PRELIMINARY SEISMIC EVALUATION OF HIGHWAY BRIDGES IN ISTANBUL

by

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ABSTRACT

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In spite of the considerable developments in bridge engineering in recent years, substantial damages of highway bridges in recent earthquakes, have lead to an increasing awareness on the seismic performance of bridges.

Seismic vulnerability of a bridge can be defined as the potential of a bridge to sustain significant damage or collapse. The seismicity at the location of the bridge is essential in the determination of its vulnerability.

Istanbul is one of the most crucial cities in Turkey due to its historical, industrial and commercial importance.

The main objective of the current study is to establish an inventory for the highway bridges on the O1 and O2 peripheral routes and the corresponding link roads in Istanbul and evaluate their seismic vulnerabilities according to a certain screening procedure. Such a preliminary screening procedure contemplates only the technical aspects of the problem and does not include political and economic considerations.

In the preliminary screening process, Structural

characteristics of the bridge, Importance of the bridge as a vital transportation link, Foundation and site characteristics of the bridge, are taken into consideration.

In addition to the above studies, truck loading testing and a detailed analysis on a typical representitive bridge are presented.

The results of this investigation are to be considered as the essential and basic step for the maintenance and rehabilitation of the highway bridges in İstanbul under a future seismic activity.

ÖZET

İSTANBUL'DAKİ OTOYOL KÖPRÜLERİNİN DEPREM YÖNÜNDEN HASAR GÖREBİLİRLİĞİNİN DEĞERLENDİRİLMESİ

Bu çalışmada amaç, İstanbul'da O1 ve O2 çevreyolları üzerinde bulunan otoyol köprüleri için bir veritabanı oluşturmak, ve bir önincelеme metodu kullanarak, bu köprülerin deprem etkisi yönünden hasar görebilirliğini değerlendirmektir. Kullanılan öninceleme metodu konuyu sadece mühendislik açısından değerlendirir, ekonomik ve idari yaklaşımları gözönüne almaz. Bir köprünün deprem yönünden hasar görebilirliğinin puanlanmasında, köprünün yapısal karekteristiği, önemi ve zemin yapısı göz önüne alınır. Bu çalışmada ayrıca seçilen bir tipik köprü için kamyon yükleme testi ve detaylı analiz yapılmıştır. Bu çalışmanın sonuçları, köprülerin ileride deprem yönünden takviye ve güçlendirme çalışmaları için gerekli görülmektedir.

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A	The cross-sectional area of the frame element.
(A)	Acceleration coefficient.
A_b	Area of spliced bar.
A_g	Gross area of column.
$A_{tr}(c)$	Area of transverse reinforcing.
BVR	Base vulnerability rating.
b_{max}	Transverse column dimension.
b_{min}	Minimum width of the column cross section.
c	Clear cover over the bars or half the clear spacing between adjacent bars.
C/D	Capacity demand ratio.
CVR	Column vulnerability rating.
d_b	Nominal bar diameter.
d(c)	Abutment displacement capacity.
d(d)	Abutment displacement demand.
E	Modulus of elasticity.
F	Framing factor.
f_c'	Compressive strength of concrete.
f_y	Yield stress in longitudinal reinforcement.
f_{yt}	Yield stress of transverse reinforcement.
g	Gravitational acceleration.
H	Average height of columns supporting the bridge deck to the next expansion joint.
IC	Importance classification
I_x	Moment of inertia in -X- direction.
I_y	Moment of inertia in -Y- direction.
J	Torsional inertia.
k	Stiffness matrix of the mathematical model.

k_3	Effectiveness of transverse bar anchorage.
k_s	A constant for reinforcing steel with a yield stress of f_y
L	Length of the bridge deck to the expansion joint or to the end of the bridge deck.
L_c	Effective column length.
$l_a(c)$	Effective anchorage length of longitudinal reinforcement.
$l_a(d)$	Required effective anchorage length reinforcement.
l_s	Splice length.
m	Mass matrix of the mathematical model.
M_i	Generalized mass.
M	Mass per unit length for the frame element.
M_u	Ultimate moment capacity of a column.
N(c)	The support length provided at a bearing seat.
N(d)	The minimum support length according to AASHTO.
P_c	Axial compressive load on the column.
$P_i(t)$	The effective modal load.
P_s	Percent main reinforcing steel.
R	Response modification factor.
r_{ad}	C/D ratio for abutments.
r_{ca}	C/D ratio for anchorage of longitudinal reinforcement.
r_{bd}	C/D ratio for bearing displacement.
r_{bf}	C/D ratio for bearing force.
r_{ec}	Ultimate moment capacity/elastic moment demand ratio.
r_{cs}	C/D ratio for splices in longitudinal reinforcement.
r_{cv}	C/D ratios for column shear.
r_{sl}	C/D ratios for liquefaction.
s	Spacing of transverse reinforcement.
S	Site coefficient.
SPC	Seismic Performance Category.
v	displacement amplitude vector.

$V_b(c)$	Force capacity at the bearings.
$V_b(d)$	Force demand at the bearings.
v_c	The shear stress carried by the concrete.
$V_e(d)$	The maximum calculated elastic shear force.
$V_i(c)$	The initial shear resistance of the undamaged column
V_u	Column shear forces.
Y_i	The mode amplitude.
ω	frequency.
ξ	damping ratio.
$\rho(c)$	Volumetric ratio of existing transverse reinforcement.
$\rho(d)$	Required volumetric ratio of transverse reinforcement.
μ	The ductility indicator.

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CHAPTER 1

INTRODUCTION

Bridges are indispensable components in transportation networks. Two factors determine bridge vulnerability to seismic effects:

First, their ability to resist earthquake forces; and second, their ability to tolerate large superstructure movements. Earthquake forces are generally higher in bridges supported on stiff substructures (i.e. in short period bridges) and deflections are usually larger in the heavier decks on flexible substructures (i.e. in long period bridges) (Buckle,I.,1991).

Bridges with the greatest vulnerability for seismic damage are therefore multi-span structures that have one or more of the following:

- Simply supported spans which have deficient bearings and inadequate seatwidths.
- Continuous spans which have intermediate hinges with deficient bearings and inadequate seatwidths.
- None ductile substructures
- Under-reinforced footings
- Under-reinforced abutment, backwalls and wingwalls
- Unusual geometry (severe curvature, severe skew, tall piers, piers with different heights, long continuous spans, piers in deep water)
- Hazardous site conditions (near active faults, on or near unstable slopes, on liquefiable foundations, on deep soft soil sites)

On the other hand, bridges with the least seismic vulnerability include:

- Single span bridges with either integral abutments or general seatwidths and adequate connection details at the abutments

- Continuous bridges with either integral abutments or general seatwidths and adequate connection details at the abutments, that have redundant substructurs and no internal hinge seats

- Bridges with earthquake protective systems such as base isolation devices which reduce seismic forces and control large superstructure movements.

In the last two decades, damages on highway bridges caused by earthquakes has led the researchers to develop several methodologies for evaluating and increasing the seismic resistance of highway bridges.

1.1 The Retrofitting Process

The Seismic Retrofitting Process can be divided into three major steps. These are:

- Preliminary Screening
- Detailed Evaluation
- Design of Retrofit Measures

"Preliminary Screening" of seismically deficient bridges is necessary to identify bridges which are potentially candidates for retrofitting.

"The detailed seismic evaluation" begins with a quantitative evaluation of the individual bridge components. The results from an elastic spectral analysis are used for this purpose. The design earthquake loading is used in the analysis. The force and displacement results, known as "demands-D", are compared with the "capacities-C" of each component.

A "Capacity/Demand-C/D" ratio less than one indicates that component failure may occur during the design earthquake and retrofitting is needed.

Retrofitting should be considered when the detailed assessment indicates that local component failure will result in an unacceptable overall performance. The effectiveness of retrofitting may be assessed by performing a detailed re-evaluation of the retrofitted bridge.

A flow chart of the retrofit process as it applies to bridges in different seismic performance categories is shown in Figure 1.1.

1.2 Objective of this study

The main objective of this study is to develop the first step of the retrofitting process which is the "preliminary screening" of the existing highway bridges on the two main highway routes in İstanbul, namely O1 and O2 routes and the corresponding link roads (Figure 1.2). There are altogether 123 bridges along those two routes. The current study will cover only 72 bridges out of these 123. Out of 72 bridges studied, 27 are located on O1 route and 34 in O2 route, and 13 in the link roads.

The study will cover the following:

1. Collection of information on the existing bridges in the populated areas of Istanbul (within the control of the 17th. Regional Office of Highway Department)

2. Seismic vulnerability rating and evaluation of the 72 bridges according to two different well-established preliminary screening methods.

3. Illustration of the "Detailed evaluation" process on a newly

constructed bridge, and analyze the bridge under the effect of a design earthquake and determine the Capacity/Demand Ratios of components.

4. Truck load testing of the bridge analyzed in item 3.

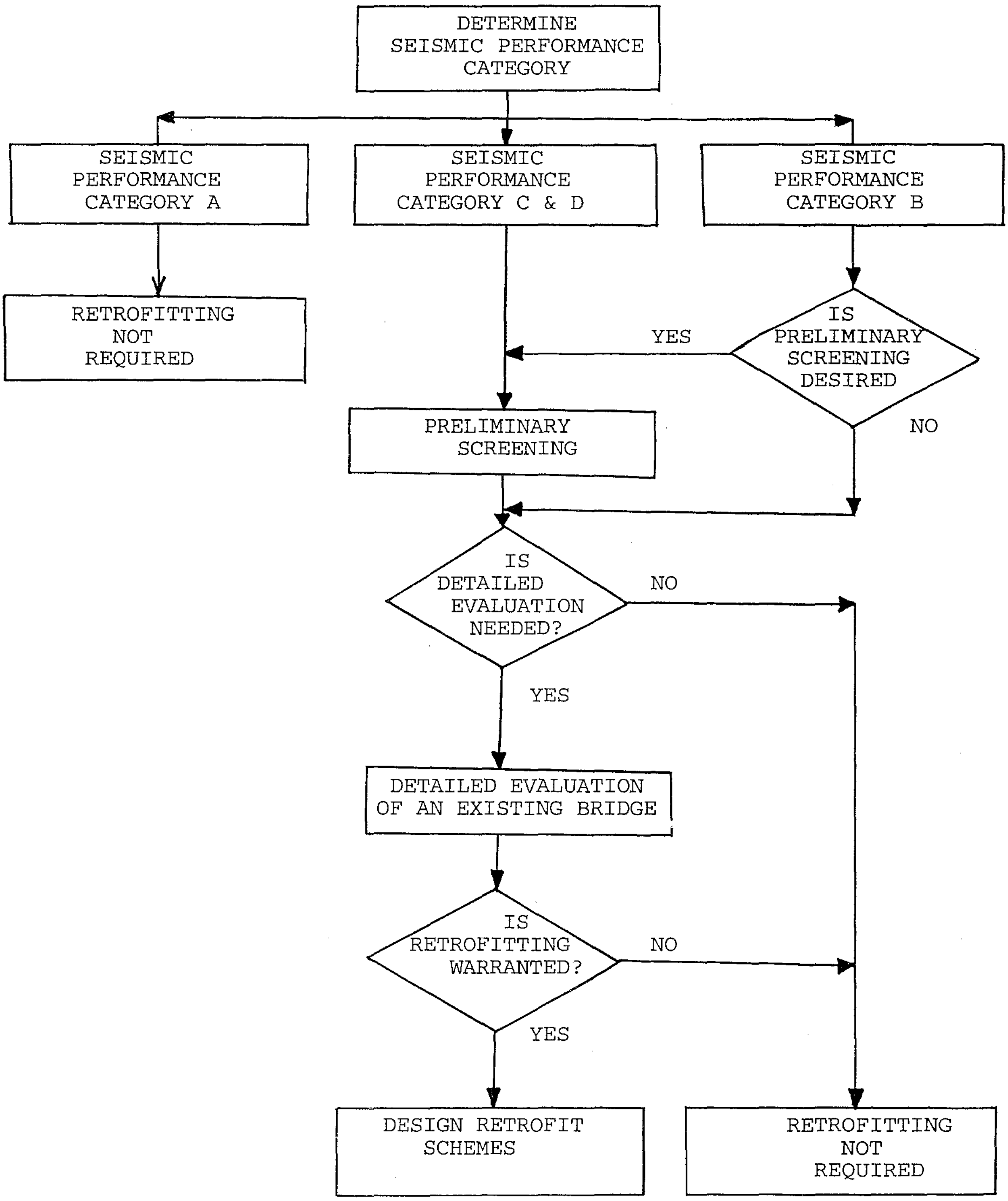

FIGURE 1.1 Seismic Retrofitting Process (ATC, 1983)

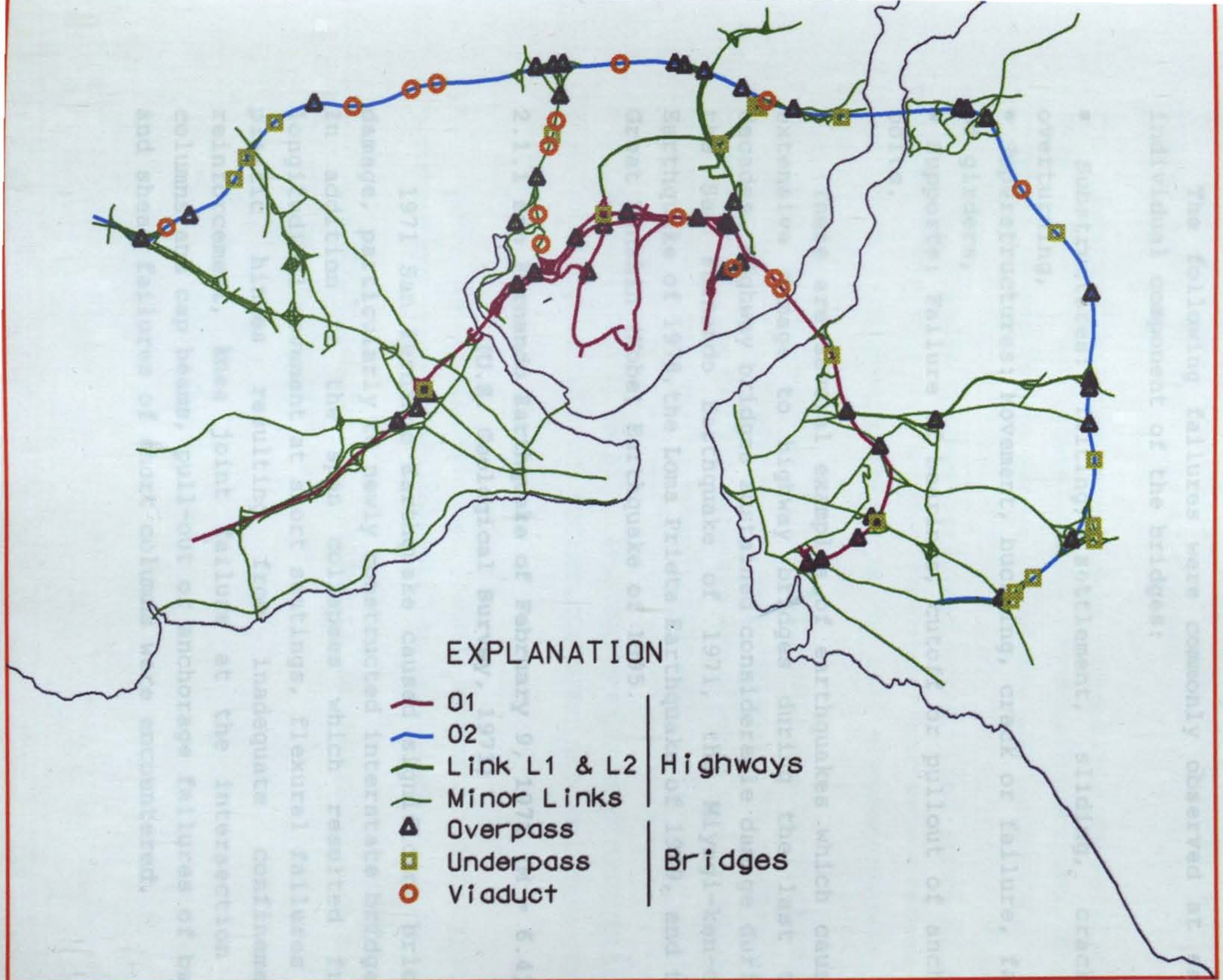

FIGURE 1.2 01-02 peripheral routes and the link routes

CHAPTER 2

HISTORICAL BACKGROUND

2.1 Examples from Recent Earthquakes

The following failures were commonly observed at each individual component of the bridges:

- Substructures: Tilting, settlement, sliding, cracks, overturning,
- Superstructures: Movement, buckling, crack or failure, fall of girders,
- Supports: Failure of bearings, cutoff or pullout of anchor bolts.

There are several examples of earthquakes which caused extensive damage to highway bridges during the last two decades. Highway bridges sustained considerable damage during the San Fernando Earthquake of 1971, the Miyagi-ken-oki Earthquake of 1978, the Loma Prieta Earthquake of 1989, and the Great Hanshin (Kobe) Earthquake of 1995.

2.1.1 San Fernando Earthquake of February 9, 1971 (M = 6.4) (U.S. Geological Survey, 1971)

1971 San Fernando Earthquake caused significant bridge damage, particularly to newly constructed interstate bridges. In addition to the span collapses which resulted from longitudinal movement at short seatings, flexural failures of plastic hinges resulting from inadequate confinement reinforcement, knee joint failure at the intersection of columns and cap beams, pull-out or anchorage failures of bars and shear failures of short columns were encountered.

The lessons learned from San Fernando Earthquake can be summarized as follows:

- The earthquake force level in San Fernando greatly exceeded the earthquake forces specified by the design criteria,

- The vertical acceleration of the earthquake possibly played a part in the cause of damages.

- Skewed structures were highly susceptible to rotational displacement toward acute corners. At some structures, the rotation caused severe damage to columns and abutments.

- Tall slender columns performed better than short stiff columns. Shearing and bending fractures that were evident on the short columns were absent in the tall slender columns.

- The vibrating action of the earthquake shattered concrete at the base and footings of many columns. This shattered concrete lost its bonding strength and allowed the column bars to be pulled out causing some structures to collapse.

- Deficiencies in details, especially at connections, placed a major role in all of the spectacular and collapse type failures.

- There was considerable ground movement. The ground movement was large enough sometimes to allow spans to drop off.

- The fill behind the abutments of many, if not all, structures in the area settled.

2.1.2 Miyagi-ken-oki Earthquake of June 12, 1978 (M = 7.4) (The Japan Society of Civil Engineers, 1988)

Highway bridges sustained considerable damage during Miyagi-ken-oki earthquake, among which The Sendai Bridge, The Kin-noh Bridge, The Eai Bridge, The Yuriage Bridge, and The Date Bridge can be mentioned (see Appendix A).

The lessons learned from the Miyagi-ken-oki Earthquake can be summarized as follows:

- Damage to superstructures concentrated on bearing supports and adjoining portions. On the other hand, most of the damage to substructures were cracking and separation of concrete at the columns of the piers and at the abutments.

- Damage to bearing supports was frequent. The failure of the bearing supports have reduced failure of bridge girders. Therefore, it is not always advised to design too strong bearings.

- A number of older bridges sustained relatively severe damage. In most of these bridges, either Gerber-type or simply supported type was used, with narrow pier caps and having no special consideration to prevent them from falling down.

2.1.3 Loma Prieta Earthquake of October 17, 1989 (M = 7.1) (Earthquake Spectra, 1990)

During the Loma Prieta Earthquake, more than 80 bridges suffered minor damage, 10 needed temporary supports, and 10 were closed due to major structural damage (Loma Prieta Eathquake Reconnaissance Report, 1990). Three bridges suffered one or more collapsed spans.

The greatest damage occured to older structures on poor ground, such as the Cypress Street Viaduct and the San Francisco-Oakland Bay Bridge.

The Cypress Street Viaduct was a two-level, elevated freeway structure built on poor soil in West Oakland. Each level was a multicell concrete box girder bridge supported on reinforced concrete frames (bents). Designed in 1951 and completed in 1957. It collapsed catastrophically during the earthquake, crushing cars and trucks as the second level pancaked onto the first level.

Factors affecting the performance of the viaduct include unconfined shear keys; inadequate joint steel; the soft-soil site; variable soil conditions; and variations in lateral stiffness due to some bents having flexural pins, some being skewed, and some having three columns at the lower level.

The collapse of the link span of the Bay Bridge was due primarily to a connection failure, followed by inadequate seat widths for the deck girders. Failure was initiated when the holding-down bolts of the eastern truss at Tower E9 failed, which permited this truss to move eastward independently of the western truss at E9. The girders supporting the link span over tower E9 became unseated, and the span collapsed.

Other bridges of similar design to the Cypress Street Viaduct probably would have collapsed if the ground shaking had lasted longer.

Damage to a bent in the connector structure in West Oakland appears to be due to the inadequate shear strength of the reinforced concrete knucle joint in this bent.

The twin bridges across Struve Slough west of Watsonville were also severely damaged, one catastrophically. These bridges have approximately 20 spans of skewed reinforced concrete T-beams supported on monolithic pile bents. Shear failures in the tops of some piles and gross relative movement of the superstructure led to hte spans falling onto the piles, some of which subsequently punched through the deck.

Many approach fills behind abutment walls settled or slumped, temporarily interrupting bridge access. In most instances access was restored immediately and the approach roads were repaved within a few days.

2.1.4 The Great Hanshin (Kobe) Earthquake of January 17,1995 (M = 6.8) (Aydınoğlu, N., Erdik, M., 1995), (NCEER, 1995)

In January 17, 1995 in Kobe an earthquake with a magnitude of 6.8 caused damage on a large number of bridges. The predominant type of bridge in Japan is the steel girder superstructure (simple and/or continuous spans) supported by bearings on concrete columns and foundations. Although bridges in this region are designed for seismic loads, the design coefficients are considerably lower than those recorded during this earthquake (Buckle, I., 1995).

Typical damage sustained by the bridges includes shear and flexural failures in nonductile concrete columns, flexural and buckling failures in steel columns, steel bearing failures under lateral load, and foundation failures due to liquefaction. In addition, there was pounding between spans, failure of several earthquake couplers, and settlement of many approach fills.

Some of the lessons learned from Kobe Earthquake summarized as follows:

- Capacity design procedures, ductile details and generous seat widths are necessary to prevent catastrophic collapse of the bridges during large earthquakes.

- Minimum connection forces need to be enforced for all seismic zones unless such connections can be shown to be fully protected by acceptable yielding of the substructures. Redundancy in connection detailing is particularly important for essential bridges. Alternative load paths are necessary if the primary load path fails due to unforeseen circumstances.
- Critically important bridges must be designed to a higher level of performance than that provided by current specifications, if full service is to be maintained after a large earthquake. Dual-level performance criteria and corresponding design strategies are necessary for important bridges.
- Retrofit measures reduce damage but inappropriate use and/or installation can defeat their purpose and perhaps even trigger collapse.
- Lateral spreading due to liquefaction can lead to span collapse even in modern bridges with massive foundations(caissons) and well-engineered fills.
- Premature failure of some bearings appear to have reduced the seismic loads in their supporting substructures by uncoupling the superstructure from its supports. This fuse-like action may have saved a number of spans from collapse and columns from shear and flexural failure.
- Accelerations in isolated superstructures are less than in conventional structures.
- Skewed bridges are susceptible to in-plane rotation leading to displacements at supports that are larger than anticipated and subsequent collapse.

Damage to highway bridges was both widespread and catastrophic. Most of this trouble was confined to older structures built more than 30 years ago and before the introduction of modern seismic codes. Some new bridges also suffered serious damage which suggests a need to re-evaluate the design loads and procedures for these structures. There is strong evidence to indicate that the peak ground accelerations were considerably higher than the seismic coefficients used for bridge design in the region. The occurence of this damaging but "rare" earthquake raises doubth, about the correct level of the design load and reinforces the need for dual-level performance criteria. These criteria should clearly state the expected performance under both large and small, rare and frequent, earthquakes and identify design strategies and procedures that will satisfy these criteria.

Damages to highway bridges in recent earthquakes are illustrated in photographs given in Appendix B.

CHAPTER 3

SEISMIC RETROFITTING PROCESS OF HIGHWAY BRIDGES

3.1 Inventory for Highway Bridges in Istanbul

3.1.1 Introduction

In total, there are 123 bridges in the study area (O1 and O2 Peripheral routes and link routes). The number of bridges on each route and the number of bridges which were screened in each route are given in Tables 3.1 and 3.2. In this study, a total of 72 bridges on E5 (designated as O1) and TEM (designated as O2) peripheral routes, and link roads in between them were screened for their preliminary vulnerability evaluation under seismic loading. The above mentioned routes are illustrated in Figure 3.1. The purpose of this section is to develop an inventory to identify the bridges on the above mentioned routes in İstanbul. The inventory contains information on: year built, geometry, number of expansion joints, skew angle, superstructure type, number of spans, substructure type, pier height, support length, bearing type, foundation type of the bridges. The above mentioned data for each bridge are given in Table 3.3.

	O1 Route	O2 Route	Link Roads	Total
Number of Bridges	45	51	27	123
Number of Screened Bridges	25	34	13	72

Table 3.1- Bridges on each route in the study area

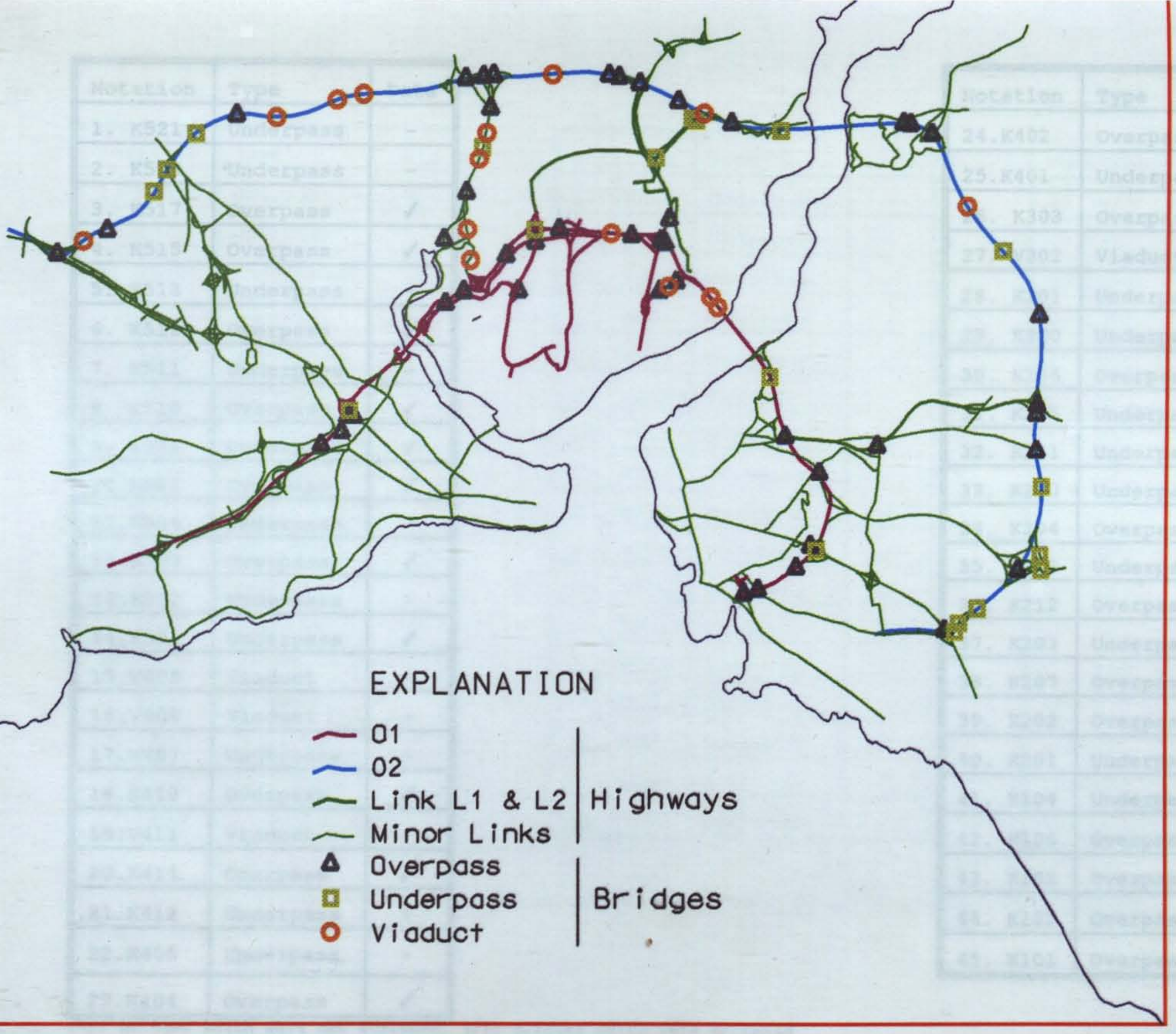

FIGURE 3.1 O1-O2 peripheral routes and the link routes

Notation	Type	Data
1. K521	Underpass	-
2. K518	Underpass	-
3. K517	Overpass	✓
4. K515	Overpass	✓
5. K513	Underpass	-
6. K512	Overpass	✓
7. K511	Underpass	-
8. K510	Overpass	✓
9. K509	Underpass	✓
10.K505	Overpass	✓
11.K504	Underpass	-
12.K503	Overpass	✓
13.K502	Underpass	-
14.K501	Underpass	✓
15.V409	Viaduct	-
16.V408	Viaduct	-
17.K407	Underpass	-
18.K410	Overpass	✓
19.V411	Viaduct	-
20.K414	Overpass	✓
21.K412	Underpass	-
22.K405	Underpass	-
23.K404	Overpass	✓
24.K402	Overpass	✓
25.K401	Underpass	-
26. K303	Overpass	✓
27. V302	Viaduct	-
28. K301	Underpass	-
29. K300	Underpass	✓
30. K206	Overpass	✓
31. K205	Underpass	✓
32. K211	Underpass	-
33. K210	Underpass	-
34. K204	Overpass	✓
35. K208	Underpass	-
36. K212	Overpass	✓
37. K203	Underpass	-
38. K207	Overpass	✓
39. K202	Overpass	✓
40. K201	Underpass	-
41. K104	Underpass	✓
42. K106	Overpass	✓
43. K103	Overpass	✓
44. K102	Overpass	✓
45. K101	Overpass	✓

(-): Bridges which were not screened, (√): Bridges which were screened

Table 3.2- Highway bridges on 01 route

Notation	Type	Data
1. KMO1	Overpass	✓
2. KMV1	Overpass	-
3. KMU4	Underpass	✓
4. KMU3	Underpass	✓
5. M5U1	Underpass	✓
6. OWO	Overpass	-
7. NMO1	Overpass	✓
8. NMU4	Underpass	✓
9. U208A	Underpass	-
10.NMU2	Underpass	-
11.NMU3	Underpass	✓
12.NMU1	Underpass	-
13.M5U2	Underpass	✓
14.M5O1	Overpass	✓
15.RMO2	Overpass	✓
16.RMU4	Underpass	-
17.RMO3	Overpass	-
18. RMU3	Underpass	-
19. RMU1	Underpass	-
20. RMO1	Overpass	✓
21. M402	Overpass	✓
22. M401	Underpass	✓
23. M3V1	Viaduct	-
24. VMO1	Overpass	✓
25. M301	Overpass	✓
26. M302	Overpass	✓
27. M2U2	Underpass	✓
28. M101	Overpass	✓
29. LMV1	Viaduct	-
30. M102	Overpass	✓
31. BF2	Overpass	-
32. BRO	Overpass	✓
33. ICO	Overpass	✓
34. B14	Overpass	✓
35. B13	Overpass	✓
36. V1	Viaduct	✓
37. B12	Overpass	✓
38. B11	Overpass	✓
39. B10	Overpass	✓
40. B9	Underpass	-
41. V5	Viaduct	-
42. V6	Viaduct	-
43. V7A	Viaduct	-
44. B6	Overpass	✓
45. B5	Underpass	✓
46. B3	Overpass	✓
47. B3B	Underpass	✓
48. B3C	Underpass	✓
49. B2	Overpass	✓
50. V7	Viaduct	-
51. B1	Overpass	✓

(-): Bridges which were not screened, (✓): Bridges which were screened

Table 3.2- (contd.) Highway bridges on O2 route

Notation	Type	Data
1.O207	Overpass	-
2.UM-U8	Underpass	-
3.UM-U7	Underpass	-
4.K3	Overpass	-
5.UM-U5	Underpass	-
6.UM-O7	Overpass	-
7.K2	Overpass	-
8.UM-O6	Overpass	-
9.UM-O5	Overpass	-
10.UMU3A	Underpass	-
11.K1	Overpass	✓
12.K305	Overpass	✓
13.K304	Underpass	-
14.BLU1	Underpass	✓
15.BF1	Overpass	-
16.L102	Overpass	✓
17.M1U1	Underpass	✓
18.M1U2	Underpass	✓
19.V4	Viaduct	✓
20.V3	Viaduct	-
21.B21	Overpass	✓
22.B19	Overpass	✓
23.V2	Viaduct	✓
24.B17	Underpass	✓
25.V2A	Viaduct	✓
26.B16	Overpass	✓
27.B15	Underpass	-

(-): Bridges which were not screened, (✓): Bridges which were screened

Table 3.2- (contd.) Highway bridges on the link roads

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
1. K517 Overpass (1972)	Length :53.0 m 4 spans , Width :12.9 m Ex. Joint : 0 Skew Angle:3.6^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Column Piers : Wall(multi) Pier Height : 8.5 m Abut. Height : 8.5 m Sup. Length : 90 cm	Elastomeric,	Shallow Soil Type : E
2. K515 Overpass (1972)	Length :68.4 m 4 spans , Width :14.0 m Ex. Joint : 0 Skew Angle :9^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 9.5 m Abut. Height : 4.5 m Sup. Length : 90 cm	Elastomeric, Freyssinet	Shallow Soil Type : E
3. K512 Overpass (1972)	Length :68.4 m 4 spans , Width :7.5 m Ex. Joint : 0 Skew Angle :12^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 7.0 m Abut. Height : 8.0 m Sup. Length : 50 cm	Elastomeric, Freyssinet	Shallow Soil Type : E
4. K510 Overpass (1972)	Length :67.8 m 4 spans , Width :8.9 m Ex. Joint : 2 Skew Angle :0^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(single) Pier Height : 7.0 m Abut. Height : 1.0 m Sup. Length : 90 cm	Freyssinet, Neopren	R.C. Piled Soil Type : B
5. K509 Underpass (1972)	Length :80.4 m 2 spans , Width :13.5 m Ex. Joint : 2 Skew Angle: 15^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(single) Pier Height : 9.5 m Abut. Height : 2.0 m Sup. Length : 60 cm	Freyssinet, Neopren	R.C. Piled Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3- Inventory for the bridges under study

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
6. K505 Overpass (1971)	Length :69.4 m 4 spans , Width :7.0 m Ex. Joint : 0 Skew Angle :0^0	Post-tensioned ; Continuous Plate No. of cell : 1	Abutments : Wall Piers : Wall(single) Pier Height : 8.5 m Abut. Height : 2.7 m Sup. Length : 60 cm	Elastomeric, Freyssinet	Shallow Soil Type :A,C
7. K503 Overpass (1971)	Length :80.9 m 4 spans , Width :7.1 m Ex. Joint : 0 Skew Angle :15^0	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 8.5 m Abut. Height : 2.5 m Sup. Length : 80 cm	Elastomeric, Freyssinet	Shallow Soil Type :A,C
8. K501 Underpass (1974)	Length :39.2 m 2 spans , Width :14.5 m Ex. Joint : 2 Skew Angle: 22^0	R.C. Plate No. of cell : 3	Abutments : Wall Piers : - Pier Height : 11.0 m Abut. Height : 11.0 m Sup. Length : 90 cm	Elastomeric, Freyssinet	R.C. Piled Soil Type : B
9. K410 Overpass (1972)	Length :34.8 m 2 spans , Width :7.0 m Ex. Joint : 0 Skew Angle :27^0	R.C. Plate No. of cell : 3	Abutments : Wall Piers : Wall(single) Pier Height : 13.8 m Abut. Height : 2.0 m Sup. Length : 115 cm	Neopren	Shallow Soil Type : B
10. K414 Overpass (1972)	Length :69.2 m 3 spans , Width :7.0 m Ex. Joint : 2 Skew Angle :9^0	R.C. Plate No. of cell : 3	Abutments : Wall Piers : Wall(single) Pier Height : 12.0 m Abut. Height : 2.0 m Sup. Length : 50 cm	Neopren	Shallow Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
11. K404 Overpass (1972)	Length :81.4 m 4 spans , Width :9.5 m Ex. Joint : 2 Skew Angle :2^{0}	Post-tensioned ; Continuous Box Girder No. of cell : 1	Abutments : Wall Piers : Wall(multi) Pier Height : 7.0 m Abut. Height : 3.5 m Sup. Length : 70 cm	Neopren	Shallow , Soil Type : B
12. K402 Overpass (1972)	Length :75.6 m 4 spans , Width :16.6 m Ex. Joint : 2 Skew Angle :13^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 9.5 m Abut. Height : 2.0 m Sup. Length : 80 cm	Elastomeric, Freyssinet	Shallow Soil Type : B
13. K303 Overpass (1974)	Length :73.9 m 3 spans , Width :6.6 m Ex. Joint : 2 Skew Angle :0^{0}	Post-tensioned ; Continuous Box Girder No. of cell : 3	Abutments : Wall Piers :Column(single) Pier Height : 4.5 m Abut. Height : 4.5 m Sup. Length : 70 cm	Elastomeric, Freyssinet	Shallow Soil Type : B
14. K300 Underpass (1974)	Length :49.0 m 2 spans , Width :15.0 m Ex. Joint : 1 Skew Angle: 50^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 9.0 m Abut. Height : 9.0 m Sup. Length : 90 cm	Elastomeric, Freyssinet	R.C. Piled Soil Type : D
15. K205 Underpass (1974)	Length :67.5 m 4 spans , Width :17.6 m Ex. Joint : 2 Skew Angle: 0^{0}	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(multi) Pier Height : 8.5 m Abut. Height : 8.0 m Sup. Length : 60 cm	Elastomeric, Freyssinet	R.C. Piled Soil Type : D

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
16. K206 Overpass (1972)	Length :67.5 m 4 spans , Width :10.0 m Ex. Joint : 0 Skew Angle :9^0	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 8.8 m Abut. Height : 4.0 m Sup. Length : 80 cm	Freyssinet Neopren	Shallow Soil Type : B
17. K204 Overpass (1972)	Length :35.0 m 2 spans , Width :10.0 m Ex. Joint : 2 Skew Angle :0^0	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 7.9 m Abut. Height : 8.2 m Sup. Length : 80 cm	Elastomeric, Freyssinet	Shallow Soil Type : B
18. K212 Overpass (1972)	Length :58.6 m 4 spans , Width :7.5 m Ex. Joint : 0 Skew Angle :9^0	Post-tensioned ; Continuous Plate No. of cell : 1	Abutments : Wall Piers : Wall(multi) Pier Height : 8.5 m Abut. Height : 6.0 m Sup. Length : 60 cm	Freyssinet Neopren	Shallow Soil Type : B
19. K207 Overpass (1972)	Length :69.8 m 4 spans , Width :7.9 m Ex. Joint : 0 Skew Angle:4.5^0	Post-tensioned ; Continuous Plate No. of cell : 3	Abutments : Wall Piers : Wall(single) Pier Height : 8.2 m Abut. Height : 3.1 m Sup. Length : 80 cm	Freyssinet Neopren	Shallow Soil Type : B
20. K202 Overpass (1972)	Length :72.7 m 4 spans , Width :10.9 m Ex. Joint : 2 Skew Angle :27^0	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 10.5 m Abut. Height : 4.5 m Sup. Length : 80 cm	Freyssinet, Neopren	Shallow Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
21. K104 Underpass (1974)	Length :69.2 m 4 spans , Width : 17.8 m Ex. Joint : 2 Skew Angle:29°	Post-tensioned ; Continuous Plate No. of cell : 1	Abutments : Open Piers : Wall(multi) Pier Height : 7.5 m Abut. Height : 3.5 m Sup. Length : 70 cm	Neotopf, Neopren	R.C. Piled , Soil Type : D
22. K106 Overpass (1973)	Length :67.4 m 4 spans , Width :8.9 m Ex. Joint : 2 Skew Angle :19°	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 12.8 m Abut. Height : 4.5 m Sup. Length : 85 cm	Elastomeric, Freyssinet	R.C. Piled Soil Type : C
23. K103 Overpass (1973)	Length :60.1 m 4 spans , Width :8.9 m Ex. Joint : 2 Skew Angle :10°	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 8.0 m Abut. Height : 3.0 m Sup. Length : 80 cm	Elastomeric, Freyssinet	R.C. Piled , Soil Type : C
24. K102 Overpass (1973)	Length :72.0 m 4 spans , Width :7.0 m Ex. Joint : 1 Skew Angle :18°	Post-tensioned ; Continuous Plate	Abutments : Wall Piers : Wall(multi) Pier Height : 9.0 m Abut. Height : 2.5 m Sup. Length : 80 cm	Freyssinet, Neopren	Shallow Soil Type : C
25. K101 Overpass (1973)	Length:125.5 m 4 spans , Width :9.0 m Ex. Joint : 2 Skew Angle : 0°	Post-tensioned ; Continuous Box Girder	Abutments : Open Piers : Wall(single) Pier Height : 4.0 m Abut. Height : 3.0 m Sup. Length : 80 cm	Neotopf, Neopren	R.C. Piled , Soil Type : C

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
26. KMO1 Overpass (1990)	Length :69.4 m 4 spans , (2×14.4) m (2×18.0) m Width : 21.0 m Ex. Joint : 0 Skew Angle: 23^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments :Wall Piers : Wall(single) Pier Height : 11.0 m Abut. Height : 11.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type : D
27. KMU4 Viaduct (1990)	Length :18.5 m 1 spans , (1×15.0) m Width :10.5 m Ex. Joint : 0 Skew Angle: 20^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 9.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :D
28. KMU3 Viaduct (1990)	Length :18.5 m 1 spans , (1×15.0) m Width :11.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 9.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type :D
29. M5U1 Underpasses (1990)	Length :74.2 m 4 spans , (4×18.3) m Width : 21.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Open Piers : Wall (single) Pier Height :12.0 m Abut. Height :11.5 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type :E

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
30. NM01 Overpass (1990)	Length :51.1 m 2 spans , (2×23.8) m Width : 21.0 m Ex. Joint : 0 Skew Angle: 22^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 11.0 m Abut. Height :12.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : E
31. NMU4 Viaduct (1990)	Length:114.0 m 1 spans , (1×14.0) m Width :7.0 m Ex. Joint : 2 Skew Angle:	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 10.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type : A,B,D
32. NMU3 Viaduct (1990)	Length :47.8 m 1 spans , (1×14.0) m Width :7.4 m Ex. Joint : 2 Skew Angle: 43^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 10.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type : A,B,D
33. M5U2 Underpasses (1990))	Length :47.4 m 1 spans , Width : 7.0 m Ex. Joint : 2 Skew Angle: 11^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height :8.5 m Sup. Length : 75 cm	Elastomeric laminated	Shallow Soil Type :A,B
34. M501 Overpass (1991)	Length :80.0 m 4 spans , (4×19.0) m Width :14.0 m Ex. Joint : 0 Skew Angle: 14^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 8.4 m Abut. Height :2.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :A,B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
35. RMO2 Viaduct (1990)	Length :52.0 m 2 spans , (2×25.5) m Width : 14.0 m Ex. Joint : 0 Skew Angle: 9^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 8.4 m Abut. Height : 9.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : A,B,D
36. RMO1 Viaduct (1990)	Length :67.1 m 2 spans , (2×29.7) m Width :21.0 m Ex. Joint : 0 Skew Angle: 28^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 8.0 m Abut. Height : 11.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : A,B,D
37. M402 Overpass (1990)	Length :49.6 m 2 spans , (2×24.0) m Width : 4.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 7.7 m Abut. Height :8.5 m Sup. Length : 50 cm	Elastomeric laminated	Shallow Soil Type :A,B
38. M401 Overpass (1990)	Length :48.5 m 2 spans , (2×24.0) m Width : 4.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 11.5 m Abut. Height :12.5 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : D

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
39. VMO1 Viaduct (1990)	Length :49.0 m 2 spans , (1×25.5) m (1×22.5) m Width : 18.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 9.0 m Abut. Height : 10.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : A,B,C,D
40. M301 Overpass (1990)	Length:111.0 m 4 spans , (2×19.0) m (2×24.0) m Width : 11.0 m Ex. Joint : 2 Skew Angle: 40^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 15.0 m Abut. Height : 7.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :B,C
41. M302 Overpass (1990)	Length :49.1 m 2 spans , (2×24.0) m Width : 11.0 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 12.0 m Abut. Height : 5.0 m Sup. Length : 50 cm	Elastomeric laminated	Shallow Soil Type :B,C
42. M2U2 Underpasses (1989)	Length :41.0 m 1 spans , Width : 14.0 m Ex. Joint : 2 Skew Angle: 47^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height :8.5 m Sup. Length : 70 cm	Elastomeric laminated	Shallow Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
43. M101 Overpass (1988)	Length :70.0 m 2 spans , (2×35.0) m Width : 7.0 m Ex. Joint : 0 Skew Angle: 0^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 11.0 m Abut. Height : 12.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B
44. M102 Overpass (1988)	Length :70.0 m 2 spans , (2×35.0) m Width : 7.0 m Ex. Joint : 2 Skew Angle: 23^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 14.0 m Abut. Height : 20.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B
45. BRO Overpass (1989)	Length :42.0 m 2 spans , (2×20.5) m Width : 11.7 m Ex. Joint : 2 Skew Angle: 0^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 8.0 m Abut. Height : 11.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : D
46. ICO Overpass (1989)	Length :64.6 m 2 spans , (2×31.7) m Width : 7.0 m Ex. Joint : 2 Skew Angle: 40^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 12.0 m Abut. Height : 14.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
47. B14 Overpass (1988)	Length :46.8 m 2 spans , (2×22.4) m Width : 13.0 m Ex. Joint : 0 Skew Angle: 17^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 12.0 m Abut. Height :13.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : D
48. B13 Overpass (1988)	Length :44.3 m 2 spans , (2×21.4) m Width : 9.0 m Ex. Joint : 2 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 11.5 m Abut. Height :16.0 m Sup. Length : 55 cm	Elastomeric laminated	Shallow Soil Type : E
49. V1 Viaduct (1988)	Length:402.0 m 10 spans , (10×40) m Width : 36.0 m Ex. Joint : 3 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers:-H- Section (single) Pier Height : 35.0 m Abut. Height : 2.0 m Sup. Length : 200 cm	Elastomeric laminated	R.C. Piled , Soil Type :B,E
50. B12 Overpass (1988)	Length :65.7 m 2 spans , (2×31.3) m Width : 9.0 m Ex. Joint : 0 Skew Angle: 54^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 10.0 m Abut. Height :12.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : D

