Research Report
KTC-06-23/SPR-206-00-4F

KENTUCKY TRANSPORTATION CENTER

DETAILED SEISMIC EVALUATION OF BRIDGES ALONG I-24 IN WESTERN KENTUCKY

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Research Report
KTC-06-23/SPR206-00-4F

DETAILED SEISMIC EVALUATION OF BRIDGES ALONG I-24 IN WESTERN KENTUCKY

by

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16. Abstract

This report presents a seismic rating system and a detailed evaluation procedure for selected highway bridges on/over I-24 in Western Kentucky near the New Madrid Seismic Zone (NMSZ). The rating system, based upon structural vulnerability, seismic and geotechnical hazards, and socioeconomics factors, was used to rank 127 (82 on and 45 over I-24) bridges on/over I-24.

A total of 14 bridges were selected and subsequently evaluated using a capacity/demand ratio method outlined in the *Seismic Retrofitting Manual for Highway Bridges* by the Federal Highway Administration. The focus was on four bridge components; namely the expansion joints, bearings, columns, and footings. The process involved creating a finite element model of all bridges, and proceeded by a dynamic analysis based on a given time-history spectra response of a 250-year event. The methods and results of the analyses are presented herein.

The results indicate that the rating system is an effective means in terms of determining and prioritizing highway bridges for seismic evaluation and retrofit.

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EXECUTIVE SUMMARY

Interstate-24 (I-24) in Western Kentucky lies just east of the New Madrid Seismic Zone (NMSZ). The last *major* earthquake near this region was the Great New Madrid Earthquake of 1811-1812 with a magnitude of 7.5 or greater on the Richter scale. The NMSZ remains active, recording about 200 earthquakes per year, though most of them are too small to be felt by humans. Seismologists, however, believe that there is a high probability of a major earthquake event in the near future. Due to locality and socioeconomic factors, I-24 is listed as one of the high priority and emergency routes in the region. Hence, it is essential that I-24 remains functional and operational during a major earthquake event. Therefore, the objective of this study is to perform detailed seismic evaluation on 14 selected highway bridges along I-24 that are deemed susceptible to severe damage in a major earthquake event.

A seismic rating system and a detailed evaluation procedure for I-24 bridges are presented in this report. The seismic rating system, which is based on structural vulnerability, seismic and geotechnical hazards, and socioeconomics factors, was used to rank 127 (82 on and 45 over I-24) bridges along I-24. A total of 14 bridges were selected and was subsequently evaluated based on a capacity/demand ratio method outlined in the *Seismic Retrofitting Manual for Highway Bridges* (Publication No. FHWA-RD-94-052). The rating system and evaluation procedure are presented herein.

The detailed evaluation focused on four distinct bridge components; namely the expansion joints, bearings, columns, and footings. Two important aspects of a bridge which include embankment and foundation stability were not considered (i.e. seismic performance of embankment and foundation stability was performed separately and reported in a different report). The evaluation procedure involved creating a finite element model in of all 14 bridges using SAP 2000. The process was then proceeded by a dynamic analysis based on a given time history spectra response of a 250-year event. Details of finite element model generation, essential, and results are presented in this report. Deficiency of these bridges due to the dynamic load was documented, and retrofit recommendations are presented. The results indicate that the rating system is an effective means in terms of identifying and prioritizing highway bridges for seismic evaluation and retrofit. Tables E.1 and E.2 provide a summary of the detailed evaluation of the selected bridges.

Table E.1 Summary of Seismic Deficiencies of the Selected Bridges along I-24 for projected 250-Year Seismic Events.

Bridge Number (BIN)	Ranking	Seismic Deficiencies
73-0024-00112 73-0024-00112 P	14	- Bearing seat capacity
73-0068-00060 73-0068-00060 P	24	- Column flexural capacity
73-0024-00107 73-0024-00107 P	36	- Column flexural capacity
73-0024-00115 73-0024-00115 P	36	- Bearing seat capacity - Column flexural capacity - Footing flexural capacity
73-3075-00065	48	- Bearing seat capacity - Column flexural capacity
73-0024-00113	48	- Bearing seat capacity - Column flexural capacity - Column shear capacity - Column transverse confinement

Table E.2 C/D ratios of the Selected I-24 Bridges along I-24 in Western Kentucky for 250-Year Event

C/D Ratios / BIN	Joints and/or Bearings		Columns and/or Footings									Bridge ranks
	r_{bd}	r_{bf}	r_{ec}	r_{ef}	r_{ca} (cap)	r_{ca} (footing)	r_{sc} (cap)	r_{sc} (footing)	r_{cv}	r_{cc}	r_{fr}	
73-0024-00112 73-0024-00112 P	**0.61**	4.42	1.30	1.03	1.0	1.0	-	-	2.60	1.97	-	14
73-0068-00060 73-0068-00060 P	1.50	1.23	**0.56**	10.2	1.0	1.0	-	-	1.12	1.57	-	24
73-0024-00102 73-0024-00102 P	1.63	1.90	-	-	-	-	-	-	-	-	-	29
73-0024-00120 73-0024-00120 P	1.07	2.50	1.20	1.74	1.0	1.0	-	-	2.40	2.59	-	29
73-0024-00107 73-0024-00107 P	1.78	8.24	**0.69**	-	-	-	-	-	1.38	3.44	-	36
73-0024-00115 73-0024-00115 P	**0.61**	4.64	**0.69**	**0.96**	1.0	1.0	-	-	1.38	1.41	-	36
73-3075-00065	**0.74**	3.81	**0.81**	1.05	1.0	1.0	-	-	1.62	1.92	-	48
73-0024-00113	**0.67**	1.0	**0.35**	1.13	1.0	1.0	-	-	**0.7**	**0.92**	-	48

Note: When C/D ratio is less than 1.0, retrofitting measure must be performed

NOTE: This report is the fourth (4th) in a series of seven reports for Project SRP 206: “Seismic Evaluation of I-24 Bridges”. The seven reports are:	
Report Number:	**Report Title:**
(1) KTC-06-20/SPR206-00-1F	Seismic Evaluation of I-24 Bridges and Embankments in Western Kentucky – Summary Report
(2) KTC-06-21/SPR206-00-2F	Site Investigation of Bridges along I-24 in Western Kentucky
(3) KTC-06-22/SPR206-00-3F	Preliminary Seismic Evaluation and Ranking of Bridges along I-24 in Western Kentucky
(4) KTC-06-23/SPR206-00-4F*	Detailed Seismic Evaluation of Bridges along I-24 in Western Kentucky
(5) KTC-06-24/SPR206-00-5F	Seismic Evaluation of the Tennessee River Bridges on I-24 in Western Kentucky
(6) KTC-06-25/SPR206-00-6F	Seismic Evaluation of the Cumberland River Bridges on I-24 in Western Kentucky
(7) KTC-06-26/SPR206-00-7F	Seismic Evaluation and Ranking of Bridge Embankments along I-24 in Western Kentucky

* Denotes current report

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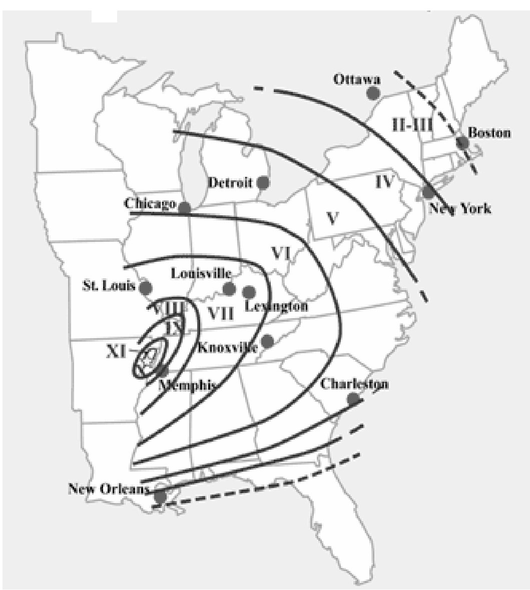
1 INTRODUCTION

1.1 THE NEW MADRID SEISMIC ZONE

The New Madrid Seismic Zone (NMSZ) extends more than 120 miles southward from Cairo, Illinois, at the junction of the Mississippi and Ohio rivers, into Arkansas and parts of Kentucky and Tennessee.

The greatest earthquake risk east of the Rocky Mountains is along the NMSZ. Damaging earthquakes are not as frequent as in California, but when they do occur, the destruction covers more than 15 times the area because of the underlying geology and soil conditions prevalent in the region (National Earthquake Information Center, 2003). The zone is active, averaging about 200 earthquakes per year, though most of them are too small to be felt by humans.

A *damaging* earthquake in this area (6.0 or greater on the Richter scale) occurs , on average, once every 80 years – an estimated magnitude 6.4 occurred near Marked Tree, Arkansas, in 1843, and another earthquake with an estimated magnitude of 6.8 occurred near Charleston, Missouri, in 1895. A *major* earthquake (7.5 or greater) occurs every 200-300 years. It is believed that there is a 10% chance of such a disaster by the year 2000 and a 25% chance by 2040. The last major earthquake was the Great New Madrid Earthquake of 1811-1812. This earthquake occurred over a series of over 2000 tremors in five months, five of which were 8.0 or more in magnitude (National Earthquake Information Center, 2003). Fig. 1.1 below shows the Modified Mercalli intensity for the first event of the 1811-1812 New Madrid earthquakes (Bolt, 1993).



Modified Mercalli Intensity Scale

INTENSITY	EFFECTS	AVE. PEAK ACCEL.
VI Strong	Felt by all. Damage slight	0.06-0.07g
VII Very Strong	Everybody runs outdoors. Considerable damage to poorly designed buildings	0.10-0.15g
VIII Destructive	Considerable damage to ordinary buildings	0.25-0.30g
IX Ruinous	Great damage to ordinary buildings	0.50-0.55g
X Disastrous	Many buildings destroyed	> 0.60g
XI Disastrous	Few, if any, structures remain standing	

Fig. 1.1. Isoseismal map for the Arkansas earthquake of December 16, 1811 (Bolt, 1993).

1.2 INTERSTATE 24 IN WESTERN KENTUCKY

Due to close proximity to the New Madrid Seismic Zone, counties in the western part of Kentucky are especially vulnerable to a major earthquake. In fact, many bridges along I-24 are inadequately designed to resist seismic loadings. Fig. 1.2 shows where I-24 highway is located.

