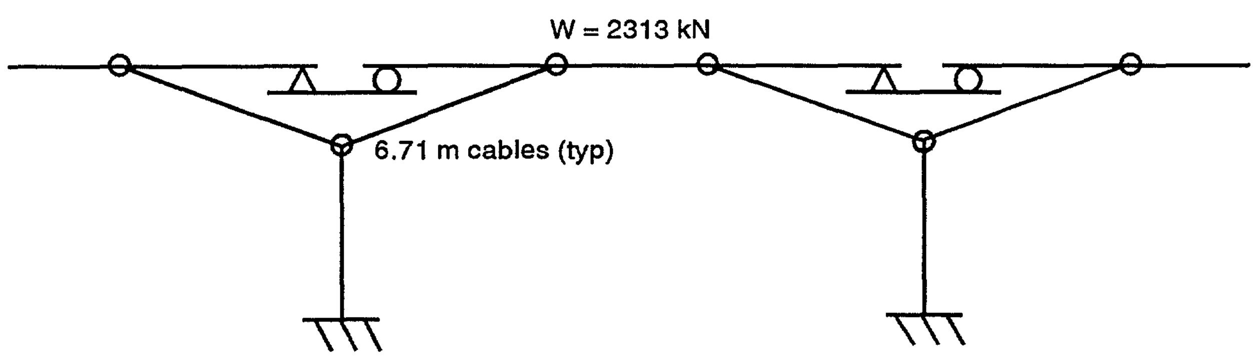
10 x 8 = 80 cables with 1.524 m
10 x 9 = 90 cables with 2.18 m

$$K_r = \frac{F_y N_r A_r}{D_r} = \frac{174 \times 90}{57} = 275 \text{ kN/m (2.18 m cables)}$$
$$= \frac{174 \times 80}{46} = 303 \text{ kN/m (1.524 m cables)}$$

Hinge	W	K_u	K_r	$K_t = K_u + K_r$	$T = 0.063\sqrt{\frac{W}{K_t}}$	ARS
1	16903	346	303	649	0.32	1.82g
2	19572	320	275	595	0.36	1.78g

$$D_t = \frac{ARS(W)}{K_t} = \frac{1.82(16903)}{649} = 47 \text{ mm} > 46 \text{ mm } D_r \quad \text{OK}$$
$$= \frac{1.78(19572)}{595} = 59 \text{ mm} > 57 \text{ mm } D_r \quad \text{OK}$$

Example 2 — Multiple Simple Spans, Retrofit

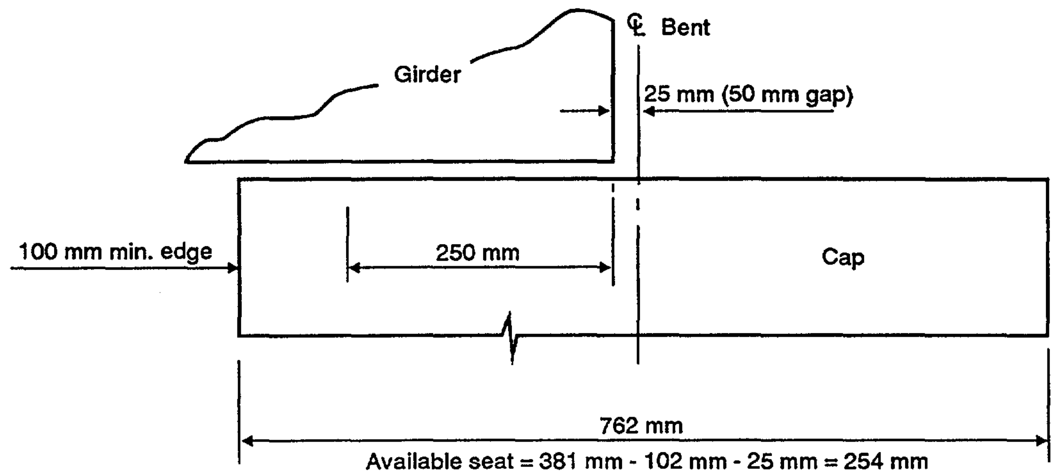


Seismic data: A = 0.7g, 3 m to 24 m alluvium

Bearings: Assume fixed bearings are no good in longitudinal direction. (Note: If bearings were okay, they could be used to add to stiffness of system in longitudinal direction.) Assume keys will be checked/strengthened in transverse direction.

Restrainers: 6.71 m long with 13 mm gap.

Available seat width:



Maximum restrainer deflection (D_y):

$$D_y = \frac{F_y L}{E} = \frac{1.21(6.71)(1000)}{69.57} = 117 \text{ mm}$$

$$\text{Add gap } D_g = \underline{\ 13 \text{ mm}}$$

$$D_r = 130 \text{ mm} < 254 \text{ mm} \quad \text{OK}$$

Total stiffness is from restrainers - try 20 cables:

$$K_t = \frac{F_y N_r (A_r)}{D_r} = \frac{1.21(20)(143)}{130} = 26.62 \text{ kN/mm}$$

$$T = 0.063\sqrt{\frac{2313}{26.62}} = 0.59 \text{ sec; ARS} = 1.62g$$

$$D_t = \frac{ARS(W)}{K_t} = \frac{1.62(2313)}{26.62} = 141 \text{ mm} > 130 \text{ mm } (8\%)$$

Too large; add more cables (or lengthen).

Try 24 cables:

$$K_t = \frac{1.21(24)(143)}{130} = 31.94 \text{ kN/mm}$$

$$T = 0.063\sqrt{\frac{2313}{31.94}} = 0.54\ \text{sec}; \text{ARS} = 1.7\,g$$

$$D_t = \frac{1.7(2313)}{31.94} = 123\ \text{mm} < 130\ \text{mm OK}$$

Use 24 cables total, 6.71 m long.

Note: Because D_t is less, these cables could be shortened - try 6.10 m long cables, total 24:

D_y = 6.10/6.71(117)	=	106 mm
Add gap	=	13 mm
D_r	=	119 mm

$$K_t = \frac{1.21(24)143}{119} = 34.90\ \text{kN/mm}$$

$$T = 0.063\sqrt{\frac{2313}{34.90}} = 0.52\ \text{sec}; \text{ARS} = 1.78\,g$$

$$D_t = \frac{1.78(2313)}{34.90} = 118\ \text{mm} < 119\ \text{mm OK}$$

Use 24 cables total, 6.10 m long.

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