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
51. B11 Overpass (1988)	Length :65.7 m 2 spans , (2×31.3) m Width : 9.0 m Ex. Joint : 0 Skew Angle: 54^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 11.0 m Abut. Height :13.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : D
52. B10 Overpass (1989)	Length :78.9 m 3 spans , (1×29.5) m (1×22.0) m (1×19.0) m Width : 12.0 m Ex. Joint : 0 Skew Angle: 26^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 10.0 m Abut. Height :12.0 m Sup. Length : 100 cm	Elastomeric laminated	Shallow Soil Type : B
53. B6 Overpass (1989)	Length :45.6 m 2 spans , (2×22.5) m Width : 9.0 m Ex. Joint : 0 Skew Angle: 9^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 12.0 m Abut. Height :12.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B
54. B5 Underpasses (1988)	Length :43.8 m 1 spans , Width : 11.5 m Ex. Joint : 0 Skew Angle: 14^{0}	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height :8.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
55. B3 Overpass (1989)	Length:103.1 m 3 spans , (3×31.5) m Width : 9.0 m Ex. Joint : 0 Skew Angle: 23^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Open Piers : Wall (single) Pier Height : 11.0 m Abut. Height :1.0 m Sup. Length : 180 cm	Elastomeric laminated	R.C. Piled Soil Type : B
56. B3B Underpasses (1988)	Length :38.2 m 2 spans , (2×16.0) m Width : 12.0 m Ex. Joint : 0 Skew Angle: 35^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 10.0 m Abut. Height :10.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B
57. B3C Underpasses (1987)	Length :45.0 m 1 spans , Width : 11.5 m Ex. Joint : 0 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height :8.0 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type : B
58. B2 Overpass (1989)	Length :40.5 m 2 spans , (2×19.5) m Width : 9.0 m Ex. Joint : 0 Skew Angle: 14^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 9.0 m Abut. Height :10.5 m Sup. Length : 40 cm	Elastomeric laminated	Shallow Soil Type : B
59. B1 Viaduct (1989)	Length :68.9 m 2 spans , (2×33.9) m Width :11.0 m Ex. Joint : 0 Skew Angle: 54^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 8.0 m Abut. Height : 9.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :D

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
60. K1 Viaduct (1993)	Length:207.5 m 8 spans , (8×26.05) m Width :14.0 m Ex. Joint : 2 Skew Angle: 3^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Open Piers : Column(multi) Pier Height : 11.0 m Abut. Height : 6.5 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :
61. K305 Overpass (1974)	Length:159.6 m 5 spans , Width :8.0 m Ex. Joint : 0 Skew Angle :0^0	Post-tensioned ; Continuous Box Girder No. of cell : 3	Abutments : Wall Piers :Column(single) Pier Height : 5 m Abut. Height : 3.0 m Sup. Length : 75 cm	Neotopf	Shallow Soil Type : B
62. M1U1 Viaduct (1988)	Length :47.6 m 1 spans , (1×13.0) m Width :7.0 m Ex. Joint : 2 Skew Angle: 47^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 12.0 m Sup. Length : 70 cm	Elastomeric laminated	Shallow Soil Type :B
63. M1U2 Viaduct (1988)	Length :42.6 m 1 spans , (1×13.0) m Width :7.0 m Ex. Joint : 2 Skew Angle: 47^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 8.5 m Sup. Length : 70 cm	Elastomeric laminated	Shallow Soil Type :B
64. L1O2 Viaduct (1988)	Length :40.0 m 1 spans , (1×12.5) m Width :7.0 m Ex. Joint : 2 Skew Angle:	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 8.2 m Sup. Length : 60 cm	Elastomeric laminated	Shallow Soil Type :B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
65. BLU1 Viaduct (1988)	Length :40.2 m 1 spans , (1×17.8) m Width :7.0 m Ex. Joint : 2 Skew Angle: 30°	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 9.5 m Sup. Length : 70 cm	Elastomeric laminated	Shallow Soil Type :B
66. B16 Viaduct (1988)	Length :47.0 m 2 spans , (2×23.0) m Width : 8.0 m Ex. Joint : 0 Skew Angle: 11°	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 9.2 m Abut. Height : 10.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :B
67. V2A Viaduct (1989)	Length:242.0 m 6 spans , (6×40.0) m Width : 11.0 m Ex. Joint : 2 Skew Angle: 0°	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Open Piers:-H- Section (single) Pier Height : 32.0 m Abut. Height :15.0 m Sup. Length : 200 cm	Elastomeric laminated	Shallow Soil Type :B
68. B17 Viaduct (1988)	Length :39.9 m 1 spans , (1×11.0) m Width : 4.0 m Ex. Joint : 2 Skew Angle: 9°	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : - Pier Height : Abut. Height : 11.5 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type :B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

Bridge Name (Year Built)	Geometry	Superstructure Type	Substructure Type	Bearing Type	Foundation Type
69. V2 Viaduct (1988)	Length:402.0 m 10 spans , (10×40.0) m Width : 22.0 m Ex. Joint : 3 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers:-H- Section (single) Pier Height : 40.0 m Abut. Height :15.0 m Sup. Length : 200 cm	Elastomeric laminated	Shallow Soil Type :B
70. B19 Overpass (1989)	Length :67.0 m 2 spans , (2×32.8) m Width : 8.0 m Ex. Joint : 0 Skew Angle: 45^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers : Wall (single) Pier Height : 9.0 m Abut. Height :11.0 m Sup. Length : 80 cm	Elastomeric laminated	Shallow Soil Type : B
71. B21 Underpasses (1989)	Length:150.0 m 6 spans , (4×25.0) m (2×24.0) m Width : 19.0 m Ex. Joint : 3 Skew Angle: 11^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Open Piers: Column(multi) Pier Height : 11.0 m Abut. Height :12.0 m Sup. Length : 120 cm	Elastomeric laminated	R.C. Piled Soil Type :D
72. V4 Viaduct (1989)	Length:161.5 m 4 spans , (2×40.0) m (1×39.5) m (1×39.0) m Width : 22.0 m Ex. Joint : 2 Skew Angle: 0^0	Precast Pretensioned ; Simple Supported Box Girder	Abutments : Wall Piers:-H- Section (single) Pier Height : 21.0 m Abut. Height :19.0 m Sup. Length : 200 cm	Elastomeric laminated	Shallow Soil Type :B

(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.3-(contd) Inventory for the bridges under study

3.1.2 Classification

The bridges given in Table 3.3 were classified according to their type of superstructure, type of substructure, type of bearing and type of foundation as follows:

Superstructure : 6 different types of superstructures were identified. (see Table 3.4)

*Type	I	II	III	IV	V	VI
01 route	3	21	18	-	3	-
02 route	-	-	-	47	-	4
link routes	1	-	1	24	-	1
Max. Bridge Length(m)	861.0	81.4	69.2	402.0	411.0	788.8
Min. Bridge Length(m)	73.9	35.0	28.0	15.0	272.0	324.8
Max. Deck Width (m)	25.0	17.8	14.5	36.0	32.8	36.0
Min. Deck Width (m)	6.6	7.0	6.0	4.0	21.5	22.0
Max. Span Length(m)	-	-	-	40.4	45.0	58.0
Min. Span Length(m)	-	-	-	11.0	31.0	40.0
Max. Number of Span	-	4	3	10	9	14
Min. Number of Span	3	2	2	1	6	6
Max.Support Height (m)	10.6	-	-	40.5	43.0	70.5
Min.Support Height (m)	-	-	-	6.2	32.0	12.5

Table 3.4- Classification of superstructures

*Type I : Post-tensioned Continuous Box Girder,
Type II : Post-tensioned Continuous Plate Girder(Figure 3.2),
Type III : Reinforced Concrete Plate Girder,
Type IV : Precast, Prestressed, Simple Supported Box Beam (Figure 3.3),
Type V : Post-tensioned Simple Supported Girder,
Type VI : Precast, Prestressed Continuous Box Girder,

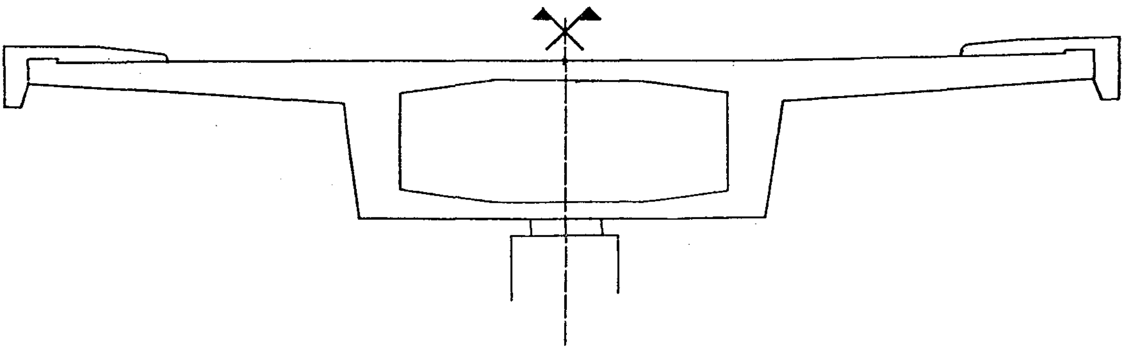

FIGURE 3.2 Typical cross-section of post tensioned continuous plate girder.

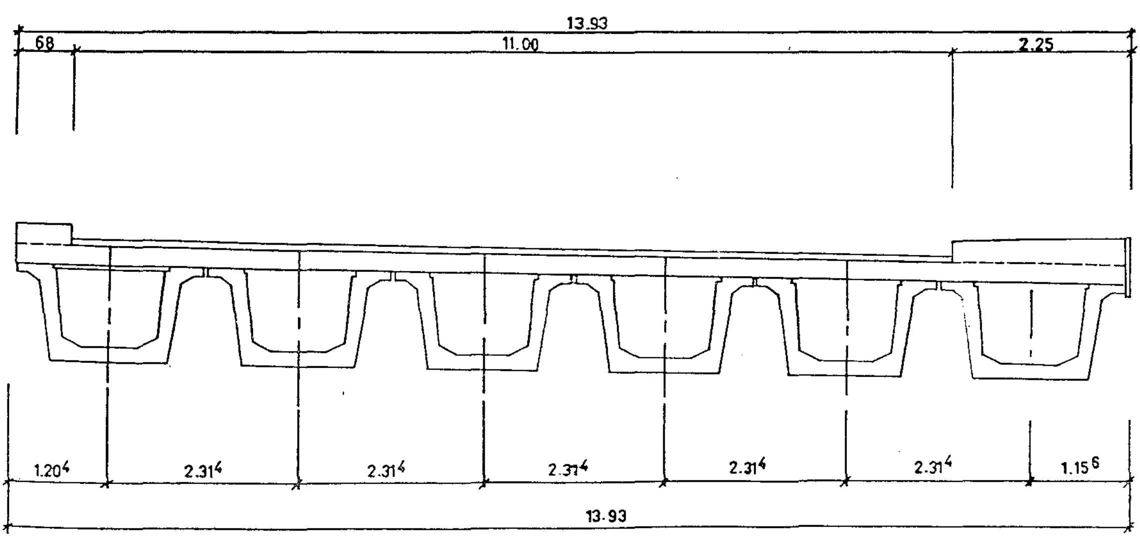

FIGURE 3.3 Typical cross-section of precast prestressed simple supported box beam.

Substructure :

Abutments : 2 types of abutments were commonly encountered in the study area :

Type I : Wall type (Figure 3.4)
Type II : Open type (Figure 3.5)

Piers : Piers which were encountered in the study area are:

Type I : Wall type (Figure 3.6)
Type II : Column type (Figure 3.7)

Bearings : Almost all the bridges in the study area have Elastomer Bearings of different types, namely Neopren, Freyssinet, Elastomer laminated, except a few Neotopf Bearings which are steel. A typical elastomeric laminated bearing is illustrated in Figure 3.8.

Foundation : The most common types of footings which were encountered in the study area are Piled Footings & Shallow Footings.

3.1.3 Details About Some Important Viaducts and Interchanges

O1 route contains 45 bridges. Most of the bridges have Post-tensioned Continuous Plate Girder type of superstructures (Table 3.4), a typical cross-section of which is given in Figure 3.2. On O2 route, all the bridges except four viaducts (which were constructed by incremental launching method), have Precast Prestressed Simple Supported box beam type of superstructures, a typical cross section of which is given in Figure 3.3. On the link roads, there are 27 bridges, 24 of which have Precast Prestressed Simple Supported box beam type of superstructure.

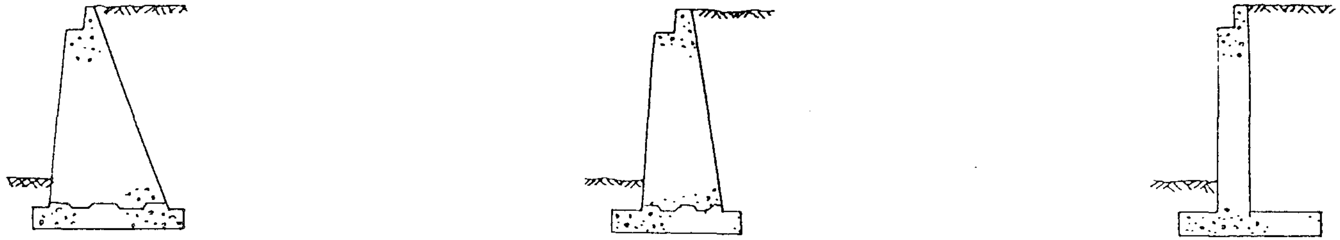

FIGURE 3.4 Wall type abutments.

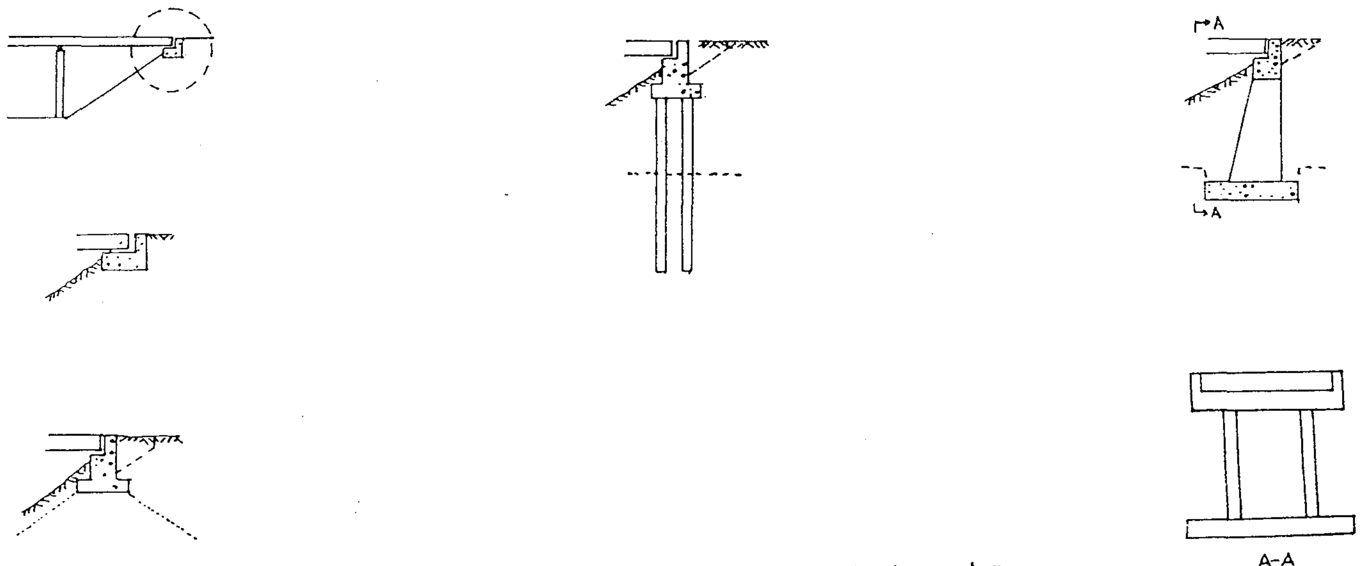

FIGURE 3.5 Open type abutments.

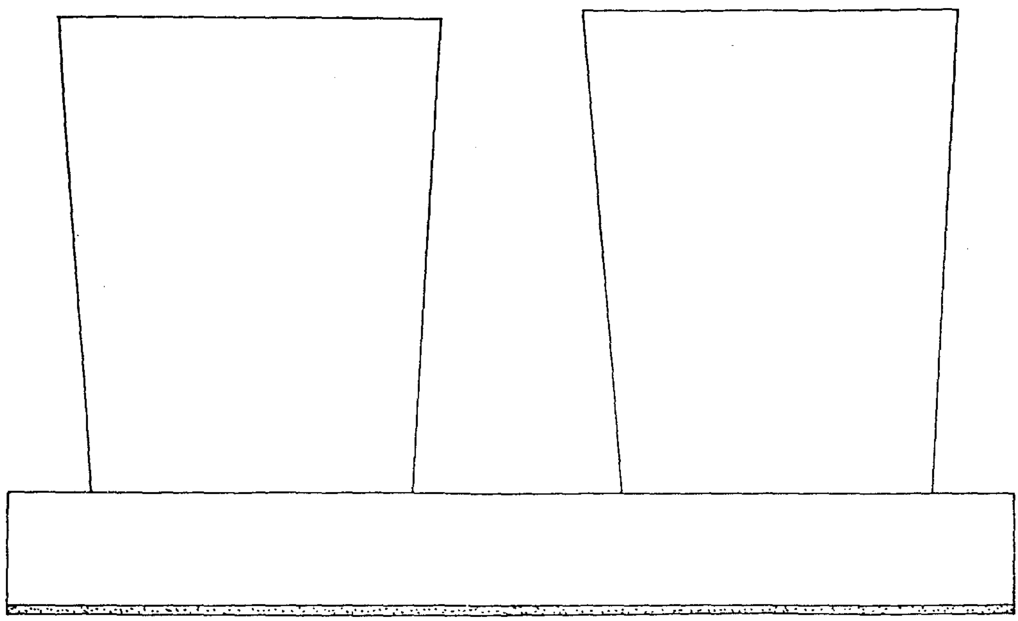

FIGURE 3.6 A typical wall type pier.

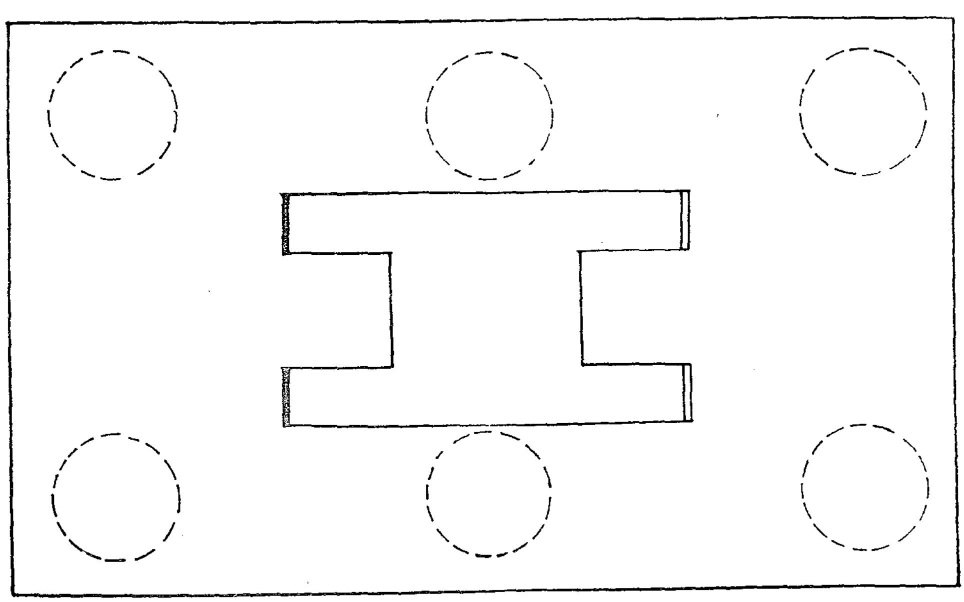

FIGURE 3.7 Plan view of a -H- section column.

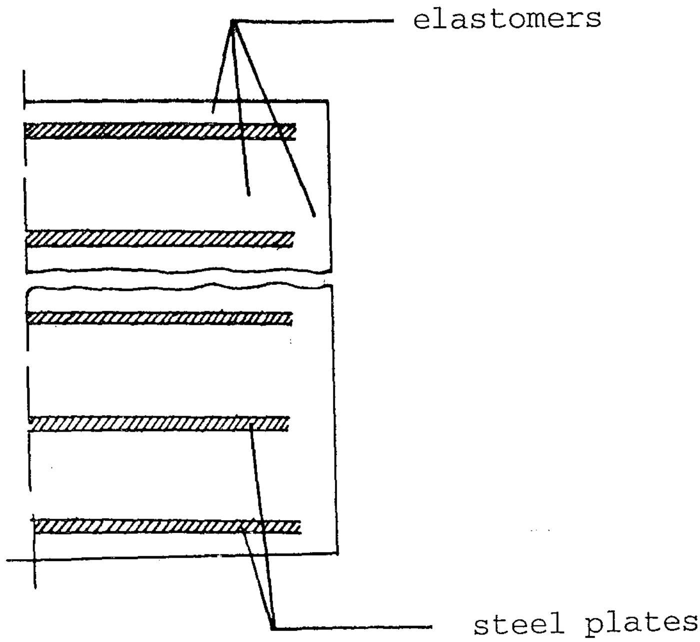

FIGURE 3.8 A typical elastomeric laminated bearing.

Viaducts :

Viaducts on 01 route

		Length
Ortaköy Viaduct	(V-408)	411.0 m
Ortaköy Viaduct	(V-409)	362.0 m
Beşiktaş Viaduct	(V-411)	272.0 m
Mecidiyeköy Viaduct	(V-302)	861.0 m

On 01 route, there are four viaducts (totaling to 1906 m of length):

The superstructure of "Ortaköy Viaducts (V-408),(V-409)" and the "Beşiktaş Viaduct (V-411)" consist of post-tensioned, precast- simple supported girders. On the other hand, the "Mecidiyeköy Viaduct (V-302)" has post-tensioned, continuous box section girders.

The abutments of (V-408),(V-409) and (V-411) viaducts are shear-wall type and the piers are consisted of concrete box-section columns. The viaduct (V302) has concrete box-section columns and abutments.

Beşiktaş Viaduct (V-411) is the tallest with a height of 43m and Mecidiyeköy Viaduct (V-302) is the longest with a length of 861 m.

Viaducts on 02 route

		Length
Mahmutbey Viaduct	(V7)	399.8 m
Gaziosmanpaşa Viaduct	(V7A)	120.0 m
Akşemsettin Viaduct	(V6)	604.0 m
Hasdal Viaduct	(V5)	324.8 m
Sadabat II Viaduct	(V1)	400.0 m
Levent Viaduct	(LMV1)	363.6 m
Molla Gürani Viaduct	(M3V1)	498.8 m

On the O2 route, there are 11 viaducts (totaling to 4301 m of length). Seven of which are located on the main route itself and the remaining four on Hasdal-Okmeydanı link road.

Viaducts on Hasdal Okmeydanı Link Road

		Length
Gedik Ahmet Paşa Viaduct	(V2A)	240.0 m
Nurtepe Viaduct	(V2)	400.0 m
Sadabad I Viaduct	(V3)	788.8 m
Okmeydanı Viaduct	(V4)	161.5 m

The "Akşemsettin", "Hasdal", "Sadabad I" and "Molla Gürani Viaducts" have a single continuous post-tensioned concrete box girder deck superstructure with cantilevers at the sides. They are constructed by incremental launching method. Main spans of those viaducts are 58.00 m and the side spans are 46.40 m long. "The concrete box girder deck" has a cross section of 7 m at the base and 10 m at the top. The height of the box is 5.05 m. The cantilevers are 4.5 m and 4.8 m long which are located at the two sides. The girder which has a 20 m deck width is cast in segments of 29 m. They were post-tensioned and launched forward.

The superstructure of the remaining viaducts consist of 5 "U" shaped precast prestressed 2 m high simple beams of 37 m span.

The piers of the majority of the viaducts have "H" cross section. Only Levent viaduct has circular piers and a few viaducts which have piers with a box cross-section.

Molla Gürani Viaduct is the tallest with a height of 77 m to the top of the deck and Sadabad I is the longest with a length of 788.8 m.

Interchanges :

There are 10 interchanges on 02 route, namely:

Mahmutbey, Metris, Kumburgaz, Hasdal, Military Academy campus, Büyükdere-Levent, Levent, Ümraniye, Anatolian and Kozyatağı interchanges.

In Table 3.5 the percentages of some bridge components for the bridges in the study area are given.

Components		Routes		
		route 01	route 02	link roads
Superstructure	Type I Type II Type III Type IV Type V Type VI	7 50 38 - 5 -	- - - 90 - 10	4 - 4 88 - 4
Abutment	Wall Type Open Type	88 12	94 6	75 25
Pier	Wall Type Column Type	88 8	74 3	8 17
Bearing	Neotopf Elastomeric laminated (Neopren, Freyssinet, Elastomeric)	4 96	- 100	- 100
Foundation	R.C. Piled Foundation Shallow Foundation	35 65	6 94	8 92

Table 3.5- Percentage of some bridge components in the study area

If we summarize the results obtained :

01 route (25 bridges)

- 100% of the bridges have a Continuous Superstructure,
- 44% of the bridges have 1 or no expansion joint,

- 56% of the bridges have 2 expansion joints,
- 24% of the bridges have straight alignment,
- 56% of the bridges are skewed with skew angle less than 20^0,
- 20% of the bridges are skewed with skew angle more than 20^0,
- 100% of the bridges are built before the adoption of an Earthquake Code,
- All the bridges have actual support lengths greater than minimum required by the Code,
- 64% of the bridges have shallow foundation,
- 36% of the bridges have piled foundation,

O2 route (34 bridges)

- 100% of the bridges have a Non-Continuous Superstructure,
- 71% of the bridges have 1 or no expansion joint,
- 26% of the bridges have 2 expansion joint,
- 3% of the bridges have 3 expansion joint,
- 33% of the bridges have straight alignment,
- 24% of the bridges are skewed with skew angle less than 20^0,
- 43% of the bridges are skewed with skew angle more than 20^0,
- 100% of the bridges are built after the adoption of an Earthquake Code,
- All the bridges have actual support lengths greater than minimum required by the Code,
- 94% of the bridges have shallow foundation,
- 6% of the bridges have piled foundation,

Link roads (13 bridges)

- 8% of the bridges have a Continuous Superstructure,
- 92% of the bridges have a Non-Continuous Superstructure,
- 23% of the bridges have 1 or no expansion joint,
- 61% of the bridges have 2 expansion joint,
- 16% of the bridges have 3 expansion joint,
- 33% of the bridges have straight alignment,
- 33% of the bridges are skewed with skew angle less than 20^0,
- 34% of the bridges are skewed with skew angle more than 20^0,

- 8% of the bridges are built before the adoption of an Earthquake Code,
- 92% of the bridges are built after the adoption of an Earthquake Code,
- All the bridges have actual support lengths greater than minimum required by the Code,
- 92% of the bridges have shallow foundation,
- 8% of the bridges have piled foundation,

3.2 Preliminary Screening Process

3.2.1 Introduction

The preliminary screening is used to obtain information and assess vulnerability of highway bridges depending on their structural elements, site, foundation, and importance. The screening process is also the starting point for a retrofitting program. An efficient and comprehensive retrofitting program requires that structures are first rated according to their need for seismic retrofitting by a preliminary screening process. A preliminary screening process is recommended for all bridges classified as Seismic Performance Category (SPC) B or greater (ATC-6-2, 1983). Bridges in SPC A generally do not have to be considered for seismic retrofitting. The detailed explanation for Seismic Performance Categories is given in Appendix C.

The preliminary screening process involves;

- An examination of bridge plans,

- Design specifications,

- Site factors,

For example, the site factors can be used to assign an importance rating to the bridge in terms of the following parameters;

- The distance of the bridge from the causitive faults,

- The probable earthquake magnitude,

- Maximum credible site acceleration,

- Soil conditions,

Currently in U.S.A, there are at least four agencies which have developed preliminary screening methodologies. These are;

- Caltrans (Maroney,1988)

- Applied Technology Council (ATC-6-2)

- Washington State Transportation Center, WSDOT (Babaei,et al.,1990)

- Illinois Department of Transportation, IDOT (Gilbert,1993)

A brief review for each of the above mentioned four methods is given below.

CALTRANS uses a risk algorithm to identify the structures most susceptible to collapse during a large earthquake. The attributes in this risk algorithm include the soil conditions, number and type of hinges, number of columns per bent, column height, skew, bridge length, abutment type, year of construction, traffic exposure, route type, detour length, and facility crossed. In a conceptual long-term ranking method, Caltrans defines the risk number as the multiplication of the weighted factors for seismicity, vulnerability, and importance.

ATC-6-2 method provides a preliminary screening as the first major step of the seismic retrofitting process that identifies and rates the bridges according to their need for seismic retrofitting. Three major components considered in seismic rating are seismicity, vulnerability and importance. The vulnerability rating depends on the most vulnerable components of a bridge identified as bearings, columns, piers, footings, abutments and foundations. The seismicity rating is directly related to the peak ground acceleration (PGA). The importance rating considers traffic exposure, detour length over and under the bridge, length and width of the bridge and function of the bridge following a major earthquake.

WSDOT provides a procedure with cost estimates for a seismic risk reduction program. Factors representing the importance and vulnerability of the bridge to seismic failure are considered in ranking. Attributes considered in the importance factor include route and utility carried by the bridge, routes crossed, traffic exposure of the route carried and crossed by the bridge, and the bridge as a structure. The last item adresses the worth of the structure as a ratio of the retrofit cost to the cost of a new structure.

IDOT uses a seismic risk method to rank bridges based on the need for a detailed seismic evaluation and potential retrofitting. Risk is expressed as the product of two components. The first is the probability of failure of a bridge which includes the characterization of seismic hazard, the probabilistic evaluation of structural failure, and the probabilistic evaluation of ground failure. The latter is the consequences of such a failure that is evaluated by a multi-attribute value function calibrated using acceptable tradeoffs among different measures. A priority score for each bridge is computed.

In the following Sections, the ATC-6-2 methodology (which was developed by the Applied Technology Council) and a Combined method (which was developed by Memphis State University; a combination of the above mentioned four methodologies) will be studied in detail.

3.2.2 ATC-6-2 Method for Preliminary Screening of Highway Bridges

The flow chart shown in Figure 3.9 illustrates the preliminary screening procedure as it applies to bridges in different Seismic Performance Categories.

To calculate the seismic rating of a bridge, consideration is given to structural vulnerability, seismicity of the bridge site, and bridge's importance as a vital transportation link. Each of these three areas are assigned a rating, weight, and score. The scores are added to arrive at an overall seismic rating according to the following procedure:

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

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

Vulnerability Rating : Vulnerability ratings are between 0 and 10. In general, 0 rating means a very low vulnerability to unacceptable seismic damage, 5 means a moderate vulnerability to collapse or a high vulnerability to loss of access, and 10 means a high vulnerability to collapse. The vulnerability rating is determined as a function of the vulnerability of the individual bridge components, as follows:

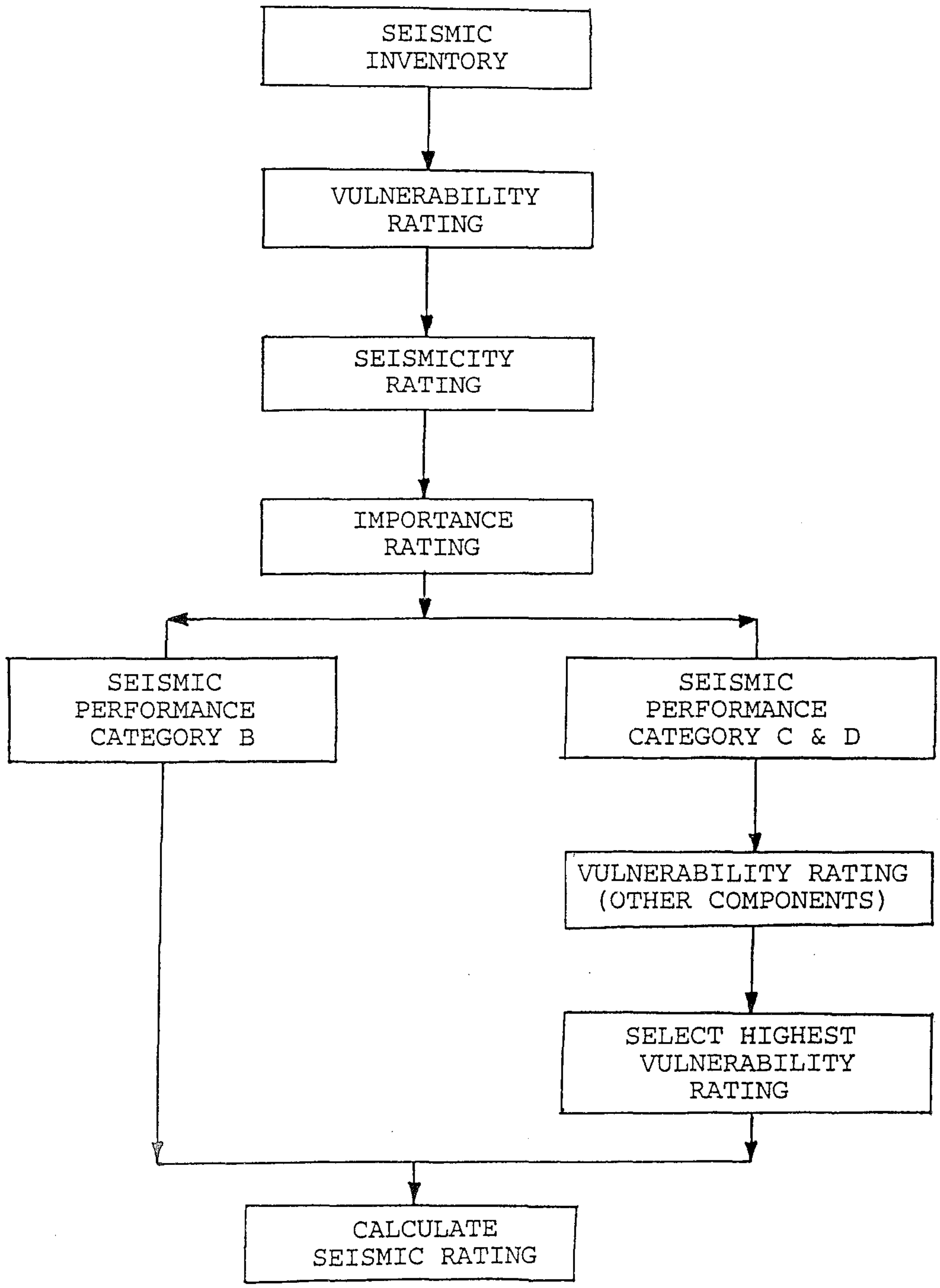

FIGURE 3.9 ATC preliminary screening process.

1- Vulnerability of bearings

2- Vulnerability of columns, piers and footings

3- Vulnerability of abutments

4- Liquefaction vulnerability of the bridge site

Seperate vulnerability ratings between 0 and 10 are assigned to both of these areas. The overall vulnerability rating of the bridge is taken as the greater of the above vulnerability ratings. The detailed explanation for the vulnerability rating of the above components is given in the Appendix D.

Seismicity Rating : Seismicity rating is obtained by multiplying the maximum acceleration coefficient by 25. If microzoning has been carried out within a jurisdiction, that jurisdiction may wish to modify the seismicity rating to yield a value of 10 at the maximum acceleration coefficient obtained from microzoning.

Importance Rating : In the Standard Specifications for Seismic Design of Highway Bridges (AASHTO) (see Appendix C) two importance classifications IC are specified. IC = I is for bridges defined as essential based on Social/Survival and Security/Defense requirements. IC = II is for all other bridges.

In the seismic rating system the importance ratings vary from 0 to 10. Bridges classified as essential IC = I may be assigned ratings between 6 to 10, while bridges classified as nonessential IC = II may have ratings between 0 to 5.

The goal of retrofitting is to minimize unacceptable damage, then the relative importance of a bridge is determined by considering the consequences of bridge failure during an earthquake. In the event of collapse of the bridge, the loss of life among individuals on or under the bridge is likely to be high. One factor which will affect the loss of life is the amount of traffic on or under the bridge at the time of the earthquake. This is likely to increase with the amount of traffic that crosses a given point during a period of time (e.g., average daily traffic) and the physical size of the bridge (e.g., length, number of lanes,etc.). Another factor that should be considered is the presence of the other facilities (e.g., buildings on, under, or near the bridge) that could be damaged or destroyed by the collapsing bridge.

The population density near the bridge site is also another important factor that it should include temporary population such as would occur in a business district. High population densities imply a concentration of people in a large number of large buildings and thus indicate a much larger potential casualty rate in the event of an earthquake. The proximity of the bridge to special types of facilities such as dams or nuclear power plants, whose failures have far-reaching consequences necessitating rapid evacuation, should also be considered in determining the importance rating.

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

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

3.2.3 Application of ATC-6-2 Method for Preliminary Screening of the Bridges in İstanbul

72 bridges were screened according to the ATC method. The overall scores are given in Table 3.6.

The weighting factor used was 3.33 for each type of rating.

Vulnerability rating : Four main components were evaluated according to their seismic vulnerability. The bearings were evaluated according to the continuity, and the skew angle of the superstructure and the minimum support length. The substructure (piers) were evaluated according to the Column Vulnerability Rating which depended upon the effective column length and percentage of the main reinforcement. The abutments were evaluated according to the fill settlement of the abutment and abutment height. Liquefaction vulnerability was evaluated according to the liquefaction susceptibility of the site, and the expected magnitude of the acceleration.

Seismicity rating : Seismicity rating was obtained by using seismic intensity (Figure 3.10) and geologic condition map (Figure 3.11) of Istanbul assuming an earthquake which will probably occur in Marmara sea.

Three different soil types were identified in Istanbul (Figure 3.11). These are:

Low Dense Soils: Alluvium, Sand, Gravel, Clay,
Medium Dense Soils: Limestone, Graywarke, Shale,
High Dense Soils: Volcanic,

The increase in the vulnerability of low dense soils during an earthquake was indirectly considered by increasing

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name	Vulnerability Rating				Seismicity Rating	Importance Rating	Total Score
	Bearings	Column	Abutment	LQ.			
1. K517	0	2	5	5	10	8.67	79
2. K515	0	2	0	5	10	8.67	79
3. K512	2	2	5	5	10	8.33	78
4. K510	0	0	0	0	6	8.67	49
5. K509	2	0	0	0	6	8.67	55
6. K505	2	0	0	3	6	8.33	58
7. K503	2	0	0	3	6	8.67	59
8. K501	3	0	0	3	6	8.33	58
9. K410	2	0	0	0	6	8.33	54
10. K414	0	0	0	0	6	8.67	49
11. K404	2	0	0	0	6	8.67	55
12. K402	2	0	0	0	6	8.67	55
13. K303	0	0	0	0	6	8.67	49(59)
14. K300	0	2	5	5	7	8.33	68
15. K205	2	2	5	5	7	9.00	70
16. K206	2	2	0	0	7	8.33	58
17. K204	0	2	5	0	7	8.33	68
18. K212	2	0	0	0	6	8.67	55
19. K207	2	0	0	0	6	8.33	54
20. K202	3	0	0	0	6	8.33	58
21. K104	3	0	0	5	7	8.67	69
22. K106	2	0	0	5	7	8.33	68
23. K103	0	0	0	5	7	8.33	68
24. K102	2	0	0	5	8	8.33	71
25. K101	2	0	0	5	8	8.67	72

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.6- The overall scores obtained from ATC Preliminary Screening Method

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name	Vulnerability Rating				Seismicity Rating	Importance Rating	Total Score
	Bearings	Column	Abutment	LQ.			
26. KMO1	5	3	5	5	8	8.33	71
27. KMU4	5	3	5	5	8	7.33	68
28. KMU3	5	3	5	5	8	7.67	69
29. M5U1	5	3	5	5	8	8.33	71
30. NMO1	5	3	5	7	8	8.00	76
31. NMU4	5	0	0	5	6	8.00	63
32. NMU3	5	0	0	5	6	8.00	63
33. M5U2	5	0	0	5	6	8.00	63
34. M501	5	0	0	5	6	8.33	64
35. RMO2	5	0	0	5	6	8.00	63
36. RMO1	5	0	0	5	6	8.00	63
37. M402	5	0	0	5	6	8.00	63
38. M401	5	0	0	5	6	8.33	64
39. VMO1	5	0	0	3	7	8.33	68
40. M301	5	0	0	3	6	8.67	65
41. M302	5	0	0	3	6	8.00	63
42. M2U2	5	0	0	0	6	8.00	63
43. M101	5	0	0	0	6	8.33	64
44. M102	5	0	0	0	6	8.00	63
45. BRO	5	3	5	5	7	8.33	68
46. İÇO	5	0	0	0	6	8.33	64
47. B14	5	3	5	5	7	8.00	67
48. B13	5	3	5	5	8	8.88	73
49. V1	5	0	0	5	8	9.00	73(83)
50. B12	5	3	5	7	7	8.33	74
51. B11	5	3	5	7	7	8.33	74
52. B10	5	0	0	0	6	8.33	64
53. B6	5	0	0	0	6	7.67	62
54. B5	5	0	0	0	6	7.67	62
55. B3	5	0	0	0	6	7.67	62
56. B3B	5	0	0	0	6	7.33	61
57. B3C	5	0	0	0	6	7.00	60
58. B2	5	0	0	0	6	7.00	60
59. B1	5	3	5	7	7	7.67	72

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.6-(contd) The overall scores obtained from ATC Preliminary Screening Method

HIGHWAY BRIDGES ON LINK ROADS

Bridge Name	Vulnerability Rating				Seismicity Rating	Importance Rating	Total Score
	Bearings	Column	Abutment	LQ.			
60. K1	5	0	0	0	6	8.00	63
61. K305	0	0	0	0	6	8.67	49(59)
62. M1U1	5	0	0	0	6	7.67	62
63. M1U2	5	0	0	0	6	7.67	62
64. L102	5	0	0	0	6	7.67	62
65. BLU1	5	0	0	0	6	7.67	62
66. B16	5	0	0	0	6	7.67	62
67. V2A	5	0	0	0	6	8.67	65(75)
68. B17	5	0	0	0	6	7.33	61
69. V2	5	0	0	0	6	8.67	65(75)
70. B19	5	0	0	0	6	8.00	63
71. B21	5	3	5	5	8	8.33	71
72. V4	5	0	0	0	6	9.33	68(78)

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)

Table 3.6-(contd) The overall scores obtained from ATC Preliminary Screening Method

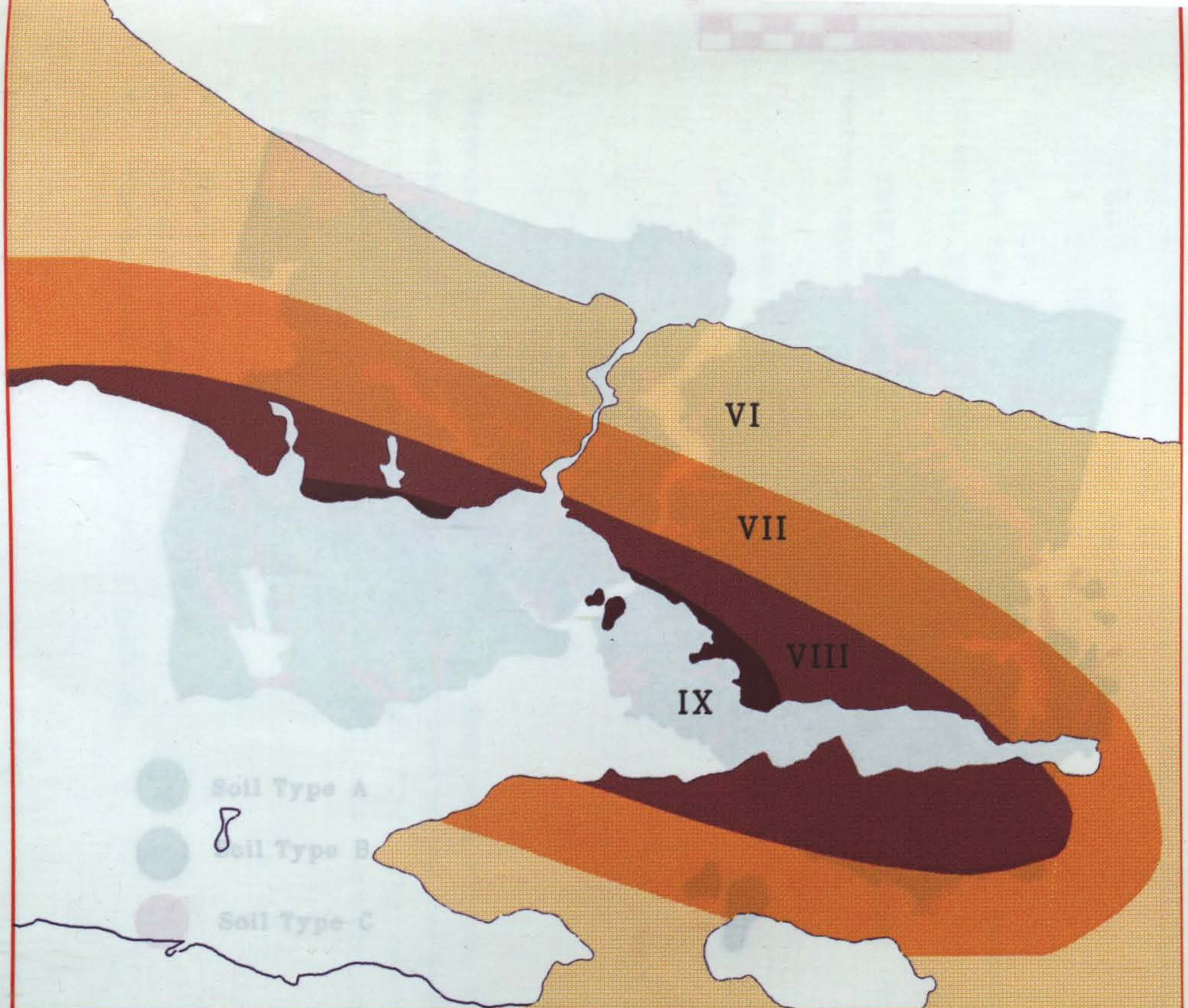

FIGURE 3.10 Seismic intensity map of Istanbul

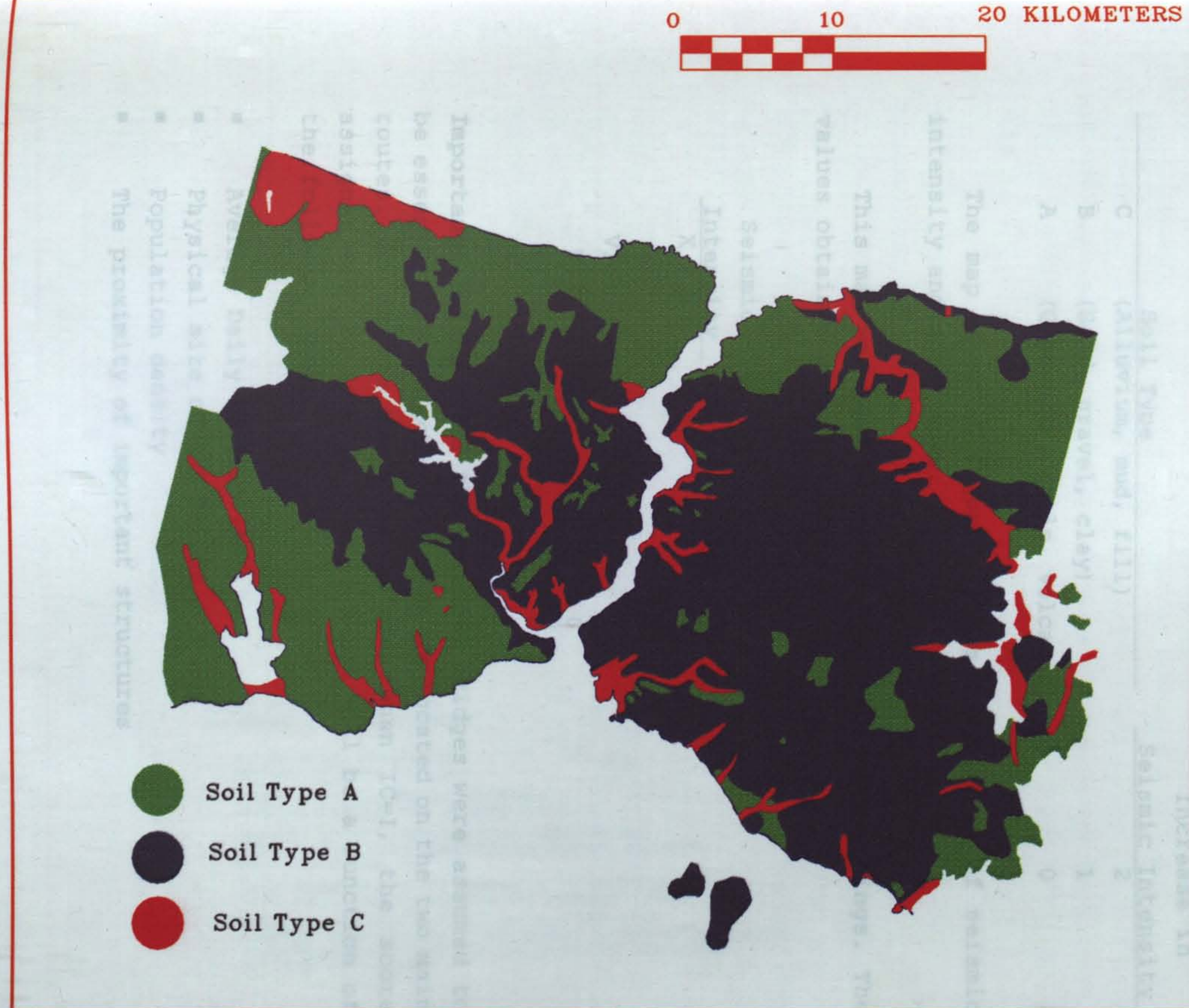

FIGURE 3.11 Geologic condition map of Istanbul

the seismic intensities. The increase in the seismic intensities as a function of the soil types were as follows:

	Soil Type	Increase in Seismic Intensity
C	(Alluvium, mud, fill)	2
B	(Sand, gravel, clay)	1
A	(Graywarke, shale, volcanic)	0

The map given in Figure 3.12 is combination of seismic intensity and soil types.