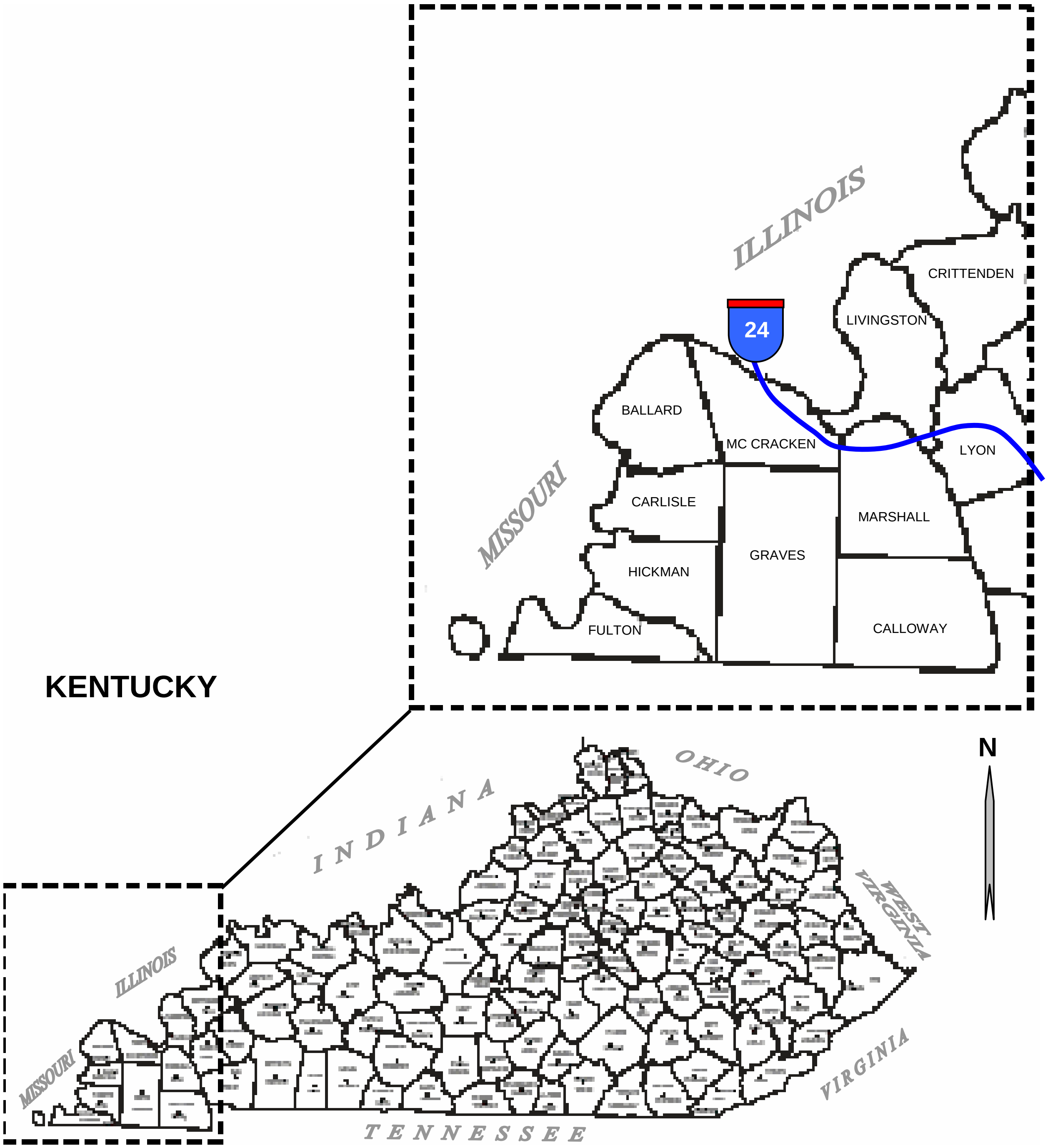


Fig. 1.2. I-24 in western part of Kentucky.

The Federal Highway Administration (FHA) has determined that I-24 is a high priority route and an emergency route for the city of Memphis, Tennessee. As a result, bridges on and over I-24 are deemed *essential* and they must remain open and provide undisrupted access during an earthquake event. It is for this reason, that the commonwealth of Kentucky has sponsored numerous efforts to analyze and examine the structural integrity of these bridges located within the danger zone, primarily those in Western Kentucky, located within the NMSZ.

The primary objective of this study is to perform a detailed seismic evaluation on selected bridges along I-24; such bridges are considered vulnerable to a seismic event based on a *Seismic Rating System*. The complete details of a Seismic Rating System and the ranking of all bridges along I-24 in Western Kentucky are presented in a separate research report. A brief summary, however, of the Seismic Rating System will be described herein. The selected bridges based on this rating system for detailed seismic evaluation will be also included.

1.3 SEISMIC RATING SYSTEM

In general, the Seismic Rating System described in this section is used as a basis for selecting bridges for detailed seismic evaluation, which will be described in Chapter 2. The information provided herein is obtained from the Seismic Retrofitting Manual for Highway Bridges (Buckle, I.G. and Friedland, I.M., 1995), published by the Federal Highway Administration (Report No. FHWA-RD-94-052). The Seismic Rating System will be explained with the aid of Fig. 1.3:

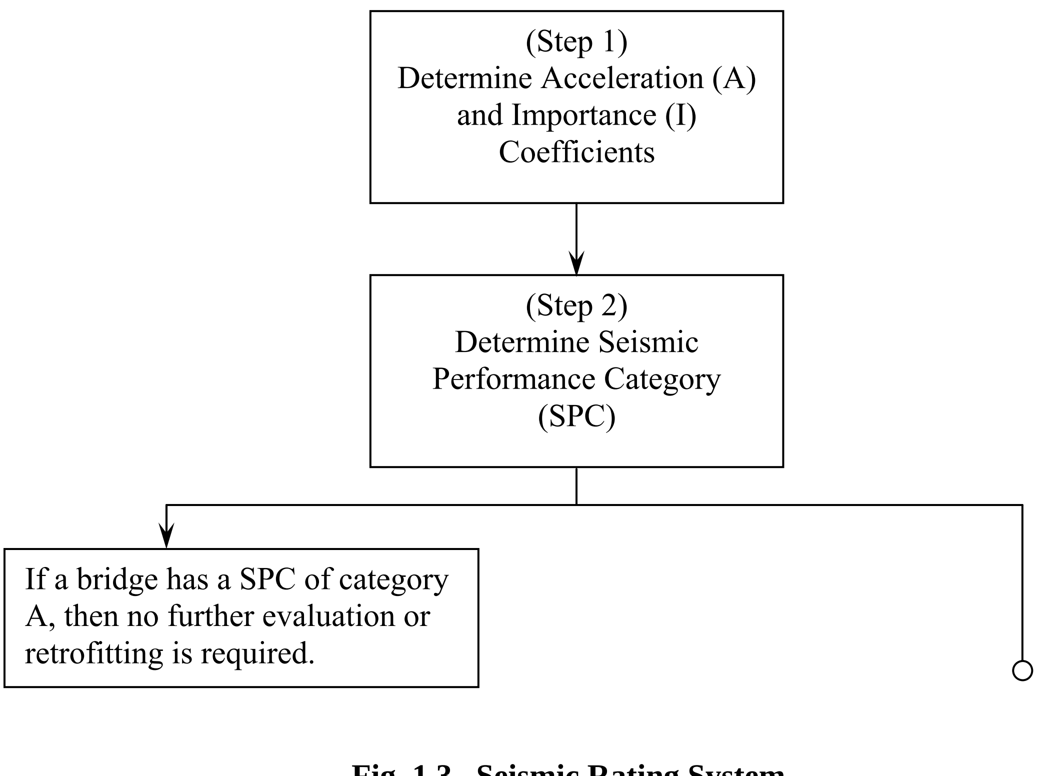


Fig. 1.3. Seismic Rating System.

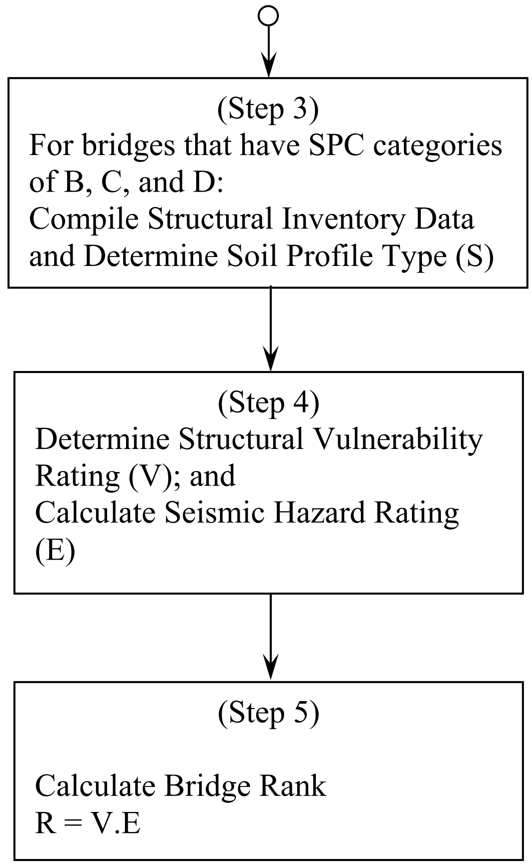


Fig. 1.3.(Cont.) Seismic Rating System.

The Seismic Rating System involves the following steps (See Fig. 1.3):

Step 1: Determination of Acceleration (A) and Importance (I) coefficients

A small particle, such as a building structure attached to the earth during an earthquake, will be moved back and forth rather irregularly. Commonly, this movement can be described as: (a) change in position, (b) change in velocity, and (c) change in acceleration, as a function of time. Most building codes prescribe how much horizontal force a building due to a design earthquake should withstand, and since this force is generally related to the ground acceleration, the ground acceleration is chosen. The peak ground acceleration (PGA) is then the maximum acceleration experienced by the building structure during the course of the earthquake motion.

Peak ground acceleration contour maps, defining seismic zones and response spectra, are given for each Kentucky county basis for the seismic design of new bridges and seismic evaluation of existing bridges. Peak ground acceleration (PGA) as a function of the acceleration (A) coefficient and gravitational acceleration constant ($g = 9.81$ m/sec^2 or 386 in/sec^2) for detailed seismic evaluation of I-24 bridges located in this region is 0.19g (where A = 0.19). This information is obtained from a Time history-response spectra (TR-250Y-0.xxg-x) identification map for a 250-year event derived by Street et al (1996).

Two categories used to describe the Importance (I) coefficient, as documented in the Seismic Retrofitting Manual (Buckle, I.G. and Friedland, I.M., 1995) are: *essential* and *standard*. Bridges classified as *essential* are bridges that must remain functional and operational after an earthquake event. All other bridges are categorized as *standard*. Since I-24 has been designated by the FHA as a priority and an emergency route, all bridges along I-24 are therefore *essential* bridges.

Step 2: Determination of Seismic Performance Category

Table 1.1 is used to determine the Seismic Performance Category (SPC) based primarily on Acceleration (A) and Importance (I) coefficients as previously described:

Table 1.1. Classification of Seismic Performance Category (SPC)
(Seismic Retrofitting Manual, Table 1)

Acceleration (A) coefficient	Importance (I) classification	
	Essential	Standard
$A \leq 0.09$	B	A
$0.09 < A \leq 0.19$	C	B
$0.19 < A \leq 0.29$	C	C
$0.29 < A$	D	C

Note that all bridges in the region of interest have a C classification.