This map was used to assign the seismicity ratings. The values obtained were as follows:

Seismic Intensity	Seismicity Rating
X - XI	10
IX	8
VIII	7
VII	6
VI	5

Importance rating : All the screened bridges were assumed to be essential IC=I, since all of them are located on the two main routes. As it was mentioned before, when IC=I, the score assigned will be between 6 to 10 which will be a function of the following items:

- Average Daily Traffic (ADT)
- Physical size of the bridge
- Population density
- The proximity of important structures

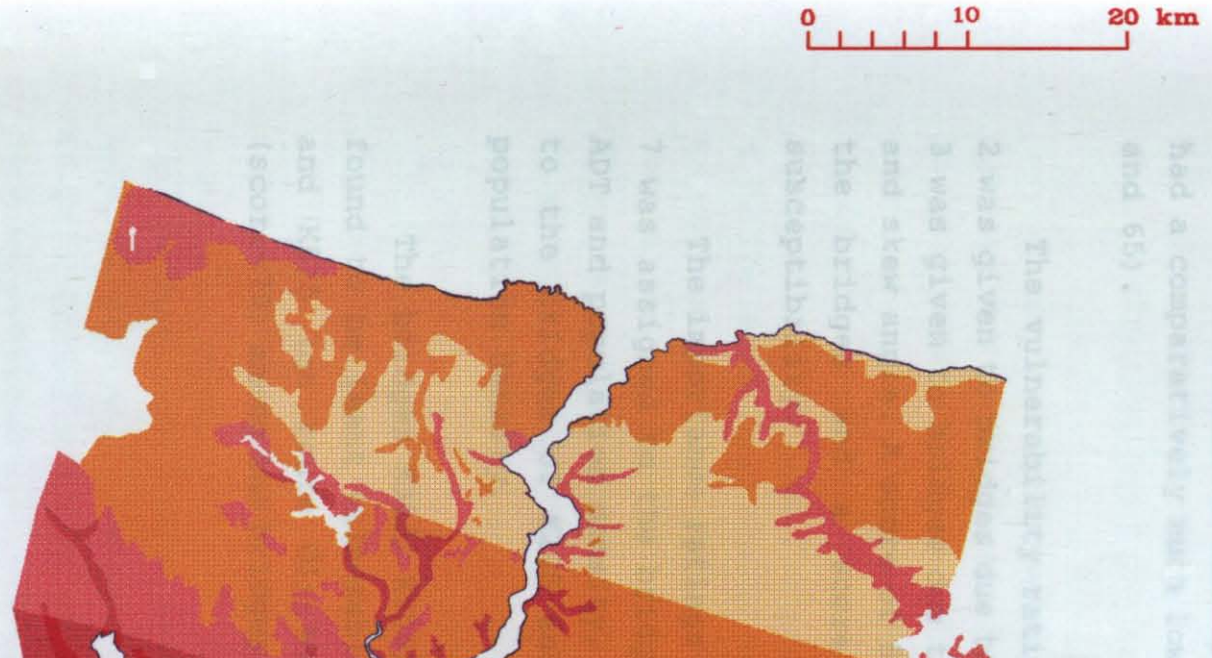

FIGURE 3.12 The combination of seismic intensity and geology condition map of Istanbul.

The overall preliminary screening scores obtained are given in Table 3.6. Higher score indicates the greater need for retrofitting the bridge for seismic action. As it is clear from Table 3.6 seismicity rating had a great influence on the total score. This may be illustrated on bridges of K512, K515, K517. Those bridges had a seismic intensity of X and they were located on alluvium soil. Their total scores which are the (highest scores encountered) are 78 and 79, whereas the bridges which had seismic intensity of VII and located on rock soil, had a comparatively much lower total score (changes between 49 and 65).

The vulnerability rating varied from 2 to 7. A score of 2 was given to bridges due to their number of expansion joints, 3 was given to bridges due to their number of expansion joints and skew angle. A score of 5 represented the discontinunity in the bridges and a score of 7 represented liquefaction susceptibility.

The importance ratings varied between 7 and 9. A score of 7 was assigned to the bridges B3C and B2, since they had low ADT and population density, whereas a score of 9 was assigned to the bridges of K205 and V1, due to their high ADT and population density.

The bridges K303, K305, K414 and K510 (on route O1) were found to be least vulnerable (score=49) and the bridges K517 and K515 (on route O1) were found to be most vulnerable (score=79) among the bridges screened according to this method.

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3.2.4 Combined Method

This method is developed by Memphis State University (Pezeshk,S., 1994). The method combines several methodologies which were mentioned in Chapter 3.2. In this method, to determine the seismic rating of a bridge, the following items are taken into account:

- Vulnerability of structural characteristics,
- Importance of the bridge as a vital transportation link,
- Foundation and site charecteristics,

Final rating is accomplished by adding the independent ratings for each of the above mentioned items. Each item is also divided into several subitems each of which has an assigned score. These scores are then summed up to arrive at an overall preliminary seismic rating. For this purpose, a "Preliminary Seismic Screening Form" is used as shown in Table 3.7. The Form consists of three major parts:
A- Structure (50 pts), B- Importance (20 pts), and C- Foundation and Site (30 pts), with a total index score of 100. The score of each criterion is determined on the basis of its relation to the effect of seismic damage caused by a moderately strong earthquake.

In this method, in contrary to ATC method, an increase in score corresponds to a decrease in vulnerability.

3.2.4.1 Vulnerability of Structural Characteristics

Recent earthquakes have indicated that the most severe type of structural damage on highway bridges is the loss of support of the girders, which, in general, was caused by the lack of continuity in the superstructure. The other causes of damage were: inadequate support lengths for the girders, skew supports, or gross movements of the superstructure at the supports due to some form of soil failure under the piers or abutments (Mayes,R.L.,1987). Accordingly the following criteria are evaluated:

Superstructure : The damage on the highway bridges during past earthquakes has shown that bridges with continuous superstructure behaved better during earthquakes since their supports could withstand large translational deformations (ATC-6-2). However, bridges with discontinuous superstructure and/or brittle supporting members were usually severely damaged especially when liquefaction took place under the piers and abutments. Accordingly, a score of 5 was assigned to bridges with continuous superstructure and 0 for bridges with discontinuous superstructure.

Number of Expansion Joints : The expansion joints increase the discontinunity of the superstructure, and affect the overall stability of the bridge. Hence, the number of expansion joints was taken into account for scoring. A score of 5 was assigned for bridges with one or no expansion joints, 4 for two or three expansion joints, and 3 for four or more expansion joints.

Type of Bearing : Bearings are the most vulnerable components of highway bridges during an earthquake. Four basic types of bearings used in the bridge construction are rocker, roller, elastomeric bearing pad, and sliding. The rocker bearing, is generally constructed of steel and rolls on a curved surface to provide for translation and/or rotational movement. It is the most seismically vulnerable among the four types since usually it has a large vertical dimension, is difficult to restrain, and can become unstable after a limited movement. Another type of bearing, the roller bearing, is also usually constructed of steel. It is stable during an earthquake, except that it can become misaligned and horizontally displaced. The third type is the elastomeric bearing pad. It is constructed of natural or synthetic elastomer. The final bearing type is the sliding bearing which relies on the sliding of one surface over another.

Accordingly a score of 0 is assigned for rocker bearings, 1 for roller bearings, 4 for elastomeric bearings and 5 for sliding bearings.

Alignment : The alignment of a bridge is a criteria which has a major effect on the performance of its bearings during an earthquake. Skew is defined as the angle between the support centerline and a line perpendicular to the bridge centerline. If the bridge has skew, the predominant mode of vibration changes into a rotation of the superstructure about a vertical axis (Imbsen,R.L.,1987). Such a rotation causes unequal distribution of forces on the bearings which are not usually accounted for in the design. Depending on the skew angle, a bridge is assigned by a low rating of 1 to 5 points.

Age : The age of a bridge was considered as a criteria to show whether the bridge construction was before or after the adoption of an earthquake code. A score of 10 for the bridges built after the adoption of the earthquake code and a score of 0 for those built before were assigned.

Regular vs Irregular : In this criteria, bridges were classified as regular or irregular. A bridge is called irregular if it has large changes in stiffness, in pier heights, or in superelevations. If a bridge is identified as irregular, then it is assigned with zero points.

Pier Height : A pier height taller than 7m is considered to be susceptible to damage during an earthquake. Accordingly a score of 5 for the pier heights between the 0-7m, and zero for the pier heights more than 7m were assigned.

Minimum Support Length : As it is explained in Appendix A (AASHTO, Standard Specifications for Seismic Design of Highway Bridges), the minimum support length of the bearing seats supporting the unrestrained expansion ends of girders must be greater than the support length provided (ATC-6-2,1983).

Otherwise, during an earthquake there is a chance that the bearing seat will lose support, resulting in failure of the superstructure. Thus a score of 10 was assigned for bridges where the minimum support length exceeded the actual support length, and a score of zero was assigned otherwise.

3.2.4.2 Importance of the Bridge as a Vital Transportation Link

The importance of the bridge as a vital transportation link plays a major role in deciding whether retrofitting is essential or not. Detour Length and Average Daily Traffic were considered to be two main factors which govern the Importance criteria:

Detour Length : The Detour Length reflects the length of the route leading to the nearest bridge adjacent to the target bridge. A score of 10 for bridges with detour lengths less than 3.5 km, 5 for those with detour lengths between 3.5 km and 7 km, zero for those with detour lengths more than 7 km were assigned.

Average Daily Traffic : The Average Daily Traffic is the other significant criteria which determines the potential loss of human life on the structure. A score of 10 for ADT less than 2000 vehicles, 5 for ADT between 2000-10000 vehicles, zero for ADT more than 10000 vehicles were assigned.

3.2.4.3 Foundation and Site Charecteristics

The excessive ground deformations, loss of stability and bearing capacity of the foundation soils are the most common failure types of the foundations. Substructures often tilt, settle, slide, or even overturn (Mayes,R.L.,1987). Scoring for the foundation and site charecteristics were based on soil conditions, seismicity of the site and abutment heights:

Soil Condition : The Soil condition has an influential effect on the duration and amplitude of the ground shaking. In Istanbul, as it is seen from the map given in Figure 3.10, 3 different soil types were identified. Hence, a score of 10 for soil types A; a score of 5 for soil type B, and a score of zero for soil type C were assigned.

Seismicity of the Site : Seismicity scoring was considered to be the same as obtained in ATC method seismicty rating (Chapter 3.2.3). Since, in the combined method, in contrary to ATC method a lower seismic rating score indicates greater need for seismic retrofitting, the seismicity rating was rearranged as follows:

Seismic Intensity	Seismicity Rating
X - XI	0
IX	2
VIII	3
VII	4
VI	5

Abutment Height : Tall abutments may lead to the failures such as tilting, settling, and shearing. Accordingly, a score of 10 for abutment heights between 0-5 m, 5 for abutment heights between 5-10 m, and zero for abutment heights exceeding 10 m were assigned.

STRUCTURAL CHARACTERISTICS					Final Score
Superstructure (5pts)	Continuous 5	Non-Continuous 0			
Number of Expansion Joints (5pts)	<=1 5	2 4	3 4	>=4 3	
Bearing Type (5pts)	Rocker 0	Roller 1	Elastomeric 4	Sliding 5	
Alignment (5pts)	Straight 5	Skewed(<20^0) 4	Skewed(>20^0) 1	Curved 1	
Age (10pts)	Before Adoption of EQ Code 0	After Adoption of EQ Code 10			
Regular vs Irregular (5pts)	Regular 5	Irregular 0			
Pier Height (5pts)	<7 m 5	>7 m 0			
Actual Support Length > Minimum Required Support Length (10pts)	Yes 10	No 0			
Final Structural Score					

IMPORTANCE AS A VITAL LINK				Final Score
Detour Length (10pts)	>4 miles 0	2-4 miles 5	<2 miles 10	
ADT (10pts)	<2000 10	2000-10000 5	>10000 0	
Final Importance Score				

FOUNDATION AND SITE CHARACTERISTICS						Final Score
Soil Condition (10pts)	A 10	B 5	C 3			
Seismicity of the site (10pts)	X-XI 0	IX 2	VIII 3	VII 4	VI 5	
Abutment Height(m) (10pts)	0-5 10	5-10 5	>10 0			
Final Foundation Score						

Table 3.7- The Preliminary Seismic Screening Form

3.2.5 Application of the Combined Method to Bridges in Istanbul

As it is clear from Table 3.7, the total score (100 points) associated with a bridge consists of three components:

a) Structure (50 pts)
b) Importance (20 pts)
c) Foundation and Site Characteristics (30 pts)

A total score of 100 represents a perfect bridge whereas zero score represents a bridge that has no resistance to earthquakes.

In Istanbul, 72 bridges on O1, O2 and link routes (25 bridges on O1; 34 bridges on O2; 13 bridges on link routes) were screened according to the Combined Method. The total score obtained for each bridge is given in Table 3.8.

If we summarize the results obtained :

- The lowest screening score was 35 (NMO1 on O2),
- The highest screening score was 68 (K101 on O1, K414 on O1, and K509 on O1),
- 30% of the bridges had a score less than 50,

In the Combined method, the bridges which have total scores under 50 may be described as, bridges with discontinuous superstructure, irregular geometry, and soil type C. On the other hand, the bridges which had a total score over 50 were, bridges with continuous superstructure, regular geometry, and constructed on soil types A and B.

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name	Structural Characteristics								Importance		Site			Total Score
	a	b	c	d	e	f	g	h	i	j	k	l	m	
1. K517	5	5	4	4	0	5	0	10	10	0	0	0	5	48
2. K515	5	5	4	4	0	0	0	10	10	0	0	0	10	48
3. K512	5	5	4	4	0	0	0	10	10	0	0	0	5	43
4. K510	5	4	4	5	0	0	0	10	10	0	10	4	10	62
5. K509	5	4	4	4	0	5	0	10	10	0	10	4	10	66
6. K505	5	5	4	5	0	0	0	10	10	0	5	4	10	58
7. K503	5	5	4	4	0	5	0	10	10	0	5	4	10	62
8. K501	5	4	4	1	0	5	0	10	10	0	10	4	0	53
9. K410	5	5	4	1	0	0	0	10	10	0	10	4	10	59
10. K414	5	4	4	4	0	5	0	10	10	0	10	4	10	66
11. K404	5	4	4	4	0	0	0	10	10	0	10	4	10	61
12. K402	5	4	4	4	0	0	0	10	10	0	10	4	10	61
13. K303	5	4	4	5	0	0	5	10	10	0	10	4	10	67(57)

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)
a-Superstructure; b-Number of Expansion Joints; c-Type of Bearing; d-Alignment; e-Age; f-Regular vs Irregular; g-Pier Height; h-Minimum Support Length; i-Detour Length; j-Average Daily Traffic; k-Soil Condition; l-Seismicity of the Site; m- Abutment Height;

Table 3.8- The Overall Scores Obtained From Combined Method

HIGHWAY BRIDGES ON O1 ROAD

Bridge Name	Structural Characteristics								Importance		Site			Total Score
	a	b	c	d	e	f	g	h	i	j	k	l	m	
14. K300	5	5	4	1	0	5	0	10	10	0	0	3	5	48
15. K205	5	4	4	5	0	5	0	10	10	0	0	3	5	51
16. K206	5	5	4	4	0	0	0	10	10	0	0	3	10	51
17. K204	5	4	4	5	0	5	0	10	10	0	0	3	5	51
18. K212	5	5	4	4	0	0	0	10	10	0	10	4	5	57
19. K207	5	5	4	4	0	5	0	10	10	0	10	4	10	67
20. K202	5	4	4	1	0	0	0	10	10	0	10	4	10	58
21. K104	5	4	1	1	0	5	0	10	10	0	0	3	10	49
22. K106	5	4	4	4	0	0	0	10	10	0	5	3	10	55
23. K103	5	4	4	4	0	5	0	10	10	0	5	3	10	60
24. K102	5	5	4	4	0	5	0	10	10	0	5	2	10	60
25. K101	5	4	1	5	0	5	5	10	10	0	5	2	10	62

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)
a-Superstructure; b-Number of Expansion Joints; c-Type of Bearing; d-Alignment; e-Age; f-Regular vs Irregular; g-Pier Height;
h-Minimum Support Length; i-Detour Length; j-Average Daily Traffic; k-Soil Condition; l-Seismicity of the Site; m- Abutment Height;

Table 3.8-(contd) The Overall Scores Obtained From Combined Method

HIGHWAY BRIDGES ON O2 ROAD

Bridge Name	Structural Characteristics								Importance		Site			Total Score
	a	b	c	d	e	f	g	h	i	j	k	l	m	
26. KMO1	0	5	4	1	10	0	0	10	10	0	0	2	0	37
27. KMU4	0	5	4	1	10	0	0	10	10	0	0	2	5	42
28. KMU3	0	5	4	5	10	0	0	10	10	0	0	2	5	46
29. M5U1	0	5	4	5	10	0	0	10	10	0	0	2	0	41
30. NMO1	0	5	4	1	10	0	0	10	10	0	0	2	0	37
31. NMU4	0	4	4	4	10	0	0	10	10	0	10	4	0	51
32. NMU3	0	4	4	1	10	5	0	10	10	0	10	4	0	53
33. M5U2	0	4	4	4	10	5	0	10	10	0	10	4	5	61
34. M501	0	5	4	4	10	5	0	10	10	0	5	4	10	62
35. RMO2	0	5	4	4	10	5	0	10	10	0	5	4	5	57
36. RMO1	0	5	4	1	10	0	0	10	10	0	10	4	0	49
37. M402	0	5	4	5	10	5	0	10	10	0	5	4	5	58
38. M401	0	5	4	5	10	0	0	10	10	0	0	4	0	48
39. VMO1	0	5	4	5	10	0	0	10	10	0	0	3	0	46
40. M301	0	4	4	1	10	5	0	10	10	0	10	4	5	58
41. M302	0	5	4	5	10	0	0	10	10	0	10	4	10	63

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)
a-Superstructure; b-Number of Expansion Joints; c-Type of Bearing; d-Alignment; e-Age; f-Regular vs Irregular; g-Pier Height;
h-Minimum Support Length; i-Detour Length; j-Average Daily Traffic; k-Soil Condition; l-Seismicity of the Site; m- Abutment Height;

Table 3.8- (contd) The Overall Scores Obtained From Combined Method

Bridge Name	Structural Characteristics								Importance		Site			Total Score
	a	b	c	d	e	f	g	h	i	j	k	l	m	
42. M2U2	0	4	4	1	10	5	0	10	10	0	10	4	5	58
43. M101	0	5	4	5	10	0	0	10	10	0	10	4	0	53
44. M102	0	4	4	1	10	0	0	10	10	0	10	4	0	48
45. BRO	0	4	4	5	10	0	0	10	10	0	0	3	0	41
46. İÇO	0	4	4	1	10	0	0	10	10	0	10	4	0	48
47. B14	0	5	4	4	10	0	0	10	10	0	0	3	0	41
48. B13	0	4	4	5	10	0	0	10	10	0	0	2	0	40
49. V1	0	4	4	5	10	0	0	10	10	0	0	2	10	50(40)
50. B12	0	5	4	1	10	0	0	10	10	0	0	3	0	38
51. B11	0	5	4	1	10	0	0	10	10	0	0	3	0	38
52. B10	0	5	4	1	10	0	0	10	10	0	10	4	0	49
53. B6	0	5	4	4	10	5	0	10	10	0	10	4	0	57
54. B5	0	5	4	4	10	5	0	10	10	0	10	4	5	62
55. B3	0	5	4	1	10	5	0	10	10	0	10	4	10	64
56. B3B	0	5	4	1	10	5	0	10	10	0	10	4	0	54
57. B3C	0	5	4	5	10	5	0	10	10	0	10	4	5	63
58. B2	0	5	4	5	10	0	0	0	10	0	10	4	0	43
59. B1	0	5	4	1	10	5	0	10	10	0	0	3	5	48

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)
a-Superstructure; b-Number of Expansion Joints; c-Type of Bearing; d-Alignment; e-Age; f-Regular vs Irregular; g-Pier Height; h-Minimum Support Length; i-Detour Length; j-Average Daily Traffic; k-Soil Condition; l-Seismicity of the Site; m- Abutment Height;

Table 3.8- (contd) The Overall Scores Obtained From Combined Method

HIGHWAY BRIDGES ON LINK ROADS

Bridge Name	Structural Characteristics								Importance		Site			Total Score
	a	b	c	d	e	f	g	h	i	j	k	l	m	
60. K1	0	4	4	5	10	5	0	10	10	0	10	4	5	62
61. K305	5	5	1	5	0	0	0	10	10	0	10	4	10	60(50)
62. M1U1	0	4	4	1	10	0	0	10	10	0	10	4	0	48
63. M1U2	0	4	4	1	10	5	0	10	10	0	10	4	5	58
64. L102	0	4	4	1	10	0	0	10	10	0	10	4	5	53
65. BLU1	0	4	4	1	10	0	0	10	10	0	10	4	5	53
66. B16	0	5	4	4	10	5	0	10	10	0	10	4	0	57
67. V2A	0	4	4	5	10	0	0	10	10	0	10	4	0	52(42)
68. B17	0	4	4	4	10	5	0	10	10	0	10	4	0	56
69. V2	0	4	4	5	10	0	0	10	10	0	10	4	0	52(42)
70. B19	0	5	4	1	10	0	0	10	10	0	10	4	0	49
71. B21	0	4	4	4	10	0	0	10	10	0	0	2	0	39
72. V4	0	4	4	0	10	0	0	10	10	0	10	4	0	47(37)

( )- The value inside the paranthesis indicates the increased vulnerability due to the single column piers.
(Bridge No's 1-25 on O1 peripheral motorway, 26-59 on O2 peripheral motorway and 60-72 on link roads)
a-Superstructure; b-Number of Expansion Joints; c-Type of Bearing; d-Alignment; e-Age; f-Regular vs Irregular; g-Pier Height; h-Minimum Support Length; i-Detour Length; j-Average Daily Traffic; k-Soil Condition; l-Seismicity of the Site; m- Abutment Height;

Table 3.8- (contd) The Overall Scores Obtained From Combined Method

3.3 Detailed Evaluation Procedure : A Case Study

Detailed evaluation for retrofitting includes the determination of the seismic Capacity/Demand (C/D) ratios for individual bridge components. A C/D ratio less than 1.0 indicates that component failure may occur during the design earthquake and retrofitting may be appropriate.

In the detailed evaluation process, C/D ratios of four main components are determined. These are:

I- C/D ratios for bearings
II- C/D ratios for reinforced concrete columns, piers and footings
III- C/D ratios for abutments
IV- C/D ratios for liquefaction induced foundation failure

The detailed treatment of the methodology according to ATC-6-2 for determining the C/D ratios of above components are given in Appendix E.

The highway bridges in Istanbul were categorized into 6 in terms of their superstructure properties. As it is clearly seen from the Table 3.4, the majority of the bridges fall into the Category D which is a bridge with a Precast, Prestressed, Simple Supported Box Girder Superstructure. Thus for this case study such a bridge, (K1) was selected on O2 route. The location of the bridge is approximately marked on the map given in Figure 3.13. Total length of the bridge is 207.55 m and it has 8 simply supported spans. The length of the spans are 26.05 m in the mid-spans and 25.625 m in the side-spans. The columns of the piers have circular cross-section with a diameter of 1.65 m and the columns of the abutments have square cross-section with a dimension of 1.5 m. The maximum height of the piers is 12.0 m. The basic structural configuration and dimensions of the K1 bridge are given in Figure 3.14.

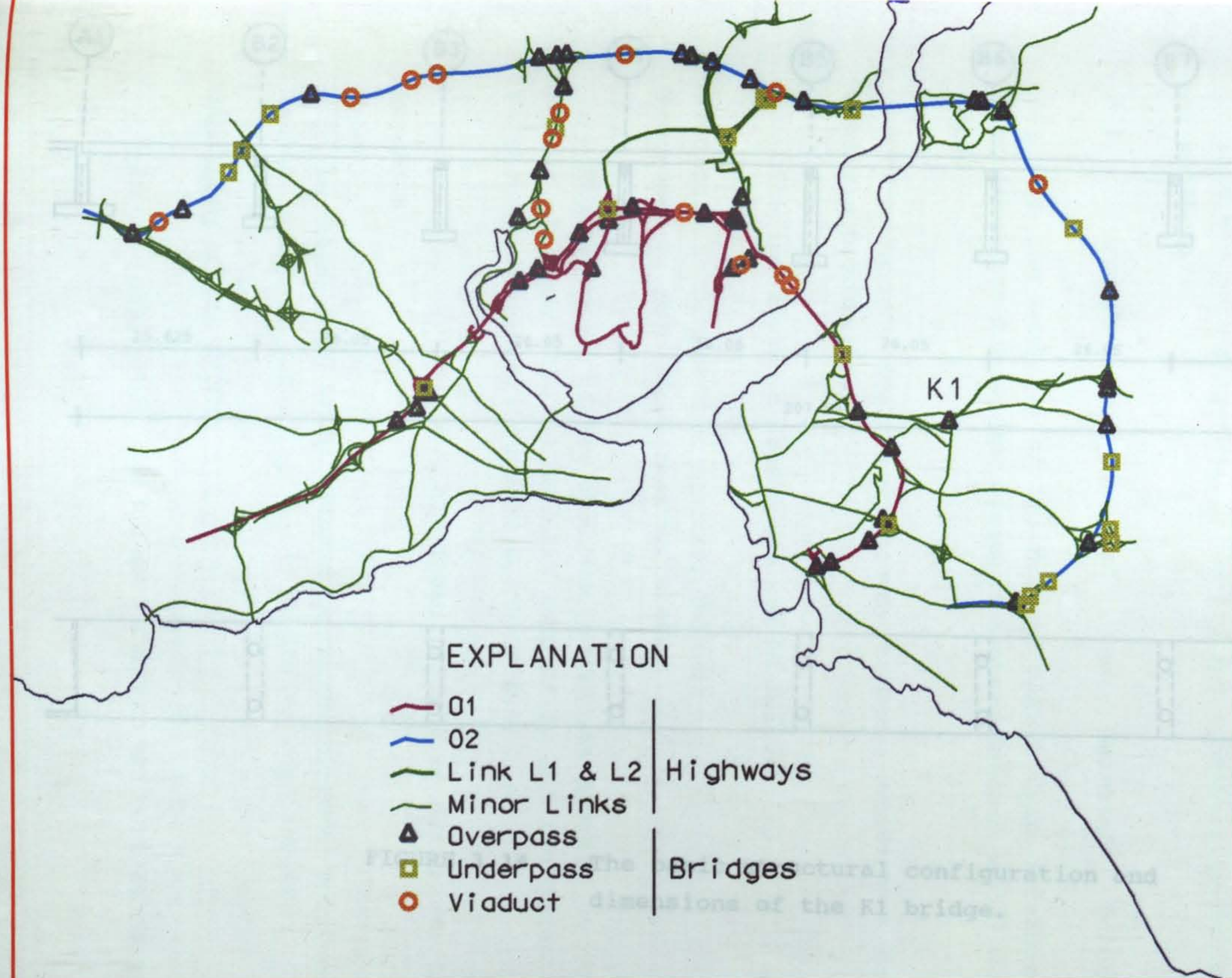


FIGURE 3.13 Location of the -K1- bridge.

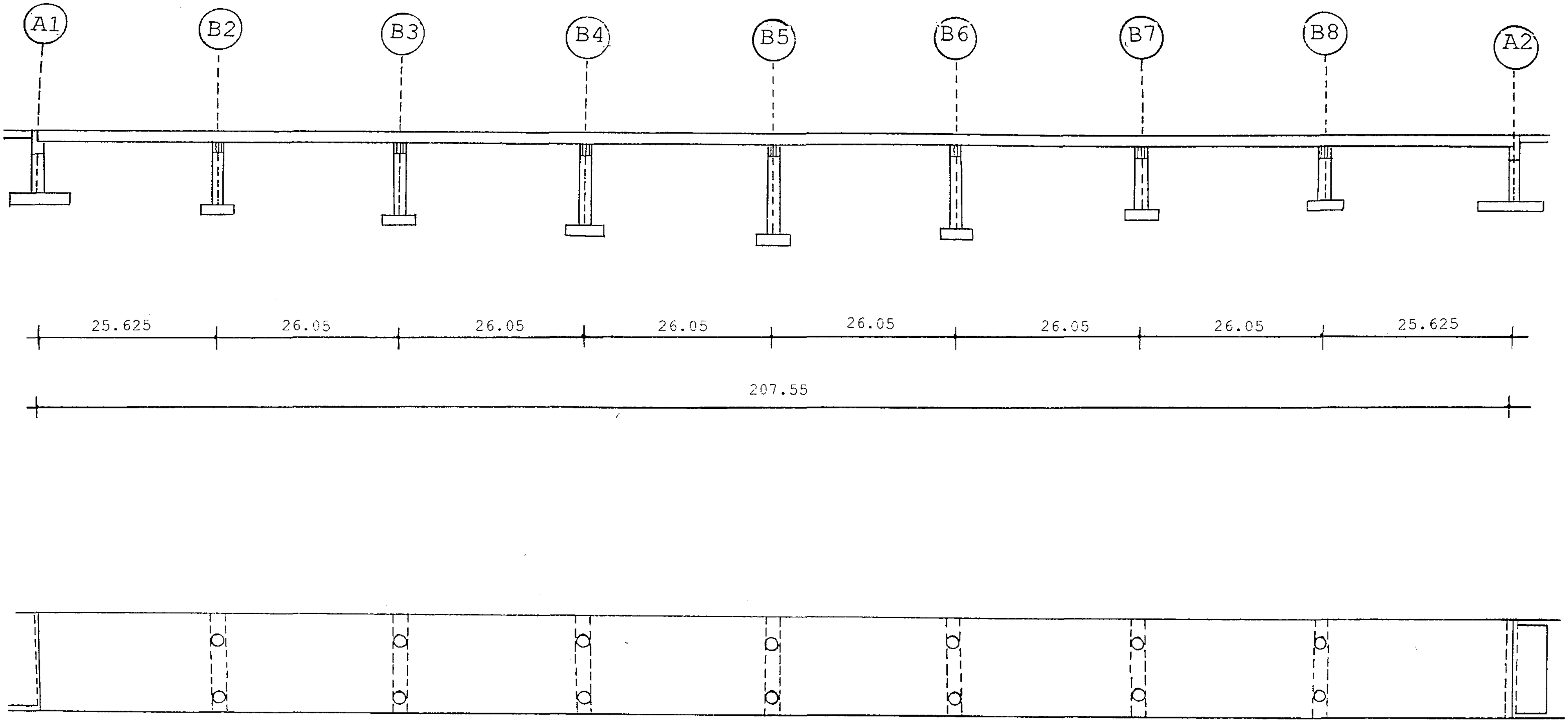


FIGURE 3.14 The basic structural configuration and dimensions of the K1 bridge.

The bridge being free of heavy traffic played an important role for this selection. Since it was then, possible to determine the dynamic properties of the bridge experimentally by truck testing, as explained in Chapter 4.

It was also taken into account that, the selected highway bridge is on a comparatively critical roadway and is expected to maintain both structural integrity and accesibility during an earthquake.

3.3.1 Seismic Analysis of the Selected Bridge

For the analysis of the bridge, AASHTO Code (Standard Specifications for Seismic Design of Highway Bridges, 1991) was used.

For Seismic Design of Highway Bridges, there are two analysis procedures.

The two analysis procedures are as follows:

Procedure 1: Single-Mode Spectral Method
Procedure 2: Multi-Mode Spectral Method

The selection of the analysis procedures depends on the number of spans, the geometrical complexity and the Seismic Performance Category (Appendix C, Table C.4)

Details of the above mentioned procedures are given in Appendix C.

In this case study, Procedure 2 (Multi-Mode Spectral Method) was used according to the following requirements:

Modelling of the Bridge

In this case study, the bridge structure was modeled using

SAP90 computer program with "Frame" elements. A three dimensional model was used with longitudinal elements representing the superstructure and the roadway, and vertical elements representing the piers and the supports, and transverse elements representing the piercaps. The Frame element model of the K1 bridge is shown in Figure 3.15.

Appropriate section properties were assigned to the Frame elements to represent the total effective stiffness properties of superstructure and substructure members.

A listing of the input data file is given in Appendix F. The consistent set of units used throughout the input file are kN, meters, and seconds, and the corresponding value for the gravitational acceleration is 9.81 m/s^2.

Superstructure : The superstructure was modeled as a single line of 54 Frame elements running along the neutral axis of the superstructure box beam. The section properties of these elements are chosen to represent the full width and depth of the box beam.

The local 2-axis is in the transverse direction which is parallel to the global Y-axis. Each span contains five roadway elements.

The translational mass for each Frame element is lumped at the joints. AASHTO,1983 recommends the location of the joints, minimum at the quarter span points.

Bearings : The bearings were modeled as frame elements between the Superstructure and the Pier Cap. The stiffness of these frame elements were assigned as the stiffnesses of the bearings. The top joint of the bearing element was connected to superstructure element and the bottom joint of the bearing element was connected to a stiff-connection element as shown in Figure 3.15 (Detail A).

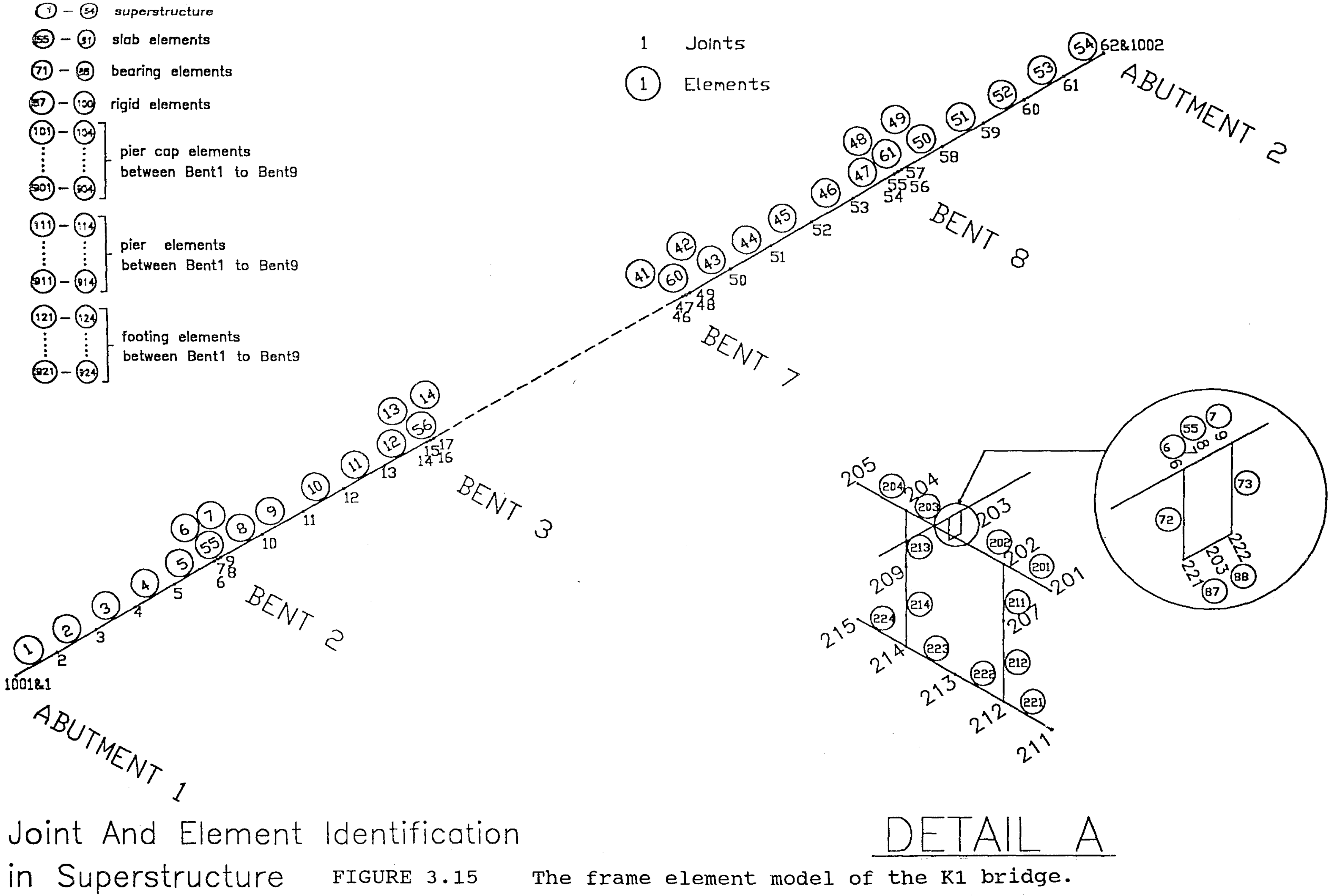


FIGURE 3.15 The frame element model of the K1 bridge.

OVERALL SYSTEM FOR K1 HIGHWAY BRIDGE

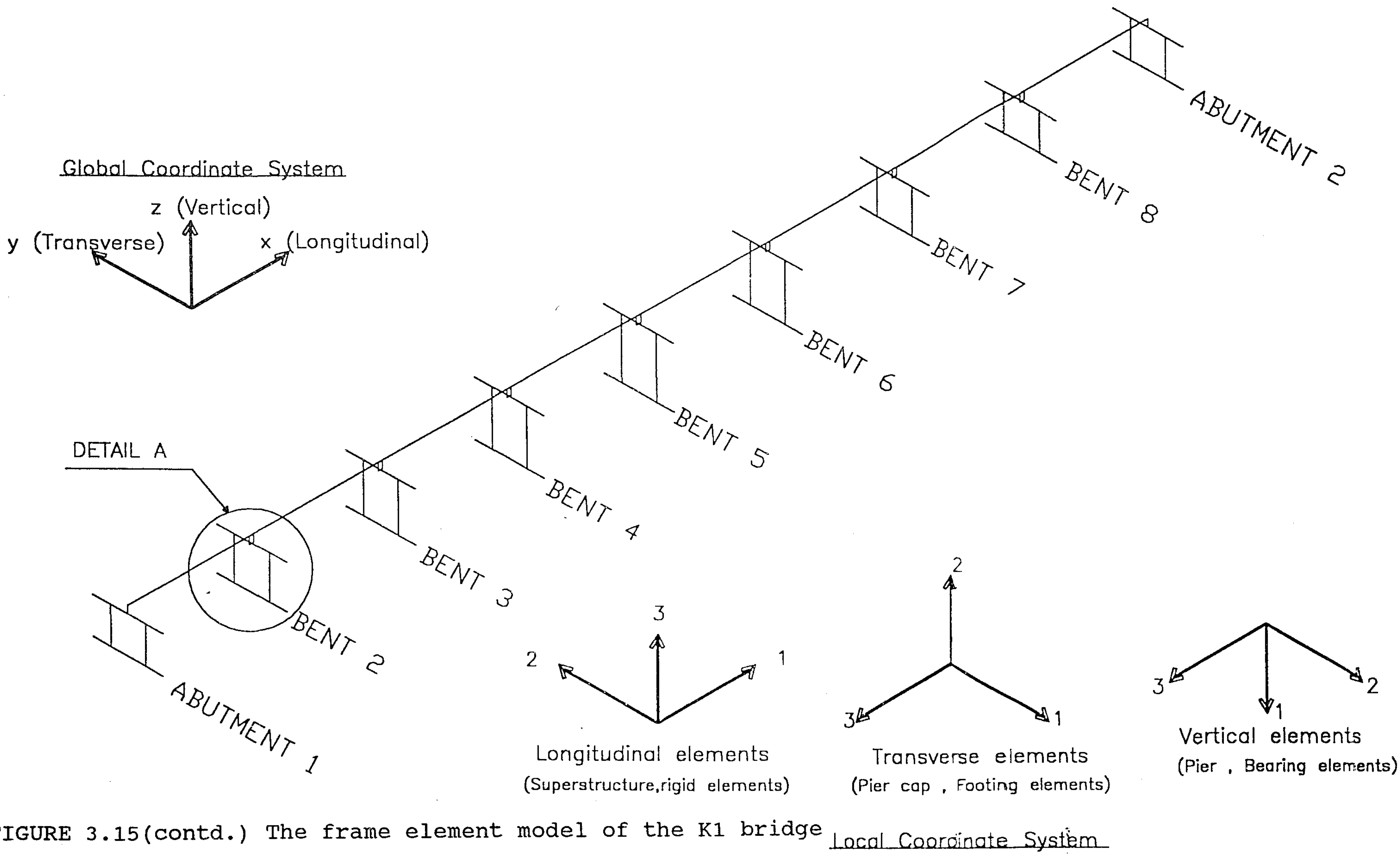


FIGURE 3.15(contd.) The frame element model of the K1 bridge

Two-Column Bents : Each column of each bent was seperately modeled with two frame elements, running along the centerlines of the columns, as shown in Figure 3.15 (Detail A).

The local 2-axis for these elements points upward, parallel to the global Z-axis.

Masses were again lumped at the joints for dynamic analysis. The AASHTO,1983 recommends that, as a maximum, joints should be located at the column third-points for long, flexible columns, e.g., columns having lengths greater than one-third of either of the adjacent span lengths. According to this criterion, every column in this structure was modeled using two frame elements.

Connection of Bents to Superstructure : The Multi-column bents were connected to superstructure with bearings and extra-stiff link elements. Two link elements were used at each bent; they connect the bent to the bottom of the bearing element.

Section Properties : All section properties for the Frame elements were computed using nominal dimensions.

Nine different section property types were identified (Table 3.9):

Section Property 1- Applies to all roadway elements, based on the cross-section of the box beam,
Section Property 2- Applies to bearings,
Section Property 3- Applies to Pier Caps,
Section Property 4- Applies to rigid link elements for artificially stiff properties,
Section Property 5- Applies to Piers,
Section Property 6- Applies to Slab,
Section Property 7- Applies to Abutment Piers
Section Property 8- Applies to Footings
Section Property 9- Applies to Abutment Footings

Property	1	2	3	4	5	6	7	8	9
A (m^2)	1.432	0.2025	2.34	10E3	2.138	3.482	2.25	7.50	12.75
I_x (m^4)	0.372	473.216	0.33	10E3	0.3638	0.018	0.4219	1.406	2.39
I_y (m^4)	142.102	0.0344	0.63	10E4	0.3638	56.313	0.4219	15.625	76.765
J (m^4)	0.729	0.0344	1.318	10E3	0.728	0.0717	0.712	4.56	8.5
M $(kN\text{-}s^2/m^2)$	21.9	0	5.96	0	5.45	8.875	5.73	19.1	32.5
E (kN/m^2)	3.58E7	2.5E6	3.455E7	3.455E7	3.18E7	3.455E7	3.18E7	3.18E7	3.18E7

Table 3.9 - The section properties for each element

All values were converted to consistent units of kN, meters and seconds. The unit weight for all types of concrete was assumed to be 25 kN/m^3.

The Link Elements were made artificially stiff by using the cross-sectional area and moments of inertia to be approximately 10^3 times larger than the corresponding largest value for the column sections. The moduli of elasticity were taken to be the same as for the superstructure.

Masses and Weights : The weight per unit length for each section was obtained by multiplying the unit weight of concrete (25 kN/m^3) by the cross-sectional area of the section.

An equivalent mass per unit length for each section is obtained dividing the weight per unit length by gravitational acceleration (g).

The link elements were assumed to be weightless and massless. It is important to include all structural masses, since the mass of the structure affects the natural period of the structure.

Foundations and Abutments : The foundation of the abutments and piers were assumed to be fully fixed in the analysis.

Acceleration coefficient

The acceleration coefficient A=0.2 was used in the analysis. This value corresponds to the value used in the original design of the bridge.

Importance Classification

The selected bridge was accepted as an essential bridge because of its social and survival requirements.

Seismic Performance Categories

The Seismic Performance Category (SPC)for the selected bridge is C, which corresponds to an Acceleration Coefficient of A=0.20 and an Importance Classification IC=I (Table C.1, Appendix C).

Site Effects

For the selected bridge, the Soil Profil Type II was accepted. It is a profile with clay or deep cohesionless conditions where the soil depth exceeds 61 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Site Coefficient

The site coefficient (S) approximates the effects of the site conditions on the elastic response coefficient or spectrum and was taken as 1.2 for Soil Profile Type II (Table C.2 in Appendix C).

Earthquake Load

A response spectrum analysis was performed to determine the seismic forces in the structure.

Dynamic Analysis : 25 eigen-modes were requested to satisfy the AASHTO recommendations. The SAP90 analysis showed that the mass participation factors were 92%, 91%, and 77% for the X-, Y- and Z- directions, respectively. The obtained Natural Frequencies and Periods for each mode are given in Table 3.10 and the mode shapes obtained for first longitudinal, transverse and vertical modes are given in Figure 3.16.

Spectral Analysis : The AASHTO normalized response spectra (Figure 3.17) for Soil Profile Type II with A=0.2g and 5%

damping was used in the spectral analysis. The response spectrum was applied independently in each of the longitudinal and transverse directions. They were then combined as given in Appendix C. The stresses obtained for each member were then used in the following section (Sec. 3.3.2) for the determination of the (Capacity/Demand) ratios of the bridge components.

K1 HIGHWAY BRIDGE

MODE NUMBER	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	1.409839	0.709301
2	1.702448	0.587389
3	1.846351	0.541609
4	1.962828	0.509469
5	2.064783	0.484312
6	2.131062	0.469250
7	2.217788	0.450900
8	2.366990	0.422477
9	2.484815	0.402444
10	3.228822	0.309710
11	3.624051	0.275934
12	3.870743	0.258348
13	3.942352	0.253656
14	4.346744	0.230057
15	4.841989	0.206527
16	4.884233	0.204740
17	5.040679	0.198386
18	5.397061	0.185286
19	5.402022	0.185116
20	5.933999	0.168520
21	6.411270	0.155975
22	6.448826	0.155067
23	6.618778	0.151085
24	7.017676	0.142497
25	7.029802	0.142252

Table 3.10- Natural Frequencies & Periods of the K1 bridge model

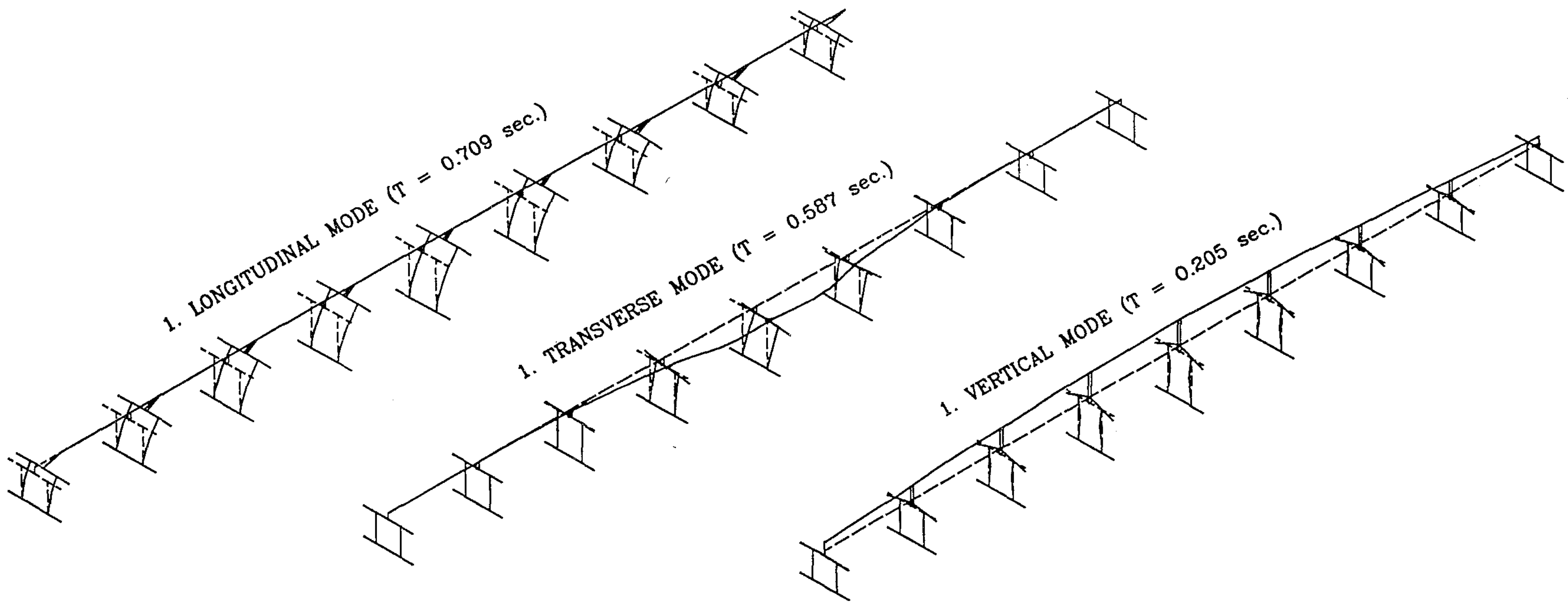


FIGURE 3.16 Mode shapes of the K1 bridge in 3 direction.

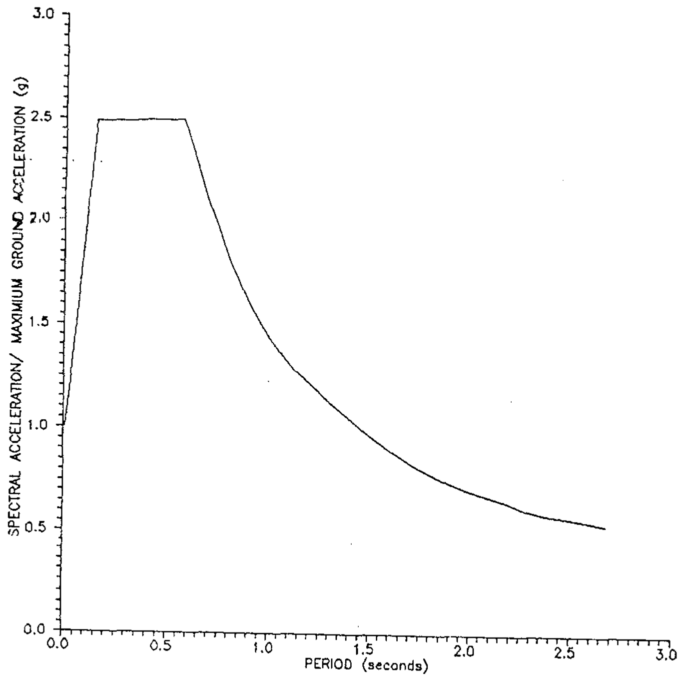


FIGURE 3.17 The AASHTO normalized response spectra.

3.3.2 Determination of the C/D ratios

The Capacity/Demand (C/D) ratios for the components of the K1 bridge were determined according to ATC-6-2 Detailed Evaluation Procedure (ATC-6-2, 1983) as follows:

Analysis Procedure : A multi-modal spectral analysis was applied using the SAP90 computer program (see Section 3.3.1).

I. C/D RATIOS FOR BEARINGS:

Here Span 1 is used for illustration purposes.

Minimum Support Lengths (Appendix C.3.4)

$$N(d) = 305 + 2.5(25.625) + 10(8.4) = 450 \text{ mm}$$

Support Length Capacity (from Figure 3.18); $N(c) = 875$ mm

C/D Ratio r_{bd} (due to support length) (Appendix E.1.2.a)

$$r_{bd} = \frac{N(c)}{N(d)} = \frac{875}{450} = 1.9$$

C/D Ratio r_{bf} (due to force) (Appendix E.1.2.b)

The bearing nominal ultimate capacity is calculated using its dimensions (0.4m×0.4m×0.12m) and the permissible shear stress (12×10^3 kN/m^2):

$$V_b(c) = (0.4)(0.4+0.12)\times12\times10^3 = 2496 \text{ kN}$$

The minimum force demand for the bearings at the Bent 2 is calculated by considering the equivalent static load to be acting on the first span.

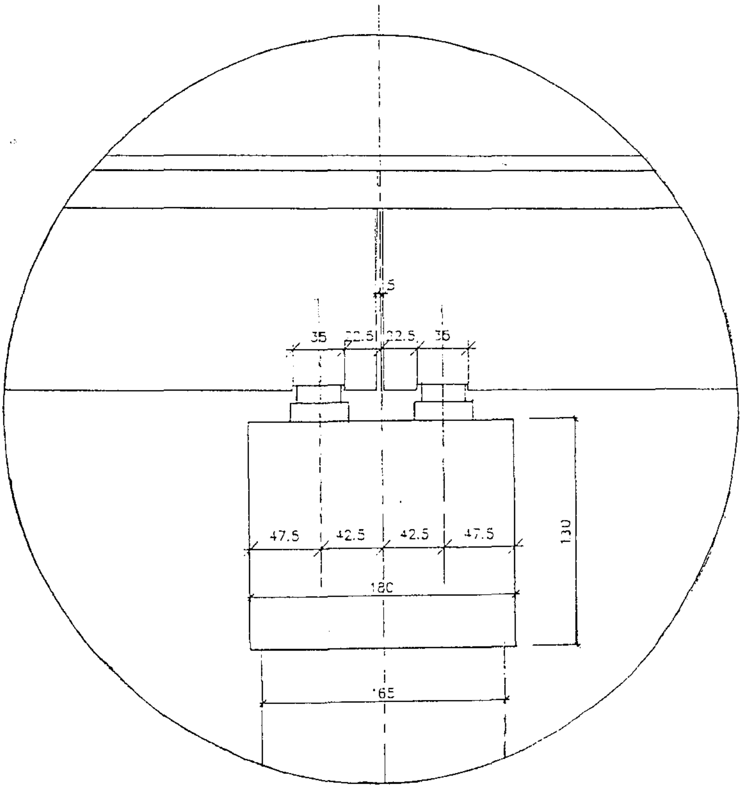


FIGURE 3.18 Support length detail of the K1 highway bridge.

Dead Loads on Superstructure :

Precast beam self weight	= 112.26 kN/m
Deck slab self weight + Panel	= 98.94 kN/m
Superimposed load	= 52.98 kN/m
Weight of the Superstructure	= 264.18 kN/m

Minimum Force Demand = 0.20(264.18(25.625/2))
= 680 kN

The bearing "demand" forces obtained from analysis (Section 3.3.1)
for Bent 2 were:

LOAD CASE	1-2 PLANE	1-3 PLANE
1	396	1287.4
2	1320	387.1

Using the maximum value,

$$V_b(d) = 1320.30 \times 1.25 = 1650 \text{ kN} > 680 \text{ kN}$$

$$r_{bf} = \frac{V_b(c)}{V_b(d)} = \frac{2496}{1650} = 1.5$$

II. C/D RATIOS FOR COLUMNS, PIERS, AND FOOTINGS:

Step 1: Elastic Moment "Demands": As explained in Section 3.3.1, in order to calculate the seismic force and displacement demands, the earthquake loading in two horizontal directions were to be applied (Load Case 1 and Load Case 2).

Load case 1 will be used for the illustration. Deadload moments, which are also included in the calculations, have been obtained from a seperate analysis. Elastic moment demands are summarized in Table 3.11.

MAXIMUM ELASTIC MOMENT DEMANDS
(kN-meters)

Location	Load Case	1-2 Plane EQ	1-2 Plane DL	1-3 Plane EQ	1-3 Plane DL	Elastic Moment Demand
BENT 2						
1 BOT	1	2080	1490	14410	0.0	14850
1 BOT	2	6910	1490	5670	0.0	10130
1 TOP	1	1620	3000	1820	0.0	4970
1 TOP	2	5380	3000	1400	0.0	8500
BENT 3						
1 BOT	1	2600	1490	13030	0.0	13660
1 BOT	2	8650	1490	5080	0.0	
1 TOP	1	2050	3000	1380	0.0	5240
1 TOP	2	6840	3000	1180	0.0	
BENT 4						
1 BOT	1	2670	1430	12140	0.0	12810
1 BOT	2	8890	1430	6080	0.0	
1 TOP	1	2140	2870	1330	0.0	5180
1 TOP	2	7120	2870	2060	0.0	
BENT 5						
1 BOT	1	3920	1350	10300	0.0	11570
1 BOT	2	13060	1350	3780	0.0	
1 TOP	1	3170	2710	680	0.0	5920
1 TOP	2	10560	2710	690	0.0	
BENT 6						
1 BOT	1	2810	1400	11500	0.0	12250
1 BOT	2	9380	1400	5590	0.0	
1 TOP	1	2260	2810	1170	0.0	5200
1 TOP	2	7530	2810	1820	0.0	
BENT 7						
1 BOT	1	1910	1580	14470	0.0	14880
1 BOT	2	6360	1580	6020	0.0	
1 TOP	1	1485	3180	1910	0.0	5040
1 TOP	2	4940	3180	1630	0.0	
BENT 8						
1 BOT	1	1800	1590	15400	0.0	15770
1 BOT	2	5990	1590	5200	0.0	
1 TOP	1	1370	3230	2250	0.0	5120
1 TOP	2	4550	3230	1020	0.0	

Table 3.11- Elastic Moment Demands of K1 highway bridge

Step 2: Ultimate Moment "Capacities": Ultimate moment capacities for the columns M_u, corresponding to Axial Load caused by Dead Load were obtained from the column interaction diagram (Figure 3.19).

Then the Overstrength Moment Capacities $M_0=1.3M_u$ were calculated for each bent as follows:

Bent	End	Axial Force due to Dead Load (kN)	$M_0=1.3M_u$ Column (kNm)
2	Top	2710	18730
2	Bottom	2710	18730
3	Top	2840	18990
3	Bottom	2840	18990
4	Top	2840	18990
4	Bottom	2840	18990
5	Top	2810	18860
5	Bottom	2810	18860
6	Top	2820	18900
6	Bottom	2820	18900
7	Top	2860	19910
7	Bottom	2860	19910
8	Top	2750	18800
8	Bottom	2750	18800

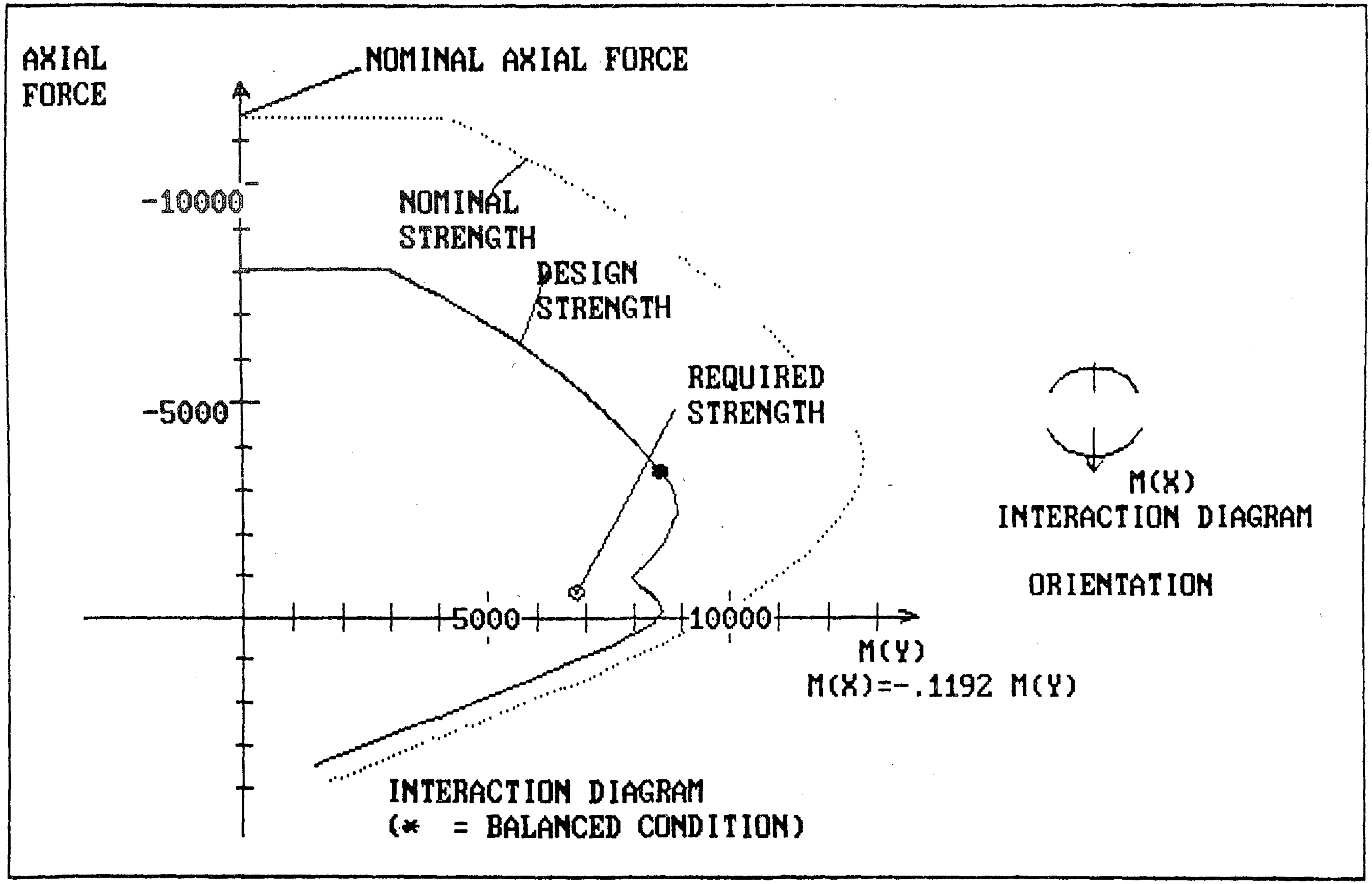


FIGURE 3.19 Interaction diagram of the K1 bridge.

The column shears were calculated using the overstrength moments at top and bottom of the each column, and the column heights:

Bent 2: $^{0}V_u$ = (18730 + 18730) ÷ 7.7 = 4860 kN

Bent 3: $^{0}V_u$ = (18990 + 18990) ÷ 8.65 = 4390 kN

Bent 4: $^{0}V_u$ = (18990 + 18990) ÷ 9.58 = 3960 kN

Bent 5: $^{0}V_u$ = (18860 + 18860) ÷ 10.95 = 3440 kN

Bent 6: $^{0}V_u$ = (18900 + 18900) ÷ 10.12 = 3730 kN

Bent 7: $^{0}V_u$ = (19910 + 19910) ÷ 7.3 = 5450 kN

Bent 8: $^{0}V_u$ = (18800 + 18800) ÷ 5.80 = 6480 kN

Axial Forces Due to Overturning in the Transverse Direction caused by the above shear forces $^{0}V_u$ were then:

Revision 1 :

Bent 2: Axial Force = 2(4860)(7.7) ÷ 7.85 = 9530 kN

Bent 3: Axial Force = 2(4390)(8.65) ÷ 7.85 = 9670 kN

Bent 4: Axial Force = 2(3960)(9.58) ÷ 7.85 = 9660 kN

Bent 5: Axial Force = 2(3440)(10.95) ÷ 7.85 = 9600 kN

Bent 6: Axial Force = 2(3730)(10.12) ÷ 7.85 = 9620 kN

Bent 7: Axial Force = 2(5450)(7.30) ÷ 7.85 = 10140 kN

Bent 8: Axial Force = 2(6480)(5.80) ÷ 7.85 = 9580 kN

By superposing the axial forces due to DL and due to Overturning in the transverse direction were revised:

Revised Column and Footing Overstrength Moments (M_1)

Bent	End	Axial Force due to Dead Load + Overturning (kN)	$M_1=1.3M_u$ Column (kNm)
2	Top	6820	18580
2	Top	12240	22210
2	Bottom	6820	18580
2	Bottom	12240	22210
3	Top	6830	18640
3	Top	12510	22050
3	Bottom	6830	18640
3	Bottom	12510	22050
4	Top	6820	18610
4	Top	12500	22030
4	Bottom	6820	18610
4	Bottom	12500	22030
5	Top	6790	20580
5	Top	12410	22340
5	Bottom	6790	20580
5	Bottom	12410	22340
6	Top	6800	20540
6	Top	12440	22550
6	Bottom	6800	20540
6	Bottom	12440	22550
7	Top	7280	20620
7	Top	13000	22560
7	Bottom	7280	20620
7	Bottom	13000	22560
8	Top	6830	18600
8	Top	12330	22300
8	Bottom	6830	18600
8	Bottom	12330	22300

Revised Column Shear Forces (^1V_u) (kN):

Bent 2: ^1V_u = (18580+18580)÷ 7.70 + (22210+22210)÷ 7.70 = 10590

Bent 3: ^1V_u = (18640+18640)÷ 8.65 + (22050+22050)÷ 8.65 = 9410

Bent 4: ^1V_u = (18610+18610)÷ 9.58 + (22030+22030)÷ 9.58 = 8480

Bent 5: ^1V_u = (20580+20580)÷10.95 + (22340+22340)÷10.95 = 7840

Bent 6: ^1V_u = (20540+20540)÷10.12 + (22550+22550)÷10.12 = 8520

Bent 7: ^1V_u = (20620+20620)÷ 7.30 + (22560+22560)÷ 7.30 = 11830

Bent 8: ^1V_u = (18600+18600)÷ 5.80 + (22300+22300)÷ 5.80 = 14100

These above computed shears (^1V_u) are not within 10 percent of the bent shears calculated in step 2 (^0V_u). Therefore the axial forces due to overturning must be recalculated.

Revision 2 :
Axial Forces Due to Overturning in the Transverse Direction

Bent 2: Axial Force = (10590)(7.7) ÷ 7.85 = 10390 kN

Bent 3: Axial Force = (9410)(8.65) ÷ 7.85 = 10370 kN

Bent 4: Axial Force = (8480)(9.58) ÷ 7.85 = 10350 kN

Bent 5: Axial Force = (7840)(10.95)÷ 7.85 = 10940 kN

Bent 6: Axial Force = (8520)(10.12)÷ 7.85 = 10980 kN

Bent 7: Axial Force = (11830)(7.30) ÷ 7.85 = 11000 kN

Bent 8: Axial Force = (14100)(5.80) ÷ 7.85 = 10420 kN

Revised Column and Footing Overstrength Moments (M_2)

Bent	End	Axial Force due to Dead Load + Overturning (kN)	M_2=1.3M_u Column (kNm)
2	Top	7680	20980
2	Top	13100	22470
2	Bottom	7680	20980
2	Bottom	13100	22470
3	Top	7530	20620
3	Top	13210	22560
3	Bottom	7530	20620
3	Bottom	13210	22560
4	Top	7510	20590
4	Top	13190	22700
4	Bottom	7510	20590
4	Bottom	13190	22700
5	Top	8130	21060
5	Top	13750	22740
5	Bottom	8130	21060
5	Bottom	13750	22740
6	Top	8160	21150
6	Top	13800	22770
6	Bottom	8160	21150
6	Bottom	13800	22770
7	Top	8140	20620
7	Top	13860	22830
7	Bottom	8140	20620
7	Bottom	13860	22830
8	Top	7670	20450
8	Top	13170	22210
8	Bottom	7670	20450
8	Bottom	13170	22210

Revised Column Shear Forces ($^{2}V_u$) (kN):

Bent 2: V_u = (20980+20980)÷ 7.70 + (22470+22470)÷ 7.70 = 11290

Bent 3: V_u = (20620+20620)÷ 8.65 + (22560+22560)÷ 8.65 = 9980

Bent 4: V_u = (20590+20590)÷ 9.58 + (22700+22700)÷ 9.58 = 9040

Bent 5: V_u = (21060+21060)÷10.95 + (22740+22740)÷10.95 = 8000

Bent 6: V_u = (21150+21150)÷10.12 + (22770+22770)÷10.12 = 8680

Bent 7: V_u = (20620+20620)÷ 7.30 + (22830+22830)÷ 7.30 = 11900

Bent 8: V_u = (20450+20450)÷ 5.80 + (22210+22210)÷ 5.80 = 14710

Since the revised bent shears ($^{2}V_u$) are within 10 percent of the previously calculated shears, no further iteration is needed.

Step 3: Ultimate Moment Capacity/Elastic Moment Demand = r_{ec}

The ratio of ultimate moment capacities and elastic moment demands (r_{ec}) at each bent are computed by using the elastic demand values given in Table 3.11 and the ultimate moment capacities ($M_u=M_2/1.3$) corresponding to the last revision of the previous step.

Bent	End	Axial Load	Demand (kNm)	Capacity (kNm)	r_{ec}
2	Top	Min	4970	16140	3.2
2	Top	Max	4970	17280	3.4
2	Bottom	Min	14850	16140	1.1
2	Bottom	Max	14850	17280	1.2
3	Top	Min	5240	15860	3.0
3	Top	Max	5240	17350	3.3
3	Bottom	Min	13660	15860	1.2
3	Bottom	Max	13660	17350	1.3
4	Top	Min	5180	15840	3.0
4	Top	Max	5180	17460	3.3
4	Bottom	Min	12810	15840	1.2
4	Bottom	Max	12810	17460	1.4
5	Top	Min	5920	16200	2.7
5	Top	Max	5920	17490	2.9
5	Bottom	Min	11570	16200	1.4
5	Bottom	Max	11570	17490	1.5
6	Top	Min	5200	16270	3.1
6	Top	Max	5200	17510	3.3
6	Bottom	Min	12250	16270	1.3
6	Bottom	Max	12250	17510	1.4
7	Top	Min	5040	15860	3.1
7	Top	Max	5040	17560	3.4
7	Bottom	Min	14880	15860	1.1
7	Bottom	Max	14880	17560	1.2
8	Top	Min	5120	15730	3.1
8	Top	Max	5120	17080	3.3
8	Bottom	Min	15770	15730	1.0
8	Bottom	Max	15770	17080	1.1

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

When r_{ec} exceeds 0.8 at bottom and or at top of the column, there will be no plastic hinging cases (ATC-6-2, 1983). In such a case the column C/D ratios for anchorage of longitudinal reinforcement and splices in longitudinal reinforcement will be calculated. In the following calculations, Bent 2 was selected for the illustration again.

C/D Ratio for Anchorage of Longitudinal Reinforcement (r_{ca}): Effective anchorage length of longitudinal reinforcement $l_a(c)$ is taken 140 cm from as-built project as seen from the Figure 3.18. Required effective anchorage length reinforcement $l_a(d)$ is calculated using the formula (eq.E-3, Appendix E.1.3.a).

$l_a(c)$ = 140 cm ~ 55 inch (Figure 3.20)

Using the following data;

All reinforcing steel to be high yield and high bond grade Y:
$f_y \geq 420$ N/mm^2 = 60000psi
Compressive strength of concrete: $f_c' = 30$ N/mm^2 = 4350 psi
c = 11/2 cm ~ 2.16 inch
d_b = ϕ25 ~ 25 mm ~ 1 inch
$A_{tr}(c)$ = 1.2 in^2
s = 6 inch

$$l_a(d) = \frac{((60000 - 11000) \div 4.8)1.0}{\sqrt{4350}\,(1 + 2.5\,(2.16 \div 1.0) + 1.5)} \qquad \text{(from eq.E-3)}$$

$l_a(d)$ = 20 inches

If the effective development length is sufficient ($l_a(c) > l_a(d)$), the C/D ratio for anchorage of longitudinal reinforcement is determined as r_{ca} = 1.0 (ATC-6-2, 1983).

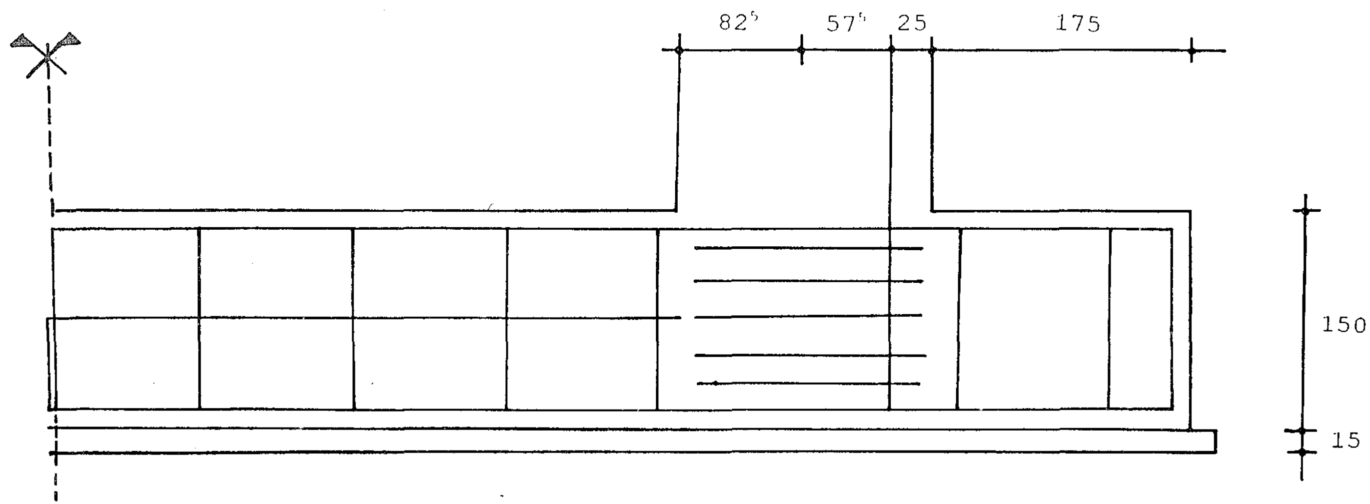


FIGURE 3.20 Cross section view of K1 bridge footing.

C/D Ratio for Splices in Longitudinal Reinforcement (r_{cs}):
Since, there is no splice in longitudinal reinforcement, this criterion will not be applied.

Step 5: C/D Ratios for Possible Hinging at the Top of the Column

$l_a(c) = 150$ cm ~ 59 inches

$$l_a(d) = \frac{((60000 - 11000) \div 4.8)1.0}{\sqrt{4350}\,(1 + 2.5\,(2.16 \div 1.0) + 1.5)} = 20 \text{ inches (from eq.E-3)}$$

$l_a(c) > l_a(d) \Rightarrow r_{ca} = 1.0$

Step 6: C/D Ratios for Column Shear (r_{cv}):

Maximum Shear Force is using M_2;

$$V_u(d) = \frac{22470 + 22470}{7.85} = 5720 \text{ kN}$$

COLUMN SHEAR FORCE DEMANDS (kN)

LOCATION	LOAD CASE	1-2 PLANE	1-3 PLANE
BENT 2	1	420	1510
(BOTTOM)	2	1410	710

$V_e(d) = 1510$ kN

$$V_i(c) = v_c\, db + \frac{A_{tr}\, f_{yt}\, d}{s}$$

v_c - The shear stress carried by the concrete (AASHTO, 1991)

$$v_c = 2\sqrt{f_c'} = 114\text{psi} \sim 800 \text{ kN/m}^2$$

$d = 160$ cm, $b = 165$ cm, $s = 15$ cm, $f_{yt} = 420$ N/mm^2

$$A_{tr} = \frac{\pi \times (1.0)^2}{4} = 0.785 \text{ cm}^2$$

$V_i(c) = 2460$ kN , $V_e(d) = 1510$ kN

$$r_{cv} = \frac{V_i(c)}{V_e(d)} = 1.60$$

III. C/D RATIO FOR ABUTMENTS (r_{ad}):

Based on experience from past earthquakes (ATC-6-2, 1983), the abutment "displacement capacities" were fixed to be 3 inches in the transverse direction and 6 inches in the longitudinal direction:

Abutment demands were based on the displacements obtained from the analysis (Section 3.3.1):

ABUTMENT DISPLACEMENT DEMANDS (in cm)

ITEM	LOAD CASE	LONGITUDINAL	TRANSVERSE
ABUT. 1	1	5.42	0.46
	2	1.63	1.54
ABUT. 2	1	5.23	0.53
	2	1.57	1.76

Transverse displacement capacity: d(c) = 3 inches ~ 7.62 cm

Abutment 1:
Transverse displacement demand: d(d) = 1.54 cm

$$r_{ad} = \frac{7.62}{1.54} = 4.9$$

Abutment 2:
Transverse displacement demand: d(d) = 1.76 cm

$$r_{ad} = \frac{7.62}{1.76} = 4.3$$

Longitudinal displacement capacity: d(c) = 6 inches ~ 15.24 cm

Abutment 1:
Longitudinal displacement demand: d(d) = 5.42 cm

$$r_{ad} = \frac{15.24}{5.42} = 2.8$$

Abutment 2:
Longitudinal displacement demand: d(d) = 5.23 cm

$$r_{ad} = \frac{15.24}{5.23} = 2.9$$

IV. C/D RATIO FOR LIQUEFACTION (r_{sl}):

Since the preliminary screening (Section 3.2.2) indicated that low liquefaction related damage for the K1 highway bridge, a C/D ratio was not applied.

SUMMARY OF FINAL C/D RATIOS:

Bearing	r_{bd} = 1.9
	r_{bf} = 1.5
Column	r_{ca} = 1.0
	r_{cs} = not applicable
	r_{cv} = 1.6
Abutments	r_{ad} = 4.9 (Abutment 1- transverse dir.)
	r_{ad} = 4.3 (Abutment 2- transverse dir.)
	r_{ad} = 2.8 (Abutment 1- longitudinal dir.)
	r_{ad} = 2.9 (Abutment 2- longitudinal dir.)
Liqufaction	r_{sl} = not applicable

As it is clearly seen from above C/D ratios since all the ratios are greater than 1, thus the capacity of the all components are sufficient to withstand the demands. Therefore, this bridge does not need any retrofitting.

CHAPTER 4

TRUCK LOADING TEST OF K1 HIGHWAY BRIDGE

4.1 Test Objective

The objective of this test was to determine the dynamic characteristics of the K1 bridge due to vibrations produced by test vehicles and compare the results obtained from the analytical solutions (see Section 3.3.2). The K1 bridge is on O2 route on a link road and at the time of the test it was closed to the traffic.

4.2 Test Set-up

The main components of the Test Set-up are shown in Figure 4.1. At the locations of the sensors, the accelerations were converted into an analog electronic signal by the sensor. The signal was then amplified and filtered before being converted to binary information, which was then stored in a Portable computer. After the completion of the test, the data was analyzed to determine the natural frequencies of the bridge. Details of the main components of the test equipment are described below.

a) Sensors : Five force-balanced AM-2 accelerometers were used. These accelerometers have 50 Hz natural frequency.

b) Strain Meter : SDA-62B 6 channel strain meter was used for signal conditioning with filtering, amplification, differentiation and integration capabilities for analog transient signals for each channel.

c) Analog to Digital Converter : To facilitate computer processing of the vibration data, the original data was converted to digital data by an Analog/Digital Converter.

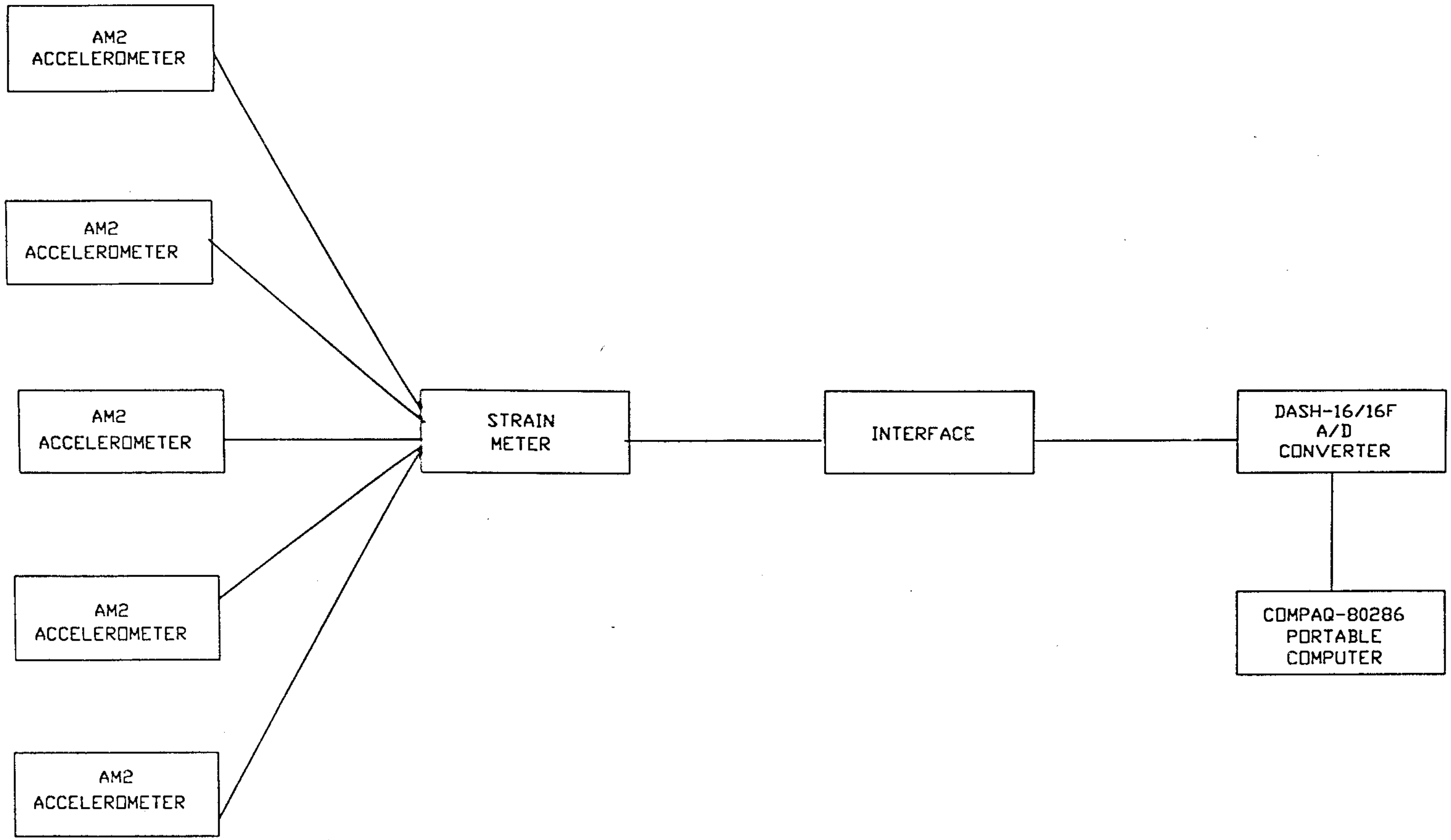


FIGURE 4.1 The main components of the testing equipment

In addition to above, sufficient amount of Cables, an Interface (which transfers the filtered records to A/D converter) and Compaq Portable Computer 80286 were also used during the test.

4.3 Test Procedure

During the tests, trucks moving at a certain speed were used to excite the bridge in the vertical direction. The average speed of the trucks were 60 km/h. Their average weight was 25 tons.

Five sensors were used during the experiment. Their locations are shown in Figure 4.2. As it can seen from this figure, the sensors were located in the midspan and near the supports of the simple span. Five tests were conducted to determine the fundamental frequencies of the bridge.

4.4 Data Processing & Test Results

Five sets of data were recorded in the vertical direction. For the first data set Sampling interval of 0.02 sec and for the others 0.01 sec were used. Each set of data was recorded for a duration of 100 seconds.

Data was analysed using two computer programs: BLA90 and FAS90. The program BLA90 was used to obtain the corrected accelerations and FAS90 was used to obtain the Smoothed Fourier Amplitude Spectra.

Data processing involves baseline correction, conversion of data from volts to cm/s^2, high pass filtering and low pass filtering. Corrected accelerations are shown in Figures 4.3 and 4.4. The F.A.S. of the channels 3,1 and 5 from test 3 and test 5 are given in Figures 4.5 and 4.6. From these plots it is concluded in the vertical direction the natural frequency of the bridge site is at about 4.8 Hz.

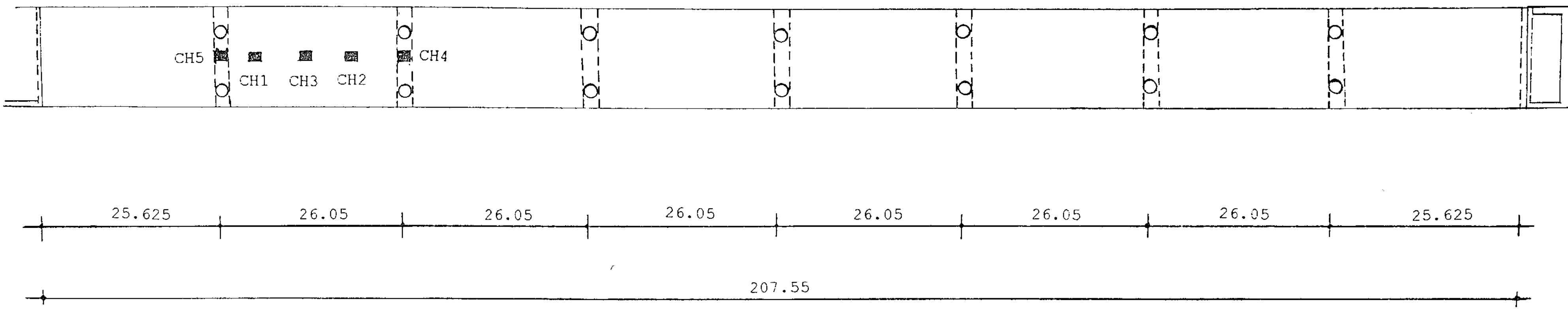


FIGURE 4.2 Location of the accelerometers on the K1 highway bridge.