Step 3: Soil Profile Type or Site (S) coefficients and Structural Inventory Data

Table 1.2 shows how the different soil profile type or site (S) coefficient is determined:

Table 1.2. Soil profile type or site (S) coefficient
(Seismic Retrofitting Manual, Table 3)

Soil Type	Soil Profile	Site (S) coefficients
I	Rock or stiff soils. Soil depth less than 60 m (200 ft)	1.0
II	Stiff cohesive or deep cohesionless soil. Soil depth exceeds 60 m (200 ft)	1.2
III	Soft to medium stiff clays and sands. Soil depth exceeds 9 m (30 ft)	1.5
IV	Soft clays or silts. Soil depth exceeds 12 m (40 ft)	2.0

The structural information of a bridge must first be collected for ranking purposes. The following represents the typical form used for data collection (Fig. 1.4):

Section	Item				Response / Notes
GENERAL	Crossing		Bridge Number:		
	Year Built		County		Detour Length (Miles)
	Latitude		Longitude		If *yes*. Please list them (Structure or load).
	Have modifications been made since the bridge was constructed? *No.* ☐ →				
	Does the bridge cross a body of water?	*Yes* ☐ *No* ☐			
	Has the bridge been seismically retrofitted?	*Yes* ☐ *No* ☐			
	Is it a rigid box culvert?	*Yes* ☐ *No* ☐			
SUPERSTRUCTURE	Is the superstructure integral with the abutments?	*Yes* ☐ *No* ☐			**Comments:**
	Does the superstructure contain box girders?	*Yes* ☐ *No* ☐			
	Is there lateral movement under traffic loading?	*Yes* ☐ *No* ☐			
	Is the bridge likely to collapse in an earthquake after toppling failure of the bearings?	*Yes* ☐ *No* ☐			
	Would gross movement of superstructure cause instability?	*Yes* ☐ *No* ☐			
	Is the bridge skewed?	*Yes* ☐ *No* ☐			
	Is there any unusual gap or offset at an expansion joint?	*Yes* ☐ *No* ☐			
BEARINGS	Type	*Rocker* ☐ *Roller* ☐ *Elastomeric Pad* ☐ *Sliding* ☐ *Multi-rotation* ☐		Condition	
	If there are pedestals, are the bearings likely to overturn in an earthquake?				*Yes* ☐ *No* ☐
	Does the bridge with less than 3 girders have exterior girder supported on the seat edge?				*Yes* ☐ *No* ☐
	Are the bearing seats, under the abutment end-diaphragm, continuous?				*Yes* ☐ *No* ☐
	Are there any girders supported on individual pedestals or columns?				*Yes* ☐ *No* ☐
	The longitudinal support length measured in a direction perpendicular to the support (cm)				
SUBSTRUCTURE	Is the abutment a cantilever earth-retaining abutment?				*Yes* ☐ *No* ☐
	Are the reinforced concrete columns monolithic with the superstructure?				*Yes* ☐ *No* ☐
	Is there horizontal or vertical movement or tilting of the abutments, columns or piers?				*Yes* ☐ *No* ☐
	Is there unusual or extensive erosion of soil at or near any of the substructure units?				*Yes* ☐ *No* ☐
	Do you think abutment-slope failures are possible in an earthquake?				*Yes* ☐ *No* ☐
OTHER					

Fig. 1.4. Structural inventory form.

Step 4: Structural vulnerability rating (V) and Seismic hazard rating (E)

Vulnerability rating (V) is determined based on four bridge components: (a) the connections, bearings, and seats; (b) columns and foundations; (c) abutments; and (d) soils. The flow chart shown in Fig. 1.5 illustrates how V is determined (for further details see the Seismic Retrofitting Manual, Section 2.3.1.1):

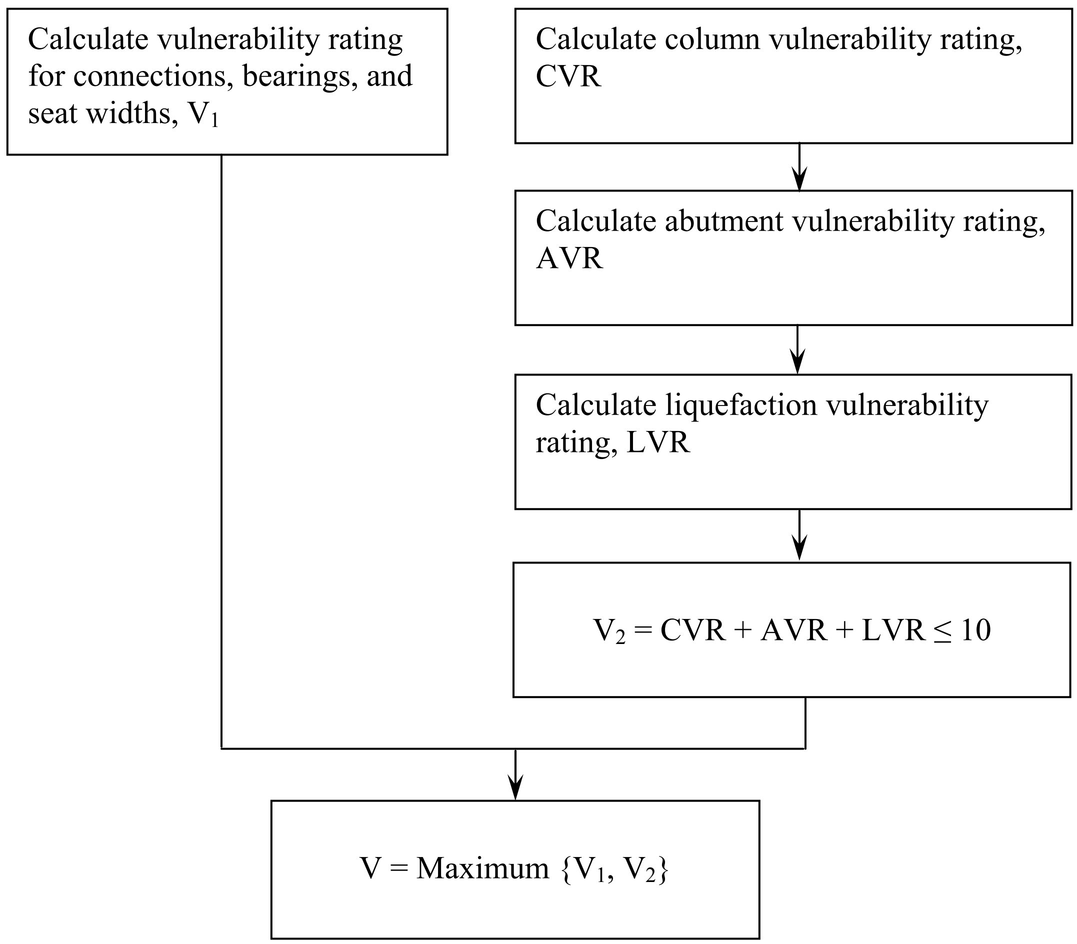


Fig. 1.5. Structural vulnerability rating (V).
(Seismic Retrofitting Manual, Figure 8)

Seismic hazard rating (E) is calculated using the following equation:

$E = 12.5 \cdot A \cdot S \leq 10$ (Seismic Retrofitting Manual, Eq. 2-4)

Step 5: Calculation of bridge rank

The bridge rank (R) is calculated based on a structural vulnerability rating (V) and a seismic hazard rating (E). Each rating (V and/or E) lies in the range of 0 to 10 and the rank (R) is found by multiplying these two ratings together:

$$R = V \cdot E \qquad \text{(Seismic Retrofitting Manual, Eq. 2-2)}$$

Since V and E, each, range from 0 to 10, the minimum and maximum values for R will then be 0 and 100, respectively. In general, the higher the R value, the greater the need for detailed seismic evaluation and potential for retrofitting needs.

1.4 I-24 HIGHWAY BRIDGES SELECTED FOR DETAILED SEISMIC EVALUATION PROCESS

The seismic rating or bridge ranking system described in the previous section was used to evaluate 127 highway bridges (82 on and 45 over I-24) on/over I-24 in Western Kentucky, near the NMSZ. The rankings (R) of these bridges fall between 0 and 48 on a scale of 100. The average ranking of all bridges is approximately 13. Based on the ranking system, the bridges, which rank 14 or higher, are selected for detailed seismic evaluation as indicated in Table 1.3:

Table 1.3. Selected Interstate-24 bridges for detailed seismic evaluation based on a 250-year event.

Bridge Identification Number	Bridge Name	Year Built	Ranking
73-0024-00112 73-0024-00112 P	I-24 over US45	1971	14
73-0068-00060 73-0068-00060 P	US68-US62 Connector	1968	24
73-0024-00102 73-0024-00102 P	Relocated Cairo Road	1969	29
73-0024-00120 73-0024-00120 P	I-24 over Clarks River	1975	29
73-0024-00107 73-0024-00107 P	Perkin Creek Channel Change	1967	36
73-0024-00115 73-0024-00115 P	I-24 over Island Creek Road	1971	36
73-3075-00065	I-24 over Sheehan Road	1966	48
73-0024-00113	I-24 over Elmdale Road	1974	48

Note that bridges designate with a letter P are parallel bridges.

2 DETAILED SEISMIC EVALUATION OF I -24 BRIDGES

2.1 GENERAL

The Seismic Retrofitting Manual for Highway Bridges (Buckle, I.G. and Friedland, I.M, 1995), SR Manual hereafter, published by the Federal Highway Administration (Report No. FHWA-RD-94-052), was used as a guide for seismic evaluation of the selected I-24 bridges.

The SR Manual proposes two methods – the Capacity/Demand (C/D) ratio method and the Lateral Strength method – for detailed seismic evaluation of bridges requiring a detailed analysis based on the their Seismic Performance Category.

In general, the Lateral Strength method treats the entire bridge system, whether individual segments or frames of the bridge between expansion joints, as a single structural system. The structural system is then evaluated using an incremental collapse mechanism approach (SR Manual, Section 3.3.3).

The Capacity/Demand (C/D) ratio method, on the other hand, evaluates the individual bridge components' (expansion joints, bearings, columns, footings, etc.) ability to resist the design earthquake. In general, the seismic demands (D) of individual components are determined from an elastic spectral analysis. The seismic capacities (C) of individual components are computed at their nominal ultimate values without capacity reduction factors, ϕ (SR Manual, Section 3.4). The capacities and demands can be forces, displacements, and other quantities that define the performance of the bridge. In this method, a calculated C/D ratio of less than 1.0 indicates that component failure may occur during the design earthquake, and consequently, retrofitting of such components may be required.

The C/D method typically results in conservative retrofitting measures, which lead to higher costs. The lateral strength method, in general, yields more accurate results, hence lower retrofitting costs (Harik et. al., 1997). However, due to the complex nature of the lateral strength method, the C/D method is often preferred, and the latter method is adopted for all bridge analyses performed in this report.

2.2 CAPACITY/DEMAND RATIO METHOD

Bridge components that may possess seismic deficiency potential during an earthquake require quantitative evaluation. Quantitative evaluation is satisfied by computing the seismic C/D ratios for the following bridge components:

(1) Expansion joints and/or bearings;
(2) Columns, piers, and/or footings;
(3) Abutments; and
(4) Foundation.

For this investigation, ONLY items (1) and (2) will be evaluated and reported. The stability analysis of the bridge abutments [Item (3)] and the liquefaction analysis of the foundation soil [Item (4)] will be presented in separate research reports.

To analyze the individual bridge components, the *demands* (forces and/or displacements) of the individual bridge components must first be calculated. In general, 3 dimensional bridge models are created for finite element analysis. This process is performed with the aid of a commercially available structural analysis computer program, e.g. SAP2000 (Wilson E.L., 1998), from which the demands of the components are derived. A schematic showing the three orthogonal directions of a bridge is presented in Fig. 2.1.