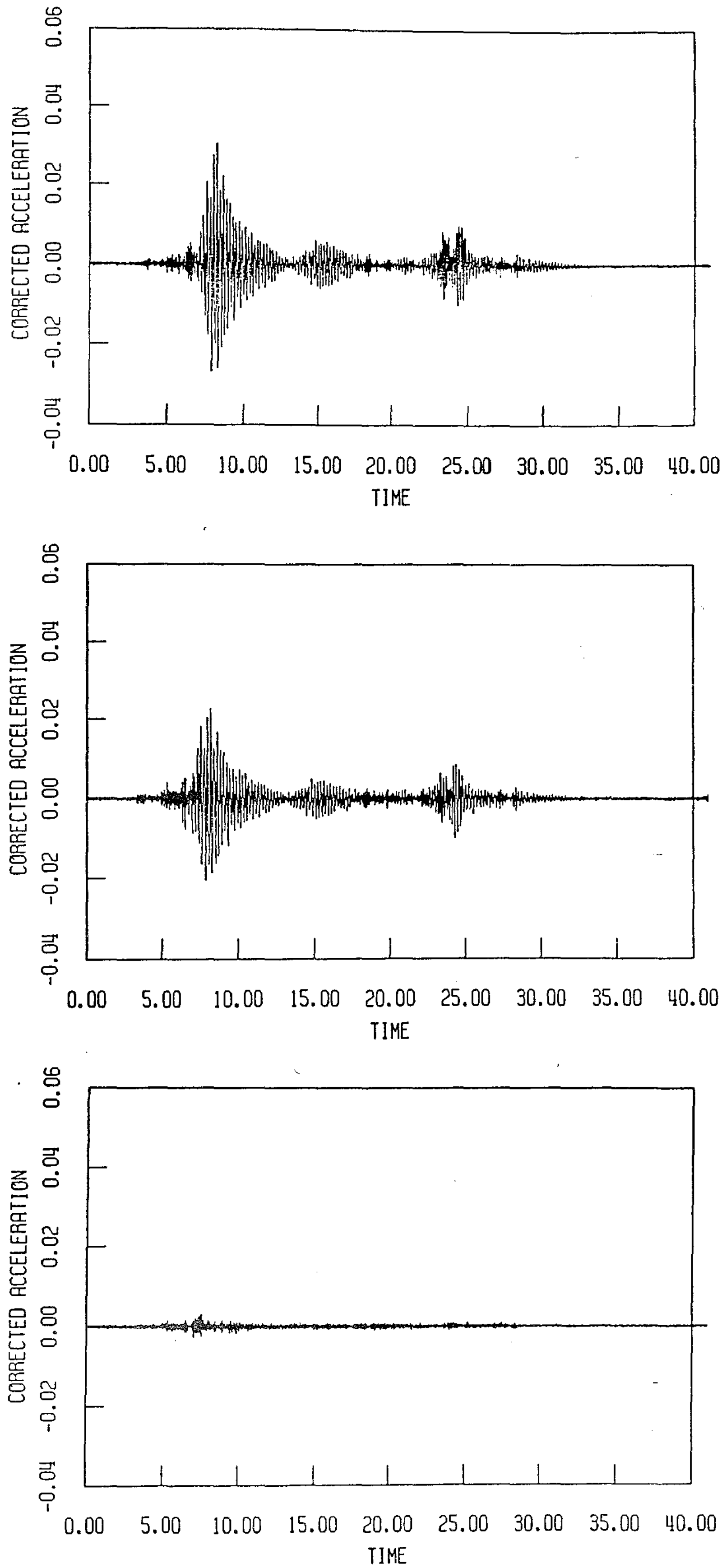
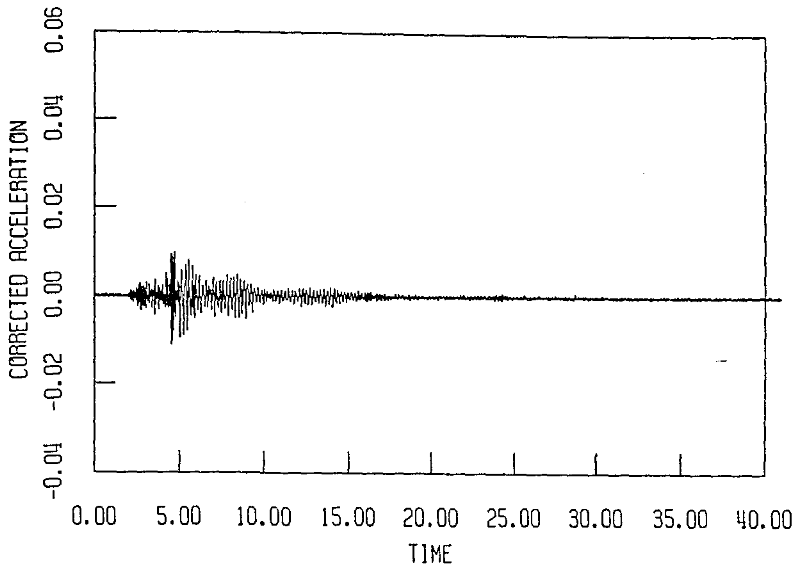
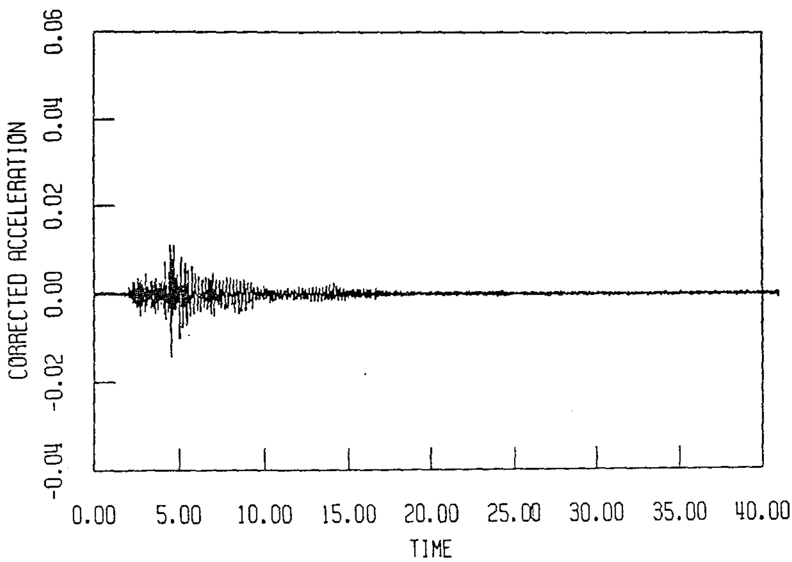


FIGURE 4.3 Corrected acceleration-time histories of test 3 for channel 3, 1 and 5 respectively.





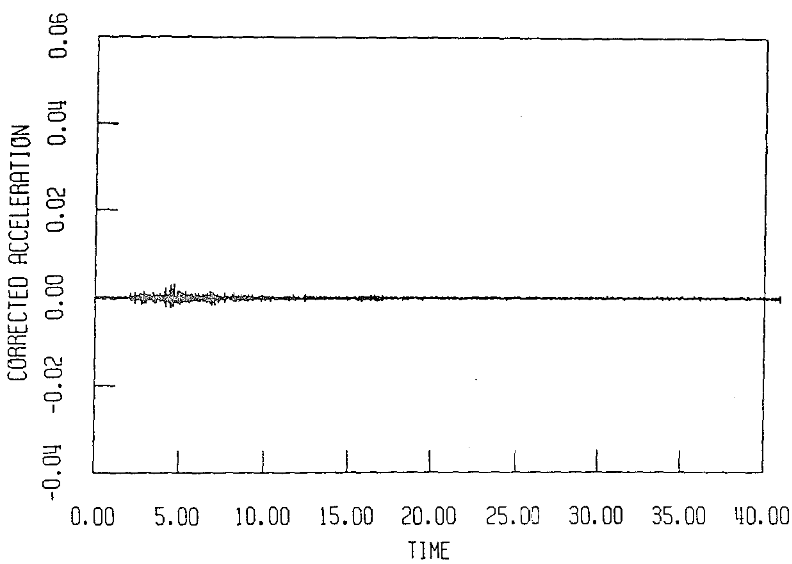
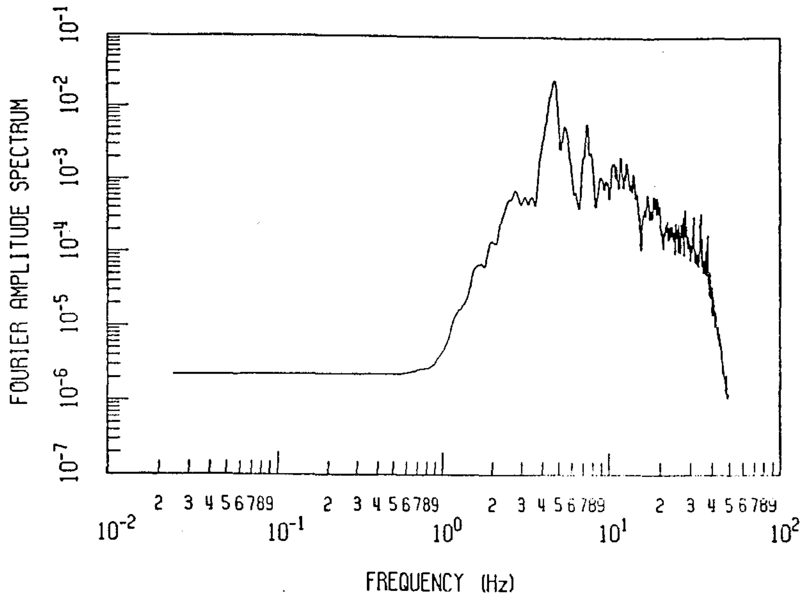
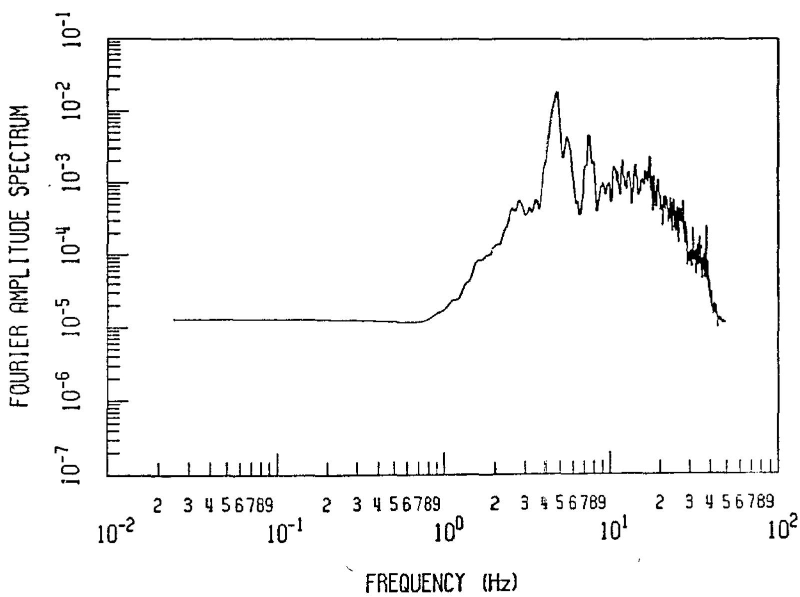


FIGURE 4.4 Corrected acceleration-time histories of test 5 for channel 3, 1 and 5 respectively.





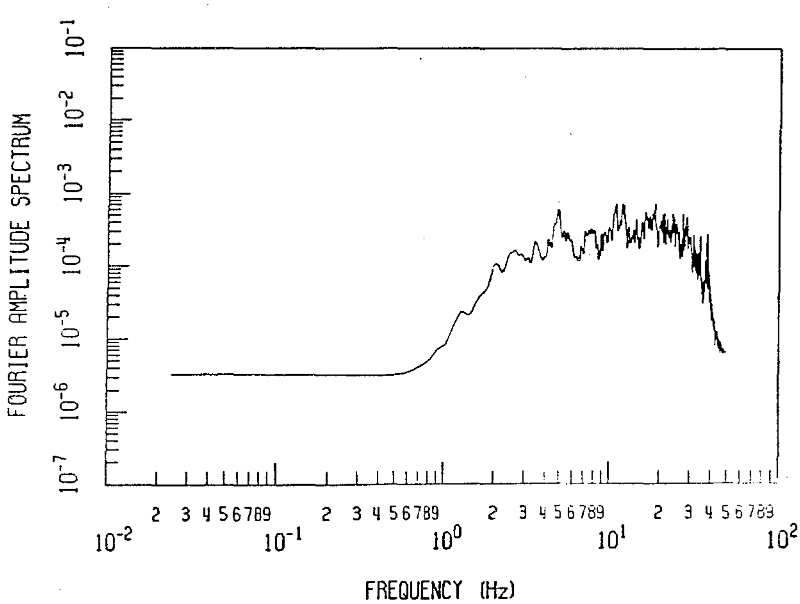


FIGURE 4.5 Fourier amplitude spectrums of test 3 for channels 3, 1 and 5 respectively.

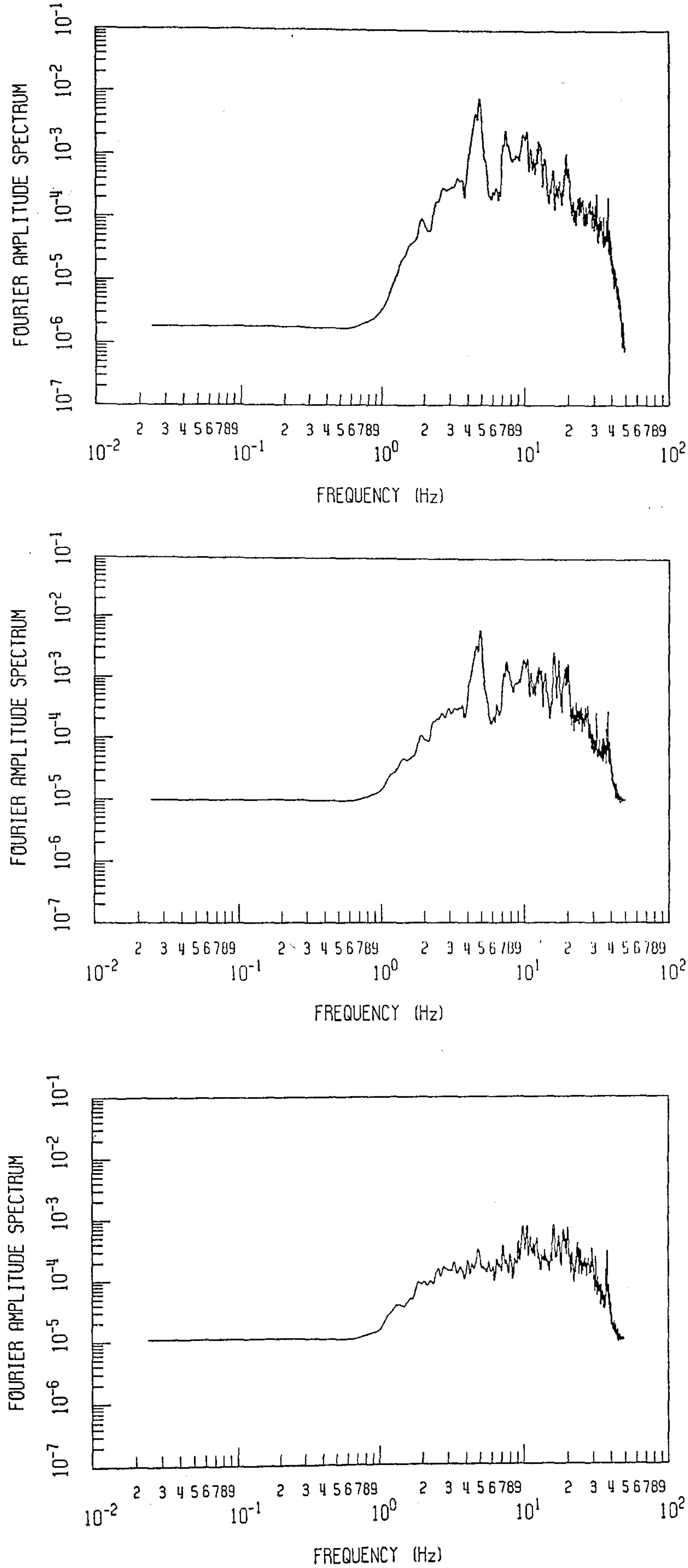


FIGURE 4.6 Fourier amplitude spectrums of test 5 for channels 3, 1 and 5 respectively.

4.5 Discussion of the Test Results

The vertical natural frequency of the K1 highway bridge obtained from the truck loading test was compared with the results of several tests which were carried out in Greece, (A. J. Karabinis) to check the reliability of the results. In Karabinis' study, 16 reinforced or prestressed concrete highway bridges under traffic and ambient excitations were tested. The purpose of these tests was to compare the effects of the geometric properties of the superstructure on the fundamental frequency contents. The results of these tests were illustrated in Figures 4.7 and 4.8. It one can observe from these figures, the vertical frequency for the bridges with span length between 25-30 m, is between 4-5 Hz. This result is confirmed with the natural vertical frequency of K1 highway bridge with 4.8 Hz. from truck loading test.

Since the longitudinal and transverse recordings could not be taken in the truck loading testing, only the vertical recordings were used in checking the analytical results. As it is seen from the analytical results given in Table 3.10, the vertical natural frequency was 4.88 Hz. This value is considered to be in good agreement with the truck loading test result which is 4.8 Hz.

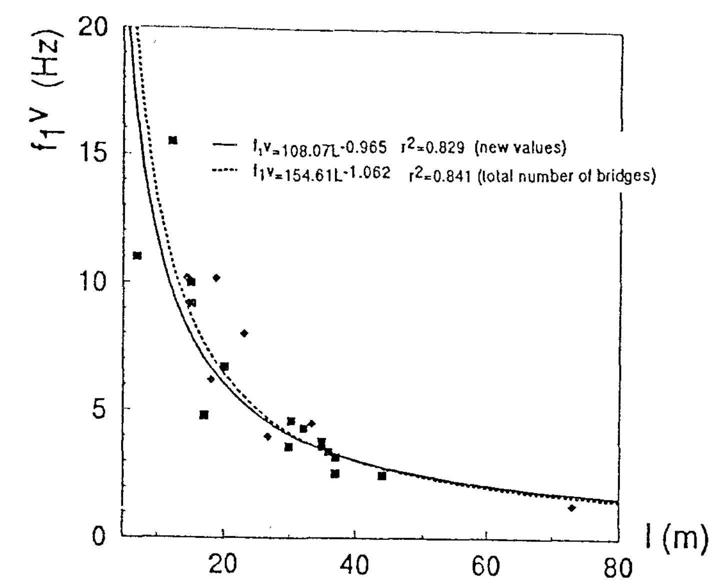


FIGURE 4.7 Experimental values of fundamental vertical frequency (f_1^v) versus length of the span, (Karabinis, 1995).

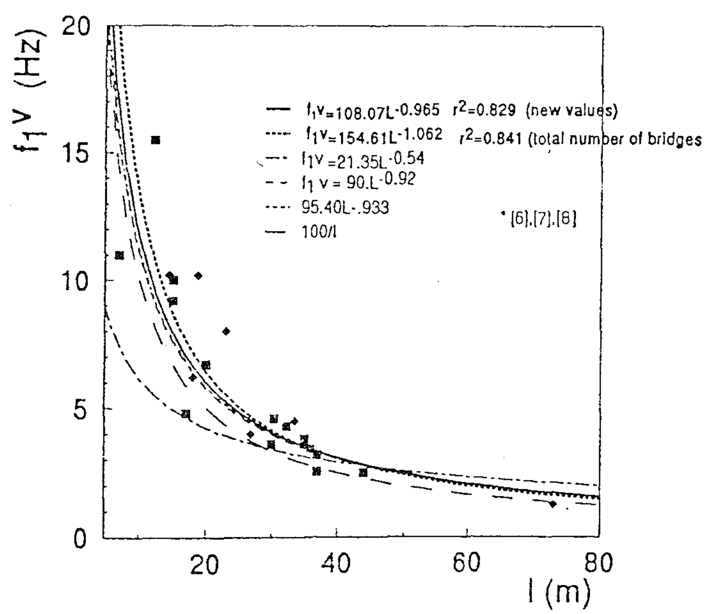


FIGURE 4.8 Correlation with experimental formulas from other researches, (Karabinis, 1995).

CHAPTER 5

SUMMARY AND CONCLUSION

Summary

Bridges are important links in a transportation system. Retrofitting of existing bridges is necessary to decrease the risk of damage induced under earthquake loading. All the bridges on the highway system cannot be retrofitted simultaneously, the most critical bridges should be retrofitted first. The selection of critical bridges requires a seismic retrofitting program. This program contains the following steps:

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

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

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

In this study, an inventory was developed for the bridges on the two peripheral motorways (O1 and O2) in İstanbul. This inventory contains information on: year built, geometry, number of expansion joints, skew angle, superstructure type, number of spans, substructure type, pier height, support length, bearing type, and foundation type of the bridges. After identifying the existing preliminary screening methods in the literature, the bridges were screened according to two different methods, namely:

- ATC-6-2 (Applied Technology Council, 1983)
- A Combined Method (Memphis State University, 1994)

Both of the above screening methods aimed to identify the bridges with high vulnerability and need to be retrofitted firstly.

As for the second step in the retrofitting program a Precast, Prestressed, Simply Supported Box girder bridge was selected for illustration of the detailed evaluation process. The bridge was modeled for and analzed by SAP90 computer program. After obtaining the natural frequencies and mode shapes Spectral analysis was applied to the bridge model as defined by the AASHTO (Standard Specifications for Seismic Design of Highway Bridges, 1991). The calculated natural frequencies were checked against the truck loading test results which were carried within the scope of this study. Spectrum analysis results were used in the ATC detailed evaluation process. Four components of the bridge, namely: Bearings, Columns, Abutments, and Liquefaction susceptibility of the bridge site were individually evaluated by comparing their Capacity/Demand (C/D) ratios. C/D ratio of any component less than 1 indicated that the component is vulnerable.

As for the final step of the retrofitting program, some retrofitting schemes were supplemented for the bridges which are found to be seismically vulnerable. The realization of the economically feasible schemes are left to the judgement of the decision makers.

Conclusions and Suggestions :

The inventory on the existing bridges in the study area lead to the following statistical evaluations:

- The most encountered bridge superstructure types on O1 route fall into the following two categories:

Post-tensioned Continuous Plate Girder (50%) and Reinforced Concrete Plate Girder (38%).

On the other hand, on O2 route 90% of the bridges and on the link routes 88% of the bridges have a Precast, Prestressed, Simple Supported Box Beam superstructure.

- On O1 route 88 percent, on route O2 94% and on the link routes 77% of the abutments are wall type .

- On O1 route 75%, on O2 route 92% and on the link routes 15% of the piers are wall type.

- All the bearings on O2 and link routes, and 96% of the bearings on O1 route are elastomeric Laminated type.

- On O1 route 36%, on O2 route 6% and on the link routes 8% of the foundations have piles.

ATC Preliminary Screening Methodology indicates that the seismic intensity and soil geology has substantial effect on the overall vulnerability scoring of bridges. Accordingly, the bridges on soil type C and have seismic intensity of X, had high vulnerability, on the soil type B and have seismic intensity of VIII, had moderate vulnerability and on soil type A and have seismic intensity of VII or VI, had low vulnerability.

The Combined Method indicates that the bridges with high vulnerability scores, in general had: simple supported superstructures, irregular geometries, skew angles greater than 20°, and built on soil type C.

The detailed retrofitting methods for the bearings, columns, abutments, and liquefaction susceptibility are given in Appendix E.

The following retrofitting methods are suggested for the bridges in İstanbul in case they are found to be potentially vulnerable:

■ Almost 64% of the bridges covered within the scope of this study have simply supported spans. There is a lack of continunuity. Thus the first priority should be given to increase the continunity of the bridge. Longitudinal steel cables which are capable of absorbing a considerable amount of energy may be used to tie the simple spans and joints together (e.g. see Figs G4-G8). Vertical restrainers are also recommended (e.g. see Fig G9).

■ The energy dissipation capacity of the bearings should be increased. The replacement of the existing elastomeric bearings by elastomeric bearing pads with a lead core seems to be the most feasable method (e.g. see Fig G.15).

■ The inventory provided by this study lack soil profile details. Therefore soil profile details should be collected for each bridge and a study should be carried out to determine the liquefaction potential for each bridge foundation, individually. Any probable liquefaction failure should either be prevented (possibly by a stabilization procedure) or its adverse effects on the overall stability should be taken into consideration and retrofitted for.

■ The screening procedures used in this study should be extended to cover all the bridges in İstanbul.

■ The screening procedures used in this study should be elaborated, even a new methodology should be developed by introducing further appropriate parameters for scoring the bridges (e.g. priority of underpass or overpass,).

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APPENDIX A

OBSERVED DAMAGES TO BRIDGES DURING MIYAGI-KEN-OKI EARTHQUAKE, (June 12, 1978 - M = 7.4)

The Sendai Bridge (1965)

Geometry : Total length is (9×33.84)= 304.56 m., Width is 6.0 m.
Superstructures : 9 span simply supported composite steel-plate girders,
Substructures : T shape column, Column height is 6.1 m.
Foundation : Rigid well foundation (9 to 18 m. deep)
Observed damages : 9 pier columns sustained damage, Piers 1 through 4 cracked horizontally at the column bases,
Piers 5 through 8 cracked horizontally near the haunches which connect columns and beams, Displacement at the pier caps of piers 1, 2 and 6 were 11 to 18 cm.
Repair work : Vertical reinforcing bars were fixed by epoxy adhesive into the well foundation, Lateral reinforcing bars were fixed to the columns, Chemical resin was placed into small cracks,

The Kin-noh Bridge (1956)

Geometry : Total length is 575.5 m., Width is 6.0 m
Superstructure : 1 span steel plate girder, 5 span simple supported steel trusses, 9 span Gerber type steel plate girders,
Substructure : RC columns on caisson foundations for truss spans RC columns on footing foundations with RC piles for the gerber plate girder span,
Foundation : Soils are of soft silts and sand, and a firm sand layer exist approximately 30 m. below the ground surface,
Observed damages : One girder fell down, Bearing supports failed, Anchor bolts of the upstream fixed bearing at pier 6 were

pulled out by about 20 cm.

The Eai Bridge

Geometry : Total length is 155 m., Width is 7.5 m.

Superstructure : 9 span simply supported steel plate girders

Substructure : 2 Abutments on pile foundations, each piers on two seperate well foundations,

Observed damages : Lower beams of the eight pier columns were severely cracked, the largest opening of the crack was 20 mm., and reinforcing bars appeared,

The Yuriage Bridge (1972)

Geometry : Total length is 541.7 m, Width is 8.0 m.

Superstructure : 3 span continuous PC box girders, 7 span simply supported post-tension PC beams (T shape)

Substructure : Two abutments are on steel pipe pile foundations, two piers are on pneumatic caisson foundation, and 7 piers are on well foundations

Observed damages : 9 pier columns sustained many cracks, mostly at the level of the ground surface

The Date Bridge (1963)

Geometry : Total length is 288.0 m., Width is 7.0 m.

Superstructures : 4 span continuous steel truss girders,

Substructures : The two abutments are on steel pipe pile foundations, and the three piers are of tall RC columns on caisson foundations embedded into gravel and sand layers,

Observed damages : A lower chord member buckled just at the fixed bearing on pier 2,Several pins at the fix bearing and one of the movable bearings were sheared off and come out

APPENDIX B

PHOTOS OF HIGHWAY BRIDGE DAMAGES IN RECENT EARTHQUAKES

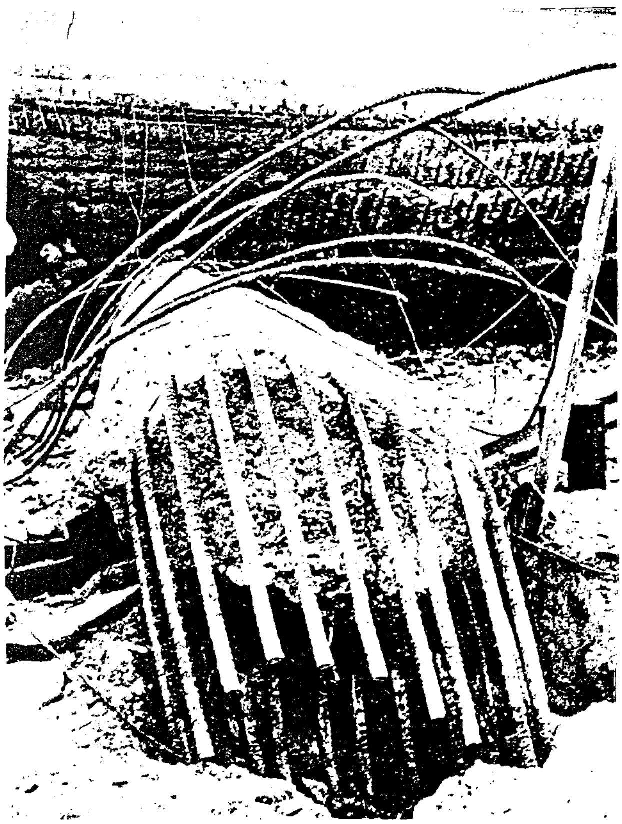


PHOTO B.1 Failure of a column due to insufficient anchorage. (San Fernando Earthquake of 1971), ATC (1979).

PHOTO B.2 Column Damage. (San Fernando Earthquake of 1971), ATC (1979).

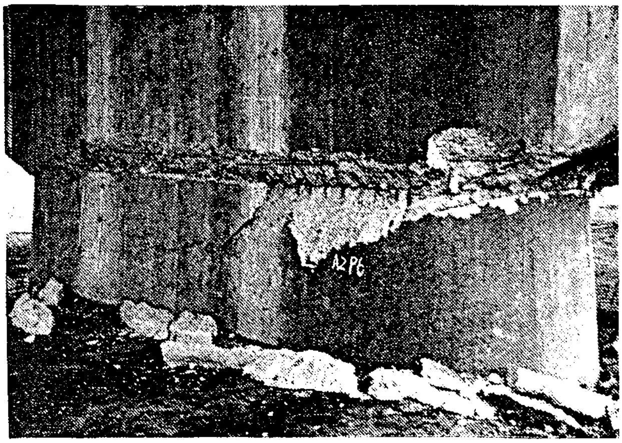


PHOTO B.3 Sendai Bridge, column damage.
(Miyagi-ken-oki Earthquake of June 12, 1978),
ATC (1979).

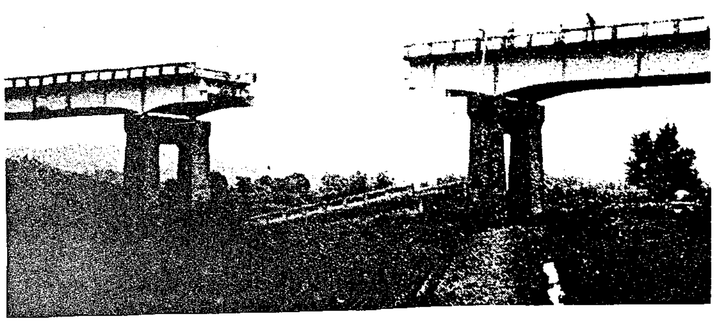


PHOTO B.4 Kin-noh Bridge, fall of suspended girder beam.
(Miyagi-ken-oki Earthquake of June 12, 1978),
ATC (1979).

PHOTO B.5 Cypress Avenue (I-880) double deck crossing.
(Loma Prieta Earthquake of October 17, 1989),
Time (October 30, 1989).

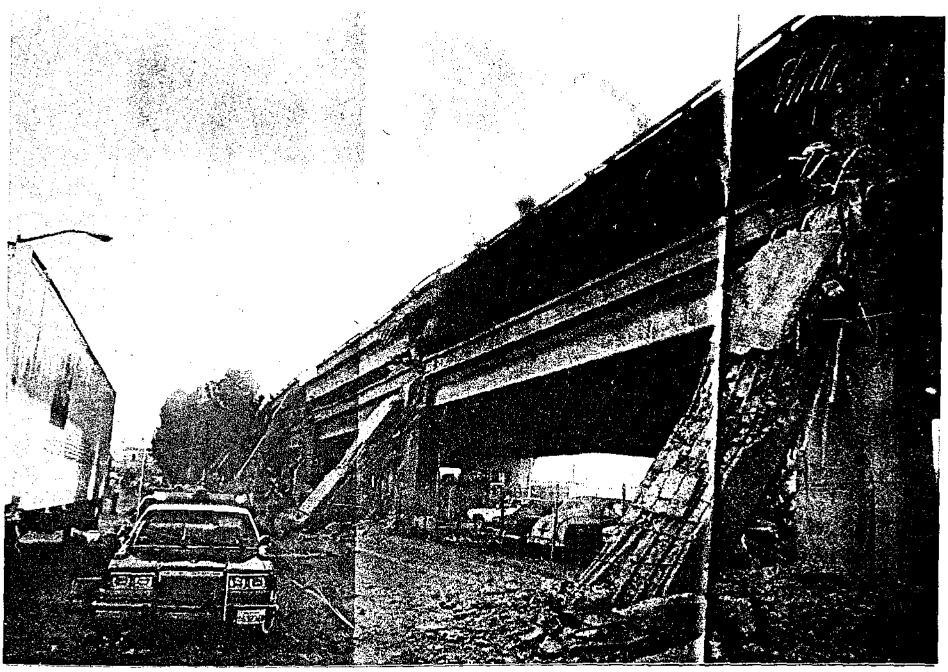


PHOTO B.6 Cypress Avenue (I-880) double deck crossing.
(Loma Prieta Earthquake of October 17, 1989),
Time (October 30, 1989).

PHOTO B.7 Collapse of Hanshin Expressway.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

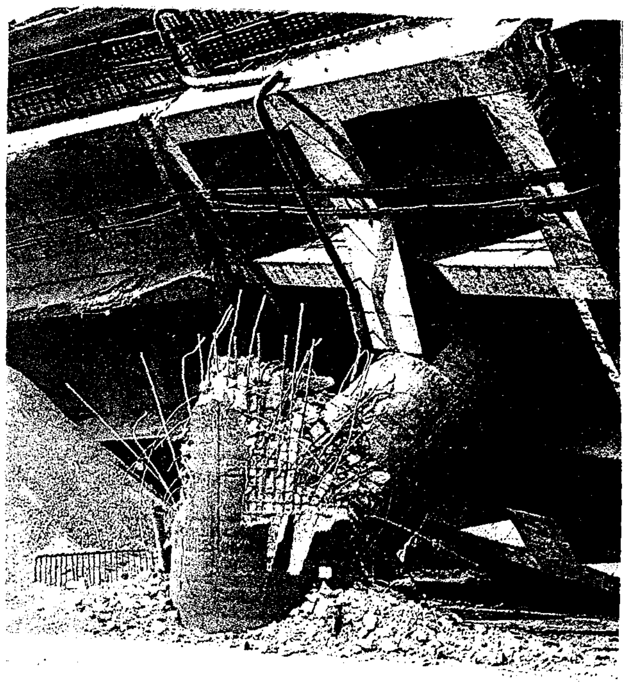


PHOTO B.8 Close view of a broken pier of Hanshin Expressway.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

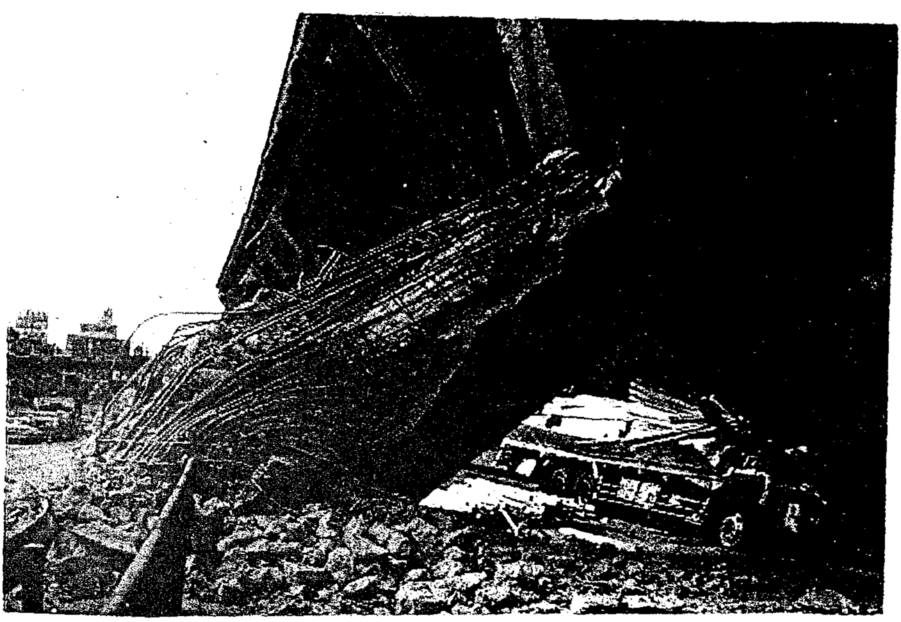


PHOTO B.9 Close view of a broken pier of Hanshin Expressway.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

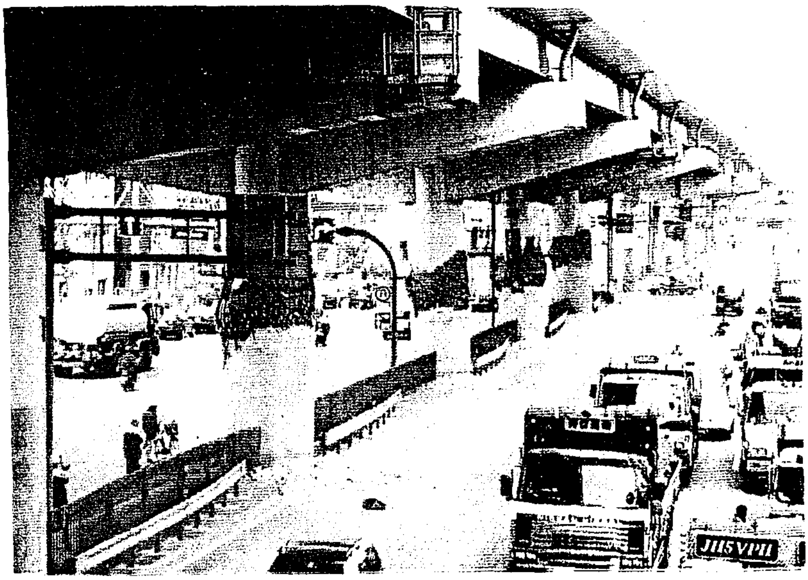


PHOTO B.10 A group of piers on Hanshin Expressway with mid-height damage.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

PHOTO B.11 A typical pier on Hanshin Expressway with mid-height damage.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

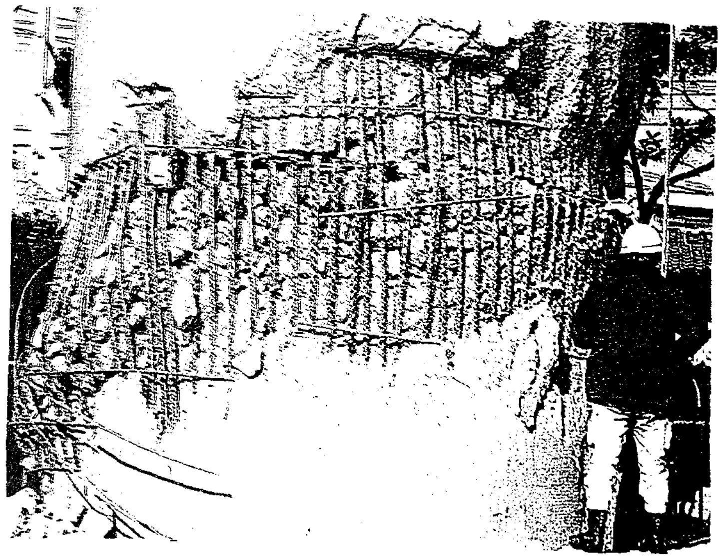


PHOTO B.12 Close view of mid-height pier damage on Hanshin Expressway.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

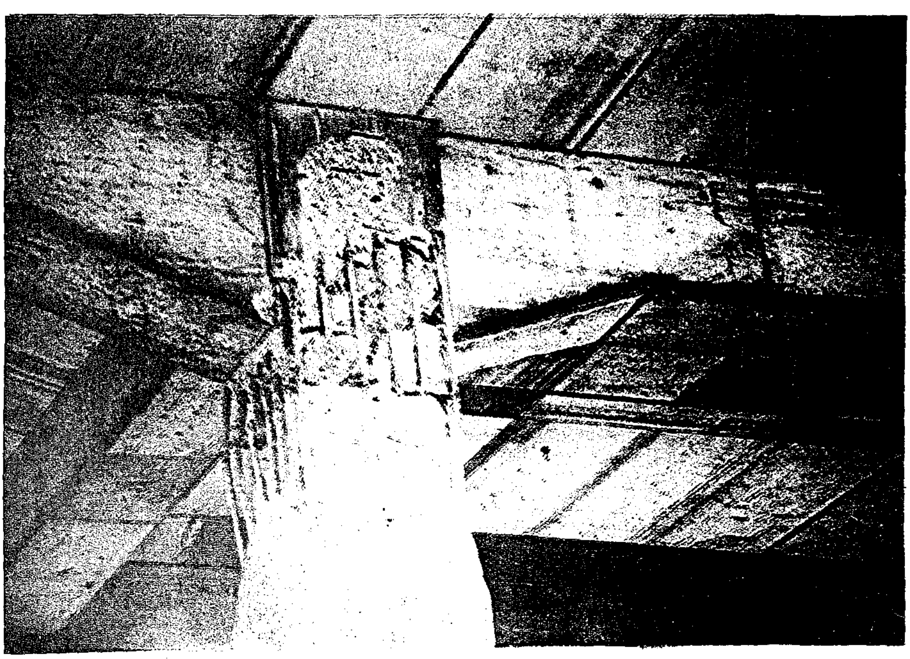


PHOTO B.13 Lack of transverse reinforcement at column-girder joints of Shinkansen Line Bridges.
(Kobe Earthquake of January 17, 1995),
(Aydınoğlu N., 1995).

APPENDIX C

AASHTO STANDARD SPECIFICATIONS FOR SEISMIC DESIGN OF HIGHWAY BRIDGES

C.1 Introduction

C.1.1 Purpose

AASHTO Standard Specifications for Seismic Design of Highway Bridges (AASHTO, 1983) establish design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes.

The AASHTO code indicates that; bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of the provisions are:

1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.

2. Realistic seismic ground motion intensities and forces are used in the design procedures.

3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accesible for inspection and repair.

C.1.2 Background

The 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges in the United States. Prior to 1971 AASHTO specifications for the seismic design of the bridges were based in part on the lateral force requirements for buildings developed by the Structural Engineers Association of California (AASHTO, 1983). In 1973 the California Department of Transportation (CALTRANS) introduced new seismic design criteria for bridges, which included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. In 1975 AASHTO adopted Interim Specifications which were a slightly modified version of the 1973 CALTRANS provisions, and made them applicable to all regions of the United States. In addition to these code changes the 1971 San Fernando earthquake stimulated research activity on seismic problems related to bridges.

C.1.3 Basic Concepts

Development of the Standards has been predicated on the following basic concepts.

- Hazard to life be minimized.
- Bridges may suffer damage but have low probability of collapse due to earthquake motions.
- Function of essential bridges be maintained.
- Design ground motions have low probability of being exceeded during normal lifetime of bridge.
- Provisions be applicable to all of the United States.
- Ingenuity of design not be restricted.

C.2 GENERAL REQUIREMENTS

C.2.1 Acceleration coefficient

In the development of the bridge standards the Effective Peak Velocity-Related Acceleration Coefficient A_v with the contour map is used to identify the Acceleration Coefficient A.

C.2.2 Importance Classification

An Importance Classification (IC) is used in conjuction with the Acceleration Coefficient (A) to determine the Seismic Performance Category (SPC) for bridges with an Acceleration Coefficient greater than 0.29.

Two importance Classifications are specified. An IC of I is assigned for essential bridges and II for all others. Essential bridges are those that must continue to function under earthquake. In the determination of the Importance Classification of a bridge the following considerations are taken into account: Social/Survival requirements, Security/Defense requirements and additionally annual daily traffic.

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

C.2.3 Seismic Performance Categories

In order to provide flexibility in specifying design provisions associated with areas of different seismic risk four Seismic Performance Categories (SPC) A through D are defined.

The four categories permit variation in the requirements for methods of analysis, minimum support lengths, column design details, foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

The Seismic Performance Category is determined from the Importance Classification and the Acceleration Coefficient. Thus, the importance of a bridge in a road network and the level of seismic exposure at a bridge site are used to determine the SPC. Different degrees of complexity in analysis and design requirements are specified for each SPC. Bridges classified as SPC D are those designed for the highest level of seismic performance and bridges classified as SPC A are those designed for the lowest level of seismic performance.

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

Table C.1- Seismic Performance Category

C.2.4 Site Effects

The effects of local soil conditions on ground motion characteristics are considered in structural design. Three different soil profile types are defined to determine the site coefficient (S).

"Soil Profile Type I" contains two parts:

1. Rock of any characteristic, whether it be shale-like or crystalline in nature. In general, such material is characterized by a shear wave velocity greater than about 762 m/sec.

2. Stiff soil conditions or firm ground including any site where soil depth is less than 61 m and the soil types overlying rock are stable deposists of sands, gravels, or stiff clays.

"Soil Profile Type II" is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 61 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

"Soil Profile Type III" is a profile with soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

C.2.5 Site Coefficient

The site coefficient (S) approximates the effects of the site conditions on the elastic response coefficient or spectrum and is given in Table C.2.

	Soil Profile Type		
	I	II	III
S	1.0	1.2	1.5

Table C.2- Site Coefficient

C.2.6 Response Modification Factor

Response Modification Factors (R) shown in Table C.3 are used to modify the component forces obtained from the elastic analysis.

The rationale used in the development of the R-Factors for columns, piers and pile bents is based on considerations of redundancy and ductility provided by the various supports.

The wall type pier is judged to have minimal ductility capacity and redundancy in its strong direction and is assigned an R-Factor of 2.

A multiple column bent with well-detailed columns is judged to have good ductility capacity and redundancy and is assigned the highest value of 5.

The ductility capacity of single columns is similar to that of columns in a multiple column bent; however, there is no redundancy and therefore a lower R-Factor of 3 is assigned to single columns to provide a level of performance similar to that of multiple column bents.

The R-Factors of 1.0 and 0.8 are assigned to connections mean that the connections are designed for the design elastic forces and for greater than the design elastic forces in the case of abutments. The reason for adopting these values is to maintain the overall integrity of the bridge structure at these important joints.

Substructure	R	Connections	R
Wall- Type Pier	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents		Expansion Joints within	
a. Vertical Piles Only	3	a Span of the Superstructure	0.8
b. One or more Batter Piles	2	Columns, Piers or Pile Bents	
Single Columns	3	to Cap Beam or Superstructure	1.0
Steel or Composite Steel and		Columns or Piers to	
Concrete Pile Bents		Foundations	1.0
a. Vertical Piles Only	5		
b. One or more Batter Piles	3		
Multiple Column Bent	5		

Table C.3- Response Modification Factor, R

C.3 Analysis Procedure

An elastic analysis procedure is used for the seismic design of bridges. The actual forces and displacements in a bridge subjected to the design ground motions may be quite different from those obtained from the elastic analysis because at these high levels of excitation the bridge may respond inelastically.

Two analytical procedures are defined and the procedure applicable for a given type of bridge, which depend on the number of spans, the geometrical complexity and the Seismic Performance Category (SPC), is given in Table C.4.

The two analysis procedures to be used are as follows:

"Procedure 1": Single-Mode Spectral Method

"Procedure 2": Multimode Spectral Method

Seismic Performance Category	Regular[1] Bridges with 2 or More Spans	Irregular[2] Bridges with 2 or More Spans
A	-	-
B	Procedure 1	Procedure 1
C	Procedure 1	Procedure 2
D	Procedure 1	Procedure 2

Table C.4- Analysis Procedure

[1] A "regular" bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports.

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

The single-mode method can be performed manually for a simple structure, such as a uniform 2-span bridge in low seismic zone. However, for all other bridge types, a hand solution is impractical and a computer-based solution becomes a necessity.

The multimode response spectrum analysis should be performed with a suitable linear dynamic analysis computer program.

In the multimode spectrum analysis, the first step is to produce a mathematical model that will represent the dynamic characteristics of the structure and produce realistic results consistent with the input parameters.

The linear dynamic analysis computer programs have the ability to calculate the mode shapes, frequencies and resulting member forces and displacements for a multimode spectral analysis. Mode shapes and frequencies are obtained from the equation

$$[k-\omega^2 m]v=0$$

using standard eigenvalue computer programs; where k and m are the known stiffness and mass matrices of the mathematical model, respectively, v is the displacement amplitude vector and ω is the frequency. This analysis will yield the dimensionless mode shapes Φ_1, Φ_2, ..., Φ_n and their corresponding circular frequencies ω_1, ω_2, ..., ω_n. The mode periods can then be obtained using

$$T_i = \frac{2\pi}{\omega_i} \quad (i=1,\ 2,\ldots,n)$$

The uncoupled normal mode equations of motion are of the form

$$\ddot{Y}_i(t) + 2\omega_i \xi_i \dot{Y}_i(t) + \omega_i^2 Y_i(t) = \frac{P_i(t)}{M_i}$$

$$(i = 1,2.....n)$$

where the subscript i refers to the mode number, Y_i, ω_i and ξ_i are the mode amplitude, frequency, and damping ratios, respectively, and

the effective modal load $P_i(t)$ and generalized mass M_i are given by

$$P_i(t) = \phi_i^T m B v_g(t)$$

$$M_i = \phi_i^T m \phi_i$$

where B is a vector containing ones and zeroes corresponding to those components in the direction of excitation $v_g(t)$ and those components in the other orthogonal directions, respectively.

The maximum absolute value of $Y_i(t)$ during the entire time-history of earthquake excitation is given by

$$Y_i(t)_{max} = \frac{T_i^2 S_a(\xi_i, T_i)}{4\pi^2} \frac{\phi_i^T mB}{\phi_i^T m\phi_i}$$

where $S_a(\xi_i, T_i)$ is the acceleration response spectral value for the prescribed earthquake excitation.

To determine the maximum value of any particular response quantity Z(t) (e.g., a shear, moment, displacement or relative displacement),

$$Z(t) = \sum_{i=1}^{n} A_i Y_i(t)$$

C.3.1 Determination of Elastic Forces and Displacements

The elastic forces and displacements are determined independently along two perpendicular axes by use of the analysis procedure. Typically the perpendicular axes are the longitudinal and transverse axes of the bridge.

C.3.2 Combination of Orthogonal Seismic Forces

A combination of orthogonal seismic forces is used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake forces in two perpendicular directions are combined to form two load cases as follows:

Load Case 1: Seismic forces and moments on each of the principle axes of a member are obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in one of the perpendicular (longitudinal) directions to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse).

Load Case 2: Seismic forces and moments on each of the principle axes of a member are obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse) to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the first perpendicular direction (longitudinal).

C.3.3 Minimum Bearing or Restrainer Force Demands

When determining the minimum bearing or restrainer force demands for the evaluation of an existing bridge, a minimum equivalent horizontal force of 0.20 times the deadload of the superstructure should be assumed.

C.3.4 Minimum Support Lengths

Minimum support lengths, N(d), for bearing seats supporting the unrestrained expansion ends of girders, as shown in Figure 3.16, are used to calculate bearing displacement C/D ratios, r_{bd}. These support lengths should be measured to normal to the face of abutment, pier, or mid-span joint. The values for minimum support length vary with the Seismic Performance Category of the bridge as given by the following formulas:

Seismic Performance Category B:

$$N(d) = 203 + 1.67L + 6.66H \text{ (mm)}$$

Seismic Performance Category C and D:

$$N(d) = 305 + 2.5L + 10H \text{ (mm)}$$

where,

L = Length of the bridge deck to the adjacent expansion joint or to the end of the bridge deck. For mid-span joints, L is the sum of L_1 and L_2, the distances to either side of the hinge. For single span bridges, L equals the length of the bridge deck. These lengths are shown in Figure C.1.

For abutments:

H = Average height of columns supporting the bridge deck to the next expansion joint. H = 0 for single-span bridges (meters).

For columns and/or piers:

H = Average height of adjacent two columns or piers (meters).

For mid-span joints:

H = Average height of adjacent two columns or piers (meters).

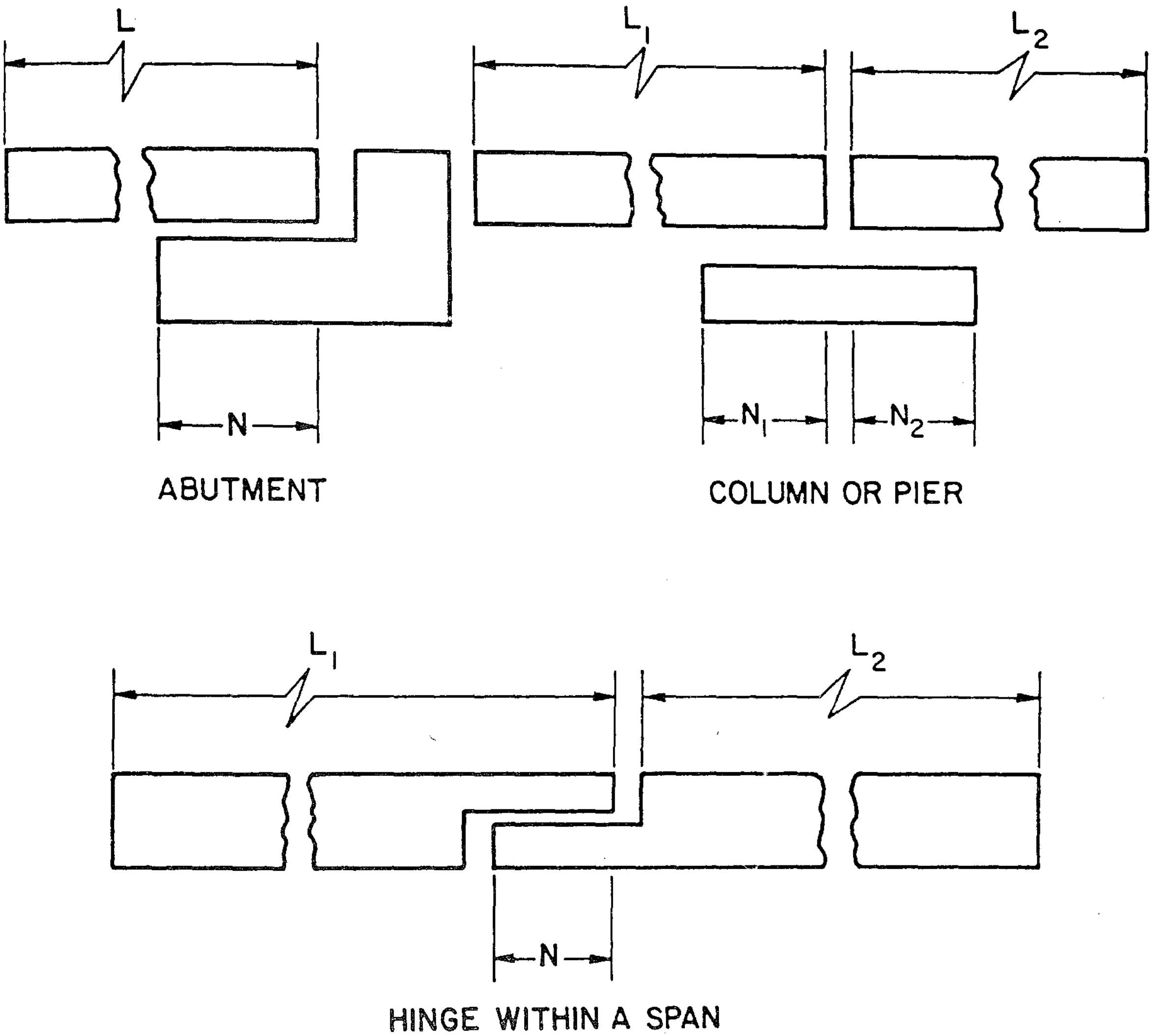


FIGURE C.1 The illustration of the minimum support length.

APPENDIX D

VULNERABILITY RATING OF THE BRIDGE COMPONENTS (ATC-6-2,1983)

Although the performance of a bridge is based on the interaction of all its components, it has been noticed in past earthquakes that certain bridge components are most vulnerable to damage. These are the bearings; columns, piers, and footings; abutments; and foundations (liquefaction damage). Bearings are the ones which are most economically retrofitted among the above components. For this reason the vulnerability rating to be used in the seismic rating system is determined by examining the bearings separately from the remainder of the structure.

I. Bearings

Bearings are used at superstructure/substructure interfaces as well as at in-span joints. The vulnerability rating for bearings will reflect the susceptibility of the bridge to a bearing failure.

Support skew has a major effect on the performance of bridge bearings. Skew is defined as the angle between the support centerline and a line perpendicular to the bridge centerline. Rocker bearings have proved to be the most vulnerable in past earthquakes. At highly skewed supports these these bearings may topple during moderate seismic shaking. When bearings may topple, it is necessary to consider the potential for collapse of the span. The potential for collapse will depend on the geometry of the bearing seat.

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

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

- Continuous structures with integral abutments.

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

1) The skew is less than 20^0, or the skew greater than 20^0 but less than 40^0 and the length-to-width ratio of the bridge deck is less 1.5.
2) Rocker bearings are not used.
3) The bearing seat on the abutment end diaphragm is continuous in the transverse direction and the bridge has in excess of three girders.
4) The support length is equal to or greater than one half the minimum required support length.

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

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

- Individual girders are supported on individual columns
- The exterior girder in a 2- or 3- girder bridge is near the edge of a continuous support.

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

Steel rocker bearings have been known to topple, resulting in a partial superstructure displacement. All bridges assigned to SPC-D are vulnerable to this type of failure. Bridges assigned to SPC-C are vulnerable only when the support skew greater than 40^0.

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

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

II. Columns, Piers, and Footings

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

<u>Step 1:</u>

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

<u>Step 2:</u>

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

Step 3:

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

Step 4:

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

$$\left(\frac{L_c}{P_s F b_{max}} \right),$$

where

L_c = Effective column length in feet.
P_s = Percent main reinforcing steel
F = Framing factor:
= 2 (multi-column bents fixed top and bottom)
= 1 (multi-column bents fixed at one end)
= 1.5 (single-column bent fixed at top and bottom-box girder)
b_{max}= Transverse column dimension (feet).

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

- $A < 0.4$ (3 points)
- Right structure-skew $\leq 20^0$ (2 points)
- Continuous structures with diaphragm abutments of approximately equal stiffness in which the length-to-width ratio of the deck is less than 4 (1 point).

Step 5:

To account for column flexural failure at a splice, the following CVR should be calculated for single-column bents supporting superstructures in excess of 90m in length or superstructure with expansion joints where the column longitudinal reinforcement is spliced at a potential plastic hinge location:

- $A \le 0.4$, CVR = 7
- $A > 0.4$, CVR = 10

Step 6:

The following CVR should be calculated for single-column bents supported on pile footings unreinforced for uplift, or poorly confined foundation shafts.

- $0.40 \le A \le 0.5$ CVR = 5.
- $A > 0.5$ CVR = 10.

III. Abutments

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

One of the major problems observed in past earthquakes has been the settlement of fill at the abutment due to excessive seismic earth pressures or seismic forces transferred from the superstructure.

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

Step 1:

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

Step 2:

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

- One percent of the fill height when $0.19 < A \leq 0.29$.
- Two percent of the fill height when $0.29 < A \leq 0.39$.
- Three percent of the fill height when $A > 0.39$.

The above settlements should be doubled if the bridge is a water crossing. When fill settlements are estimated to be greater than 15 cm, assign a vulnerability rating of 5.

Step 3:

For bridges classified as SPC-D, free-standing, earth-retaining abutments with skews greater than 40^0 where the distance between the seat and the bottom of the foundation footing exceeds 3.05 meters should be assigned a vulnerability rating of 5.

IV. Liquefaction

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

- a quantative assessment of liquefaction susceptibility,
- the magnitude of the acceleration coefficient and,
- an assessment of the susceptibility of the bridge structure itself to damage resulting from liquefaction-induced ground movement.

The observed damage has demonstrated that bridges with continuous superstructures and supports can withstand large translational deformations and usually remain serviceable (with minor repairs). However, bridges with discontinuous superstructures and/or brittle supporting members are usually severly damaged as a result of liquefaction.

The procedure is based on the following steps;

Step 1:

Determine the susceptibility of foundation soils to liquefaction.

High susceptibility is associated with conditions where:

- foundation soil providing lateral support to piles or vertical support to footings comprise on average saturated loose sands, silty sands, non-plastic silts, and
- where similar soils underly abutment fills or are present as continuous seams which could lead to abutment slope failures.

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

Low susceptibility is associated with dense soils.

Step 2:

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

- Severe liquefaction related damage :
for conditions of high susceptibility when $A > 0.29$ and for conditions of moderate susceptibility when $A > 0.4$
- Major liquefaction related damage :
for conditions of high susceptibility when $0.19 < A \leq 0.29$ or for conditions of moderate susceptibility when $0.29 < A \leq 0.39$
- Moderate liquefaction related damage :
for conditions of high susceptibility when $0.09 < A \leq 0.19$ or for conditions of moderate susceptibility when $0.19 < A \leq 0.29$
- Low liquefaction related damage :

for conditions of low susceptibility

Step 3:

A vulnerability rating of 10 is assigned for bridges subjected to severe liquefaction related damage. This rating may be reduced to 5 for single span bridges with skew less than 20^0

Step 4:

A vulnerability rating of 10 is assigned for bridges subjected to major liquefaction related damage. This rating may be reduced to between 5 and 9 for single span bridges with skew less than 40^0, and continuous multi-span bridges with skew less than 20^0 provided one of the following conditions exist:

- Reinforced concrete columns are continuous with the superstructure and have a CVR less than 5 and a height in excess of 7.5 meters.
- Steel columns (except those constructed of brittle material) are in excess of 7.5 meters.
- Columns are discontinuous with the superstructure and shifting of the superstructure will not result in instability.

Step 5:

A vulnerability rating of 5 is assigned for bridges subjected to moderate liquefaction related damage. This rating may be increased to between 6 and 10 if the vulnerability rating for the bearings is greater than or equal to 5.

APPENDIX E

PROCEDURE FOR DETAILED EVALUATION OF AN EXISTING BRIDGE (ATC-6-2, 1983)

Detailed evaluation for retrofitting includes the the determination of the seismic capacity/demand ratios for individual bridge components.

E.1 DETERMINATION OF SEISMIC CAPACITY/DEMAND RATIOS FOR BRIDGE COMPONENTS

E.1.1 Analysis Procedure

The analysis procedure is same as explained in Appendix B. AASHTO (Standard Specifications for Seismic Design of Highway Bridges, 1983) analysis procedures are used for seismic analysis of highway bridges.

E.1.2 Capacity/Demand Ratios for Expansion Joints and Bearings

a) Support Length Capacity/Demand Ratio (r_{bd})

The capacity/demand ratio for the support length 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.

Conversely, certain structural configurations are exceptionally vulnerable to collapse in the event of a loss of support at the bearings. Such structures are 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.

The support length C/D ratio is determined using the following method.

$$r_{bd} = \frac{N(c)}{N(d)} \qquad \text{(eq. E-1)}$$

where,

N(c) = The support length provided.

N(d) = The minimum support length defined in Appendix C.3.4.

The illustration of the dimensions for minimum support length requirements is shown in Appendix C, Figure C.1.

b) Force Capacity/Demand Ratio

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

$$r_{bd} = \frac{V_b(c)}{V_b(d)} \qquad \text{(eq. E-2)}$$

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.

E.1.3 Capacity/Demand Ratios for Reinforced Concrete Columns, Piers, and Footings

In the evaluation of the strength of the columns and piers, four failure modes are 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.

a) 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}:

$l_a(c)$ = Effective anchorage length of longitudinal reinforcement.

$l_a(d)$ = Required effective anchorage length reinforcement.

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

$$l_a(d) = \frac{k_s\ d_b}{\sqrt{f_c'}\,(1+ 2.5c/d_b + k_{tr})} \geq 30\ d_b \qquad \text{(eq. E-3)}$$

where

k_s = A constant for reinforcing steel with a yield stress of f_y (psi).

$$= \frac{(f_y-11000)}{4.8} \qquad \text{(eq. E-4)}$$

f_y = Yield stress in longitudinal steel reinforcement (psi)
d_b = Nominal bar diameter in inches.
f_c'= Concrete compression strength (psi).

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

and

$$k_{tr} = A_{tr}(c)\, f_{yt}/600\ sd_b \le 2.5 \qquad \text{(eq. E-5)}$$

where

$A_{tr}(c)$ = Area of transverse reinforcing

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

s = spacing of transverse reinforcement (inches).

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

b) 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 is given by the following formula:

$$A_{tr}(d) = \frac{s\ f_y}{l_s\ f_{yt}} A_b \qquad \text{(eq. E-6)}$$

where

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

If the clear spacing between spliced bars is greater than or equal to $4d_b$, where d_b is the diameter of the spliced reinforcement, $A_{tr}(c)$ will be the cross-sectional area of the confining hoop. If the clear spacing is less than $4d_b$, than $A_{tr}(c)$ will be the area of the transverse bars crossing the potential splitting crack along a row of spliced bars divided by the number of splices.

c) Column Shear

Column shear failure occurs when shear demand exceeds shear capacity. The following terms are used to calculate the C/D ratio for column shear, r_{cv}:

$V_u(d)$ = The maximum column shear force resulting from plastic hinging at both the top and bottom of the column due to yielding in the column.

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

$V_i(c)$ = The initial shear resistance of the undamaged column.

$V_f(c)$ = The final shear resistance of the damaged column.

When columns do not experience flexural yielding ($r_{ec} \geq 1.0$), the C/D ratio for column shear should be calculated using the initial shear capacity, $V_i(c)$, and the elastic shear demand, $V_e(d)$. In columns subject to yielding ($r_{ec} < 1.0$), the C/D ratio for column shear, r_{cv}, is calculated.

d) Transverse Confinement Reinforcement

Inadequate transverse confinement reinforcement in the plastic hinge region of a column can 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}:

$$r_{cc} = \mu \, r_{ec} \qquad \text{(eq. E-7)}$$

where

$$\mu = 2 + 4\left(\frac{k_1 + k_2}{2}\right)k_3 \qquad \text{(eq. E-8)}$$

where

$$k_1 = \frac{\rho(c)}{\rho(d)\left(0.5 + \dfrac{1.25P_c}{f_c'\,A_g}\right)} \leq 1 \qquad \text{(eq. E-9)}$$

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

k_3 = Efectiveness of transverse bar anchorage.
$\rho(c)$ = Volumetric ratio of existing transverse reinforcement.
$\rho(d)$ = Required volumetric ratio of transverse reinforcement.
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.

e) Footing Rotation and/or Yielding

Column footings may rotate and/or yield before columns can yield. The seismic C/D ratio for the footing rotation failure (r_{fr}) is given as follows:

$$r_{fr} = \mu \, r_{ef} \, ,$$

where μ, the ductility indicator, is taken from (Table E.1) depending on the type of footing and mode of failure.

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	3
	Reinforcing Steel Yielding in the Footing	4
	Pile Pullout at Footing	4
	Concrete Shear or Tension in the Footing	1
	Flexural Failure of Piling	4
	Shear Failure of Piling	1

Table E.1- Footing Ductility Indicators

The following procedure is used to determine the C/D ratio for columns, piers, and footings. 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.

Step 1

Determine the elastic moment demands at both ends of the column or pier for the seismic load cases.

Step 2

Calculate the nominal ultimate moment capacities for the columns.

Step 3

Calculate the set of moment C/D ratios (nominal ultimate moment capacity and elastic moment demand), r_{ec}.

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.

Step 5

Calculate the C/D ratios for the anchorage of longitudinal reinforcement, splices in the longitudinal reinforcement, 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 should also be calculated.

Step 6

Calculate the column C/D ratios for column shear.

E.1.4 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 superstructureor seismic earth pressures. Usually these types of failures alone do not result in collapse 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 oftenthe 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), is the elastic displacements at the abuments. The displacement capacity, d(c), is taken as three inches (7.5 cm) in the transverse direction and six inches (15 cm) in the longitudinal direction.

$$r_{ad} = \frac{d(c)}{d(d)}$$

E.1.5 Capacity/Demand Ratios for Liquefaction Induced Foundation Failure

Many foundation failures during earthquakes are the result of loss of foundation support occuring as a result of liquefaction.

A C/D ratio should be calculated when the preliminary screening indicates the potential exists for a major or severe liquefaction related foundation damage. To determine the C/D ratio for liquefaction failure, r_{sl}, a two-stage procedure is necessary. First The 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:

$$r_{sl} = \frac{A_L(c)}{A_L(d)} ,$$

where

$A_L(c)$ = The effective peak ground acceleration at which liquefaction failures are likely to occur.
$A_L(d)$ = A = Design acceleration coefficient for the bridge site.

It is difficult to determine the parameter $A_L(c)$, and selection of a realistic value for $A_L(c)$ requires considerable engineering judgement. Details of bridge related liquefaction failure is given in (Appendix D).

APPENDIX F

INPUT DATA OF THE K1 HIGHWAY BRIDGE MODEL

```
K1 HIGHWAY BRIDGE
SYSTEM
V=25

RESTRAINTS
111 911 100   R=1,1,1,1,1,1    : PIER FIXED END JOINTS
112 912 100   R=1,1,1,1,1,1    : PIER FIXED END JOINTS
113 913 100   R=1,1,1,1,1,1    : PIER FIXED END JOINTS
114 914 100   R=1,1,1,1,1,1    : PIER FIXED END JOINTS
115 915 100   R=1,1,1,1,1,1    : PIER FIXED END JOINTS

JOINTS
C BEAM JOINTS
1    X=  0.425   Y=0
6    X= 25.20    Y=0     G=1,6,1
7    X= 25.60    Y=0
8    X= 25.65    Y=0
9    X= 26.05    Y=0
14   X= 51.25    Y=0     G=9,14,1
17   X= 52.10    Y=0
22   X= 77.30    Y=0     G=17,22,1
25   X= 78.15    Y=0
30   X=103.35    Y=0     G=25,30,1
33   X=104.20    Y=0
38   X=129.40    Y=0     G=33,38,1
41   X=130.25    Y=0
46   X=155.45    Y=0     G=41,46,1
49   X=156.30    Y=0
54   X=181.50    Y=0     G=49,54,1
55   X=181.90    Y=0     G=7,55,8
56   X=181.95    Y=0     G=8,56,8
57   X=182.35    Y=0
62   X=207.55    Y=0     G=57,62,1
C EXPANSION JOINTS
1001  X=  0.425  Y=0
1002  X=207.55   Y=0
C BEARINGS' BOTTOM JOINTS
221   X= 25.20    Y=0   Z=-1.475
222   X= 26.05    Y=0   Z=-1.475
821   X=181.50    Y=0   Z=-1.475        G=221,821,100
822   X=182.35    Y=0   Z=-1.475        G=222,822,100
```

```
C PIER CAP JOINTS
101    X=0.425     Y=-6.965  Z=-1.475
105    X=0.425     Y= 6.965  Z=-1.475  G=101,105,1
201    X=25.625    Y=-6.965  Z=-1.475
205    X=25.625    Y= 6.965  Z=-1.475  G=201,205,1
301    X=51.675    Y=-6.965  Z=-1.475
305    X=51.675    Y= 6.965  Z=-1.475  G=301,305,1
401    X=77.725    Y=-6.965  Z=-1.475
405    X=77.725    Y= 6.965  Z=-1.475  G=401,405,1
501    X=103.775   Y=-6.965  Z=-1.475
505    X=103.775   Y= 6.965  Z=-1.475  G=501,505,1
601    X=129.825   Y=-6.965  Z=-1.475
605    X=129.825   Y= 6.965  Z=-1.475  G=601,605,1
701    X=155.875   Y=-6.965  Z=-1.475
705    X=155.875   Y= 6.965  Z=-1.475  G=701,705,1
801    X=181.925   Y=-6.965  Z=-1.475
805    X=181.925   Y= 6.965  Z=-1.475  G=801,805,1
901    X=207.550   Y=-6.965  Z=-1.475
905    X=207.550   Y= 6.965  Z=-1.475  G=901,905,1
C FOOTING JOINTS
111    X=0.425     Y=-6.965  Z=-9.05
115    X=0.425     Y= 6.965  Z=-9.05   G=111,115,1
211    X=25.625    Y=-6.965  Z=-10.23
215    X=25.625    Y= 6.965  Z=-10.23  G=211,215,1
311    X=51.675    Y=-6.965  Z=-11.41
315    X=51.675    Y= 6.965  Z=-11.41  G=311,315,1
411    X=77.725    Y=-6.965  Z=-12.59
415    X=77.725    Y= 6.965  Z=-12.59  G=411,415,1
511    X=103.775   Y=-6.965  Z=-13.77
515    X=103.775   Y= 6.965  Z=-13.77  G=511,515,1
611    X=129.825   Y=-6.965  Z=-12.95
615    X=129.825   Y= 6.965  Z=-12.95  G=611,615,1
711    X=155.875   Y=-6.965  Z=-10.13
715    X=155.875   Y= 6.965  Z=-10.13  G=711,715,1
811    X=181.925   Y=-6.965  Z=-8.81
815    X=181.925   Y= 6.965  Z=-8.81   G=811,815,1
911    X=207.550   Y=-6.965  Z=-8.99
915    X=207.550   Y= 6.965  Z=-8.99   G=911,915,1
```

```
C PIERS
107    X=0.425     Y=-3.482  Z=-4.525
109    X=0.425     Y= 3.482  Z=-4.525
207    X=25.625    Y=-3.482  Z=-5.115
209    X=25.625    Y= 3.482  Z=-5.115
307    X=51.675    Y=-3.482  Z=-5.705
309    X=51.675    Y= 3.482  Z=-5.705
407    X=77.725    Y=-3.482  Z=-6.295
409    X=77.725    Y= 3.482  Z=-6.295
507    X=103.775   Y=-3.482  Z=-6.885
509    X=103.775   Y= 3.482  Z=-6.885
607    X=129.825   Y=-3.482  Z=-6.475
609    X=129.825   Y= 3.482  Z=-6.475
707    X=155.875   Y=-3.482  Z=-5.065
709    X=155.875   Y= 3.482  Z=-5.065
807    X=181.925   Y=-3.482  Z=-4.405
809    X=181.925   Y= 3.482  Z=-4.405
907    X=207.550   Y=-3.482  Z=-4.495
909    X=207.550   Y= 3.482  Z=-4.495

CONSTRAINTS
221 821 100   C=0,0,0,203,203,203      I=0,0,0,100,100,100
222 822 100   C=0,0,0,203,203,203      I=0,0,0,100,100,100
1             C=0,1001,1001,1001,0,0
1002          C=0,62,62,62,0,0

FRAME
NM=9
C BEAMS
1 A=1.4325 J=0.729   I=0.372054,142.1018 E=3.455E7 W=35.8125*6
  M=3.65*6
C BEARINGS
2 A=0.2025 J=473.216 I=0.0344,0.0344     E=2.52E6
C PIER CAPS
3 A=2.34   J=1.3182  I=0.33,0.63         E=3.455E7   M=5.96
C RIGID ELEMENTS AT PIER CAPS
4 A=1000   J=1000    I=1000,10000        E=3.455E7  W=0   M=0
C PIERS
5 A=2.138  J=0.728   I=0.3638,0.3638     E=3.455E7   M=5.45
C SLAB OVER PIER CAP
6 A=3.4825 J=0.0717  I=0.0181,56.313     E=3.455E7   M=8.875
C ABUTMENT PIERS
7 A=2.25   J=0.712   I=0.4219,0.4219     E=3.455E7   M=5.73
C FOOTINGS
8 A=7.5    J=4.56    I=1.406,15.625      E=3.455E7   M=5.73
C ABUTMENT FOOTINGS
9 A=12.75  J=8.5     I=2.39,76.765       E=3.455E7   M=5.73
```

```
C BEAMS
1 1 2    M=1    LP=1,0     G=5,1,1,1
7 8 9                      G=6,1,1,1
14 16 17                   G=6,1,1,1
21 24 25                   G=6,1,1,1
28 32 33                   G=6,1,1,1
35 40 41                   G=6,1,1,1
42 48 49                   G=6,1,1,1
49 56 57                   G=5,1,1,1
C SLAB
55 7  8     M=6  LP=1,0  G=6,1,8,8
C BEARINGS
71  1001  103    M=2  LP=-3,0  LR=1,0,0,1,0,0
72  6     221                  LR=1,0,0,1,0,0     G=6,2,8,100
73  9     222                  LR=1,0,0,1,0,0     G=6,2,8,100
86  1002  903                  LR=1,0,0,1,0,0
C PIER CAPS
101 101 102   M=3 LP=-3,0    G=3,1,1,1
201 201 202                  G=3,1,1,1
301 301 302                  G=3,1,1,1
401 401 402                  G=3,1,1,1
501 501 502                  G=3,1,1,1
601 601 602                  G=3,1,1,1
701 701 702                  G=3,1,1,1
801 801 802                  G=3,1,1,1
901 901 902                  G=3,1,1,1
C RIGID ELEMENTS AT PIER CAPS
87 221 203   M=4 LP=1,0      G=6,2,100,100
88 203 222                   G=6,2,100,100
C PIERS
111 102 107   M=7 LP=-3,0  G=1,1,5,5
113 104 109                G=1,1,5,5
211 202 207   M=5 LP=-3,0  G=1,1,5,5
213 204 209                G=1,1,5,5
311 302 307                G=1,1,5,5
313 304 309                G=1,1,5,5
411 402 407                G=1,1,5,5
413 404 409                G=1,1,5,5
511 502 507                G=1,1,5,5
513 504 509                G=1,1,5,5
611 602 607                G=1,1,5,5
613 604 609                G=1,1,5,5
711 702 707                G=1,1,5,5
713 704 709                G=1,1,5,5
811 802 807                G=1,1,5,5
813 804 809                G=1,1,5,5
911 902 907   M=7 LP=-3,0  G=1,1,5,5
913 904 909                G=1,1,5,5
```

```
C FOOTINGS
121 111 112   M=3 LP=-3,0     G=3,1,1,1
221 211 212                   G=3,1,1,1
321 311 312                   G=3,1,1,1
421 411 412                   G=3,1,1,1
521 511 512                   G=3,1,1,1
621 611 612                   G=3,1,1,1
721 711 712                   G=3,1,1,1
821 811 812                   G=3,1,1,1
921 911 912                   G=3,1,1,1

SPEC
A=0 S=.20*9.81 D=.05
0.00       1
0.10       2
0.15       2.5
0.20       2.5
0.30       2.5
0.40       2.5
0.50       2.5
0.55       2.5
0.60       2.38027
0.70       2.10282
0.80       1.81195
0.90       1.60584
1.0        1.43956
1.1        1.31542
1.2        1.22279
1.3        1.13525
1.4        1.06484
1.5        0.9791570
1.6        0.9096805
1.7        0.8415932
1.8        0.7892542
1.9        0.7480315
2.0        0.7207040
2.1        0.6892080
2.2        0.6623437
2.3        0.6225105
2.4        0.5979620
2.5        0.5845298
2.6        0.5669292
2.7        0.5493285
2.8        0.5354330
2.9        0.5243167
3.0        0.5141270
```

APPENDIX G

SEISMIC RETROFITTING OF HIGHWAY BRIDGES

G.1 General

Retrofitting is the goal of minimizing the probability of total collapse and/or severe structural damage of the bridge.

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

When selecting appropriate measures for retrofitting, the overall capacity of the structure to resist earthquakes must be considered. An analysis of the existing structure is usually performed to identify weak links in the seismic resistance of the bridge. These weaknesses are reflected in the capacity/demand ratios for various components.

The use of expansion joint restrainers is the most popular and has proved to be an economical method of retrofitting, whereas measures such as column and liquefaction related are quite expensive in general. Economic and practical considerations are also important in the final section of a retrofit scheme.

G.2 Seismic Performance Requirements

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

1. Loss of support at the bearings which will result in a partial or total collapse of the bridge.

2. Excessive strength degradation of the supporting components.

3. Abutment and foundation failures resulting in loss of accessability to the bridge.

Once it has been decided to retrofit a component, it is recommended that the component to be retrofited should be designed in accordance with the standards for new construction. The following sections give special design requirements for each of type of the retrofiting procedure.

G.3 Bearing and Expansion Joints

The loss of support at the bearings was one of the most common types of failure on the bridges during past earthquakes. Such failures could simply and inexpensively prevented by retrofitting. Several retrofitting methods exist as explained in the following paragraphs.

G.3.1 Longitudinal Joint Restrainers

Longitudinal joint restrainers are installed to limit the relative displacement at joints and decrease the chance of a loss of support at these locations. 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.

Restrainers should be placed symmetrically to minimize the introduction eccentricities. The consequences of a premature restrainer failure should be considered. For example, the restrainer detail shown in Figure G.1 is 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.

Longitudinal restrainers should be oriented along the principal direction of expected movement. If piers are rigid in the transverse direction as shown in Figure G.2 the movement of the superstructure is along the longitudinal axis of the bridge, and restrainers should be placed accordingly. However, in a skewed bridge with transversely flexible supports, superstructure rotation can occur. In this case restrainers are more effective if placed to normal to the expansion joint as shown in Figure G.3.

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 G.4. This detail tends to prevent bearings from becoming unseated. Since each of the restrainers can only resist movement in one direction, and the closure of the expansion joint will transfer the inertia forces of one span, each restrainer must resist inertia forces of both spans. It should be noticed that in Figure G.4 the restrainers are connected to the bottom flange. This will prevent the possibility of tearing the web as mentioned earlier.

In some cases it may not be appropriate to use the positive tie to the pier. In this case adjacent spans may be tied as shown in Figure G.5.

Steel cables and bars are the most frequently used structural devices for restraining expansion joints against excessive movements. These devices do not dissipate any significant amount of energy since they are generally designed to remain elastic. Cable and bar restrainers may permit the ends of girders to be damaged, but the damage can usually be repairable and not to allow the spans to lose support. Although cables and bars do not meet all the criteria of an ideal

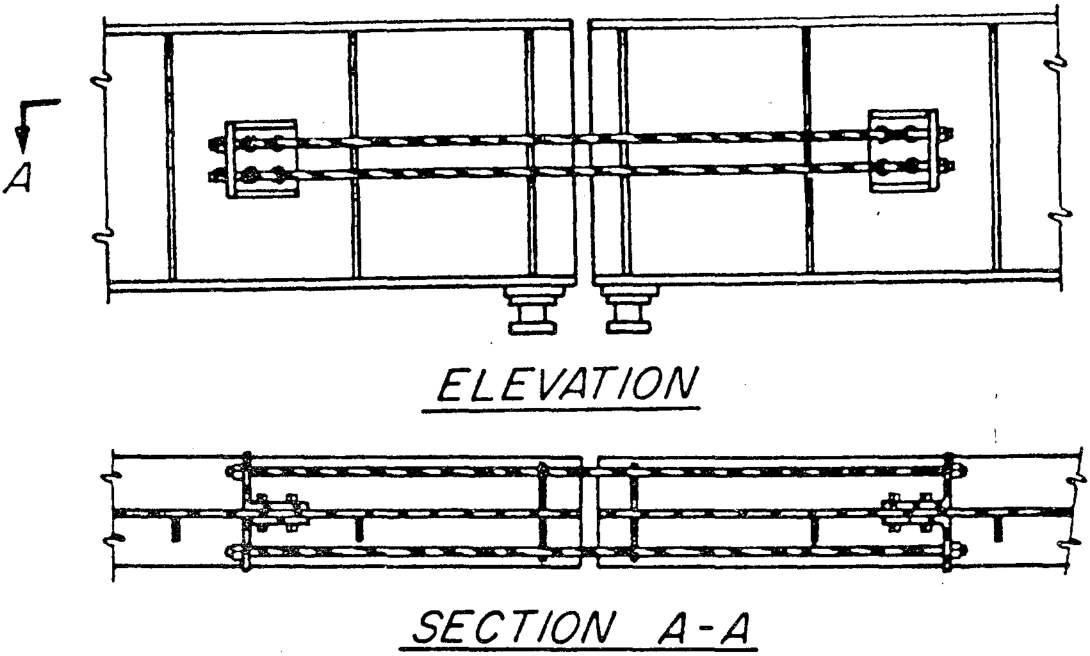


FIGURE G.1 Undesirable restrainer detail (ATC, 1983).

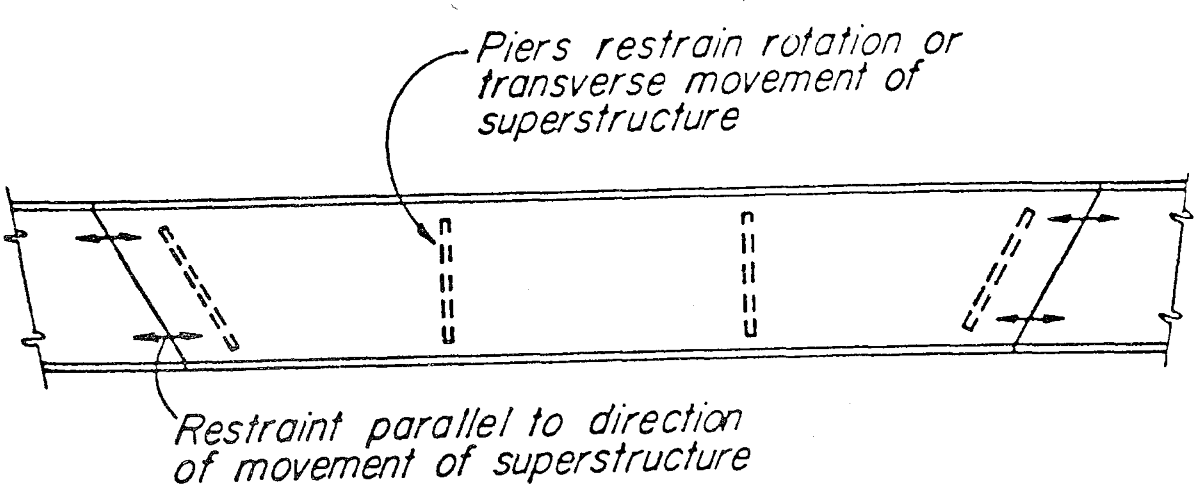


FIGURE G.2 Restrainer orientation-transversely rigid supports (ATC, 1983).

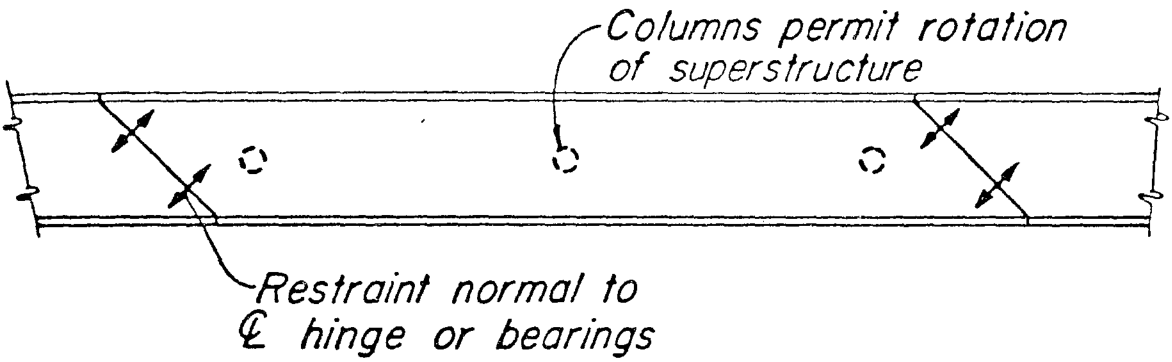


FIGURE G.3 Restrainer orientation-transversely flexible supports (ATC, 1983).

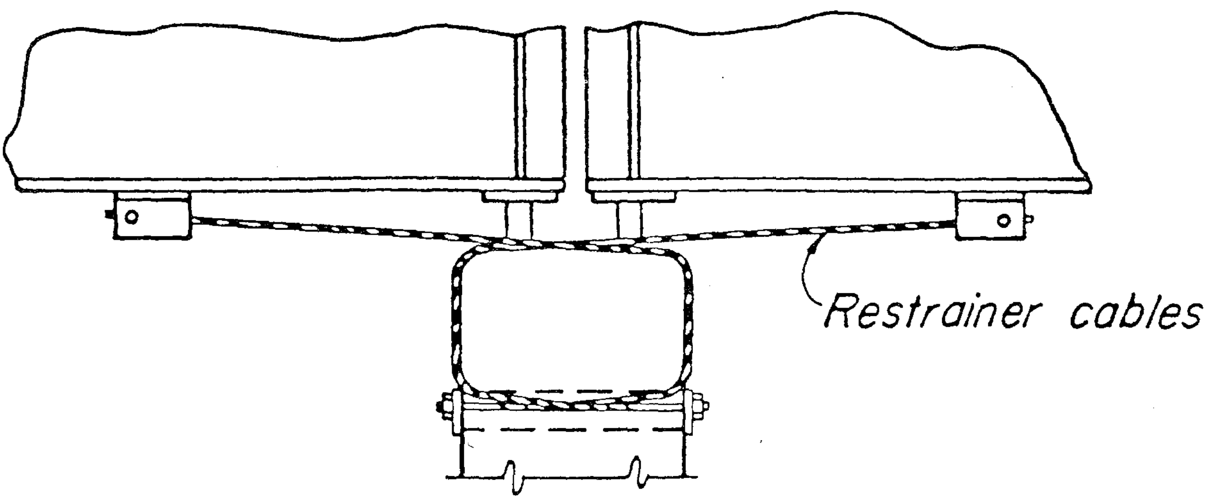


FIGURE G.4 Restrainer at pier-positive tie to pier (ATC, 1983).