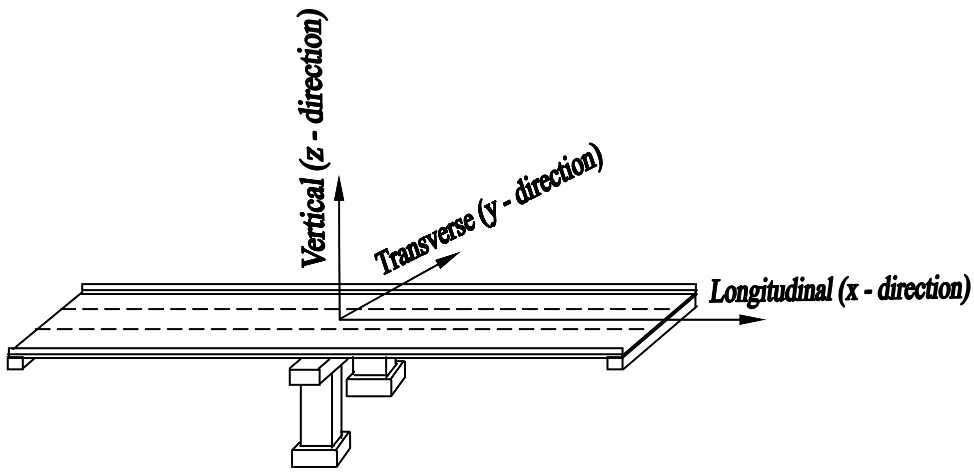


Fig. 2.1. Longitudinal, transverse, and vertical directions of a bridge

In general, the longitudinal direction is assumed to lie along the centerline of the bridge, and the transverse direction is then the perpendicular direction to the longitudinal axis, as shown in Fig. 2.1. Once seismic demands are calculated in each direction for specific individual bridge component, the demands are then combined to produce an overall demand (D) on the individual component. The combination of orthogonal seismic force and/or displacement demands is required to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquakes in two perpendicular, horizontal directions (SR Manual, Section 3.3.2.4). The larger of the following two combinations of seismic demands are used for further analysis:

- Combination (1): 100% of longitudinal demands plus 30% of transverse demands
- Combination (2): 100% of transverse demands plus 30% of longitudinal demands

Guidelines for the capacity of individual bridge components are given in Section 3.6 and Appendix A of the Seismic Retrofitting Manual. A list of the capacity/demand ratios for the detailed seismic evaluation is presented in Table 2.1.

Table 2.1. Capacity/demand ratios for detailed seismic evaluation.

No.	Symbol	Definition	Seismic Retrofitting Manual
1	r_{bd}	Displacement ratio for bearing/joint	Sections 3.6.2, & A.4.2
2	r_{bf}	Force ratio for bearing/joint	Sections 3.6.2, & A.4.3
3	r_{ec}	Force ratio for column	Sections 3.6.3, & A.5
4	r_{ef}	Force ratio for footing	Sections 3.6.3, & A.5
5	$r_{ca\ (cap)}$	Anchorage length ratio for bent cap	Sections 3.6.3, & A.5.1
6	$r_{ca\ (footing)}$	Anchorage length ratio for footing	Sections 3.6.3, & A.5.1
7	r_{sc}	Splice length ratio for column	Sections 3.6.3, & A.5.2
8	r_{cv}	Shear ratio for column	Sections 3.6.3, & A.5.3
9	r_{cc}	Confinement ratio for transverse reinforcement	Sections 3.6.3, & A.5.4
10	r_{fr}	Footing rotation and/or yielding ratio	Sections 3.6.3, & A.5.5

The following sections describe in detail the determination of seismic C/D ratios of (a) expansion joints and/or bearings, and (b) columns, piers, and/or footings:

2.3 CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS AND/OR BEARINGS

In general, two C/D ratios, the displacement C/D ratio, r_{bd}, and the force C/D ratio, r_{bf}, must be checked for expansion joints and/or bearings as proposed by the Seismic Retrofitting Manual. The procedures of determining these ratios are described as follows:

2.3.1 Displacement C/D ratio for expansion joints and/or bearing

The displacement C/D ratio, r_{bd}, calculation is explained with the aid of Fig. 2.2. Section A.4.2 of the SR Manual proposes two methods to calculate the displacement C/D ratios, methods 1 and 2. The lesser of the C/D ratios calculated by methods 1 and 2 is used for the expansion joint and/or bearing. When the calculated r_{bd} is less than 1, retrofitting measures must be taken.

2.3.2 Force C/D ratio for expansion joints and/or bearing

The force C/D ratios, r_{bf}, for bearings and expansion joint restrainers are discussed in Section A.4.3 of the SR Manual. Specifically, the force demand, $V_b(d)$, is calculated by multiplying the elastic analysis value by 1.25. For cases where elastic analysis has not been carried out, it can be assumed that the force demand is 20 percent of the dead load of the superstructure. The bearing force capacity, $V_b(c)$, depends on the type of bearing supports. For instance, the bearing capacity may be the shear resistance provided by the shear key or the frictional force provided by the bearing pads.

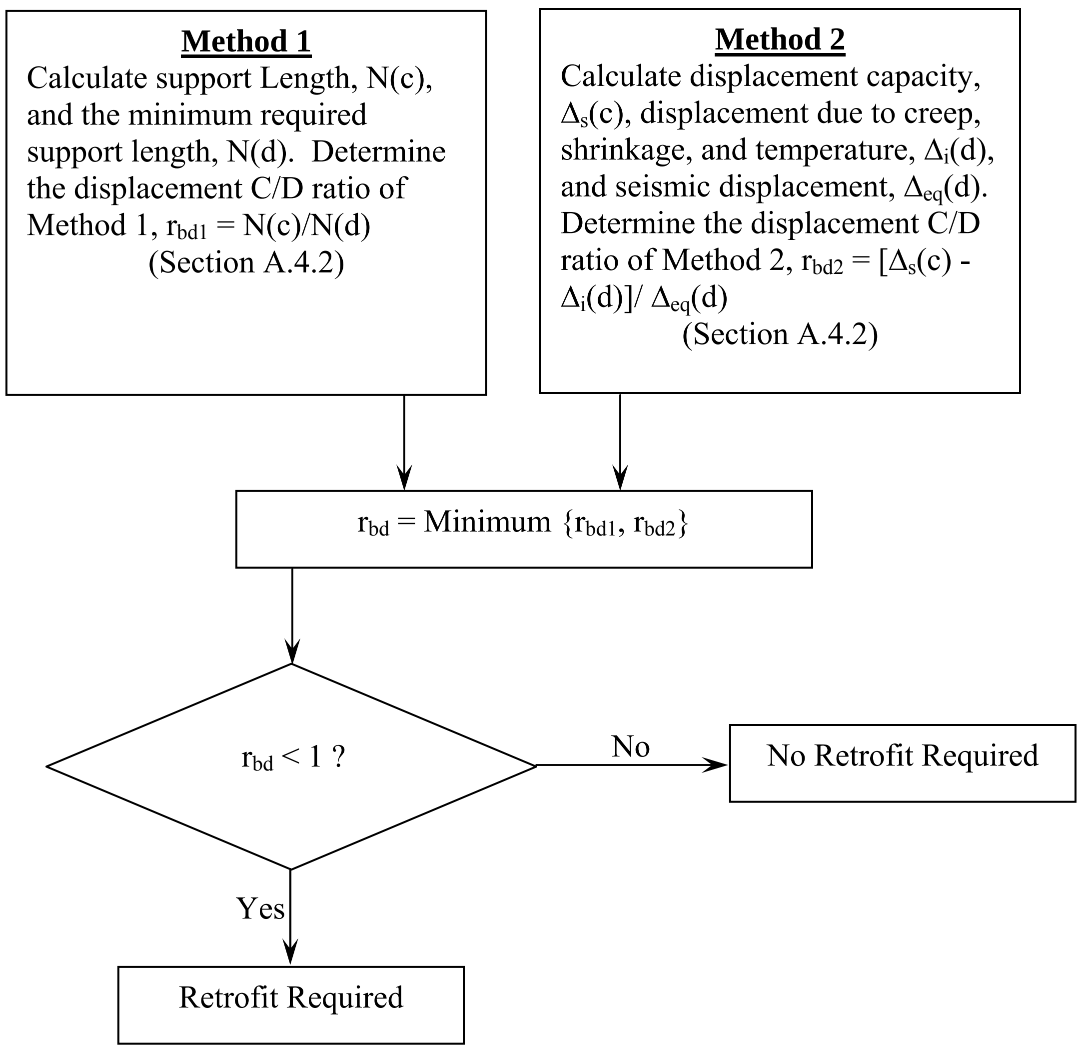


Fig. 2.2. Displacement capacity/demand ratios for expansion joints and/or bearings.

2.4 CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTINGS

2.4.1 Force C/D ratios of column and footing

The determination of the column and footing C/D ratios, r_{ec} and r_{ef}, is explained in this section. First, the moment demands of the columns and footings, $M_n(d)$ and $M_f(d)$, of substructures are determined by elastic analysis for the seismic load combinations described in Section 2.2. The elastic moment demands may be taken as the sum of the absolute values of the earthquake and dead load moments as described in the Seismic Retrofitting manual. The nominal ultimate moment capacities for both the column and footing, $M_n(c)$ and $M_f(c)$, are then calculated from the axial loads due to the earthquake and the self-weight of the structure. Lastly, the column and footing force C/D ratios can be determined using the following expressions:

$r_{ec} = M_n(c)/M_n(d)$ – Column force C/D ratio

$r_{ef} = M_f(c)/M_f(d)$ – Footing force C/D ratio

2.4.2 Anchorage of Longitudinal Reinforcement

A sudden loss of column flexural strength can occur if longitudinal reinforcement is not properly anchored. The determination of the anchorage ratio, r_{ca}, of longitudinal reinforcement is explained with the aid of Fig. 2.3:

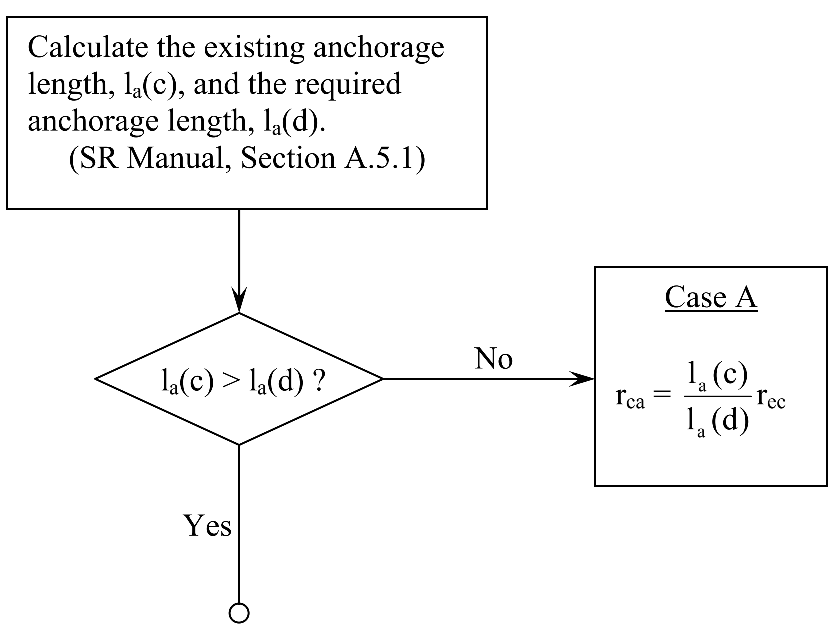


Fig. 2.3. Anchorage capacity/demand ratio of longitudinal reinforcement.
(Seismic Retrofitting Manual, Figure 78)

Case B Identify Anchorage Detail				
Detail No.	Location	AnchorageType	Top Footing Reinforcing	C/D Ratio
1	Footing	Straight	No	$r_{ca} = r_{ef}$
2	Footing	90^{o} hook away from centerline	No	$r_{ca} = 1.3r_{ef}$
3	Footing	90^{o} hook toward centerline	No	$r_{ca} = 2.0r_{ef}$
4	Footing	Straight	Yes	$r_{ca} = 1.5r_{ef}$
5	Footing	90^{o} hook	Yes	$r_{ca} = 1.0$
6	Bent Cap	–	–	$r_{ca} = 1.0$

Fig. 2.3.(Cont.) Anchorage capacity/demand ratio of longitudinal reinforcement.
(Seismic Retrofitting Manual, Figure 78)

2.4.3 Splices in Longitudinal Reinforcement

Longitudinal reinforcements that are not well confined by closely spaced transverse reinforcement have the potential of losing flexural strength near or within the yielding zone. The procedure used to determine the adequacy of splice in longitudinal reinforcement is illustrated in Fig. 2.4:

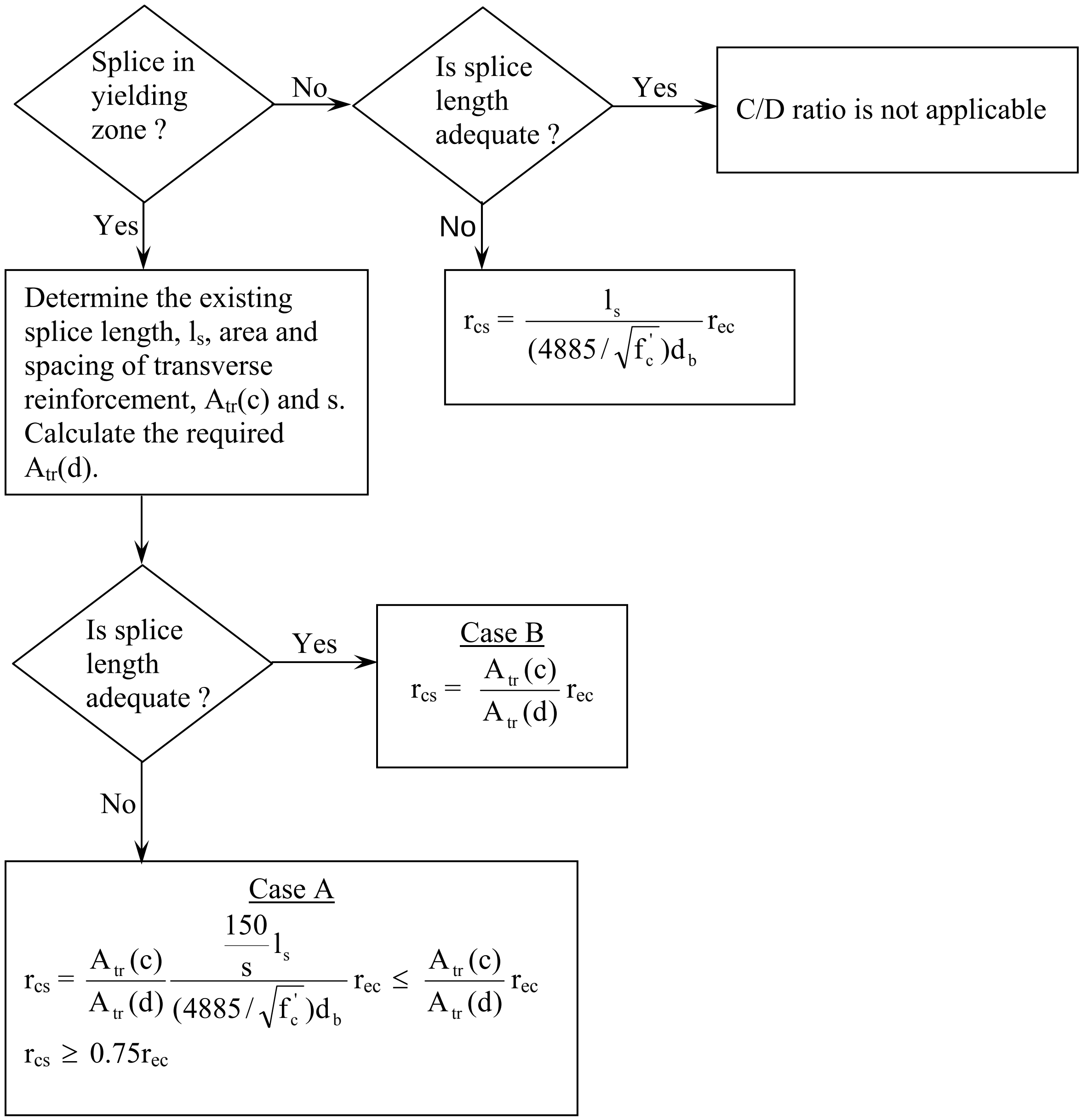


Fig. 2.4. Procedure for determining C/D ratios for splices in longitudinal reinforcement.
(Seismic Retrofitting Manual, Figure 80)

2.4.4 Column Shear

Column shear failure occurs when column shear capacity is exceeded. To illustrate how the C/D ratio of column shear is calculated, Fig. 2.5 is presented.

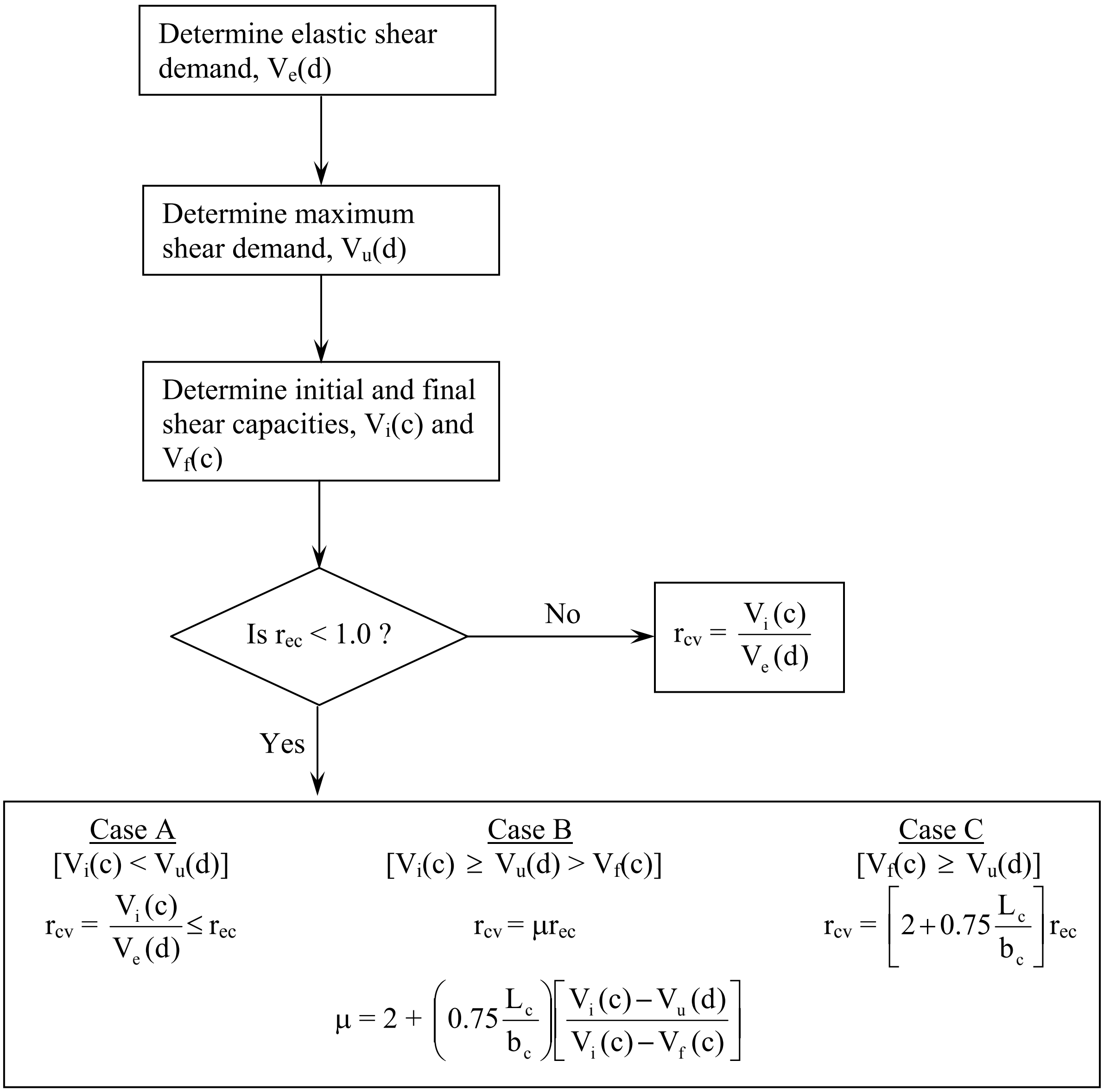


Fig. 2.5. C/D ratios for column shear.
(SR Manual, Figure 81)

2.4.5 Transverse Confinement Reinforcement

Adequate transverse confinement reinforcement in columns must be present to prevent buckling of the main reinforcement and crushing of concrete in compression, which ultimately leads to loss of strength and serviceability. The degree to which degradation is prevented depends largely on the amount and spacing of transverse reinforcement and the adequacy of the anchorage of this reinforcing. The transverse confinement C/D ratio, r_{cc}, can be determined by multiplying the C/D ratio of column, r_{ec}, with a ductility indicator, μ (for further details, see SR Manual Section A.5.4). For a conservative estimate, a ductility indicator of 2 may be used as indicated in the SR Manual. Note that the transverse confinement C/D ratio, r_{cc}, should only be investigated when the column force C/D ratio, r_{ec}, is less than 0.8, as proposed in the SR Manual (Cases III and IV).

2.4.6 Footing Rotation and/or Yielding

The seismic C/D ratio for footing rotation and/or yielding, r_{fr}, can be determined by multiplying the C/D ratio of footing, r_{ef}, with the ductility indicator, μ (SR Manual Section A.5.5). The ductility indicator, μ, is dependent on the type of footing and the mode of footing failure. The ductility indicator, μ, can be determined from Table 2.2 as proposed by the SR Manual. The ratio, r_{fr}, should only be calculated when r_{ef} is less than 0.8 (Cases II and IV in the SR Manual).

Table 2.2. Footing ductility indicator
(SR Manual, Table 8)

Type of Footing	Factor limiting the capacity	μ
Spread Footing	Soil bearing failure	4
	Reinforcing steel yielding in the footing	4
	Concrete shear or tension in the footing	1
Pile Footing	Pile overload (compression or tension)	3
	Reinforcing steel yielding in the footing	4
	Pile pullout at footing	2
	Concrete shear or tension in the footing	1
	Flexural failure of piling	4
	Shear failure of piling	1

3 FINITE ELEMENT MODELING WITH SAP 2000

3.1 Creating Models with SAP 2000

The dynamic responses (i.e. displacement and force) of all 14 bridges were calculated using SAP 2000. The 3D object based graphical modeling environment of SAP 2000 permits relatively quick generation of finite element (FE) structural models, and its wide variety of analytical options allows one to perform structural analysis with ease. The procedure, in general, follows these steps:

<u>Step 1</u>: Set up 3-D Bridge Model

New models may be created with very little effort using pre-programmed bridge template. Typically, the use of bridge template requires information such as *Number of Plans*, *Number of Girders*, *Number of Columns*, *Span Length*, *Girder Spacing*, *Column Spacing*, *Column Height* and *Skew angle*. In the analysis of the I-24 bridges, the information was obtained directly from the bridge plans.

<u>Step 2</u>: Define Material Properties

The materials properties such as the yield strength of steel, compression strength of concrete, elastic modulus, coefficient of thermal expansion, density, etc., are required in the FE analysis, and these properties were presented in and obtained from the existing bridge plans.

<u>Step 3</u>: Define Sections and Assign

The 3-D FE model is composed from several elements (i.e. frame elements such as girders, diaphragm beams, columns, etc., and shell elements such as bridge decks and pier-walls). These frame and shell elements were defined in accordance with the dimensions given in the bridge plans.

<u>Step 4</u>: Define and Assign Static Loads

In any type of analysis, dead load due to self-weight exists and must be considered. In SAP 2000, the dead load due to self-weight of the defined elements was automatically generated and calculated based on the defined material and section properties in Steps 2 and 3. Additional masses, such as those due to traffic barriers, light fixtures, etc., can be manually defined and assigned to the appropriate joints in the model.