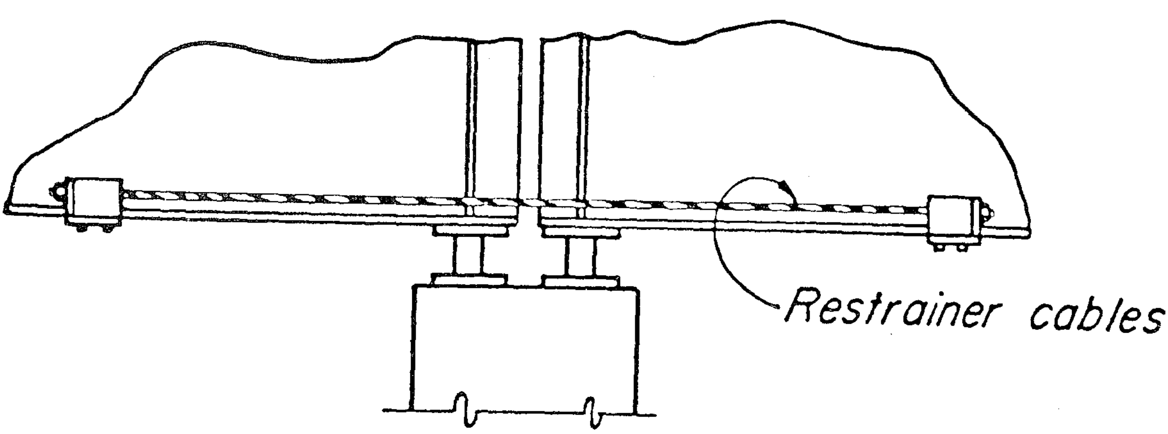


FIGURE G.5 Restrainer at pier-no positive tie to pier (ATC, 1983).

restrainer, they are relatively simple to install and economical.

Figure G.6 shows a method for retrofitting a mid-span expansion joint in a concrete box girder. Rigid steel bars may be used to prevent seperation of the joint. Concrete bolsters are sometimes necessary to strengthen the concrete diaphrams to accomodate the force transmitted from the restrainers.

An alternate method for restraining joints when the diaphram is weak, is to attach restrainers to the sides of the girders or to the underside of the deck. In this case, it is necessary to locate restrainer anchors a sufficient distance from the joint to prevent damage to the ends of the span. A detail in which restrainers are anchored to the deck is shown in Figure G.7.

G.3.2 Transverse Bearing Restrainers

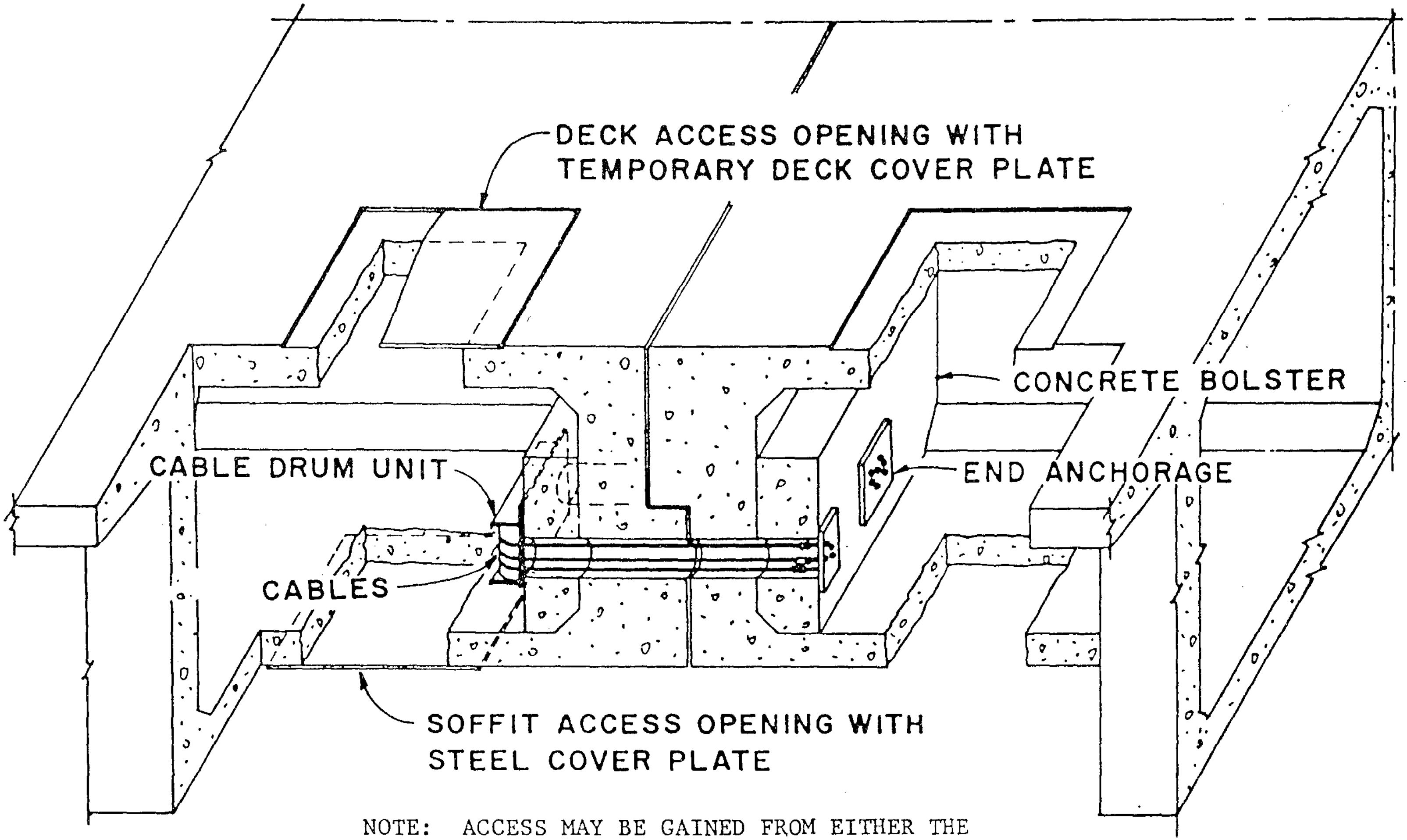
Transverse restraint at bearings is intended to prevent unacceptable damage resulting from excessive transverse motion. Transverse bearing restrainers are usually designed to resist load elastically. As it is seen from the Figure G.8 a double extra strong steel pipe filled with concrete that passes through the joint is used to provide transverse restraint.

G.3.3 Vertical Motion Restrainers

Vertical motion restrainers are desirable to prevent damage or loss of stability at the bearings. Use of vertical motion restrainers are considered only when longitudinal restrainers are contemplated and the bridge is in SPC-D. A possible hold-down detail is shown in Figure G.9.

G.3.4 Bearing Seat Extension

A bearing seat extension may be considered as a retrofit



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

FIGURE G.6 Longitudinal joint restrainer for concrete box girder (ATC, 1983).

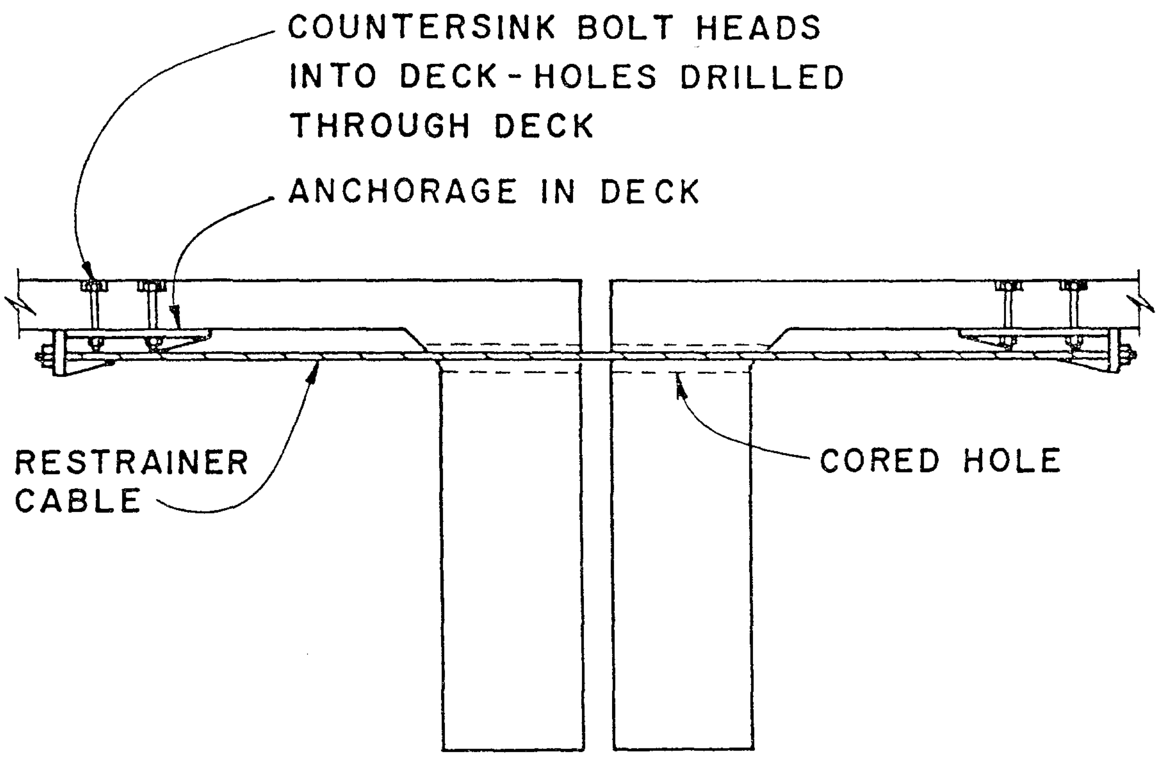


FIGURE G.7 Expansion joint restrainers tied to the concrete deck (ATC, 1983).

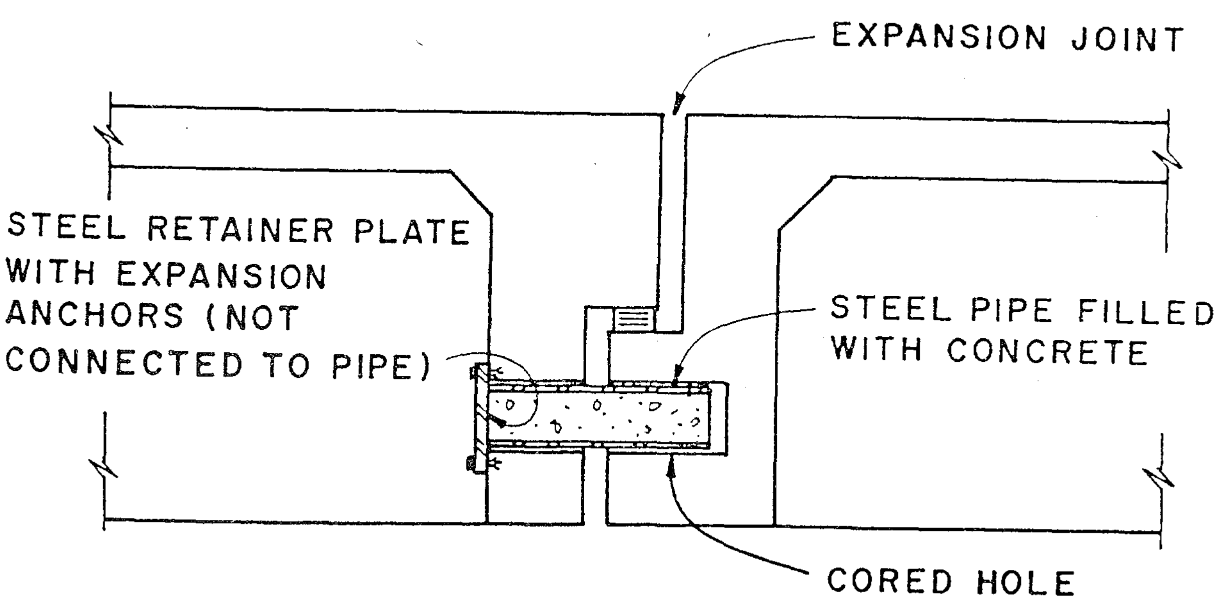


FIGURE G.8 Transverse restrainer retrofit for concrete bridge (ATC, 1983).

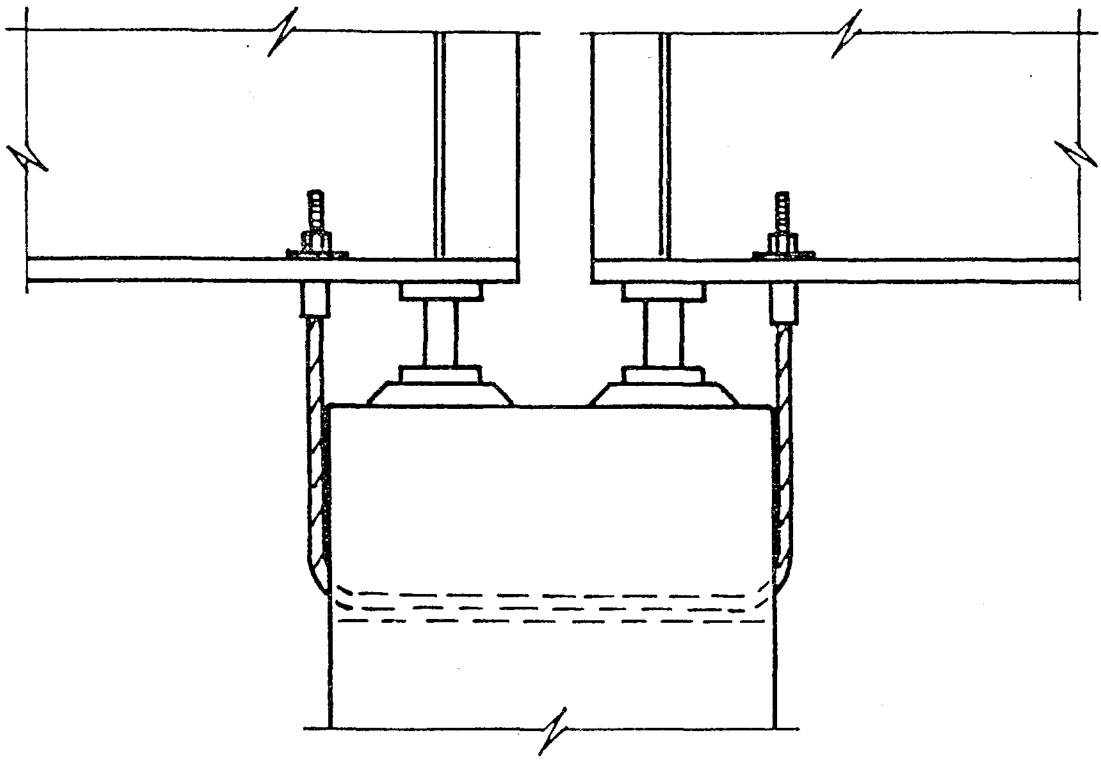


FIGURE G.9 Vertical motion restrainer retrofit (ATC, 1983).

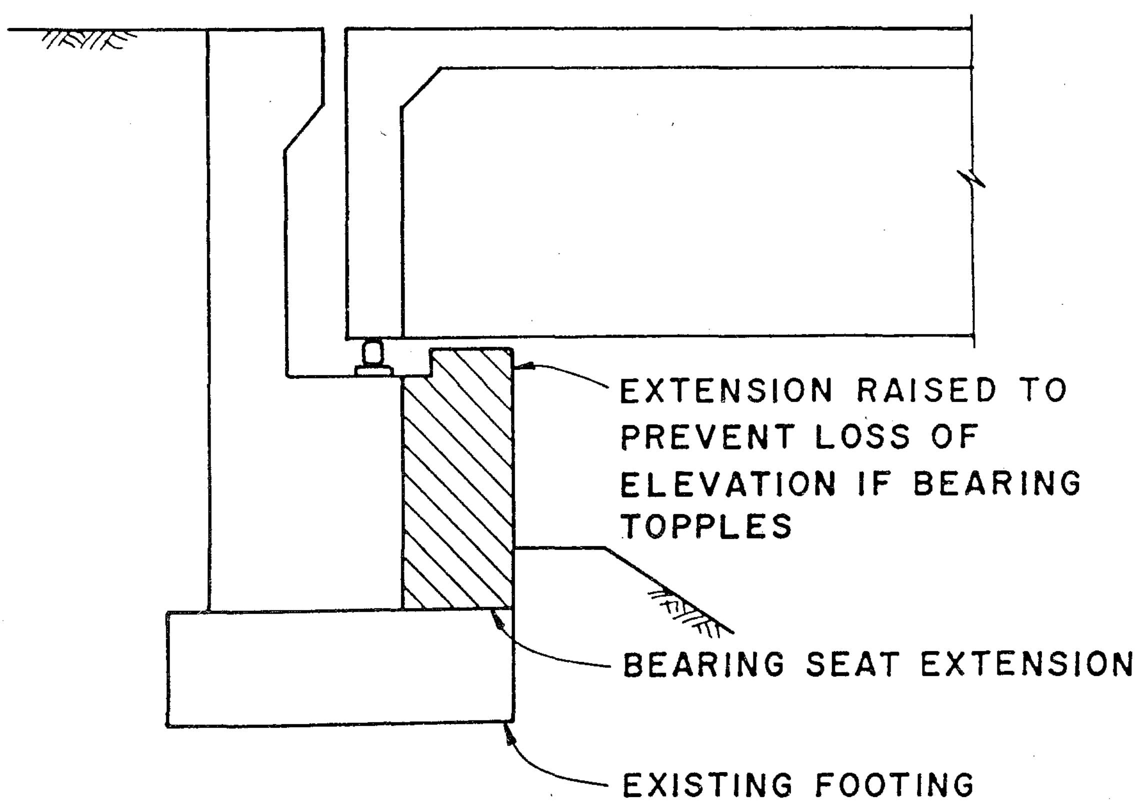


FIGURE G.10 Bearing seat extension at abutment (ATC, 1983).

measure when it is impractical to restrain movement enough to prevent loss of support at the bearings. If possible, at abutments, these extensions should be supported directly on the foundation as shown in Figure G.10.

All bearing seat extensions should provide a final minimum seat width equal to or greater than required by AASHTO (AASHTO, 1991). The design forces for bearing seat extensions are intended to consider the forces to which a bearing seat may be subjected during an earthquake large enough to cause bearings to become unseated.

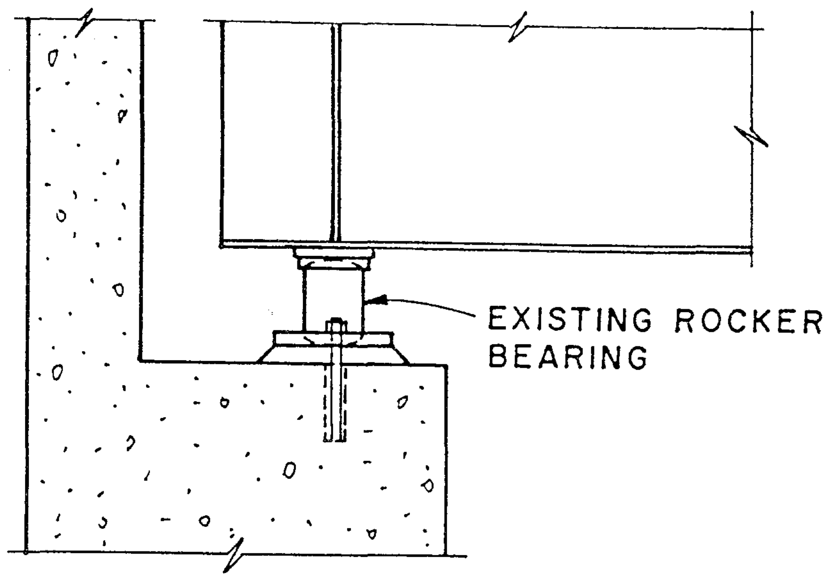
G.3.5 Replacement of Bearings

Replacement of bearings should be considered if their failure will result in collapse or loss of function of the superstructure.

Steel rocker bearings are particularly vulnerable to damage during an earthquake. This type of bearing is a prime candidate for replacement by more seismically resistant bearings such as elastomeric bearing pads.

One possibility is to replace, high rocker bearings by a prefabricated steel bearing assembly and elastomeric bearing pads. The details for such a retrofit scheme are shown in Figure G.11.

Another possible solution for replacing steel rocker bearings is shown in Figure G.12. In this case a concrete cap is used to build up the elevation difference between a replacement elastomeric bearing and the original high steel rocker bearing. With this method of replacement, the concrete cap can be constructed at a higher elevation between girders to serve as a transverse shear key. In addition, vertical motion restrainers can be anchored in the new concrete cap.



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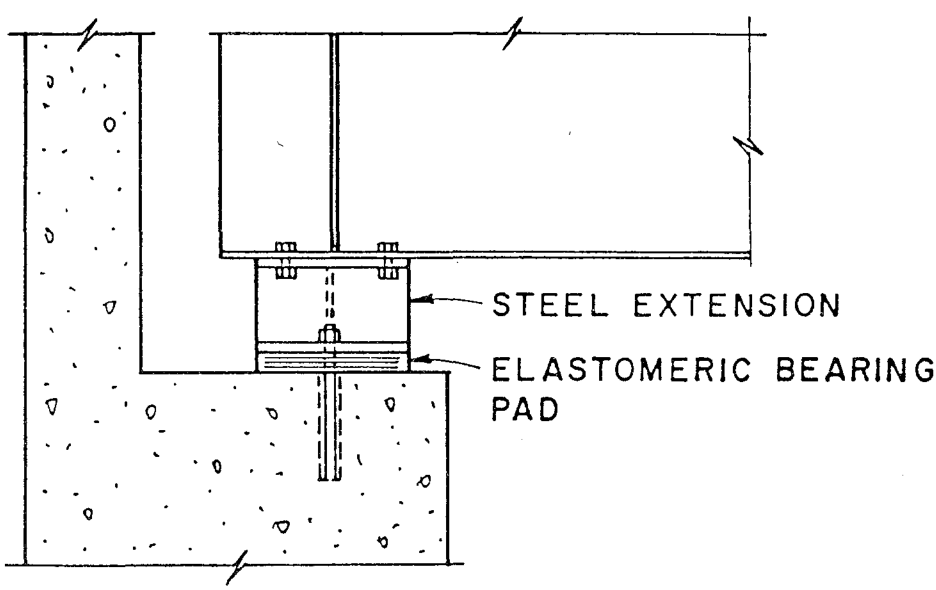
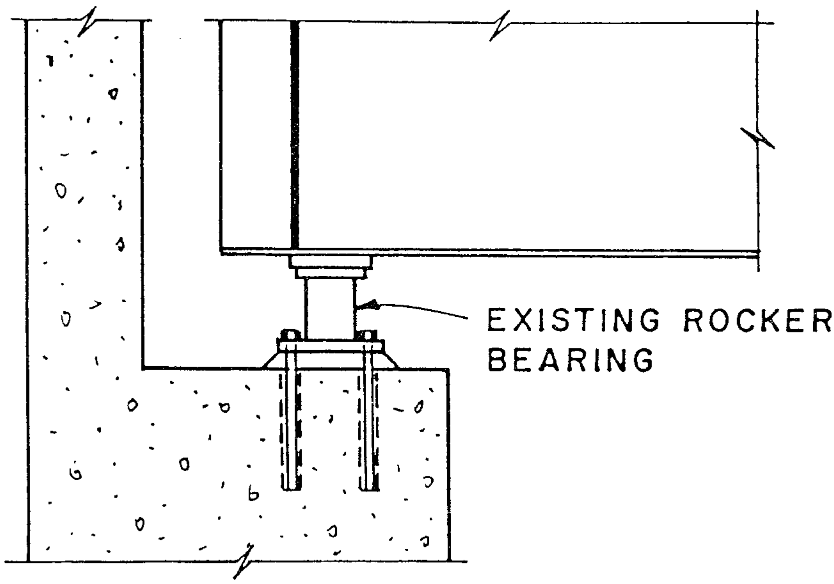


After

FIGURE G.11 Replacement of rocker bearings-steel extension (ATC, 1983).



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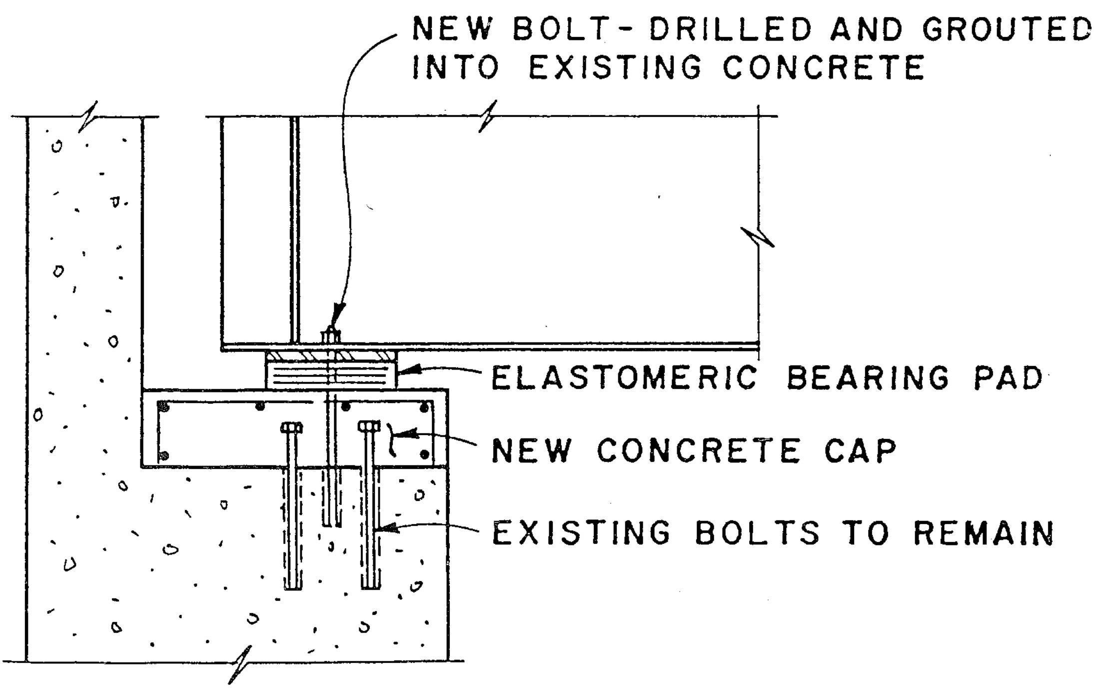


After

FIGURE G.12 Replacement of rocker bearings-concrete cap (ATC, 1983).

G.3.6 Special Earthquake Resistant Bearings and Devices

Special earthquake-resistant bearings and devices are used for isolation, energy absorbtion, and/or restraint to limit seismic forces and displacements to acceptable levels. In addition to performing under normal service conditions, an earthquake-resistant bearing should be capable of resisting seismically induced forces, restricting relative displacements within the bridge, dissipating energy, and returning the structure to its pre-earthquake position. A bearing system having these capabilities might be composed of the components shown in Figure G.13. Vertical support is provided by a flexible bearing and/or sliding support isolator. In the case of a "fixed" bearing, a fuse is used to prevent movement under service conditions, but can fail during a large earthquake. During rapid movement a motion induced arrester engages an energy dissipator or stopper. Excessive relative displacements are prevented by a restrainer with a gap to allow limited displacements. Following an earthquake, the flexible support provides a restoring force to bring the structure back to its pre-earthquake position. Since it is difficult to consider a self contained bearing with all of these capabilities, it is useful to think in terms of a "bearing system" that may be composed of bearings and other devices.

In New Zealand, several bridges have been constructed utilizing special energy-dissipating devices. Some of the devices initially considered are shown in Figure G.14. The devices shown are used to connect the bridge superstructure to the substructure and are usually installed in parallel with elastomeric bearing pads. At low levels of lateral load such as may occur in a moderate earthquake or due to wind, the devices will remain elastic and restrain movement at the bearings. During strong seismic shaking, the devices yield, allowing translation at the bearings. When the devices yield, the load transmitted from the superstructure to the substructure is limited to the ultimate capacity of the

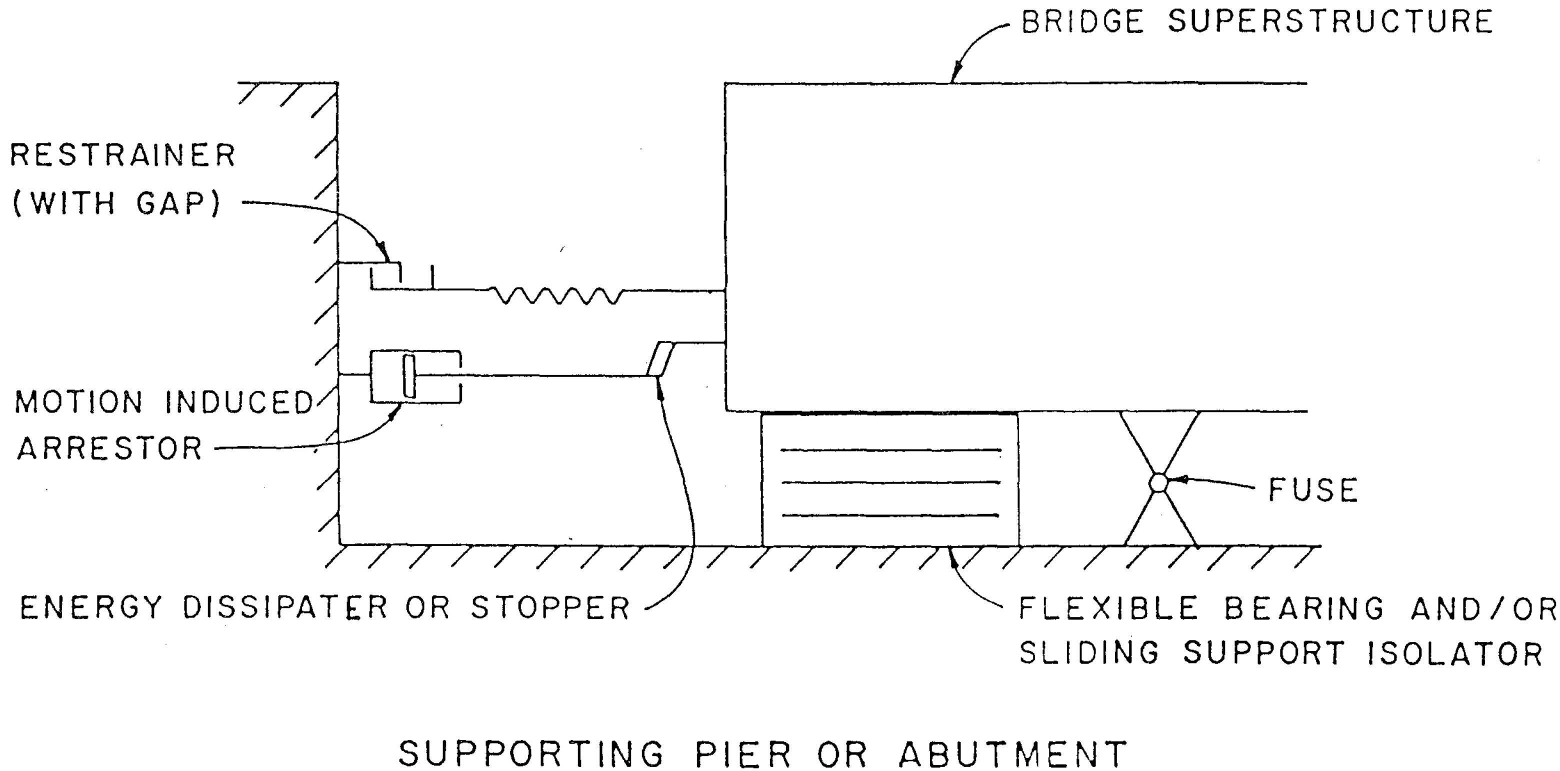


FIGURE G.13 An ideal bearing device (ATC, 1983).

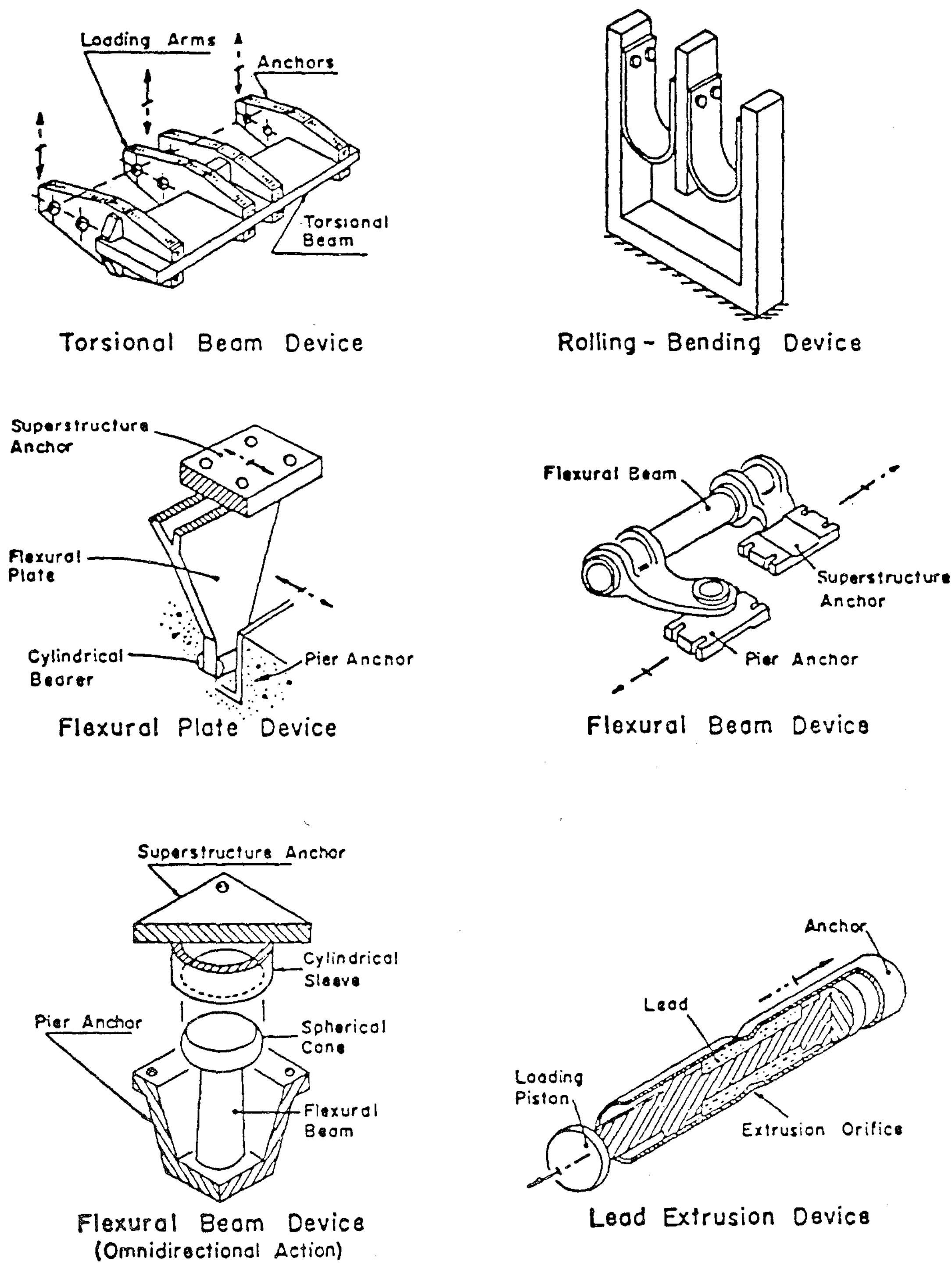


FIGURE G.14 New Zealand energy dissipation devices (ATC, 1983).

devices. In addition, energy is dissipated during yielding which tends to damp the seismic response.

An elastomeric bearing pad which has a circular core removed and replaced by lead is used in New Zealand. This concept is shown in Figure G.15. The lead can deform many times under gradual movement such as it occurs due to temperature change or creep. Under rapid movement such as it would occur during a strong earthquake, the lead resists greater loads and dissipate energy. As a retrofit technique, old bearings are replaced with elastomeric bearing pad with a lead core in New Zealand.

In Japan, viscous damping devices called "menshin devices" (Kawashima K., 1994b) are used in the expansion joints. The device allows the expansion joint to open and close during normal temperature movement but limits the relative movement of the joint during an earthquake.

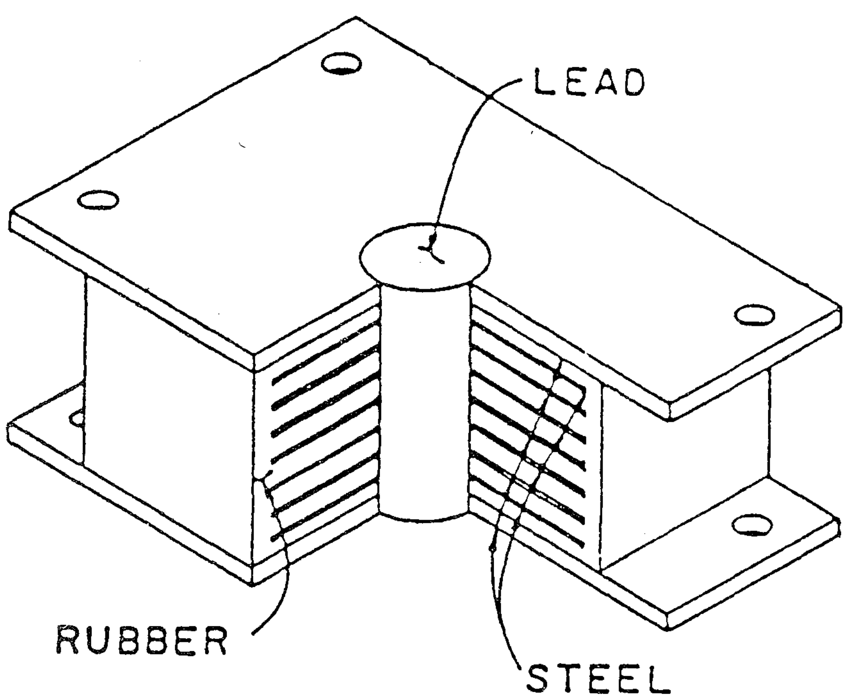


FIGURE G.15 Elastomeric bearing pad with a lead core (ATC, 1983).

G.4 Reinforced Concrete Columns, Piers, and Footings

Reinforced concrete columns, piers, and footings may fail in several ways during an earthquake. In general, it is more difficult and less cost effective to retrofit these components than it is with the bearings.

G.4.1 Force Limiting Devices

A force-limiting device provides a mechanism that limits the amount of force that can be transferred between the superstructure and supporting substructure. The TFE (Teflon) sliding bearing is the most common type of a simple force-limiting device. It provides a very small transfer of force.

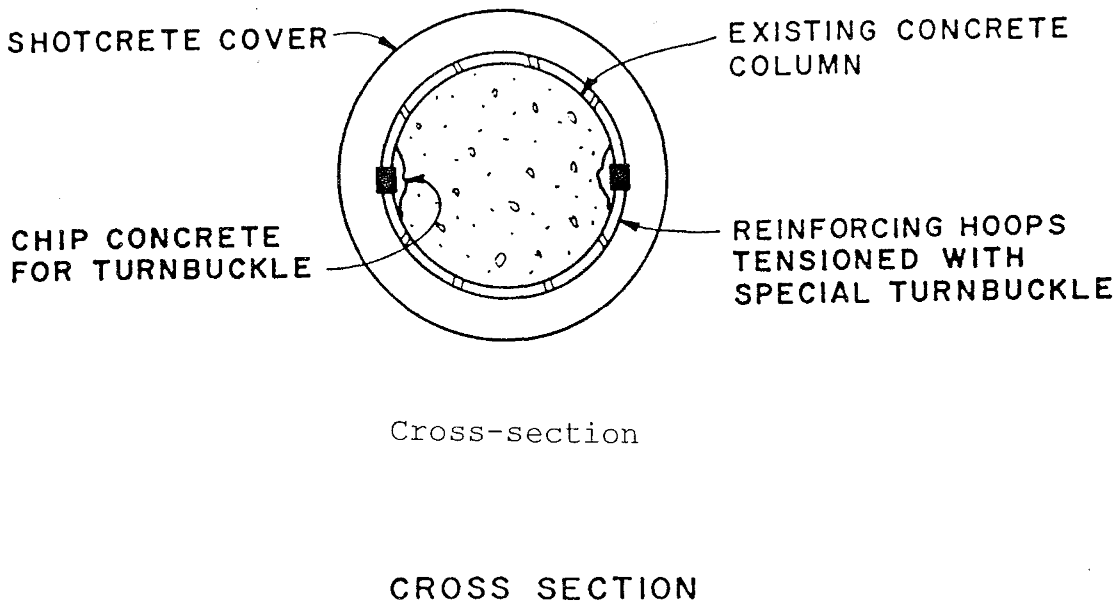
The use of force-limiting devices should be restricted to devices whose dynamic performance has been demonstrated by physical testing.

G.4.2 Increased Transverse Confinement

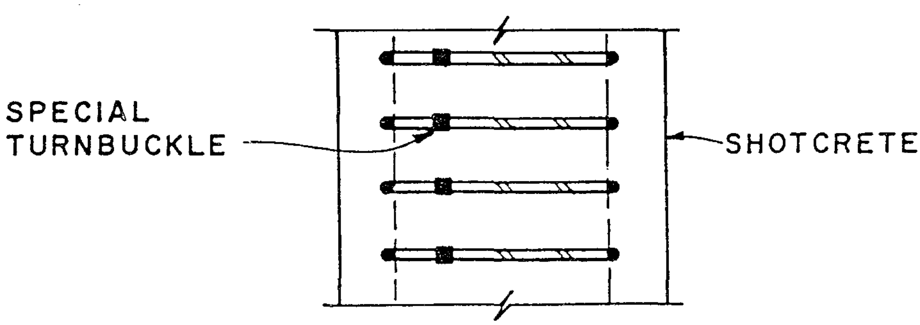
Improved confinement increases the ability of a column to withstand repeated cycles of loading beyond the elastic limit and tend to prevent column failure due to shear, loss of anchorage or splice capacity of longitudinal reinforcement, and degredation of flexural capacity.

There are several different methods to increase confinement.

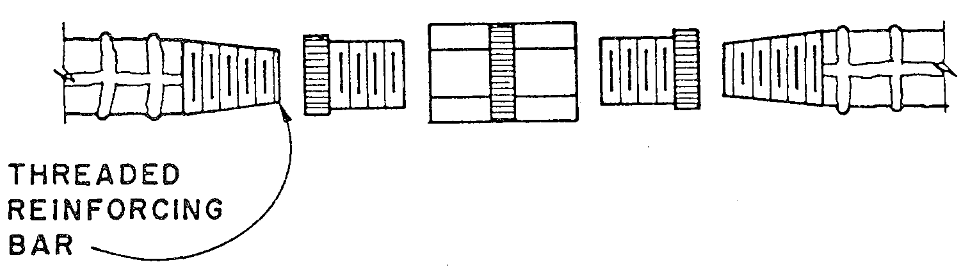
One method which utilizes half-inch steel reinforcing prestressed on the outer face of the column is shown in Figure G.16. The prestress force is provided by threading the ends of the bars so these can be connected together with a specially designed turnbuckle.



Cross-section



Elevation



Turnbuckle detail

FIGURE G.16 Column retrofit to increase confinement with steel hoops (ATC, 1983).

Another method is to use prestressing wire wrapped under tension around the column is shown in Figure G.17. The wire and anchorages are protected by a concrete cover.

A solid-steel shell placed around an existing column as shown in Figure G.18 is a retrofitting method to increase concrete confinement in columns. A small space is left between the column and the shell which is grouted later.

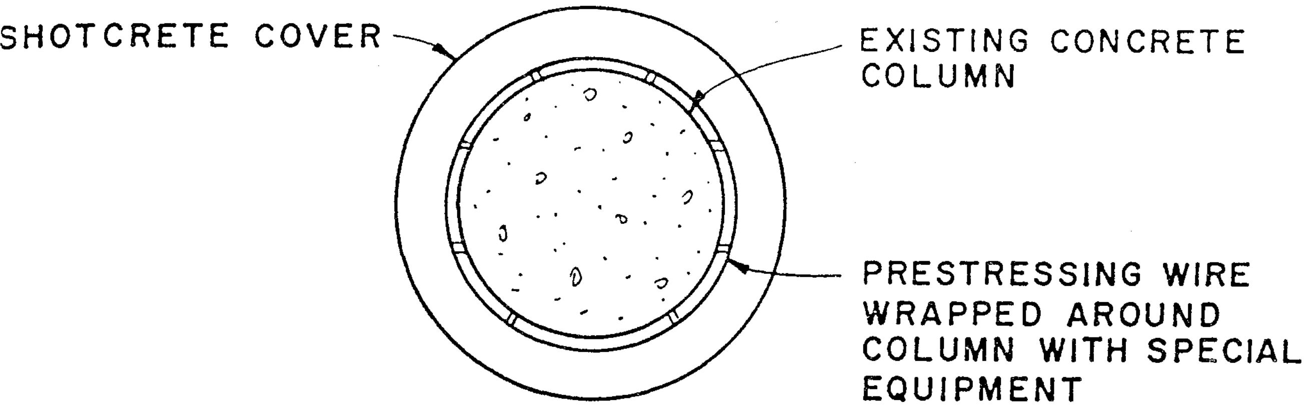
Retrofitting methods should carefully be detailed so that transverse confinement remains effective throughout the duration of the seismic loading.

G.4.3 Reduced Flexural Reinforcement

The ultimate shear force on a column can be reduced by decreasing the yield moment at one or both ends of the column. This retrofitting method should only be considered when columns are over-reinforced for flexure resulting in little or no flexural yielding during an earthquake. Reduction in flexural reinforcement should never be used when the loss of flexural capacity will result in the formation of a collapse mechanism. The simplest method for reducing flexural reinforcement is to cut some of the longitudinal reinforcing bars as shown in Figure G.19.

G.4.4 Increased Flexural Reinforcement

This retrofit technique increases the flexural capacity of the column. Increased reinforcement will not totally be effective unless the column can be made to show a ductile behaviour. Adequate transverse confinement will assure ductile behavior. This retrofitting technique should only be considered when loss of flexural strength results in a collapse mechanism and when the ultimate moment capacity/elastic moment ratio, r_{ec}, is less than 0.125.