<u>Step 5</u>: Define Time History Response Spectra

The Time History Response Spectra of a 250-year earthquake event (Fig. 3.1) was used for the dynamic analysis. These were defined and used in three orthogonal directions; namely the transverse, vertical, and longitudinal of the bridge structure as illustrated in Fig. 2.1. The load combinations as defined in the capacity/demand method by the Federal Highway Administration were used.

<u>Step 6</u>: Analysis and Output

Once Steps 1 to 5 are completed, a *Dynamic Analysis* option together with appropriate parameters can be selected and performed in SAP 2000. The analytical process will take several minutes to complete, and thereafter users can obtain the desired output such as joint displacement, forces at the bearings, element forces, and so on.

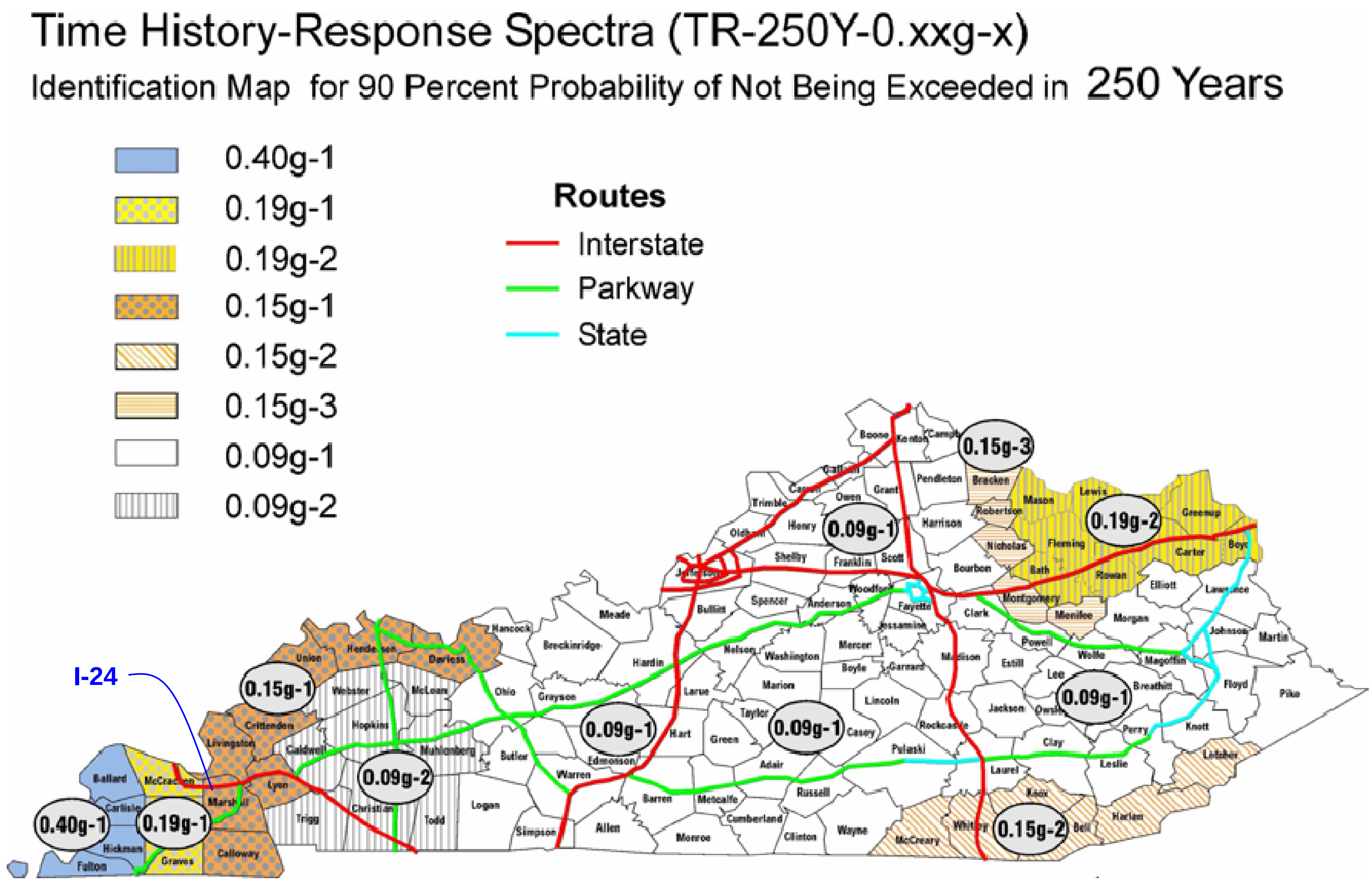


Fig 3.1. Time History Response Spectra Identification Map of 250-year Event for Kentucky

4 DETAILED SEISMIC EVALUATION OF THE US68-US62 CONNECTOR BRIDGE OVER I-24 IN McCRACKEN COUNTY, KY

The US68-US62 Connector Bridge over I-24 in McCracken County, KY, is selected to illustrate the evaluation process.

4.1 US68-US62 Bridge Description

Fig. 4.1 shows a three-dimensional view of the US68-US62 Connector Bridge over I-24 in McCracken County, KY. The continuous structure, with two equal spans of 91.5 ft, was constructed in 1968. The superstructure consists of five steel plate I-girders supporting an eight-inch concrete bridge deck. The substructure – pier – is made up of three columns supported on a pile footing (Fig. 4.2). The footing pedestal has a thickness equal to that of a column, 36 in. Soft to medium-stiff clays and sands were found at the bridge site.

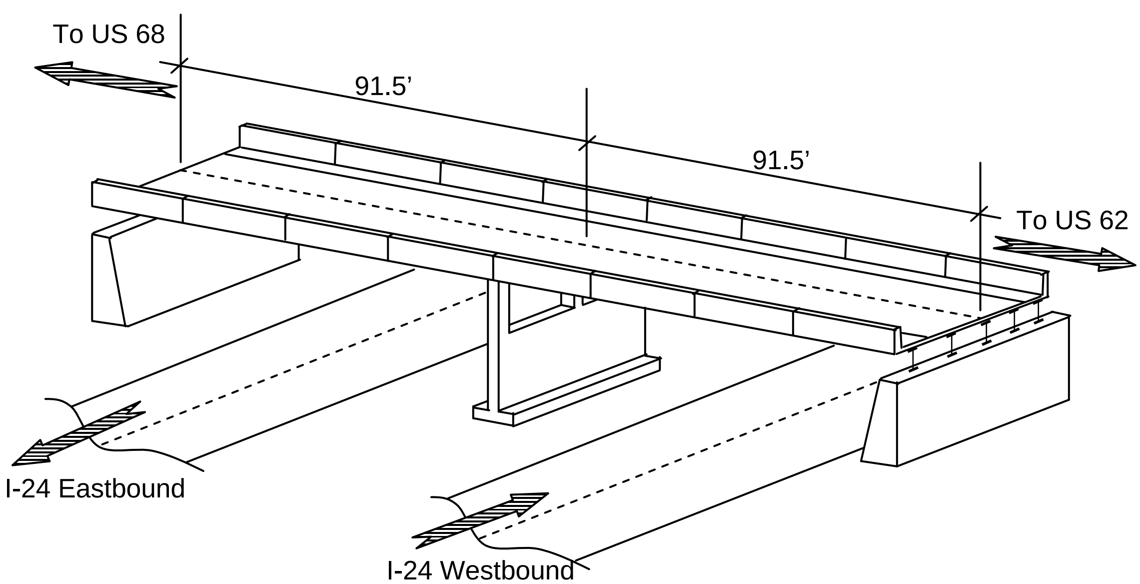


Fig. 4.1. U.S. 68- U.S. 62 Connector Bridge over I-24 in McCracken County, KY.

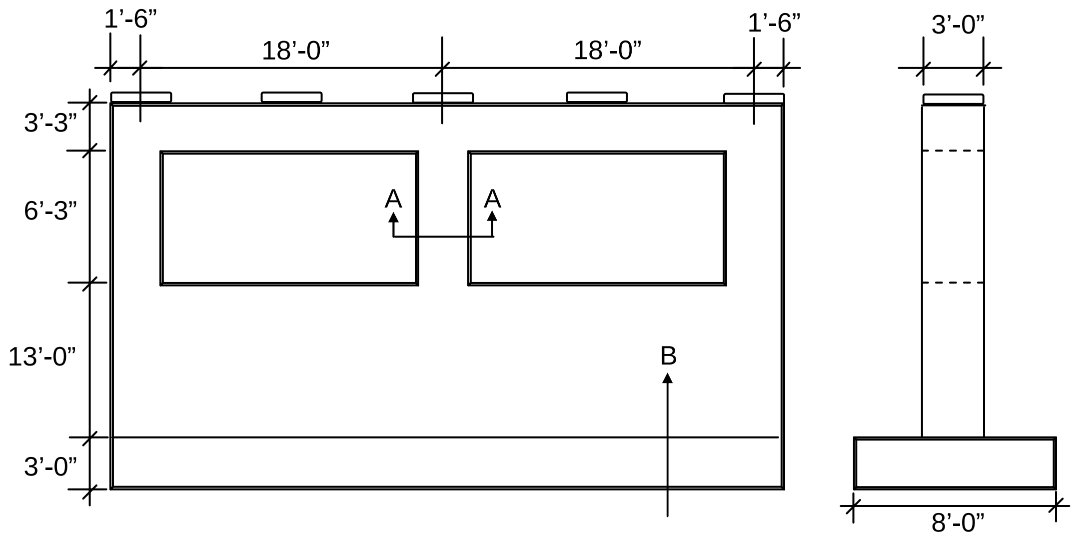


Fig. 4.2. Dimension of the substructure of the US68-US62 connector bridge.

4.2 Bridge Classification and Analysis Procedure

Based on the acceleration contour map, the 250-year design acceleration coefficient for McCracken County is A = 0.19g. Since the bridge is located along a priority route, this bridge is viewed as "Essential" based on AASHTO specifications. This combination of acceleration coefficient and importance classification gives the seismic performance category (SPC) of C (refer to SR Manual Section 1.5).

Section 3.3.2.1 of the SR Manual specifies the minimum dynamic analysis required for a bridge. US68-US62 Connector is a "regular" bridge by SR Manual definition. Based on the criterion set forth in the SR Manual, a regular bridge has less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span-to-span or support-to-support. Therefore, a uniform-load or single-mode spectral method should be specified as the minimum required analysis.

4.3 Bridge Components that Require Seismic Evaluation

Table 2.1 in Section 2.2 lists the bridge components required for seismic evaluation wherever is applicable. For this bridge, almost all C/D ratios listed in Table 2.1 will be investigated.

Seismic *demands* of individual bridge components are determined using SAP 2000. A three dimensional bridge model was built in SAP 2000 for this purpose. The general process of SAP 2000 is given in previous chapter. The mode shapes and natural frequencies of the bridge were determined and the first periods corresponding to the three orthogonal directions, determined using SAP 2000, are 0.4636 seconds (vertical), 0.4112 seconds (longitudinal), and 0.0883 seconds (transverse).

Details and results of the computer analysis are excluded in this example. The seismic demands in the subsequent section are obtained from results generated by the computer analysis.

4.4 Determination of Capacity/Demand (C/D) Ratios

4.4.1 Capacity/Demand Ratios for Expansion Joints and/or Bearings

4.4.1.1 Displacement C/D ratios (Sections 3.6.2 & A.4.2 of the SR Manual)

Two methods are outlined to determine the displacement C/D ratios, r_{bd}. The value, r_{bd}, is the lesser of the values calculated using the following two methods.