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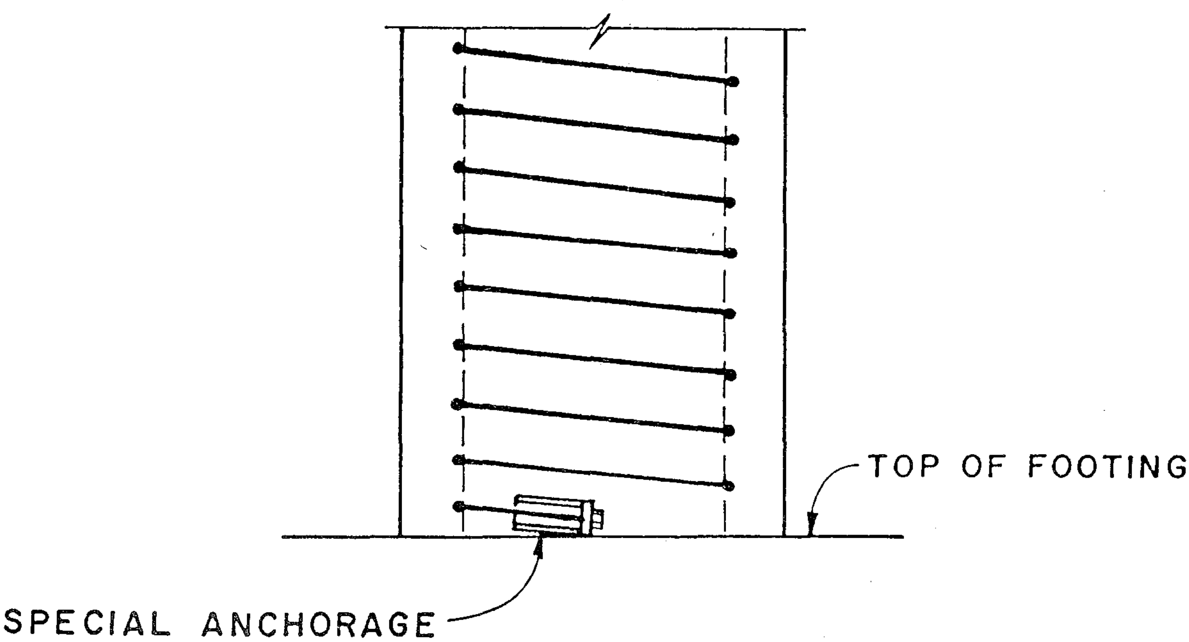
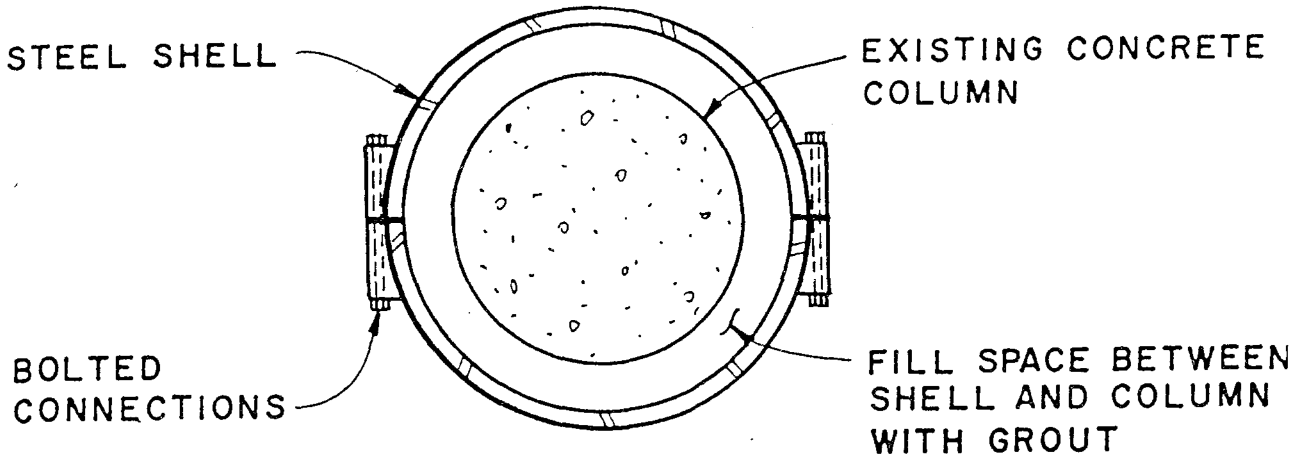
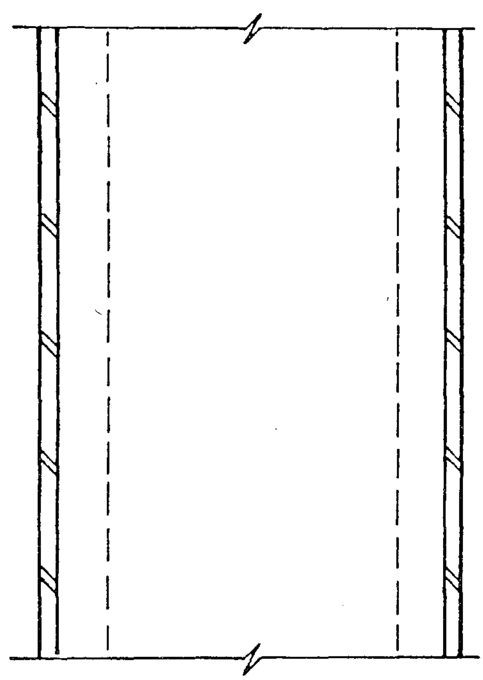


Elevation

FIGURE G.17 Column retrofit to increase confinement with prestress wire (ATC, 1983).

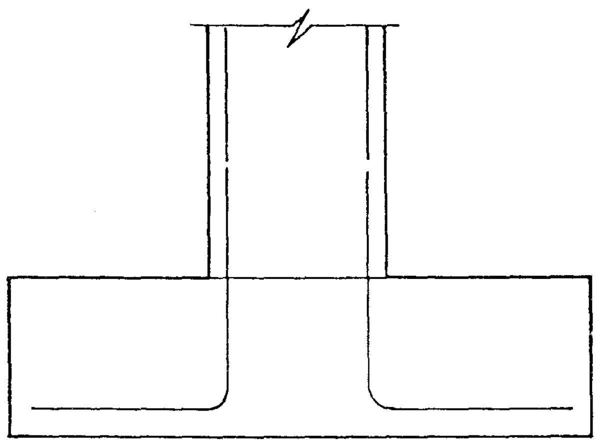


Cross-section



Elevation

FIGURE G.18 Column retrofit to increase confinement with steel shell (ATC, 1983).



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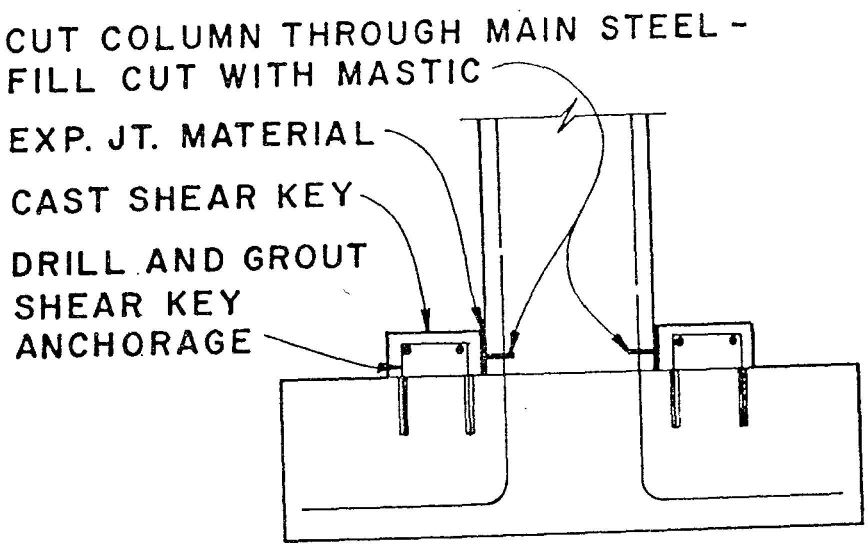


After

FIGURE G.19 Column retrofitting by cutting main reinforcement (ATC, 1983).

The Retrofitting methods to increase the flexural strength of reinforced concrete columns which are especially preferred in Japan are shown in Figures G.20 through G.22.

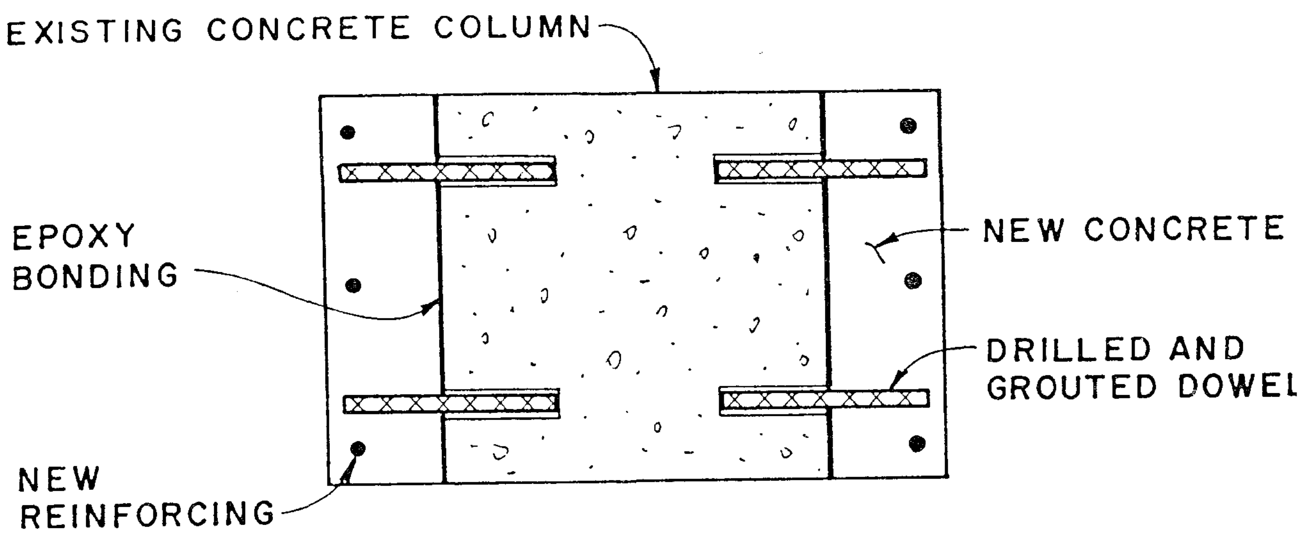
G.4.5 Infill Shear Wall

The transverse resistance of multi-column bents can be increased by constructing an infill concrete shear wall between individual columns in the bent. This technique has been used to repair earthquake damage to bridges in Japan and California, and requires that individual column footings be extended to support the shear wall. The shear wall is tied into the existing structure with grouted bars or anchors. Figure G.23 illustrates the use of an infill concrete shear wall to retrofit a multi-column bridge bent. This type of structure modification will have a large effect on the structural strength and stiffness in the transverse direction.

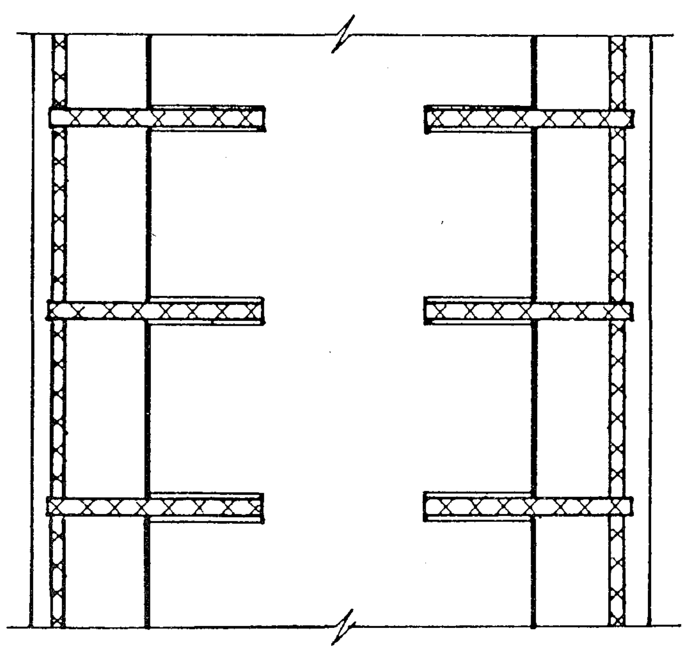
G.4.6 Strengthening of Footings

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

A method for retrofitting footings is shown in Figure G.24. A concrete cap of constant thickness is cast directly on top of the footing. Continunity with the existing footing is provided by steel dowels grouted in drilled holes.

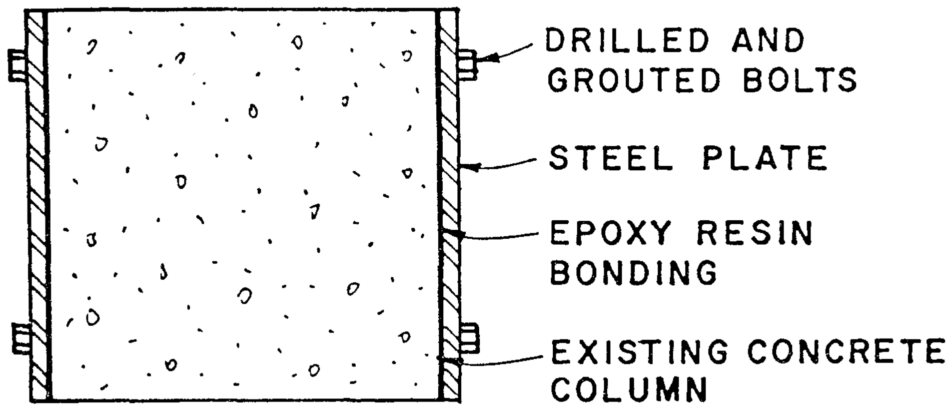


Cross-section



Elevation

FIGURE G.20 Column strengthening by concrete overlays (ATC, 1983).



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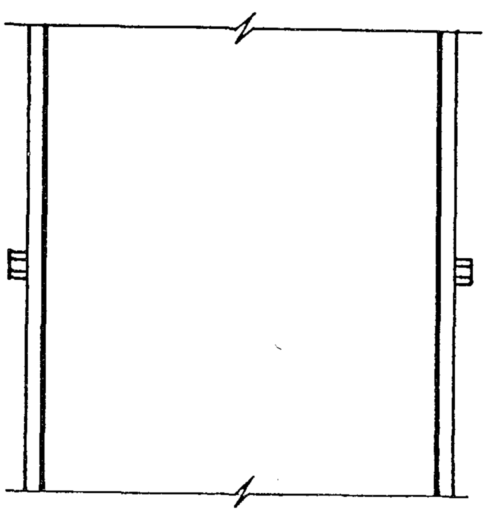

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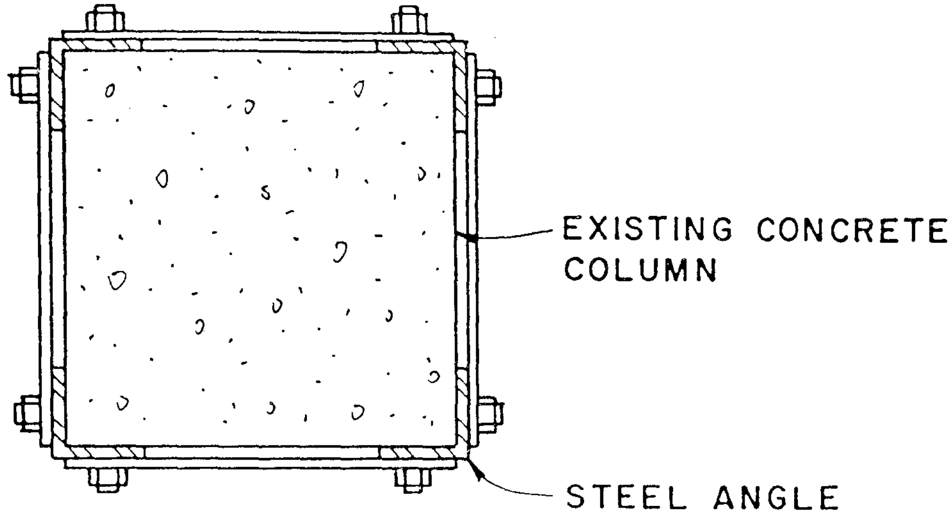
FIGURE G.21 Column strengthening by steel plates (ATC, 1983).



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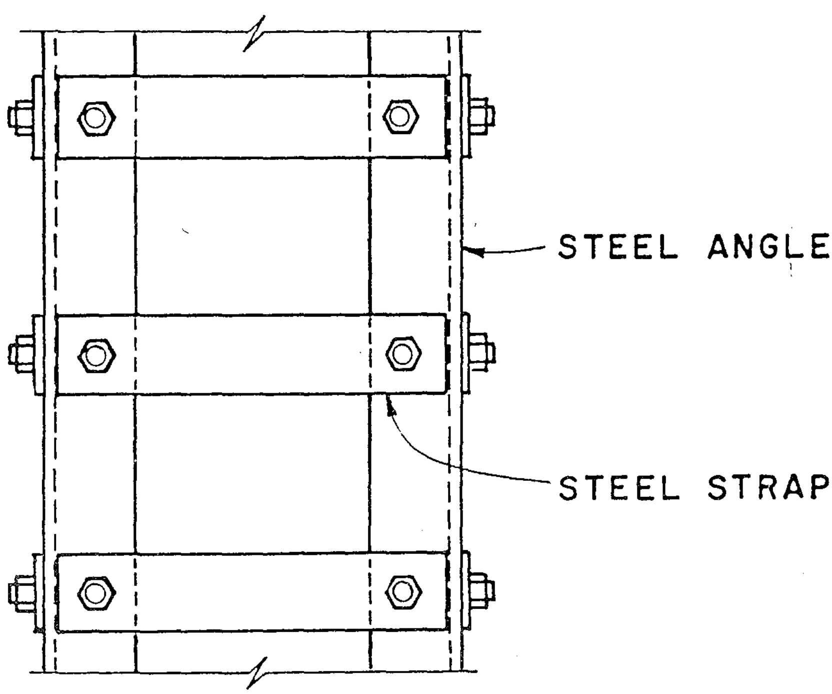


Elevation

FIGURE G.22 Column strengthening by steel angles (ATC, 1983).

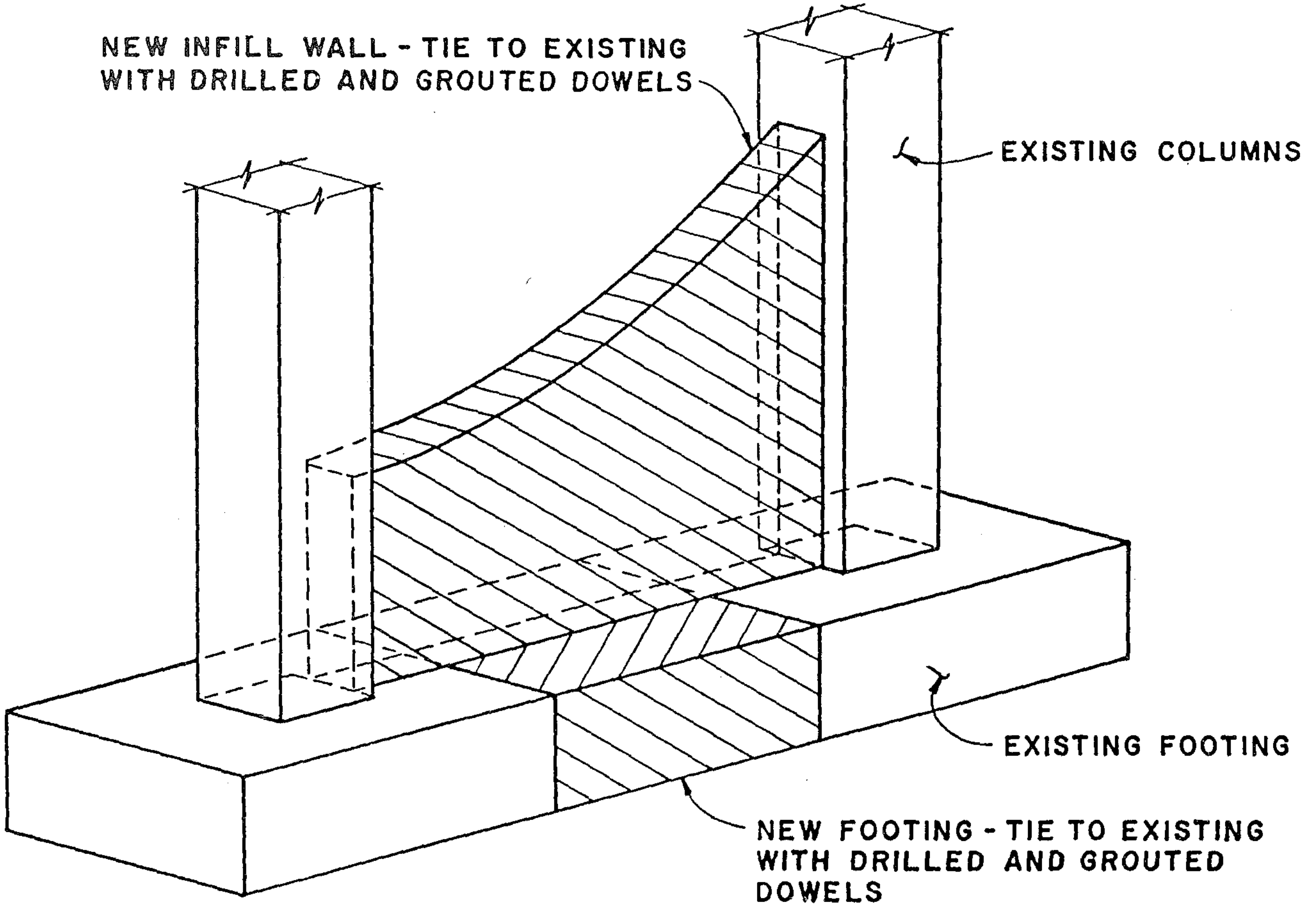
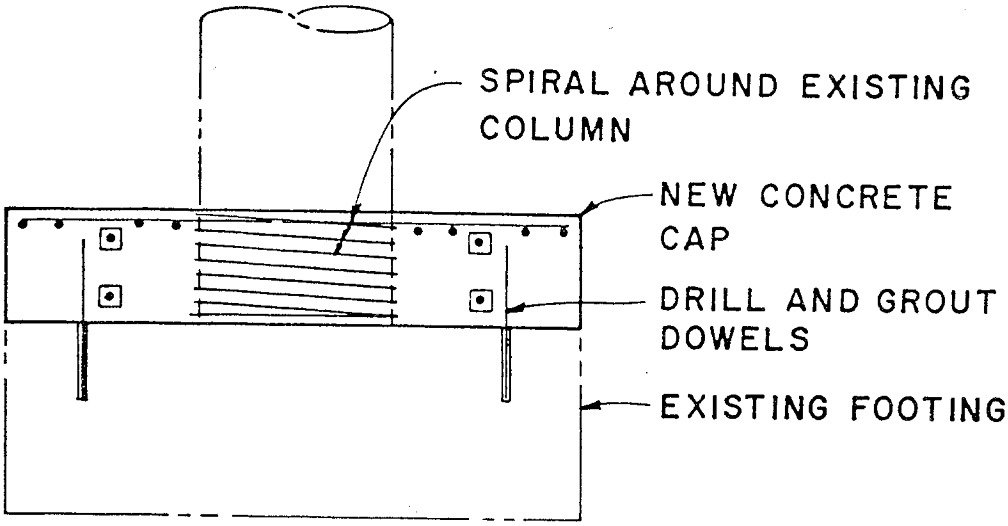


FIGURE G.23 Infill shear wall retrofit (ATC, 1983).



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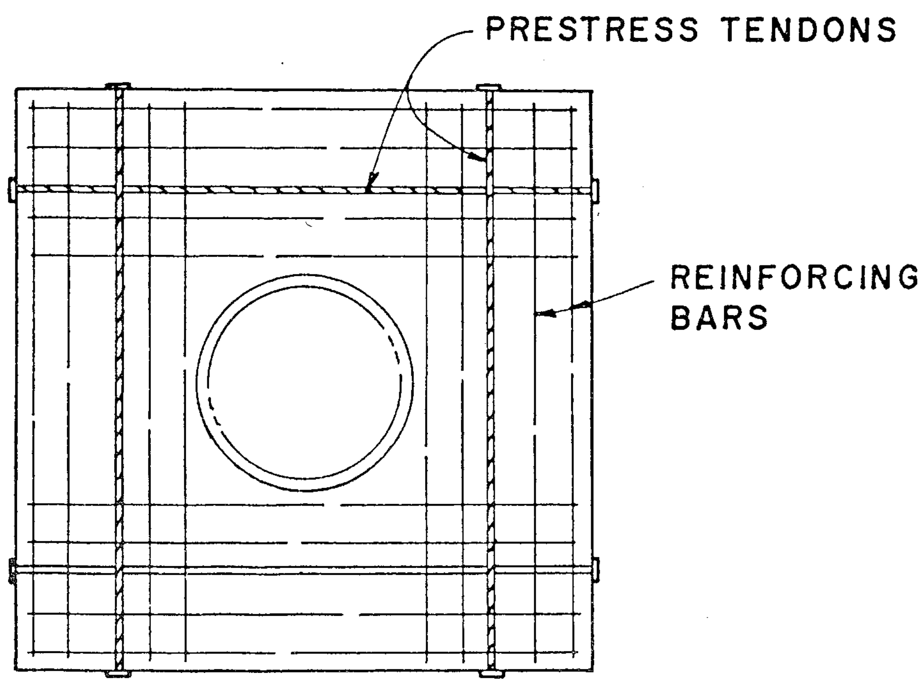

Plan

FIGURE G.24 Footing retrofit by adding concrete cap (ATC, 1983).

G.5 Abutments

Abutment failure very rarely results in the collapse of the structure unless associated with liquefaction failure. Abutment retrofitting is also rarely performed because it is considered to be less economical than bearing retrofit. The use of restrainers to limit relative displacement at the abutment bearings may result in much larger abutment forces. In this case, the abutment should be strengthened to resist the additional forces. Two possible retrofit measures to mitigate the effects of abutment failure are as follows:

G.5.1 Settlement Slabs

Settlement (or approach) slabs are designed to provide continunity between the bridge deck and the abutment fill in the case of approach fill settlement. Settlement slabs should be tied to the abutment to prevent them from pulling away. It is recommended that they are considered only for bridges classified as SPC-D with approach fills subject to excessive settlement due to either soil failure or structural failure of the abutment. Figures G.25 and G.26 show two different types of settlement slabs that have been used in the past.

G.5.2 Soil Anchors

Horizontal displacement at the abutment may cause a loss of accessability to the bridge. Displacements of the abutment normal or parallel to the abutment face may be prevented or minimized by adding soil anchors. Soil anchors similar to those shown in Figure G.27 are used as a retrofit measure.

G.6 Liquefaction and Soil Movement

Liquefaction and/or excessive movement have been the cause for the majority of bridge failures in some areas during past earthquakes. Two approaches are suggested to retrofit such type

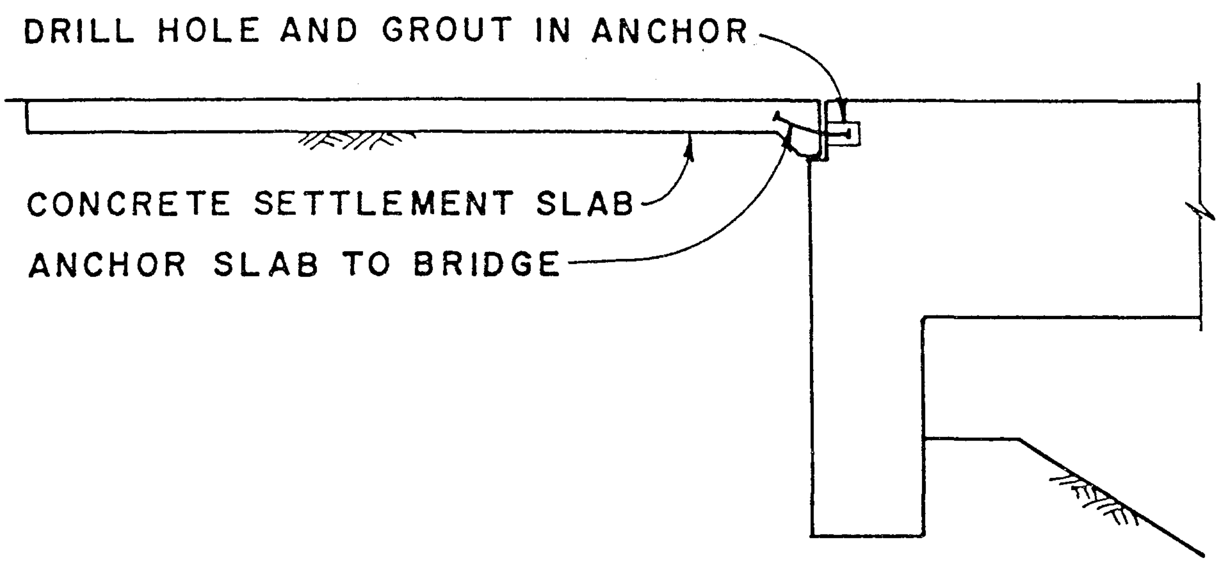

FIGURE G.25 Settlement slab-California style (ATC, 1983).

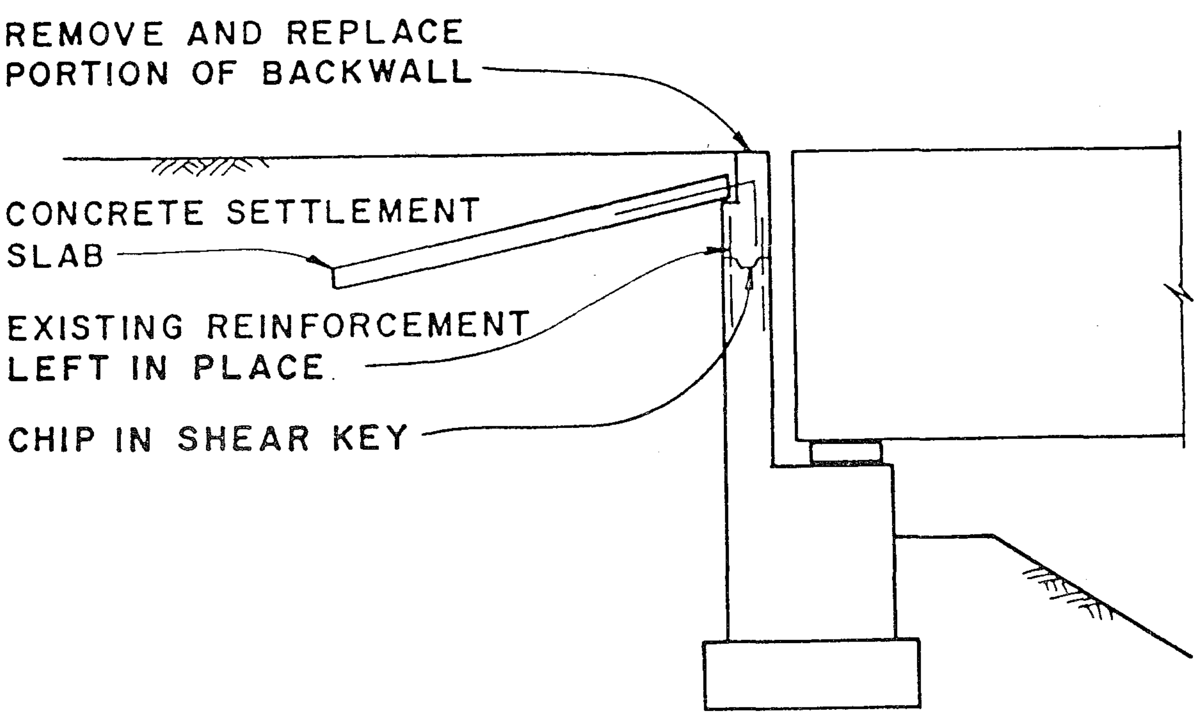

FIGURE G.26 Settlement slab-New Zealand style (ATC, 1983).

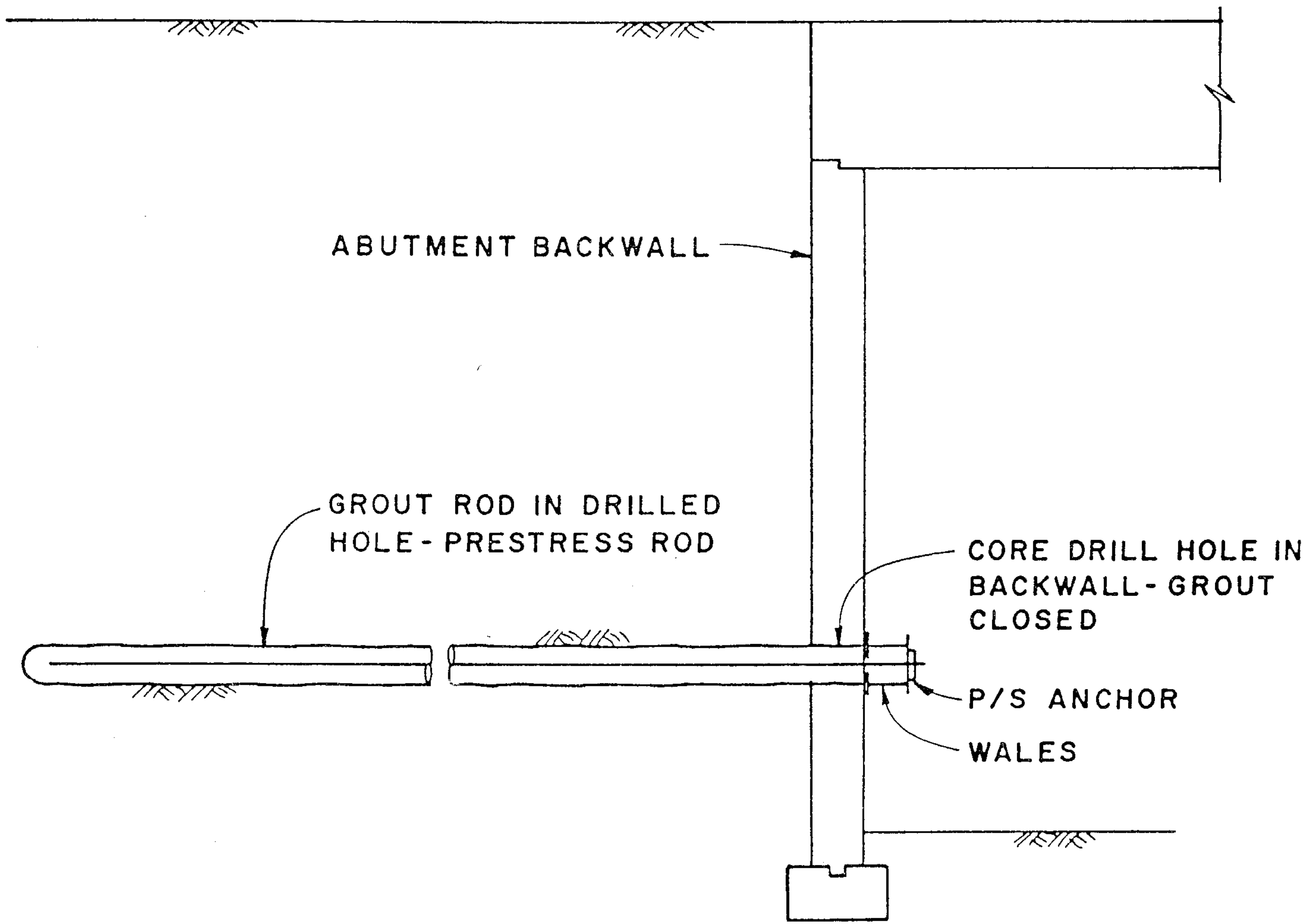

FIGURE G.27 Retrofit of abutment with soil anchors (ATC, 1983).

of failures (ATC-6-2, 1983). The first approach is to eliminate or improve the soil conditions that tend to be responsible for seismic liquefaction. The second approach is to increase the ability of the structure to withstand large relative displacements caused by liquefaction or large soil movement.

G.6.1 Site Stabilization

Several methods are available for stabilizing the soil at the site of a bridge. Some possible methods include:

a) Lowering of groundwater table

Gravity drainage and some mechanical methods are used for the lowering of groundwater table. It should be taken into account that 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.

b) Consolidation of soil or sand compaction

Densification of the soil can also be effective in reducing the potential for liquefaction. Consolidation of only the surface layer can block drainage and actually be detrimental. Soil densification through the use of vibrofloatation or sand compaction piles improves the drainage and therefore is the preferred method. Preconsolidation can result in significant settlements, and care should be taken to protect the existing structure from damage.

c) Vertical network of drains

A method which improves drainage without disrupting the existing structure is to install a network of gravel drains as shown in Figure G.28. These drains allow water to escape during an earthquake and thus prevent the build-up of pore pressure which can reduce the shear strength of the soil.

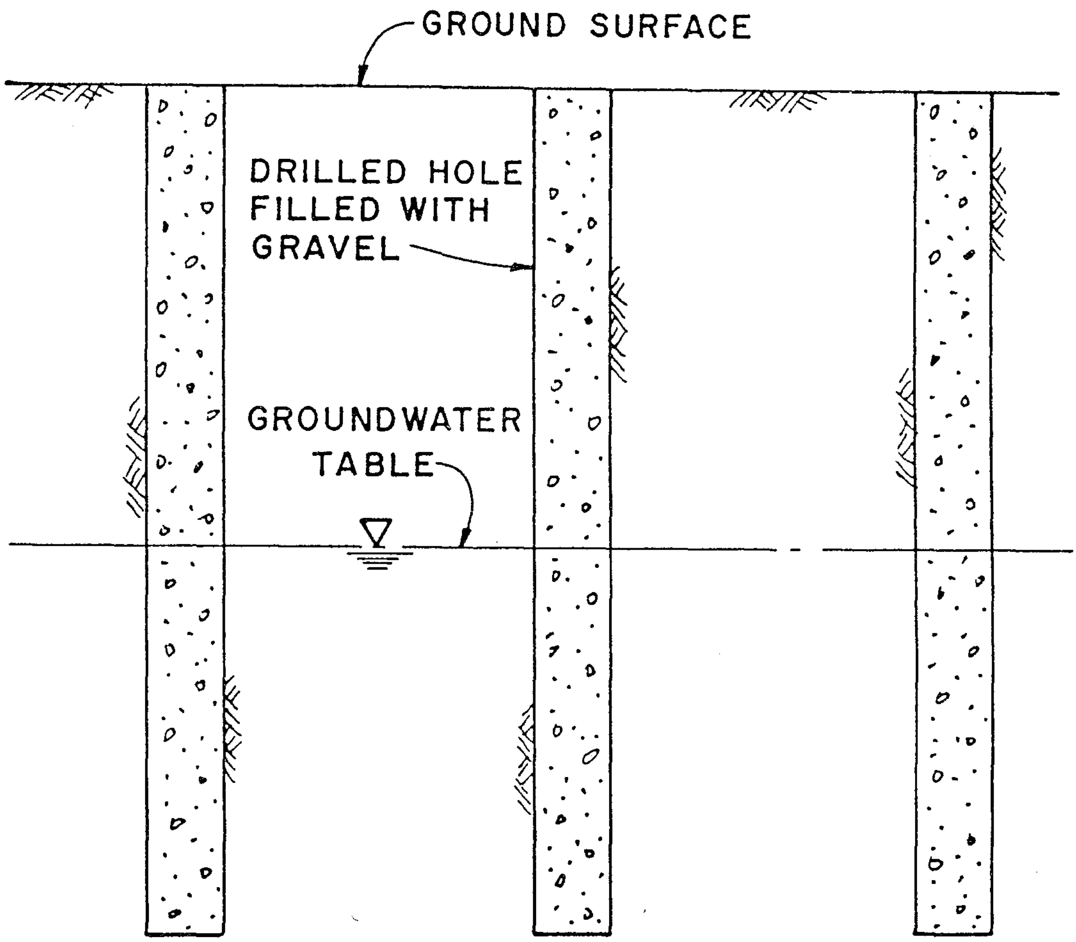

FIGURE G.28 Gravel drain system (ATC, 1983).

d) Placement of permeable overburden

The increased intergranular forces resulting from the overburden necessitate higher pore pressures to balance these forces and cause liquefaction. The permeability of the overburden prevents the build up of pore pressure. In addition the overburden results in some preconsolidation which reduces the chances of liquefaction.

e) Soil grouting or chemical injection

The use of chemicals or grouts to increase the shear strength of soil is also a possible solution. This method may reduce soil permeability and prevent build-up of pore pressure.

Because of many variables and possible disadvantages associated with above methods, primarily due to excessive settlements during construction, it is recommended that these methods should be used with caution.

G.6.2 Increased Superstructure Continunity and Substructure Ductility

In addition to site stabilization, strengthening of the structure is necessary. The strengthening methods depend on the configuration of the structure and components most susceptible to damage. These usually involve methods for tying superstructure sections together and connecting the superstructure to the bents. In some cases, column retrofitting should be considered.

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 G.29. It is preferable to fail the column in flexure rather than to lose this support. Therefore, the superstructure

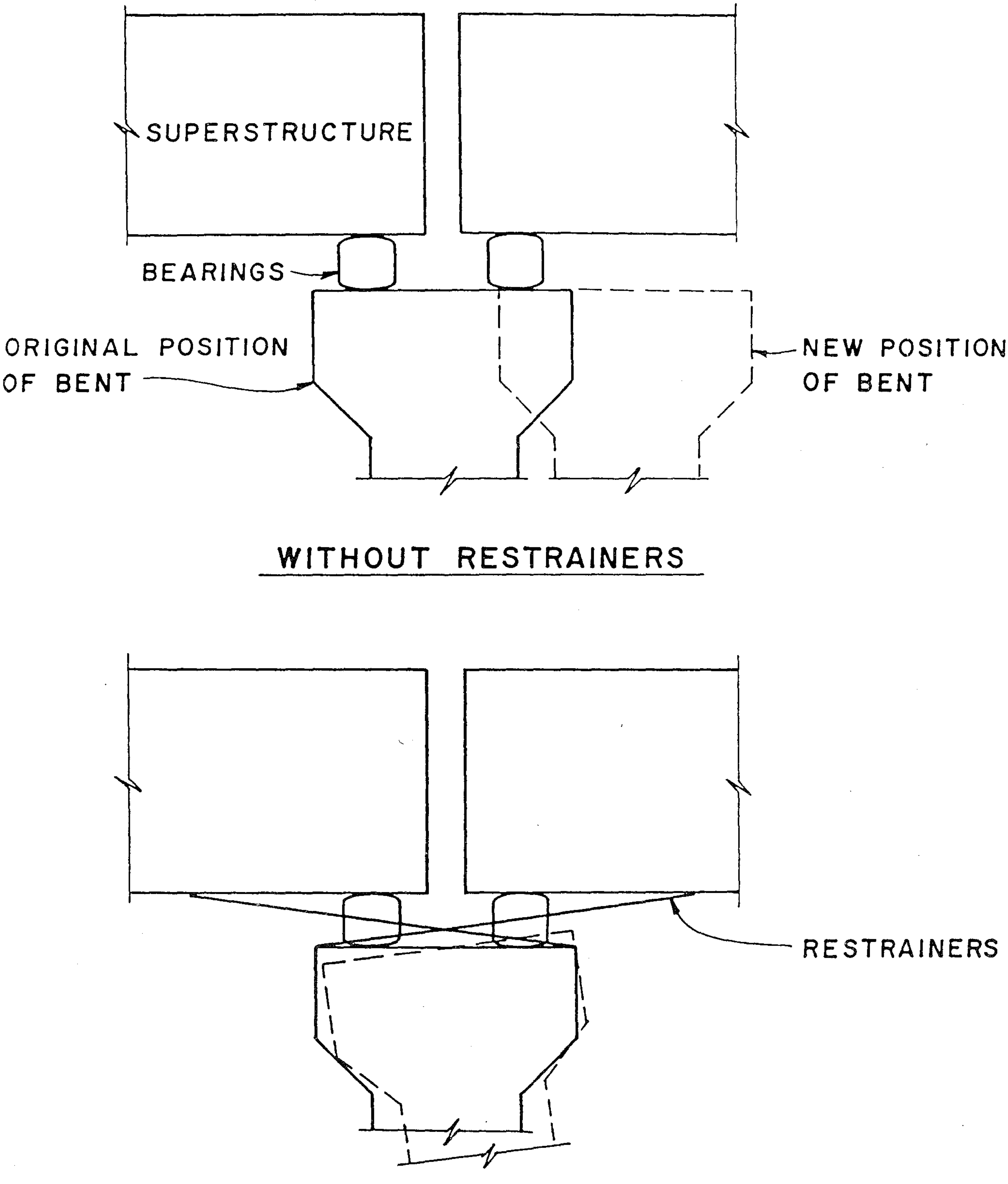

FIGURE G.29 Effects of restrainers at bent during liquefaction failure (ATC, 1983).

should be anchored to the bent. Care should be taken to provide a sufficient gap in the restrainers so that normal temperature movement or moderate earthquakes does not result in a column failure.

Transverse and vertical restrainers at the expansion joints tend to prevent the superstructure from buckling.

Any method that tends to prevent loss of support at the bearings is useful in preventing structure collapse due to excessive soil movement. Therefore most of the methods for retrofitting bearings should be considered in a structure subjected to excessive soil movement. In addition, the ability of the substructure to absorb differential movement is important.