Method 1:

$$r_{bd} = \frac{N(c)}{N(d)} = 1.50 \qquad \text{(SR Manual, Eq. A-3)}$$

where

N(c) = the support length provided = 19 in (from the bridge drawing)

N(d) = the minimum support length (see Sect. A.3 of SR Manual) = 12 + 0.03L + 0.12H

where

L = Length, in ft, of the bridge deck from the support under consideration to the adjacent expansion joint or to the end of the bridge deck = 2 x 91.5 ft = 183 ft (use length of the entire bridge deck)

H = Height, in ft, of columns supporting the bridge deck = 20.875 ft (from top of footing to the center of bent cap)

hence, N(d) = 12.67 in

Method 2:

$$r_{bd} = \frac{\Delta_s(c) - \Delta_i(d)}{\Delta_{eq}(d)} = 7.33 \qquad \text{(SR Manual, Eq. A-4)}$$

where

$\Delta_s(c)$ = available support length for movement = N(c) = 19 in

$\Delta_i(d)$ = the maximum possible movement resulting from temperature, shrinkage, and creep shortening = $\alpha L \Delta T$ = 0.143 in (assumed temperature change of 20 degree)

$\Delta_{eq}(d)$ = the maximum calculated relative displacement due to earthquake load = 2.574 in (from SAP 2000, using Response Spectral Analysis)

Thus, r_{bd} is equal to 1.5 from Method 1 (Since, r_{bd} is greater than 1.0, support lengths of the expansion joints and/or bearings are adequate)

4.4.1.2 Force C/D ratio (Sections 3.6.2 & A.4.3 of the SR Manual)

The force C/D ratio of joints and/or bearing can be determined as:

$r_{bf} = \dfrac{V_b(c)}{V_b(d)} = 1.23$ (SR Manual, Eq. A-5)

where
$V_b(c)$ = nominal ultimate capacity of expansion joints and/or bearings = μR_s = 17.67 kips
For this bridge, the bearing type is elastometric. The coefficient of friction, μ, for elastomeric type bearing is assumed to be 0.6. R_s is the average vertical reaction at supports due to self-weight of superstructure, 29.45 kips (SAP 2000).
$V_b(d)$ = Seismic force acting on joints and/or bearings = elastic force determined from analysis or 20% of R_s, whichever is larger = 14.38 kips (SAP 2000).

Since, r_{bf} is greater than 1.0, the joint and/or bearing capacity is adequate.

4.4.2 Capacity/Demand Ratios for Columns and/or Footings

4.4.2.1 Column Force C/D ratio (Sections 3.6.3 & A.5 of the SR Manual)

The column force ratio can be determined as:

$r_{ec} = \dfrac{M_n(c)}{M_n(d)} = 0.56$

where
$M_n(c)$ = nominal capacity of column = 920 kip-ft (see Fig. 4.3 for column cross section)
$M_n(d)$ = elastic moment determined from analysis using CQC method = 1634.80 kip-ft (SAP 2000)

Since, r_{ec} is less than 1.0, strengthening or retrofitting of columns is required.

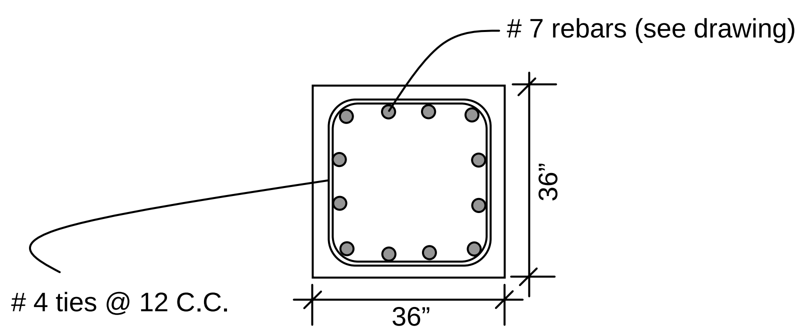


Fig. 4.3. Section A-A of columns of the US68-US62 connector bridge.

4.4.2.2 Footing Force C/D ratio (Sections 3.6.3 & A.5 of the SR Manual)

The footing force ratio can be determined as:

$$r_{ef} = \frac{M_f(c)}{M_f(d)} = 10.2$$

where
$M_f(c)$ = nominal capacity of footing = 3625 kip-ft
Mf(d) = elastic force determined from the analysis = 355.88 kip-ft (also see Fig. 4.4)

Since, r_{ec} is less than 0.8 and r_{ef} is greater than 0.8, Case III is the proper designation according to the SR Manual, Section A.5. As a result, C/D ratios of anchorage, splice, and transverse confinement of columns should also be determined.

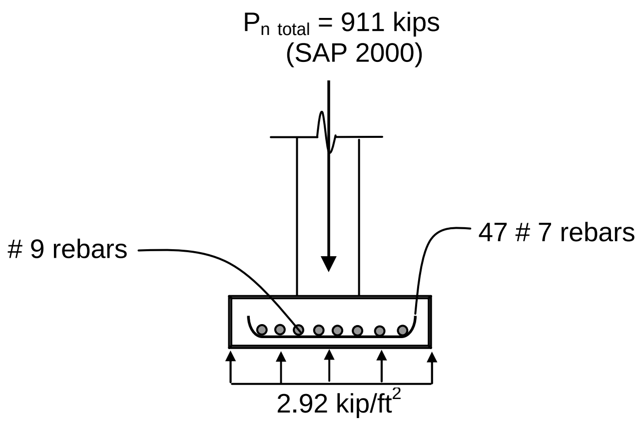


Fig. 4.4. Section B of column footing: Applied load and soil reaction.

4.4.2.3 Anchorage length C/D ratios (Sections 3.6.3 & A.5.1 of the SR Manual)

The following terms must first be calculated before determining the anchorage length ratio:

$l_a(c)$ = effective anchorage length of longitudinal reinforcement = 33 in (Bent Cap) & 156 in (Footing)

$l_a(d)$ = required effective anchorage length of longitudinal reinforcement is larger of

$$= \frac{k_s d_b}{(1 + 2.5c/d_b + k_{tr})\sqrt{f_c'}} = 15.32 \text{ in} \qquad \text{(SR Manual, Eq. A-6)}$$

or = 30 d_b = 26.25 in (controls)

where
k_s = constant of reinforcing steel = = 10208.33
d_b = nominal longitudinal bar diameter = 0.875 in (# 7 rebars shown in drawing)
f_c' = ultimate concrete compression strength = 3000 psi
c = clear concrete cover = 2.5 in
k_{tr} = conservatively assumed = 2.5

In both cases $l_a(c)$ is greater than $l_a(d)$; the anchorage length C/D ratios are 1.0 for bent cap and footing, according to Section A5.1 of the SR Manual.

4.4.2.4 Splice length C/D ratio (Sections 3.6.3 & A.5.2 of the SR Manual)

No splice is used as indicated in the bridge drawings. Longitudinal reinforcements are extended into the footing pedestal.

As a result, r_{cs} is not applicable in this particular case (see more details in Section A.5.2).

4.4.2.5 Shear Strength C/D ratio (Sections 3.6.3 & A.5.3 of the SR Manual)

Column shear failure will occur when shear demand exceeds shear capacity. According to the SR Manual, the sample columns may experience flexure yielding, as the column force ratios (r_{ec}) are less than 1.0. For this particular scenario, shear strength C/D ratio must be identified and determined from one of the three cases presented (see Fig. 81 of Section A.5.3 in the SR Manual). The following terms must first be calculated:

$V_u(d) = 1.3\sum M_u/L_c =$ 269.87 kips

where
M_u = column moment at the location where shear strength is considered
L_c = unsupported length of column
$V_e(d)$ = the maximum calculated elastic force = 214.89 kips (SAP 2000)
$V_i(c)$ = the initial shear resistance of the undamaged column (AASHTO Section 8.16.6)

$$= 3.5\sqrt{f_c^{'}}(0.8A_g) + A_v f_y \frac{d}{s} + 0.2P = 288.76 \text{ kips}$$

$V_f(c)$ = the final shear resistance of the damaged column (Section A.5.3 of SR Manual)

$$= 2\sqrt{f_c^{'}}(A_c) + A_v f_y \frac{d}{s} + 0.2P = 182.13 \text{ kips}$$

where
A_c = concrete core area confined by transverse reinforcement
A_g = gross cross section of column
A_v = leg area of transverse reinforcement
d = effective length of column cross section
s = spacing of transverse reinforcement
f_y = yield strength of transverse reinforcement
P = applied axial load on the column

Since $V_i(c) > V_u(d) > V_f(c)$, this is Case B as specified in the SR Manual.

For Case B, the column shear ratio, r_{cv}:
$r_{cv} = r_{ec} = 1.32$

Since r_{cv} is greater than 1.0, the column possesses adequate shear strength.

4.4.2.6 Confinement C/D ratio (Sections 3.6.3 & A.5.4 of the SR Manual)

Inadequate transverse confinement reinforcement will cause rapid loss of flexural capacity due to buckling of the main reinforcement and crushing of the concrete in compression. The confinement C/D ratio of transverse reinforcement shall be determined as:

$r_{cc} = \mu r_{ec}$ (SR Manual, Eq. A-21)

where

$$\mu = 2 + 4\left(\frac{k_1 + k_2}{2}\right)k_3$$ (SR Manual, Eq. A-22)

where

$$k_1 = \frac{\rho(c)}{\rho(d)\left(0.5 + \frac{1.25P}{A_g f_c^{'}}\right)} \leq 1$$

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

k3 = effectiveness of transverse bar anchorage, 1.0 can be usually be assumed

$\rho(c)$ = volumetric ratio of existing transverse reinforcement

$\rho(d)$ = required volumetric ratio of transverse reinforcement (see AASHTO Section 7.6)

b_{min} = minimum width of the column cross section = 36 in (from drawings)

For this particular case, if μ is assumed to be 2 (most conservative), the confinement ratio, r_{cc} = 1.12

Since r_{cc} is greater than 1.0, it can be concluded that the confinement provided for the columns is adequate.

4.4.2.7 Footing rotation C/D ratio (Sections 3.6.3 & A.5.5 of the SR Manual)

Since r_{ef} is greater than 0.8, the footing rotation and/or yielding ratio will not be investigated.

4.5 Summary of the US68-US62 Connector Bridge (73-0060-00060)

The C/D ratios determined in the previous sections are summarized in Table 4.1. Based upon the results of the seismic evaluation, the supporting columns of the US68-US62 bridge may be damaged during an earthquake event. Hence, it is recommended that appropriate measures be taken to overcome such potential damage. One of the recommendations, for example, is to retrofit columns in order to increase flexural capacity to a minimum of 1635 k-ft over a minimum distance of 4 ft from the top of the web wall (also see Fig. 4.5). If strengthening of columns is not a viable option, redesigning and resizing of columns or the entire bent may be necessary. Another option is to reduce lateral forces induced by earthquakes by installing seismic isolation bearings.

Table 4.1. C/D ratios for the US 68 – US 62 Connector over I-24.

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0068-00060. I-24 Bridge US 68 – US 60 Connector (McCracken County, KY). Span 1 – 91.5 ft, and Span 2 – 91.5 ft.		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.50 > 1.0	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.23 > 1.0	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.56 < 1.0	***Strengthening required*** [a]
5. Force Capacity/Demand Ratio for Footing r_{ef}	10.2 > 1.0	*Capacity is adequate*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	= 1.0	*Capacity is adequate*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	= 1.0	*Capacity is adequate*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A[b]	*Not applicable* [b]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A[b]	*Not applicable* [b]
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.12 > 1.0	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	1.57 > 1.0	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable* [c]

[a] As one possible option, the columns' capacity should to be increased to a minimum of 1635 kip-ft over a minimum distance of 4 ft from the top of the web wall shown in the shaded areas of Fig. 2.10

[b] Longitudinal reinforcement extends into the bent cap and footing pedestal

[c] Not evaluated since ref > 0.8 as proposed in the SR Manual

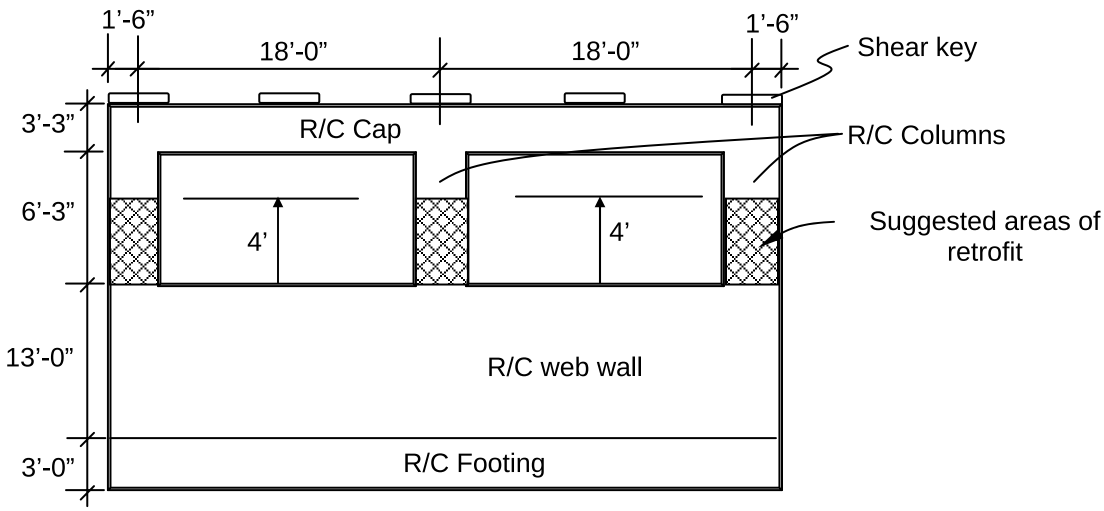


Fig. 4.5. Required areas of retrofit (an increase of flexural capacity to 1635 k-ft is recommended for all columns) for the US68-US62 connector bridge.

5 SUMMARY OF DETAILED SEISMIC EVALUATION ON SELECTED I-24 BRIDGES

5.1 Summary of the I-24 Over the Relocated Cairo Road Bridge (73-0024-00102)

Table 5.1. C/D ratios for the Cairo Road Bridge

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-00102 I-24 Over Relocated Cairo Road (McCracken County, KY) Single-span 110 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.63 > 1.0	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	1.90 > 1.0	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	N/A [a]	N/A [a]
5. Force Capacity/Demand Ratio for Footing r_{ef}	N/A [a]	N/A [a]
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	N/A [a]	N/A [a]
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	N/A [a]	N/A [a]
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A [a]	N/A [a]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A [a]	N/A [a]
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	N/A [a]	N/A [a]
11. Column Shear Capacity/Demand Ratio r_{cv}	N/A [a]	N/A [a]
12. Footing Rotation and/or Yielding Ratio r_{fr}	N/A [a]	N/A [a]

[a] 73-0024-00102 is a simply-supported single span bridge

5.2 Summary of the I-24 Over Perkin Creek Channel Change Bridge (73-0024-00107)

Table 5.2. C/D ratios for the I-24 Over Perkin Creek Channel Change Bridge

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-00107 I-24 Over Perkin Creek Channel Change (McCracken County, KY) Span 1 – 30 ft, Span 2 – 50 ft, and Span 3 – 30 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.78 > 1.0	*Capacity is adequate*
3. Force Capacity/Demand Ratio r_{bf}	8.24 > 1.0	*Capacity is adequate*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.69 < 1.0	*Strengthening required* [b]
5. Force Capacity/Demand Ratio for Footing r_{ef}	N/A [a]	N/A [a]
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	N/A [a]	N/A [a]
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	N/A [a]	N/A [a]
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A [a]	N/A [a]
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A [a]	N/A [a]
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	1.38 > 1.0	*Capacity is adequate*
11. Column Shear Capacity/Demand Ratio r_{cv}	3.44 > 1.0	*Capacity is adequate*
12. Footing Rotation and/or Yielding Ratio r_{fr}	N/A [a]	N/A [a]

[a] The substructure – pier – is made up of twelve 60-foot long reinforced concrete pre-cast concrete piles.

[b] Possible option to overcome flexural deficiency (also see Fig. 2.11)

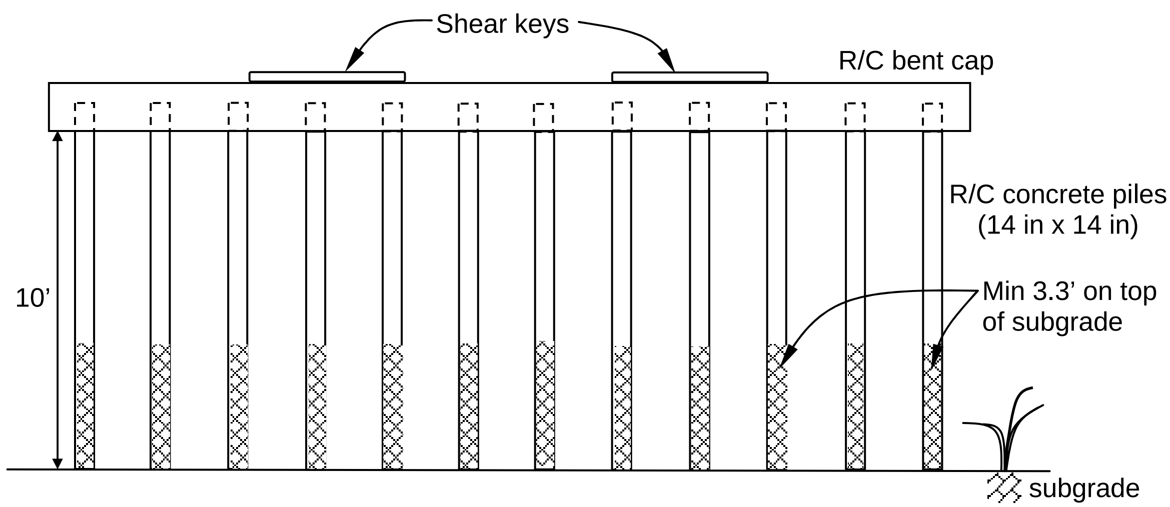


Fig. 5.1. Strengthening of concrete piles (an increase of flexural capacity to 160 k-ft is recommended for all reinforced concrete piles) for the Perkin Creek Channel Change Bridge as one possible option.

5.3 Summary of the I-24 Bridge Crossing US45 (73-0024-00112)

Table 5.3. C/D ratios for the I-24 Bridge Crossing US45

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-00112 I-24 Bridge Crossing US45 (McCracken County) Span 1 – 85 ft, and Span 2 – 85 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	0.61 < 1.0	*Capacity is not adequate.*
3. Force Capacity/Demand Ratio r_{bf}	4.42 > 1.0	*Capacity is adequate.*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	1.30 > 1.0	*Capacity is adequate.*
5. Force Capacity/Demand Ratio for Footing r_{ef}	1.03 > 1.0	*Capacity is adequate.*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.0	*Capacity is adequate.*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.0	*Capacity is adequate.*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A	
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A	
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	> 1.0	*Capacity is adequate.* [a]
11. Column Shear Capacity/Demand Ratio r_{cv}	1.97 > 1.0	*Capacity is adequate.*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable.*

[a] $r_{cc} = \mu r_{ec}$, where $2 \le \mu \le 4$.

5.4 Summary of the Sheehan Road Bridge (73-3075-B00065)

Table 5.4. C/D ratios for the Sheehan Road Bridge

1. Title Summary of the detailed seismic evaluation of bridge No.73-3075-B00065 Sheehan Road Bridge over I-24 (McCracken County) Span 1 – 92 ft, and Span 2 – 92 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	0.74 < 1.0	*Capacity is not adequate.*
3. Force Capacity/Demand Ratio r_{bf}	3.81 > 1.0	*Capacity is adequate.*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.81 < 1.0	*Capacity is not adequate.*
5. Force Capacity/Demand Ratio for Footing r_{ef}	1.05 > 1.0	*Capacity is adequate.*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.0	*Capacity is adequate.*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.0	*Capacity is adequate.*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A	
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A	
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	> 1.0	*Capacity is adequate* [a].
11. Column Shear Capacity/Demand Ratio r_{cv}	1.92 > 1.0	*Capacity is adequate.*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable.*

[a] $r_{cc} = \mu r_{ec}$, where $2 \le \mu \le 4$.

5.5 Summary of the Elmdale Road Bridge over I-24 (73-0024-B00113)

Table 5.5. C/D ratios for the Elmdale Road Bridge over I-24

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-B00113 Elmdale Road Bridge over I-24 (McCracken County) Span 1 – 60 ft, Span 2 – 105 ft, Span 3 – 105 ft, and Span 4 – 60 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	0.67 < 1.0	*Capacity is not adequate.*
3. Force Capacity/Demand Ratio r_{bf}	1.0	*Capacity is adequate.*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.35 < 1.0	*Capacity is not adequate.*
5. Force Capacity/Demand Ratio for Footing r_{ef}	1.13 > 1.0	*Capacity is adequate.*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.0	*Capacity is not adequate.*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.0	*Capacity is adequate.*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A	
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A	
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	0.7 < 1.0	*Capacity is not adequate.*
11. Column Shear Capacity/Demand Ratio r_{cv}	0.92 < 1.0	*Capacity is not adequate.*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable.*

5.6 Summary of the I-24 over Island Creek Bridge (73-0024-00115)

Table 5.6. C/D ratios for the I-24 over Island Creek Bridge

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-00115 I-24 over Island Creek Bridge (McCracken County) Span 1 – 43 ft, Span 2 – 53 ft, and Span 3 – 43 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	0.61 < 1.0	*Capacity is not adequate.*
3. Force Capacity/Demand Ratio r_{bf}	4.64 > 1.0	*Capacity is adequate.*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	0.69 < 1.0	*Capacity is not adequate.*
5. Force Capacity/Demand Ratio for Footing r_{ef}	0.96 > 1.0	*Capacity is adequate.*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.0	*Capacity is not adequate.*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.0	*Capacity is adequate.*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A	
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A	
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	> 1.0	*Capacity is adequate.* [a]
11. Column Shear Capacity/Demand Ratio r_{cv}	1.41 > 1.0	*Capacity is adequate.*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable.*

[a] $r_{cc} = \mu r_{ec}$, where $2 \le \mu \le 4$.

5.7 Summary of the I-24 over Clarks River Bridge (73-0024-00120)

Table 5.7. C/D ratios of the I-24 over Clarks River Bridge

1. Title Summary of the detailed seismic evaluation of bridge No. 73-0024-00120 I-24 over Clarks River Bridge (McCracken County) Span 1 – 140 ft, Span 2 – 200 ft, and Span 3 – 140 ft		
CAPACITY/DEMAND RATIOS FOR EXPANSION JOINTS/BEARINGS		
		Comment:
2. Displacement Capacity/Demand Ratio r_{bd}	1.07 > 1.0	*Capacity is adequate.*
3. Force Capacity/Demand Ratio r_{bf}	2.50 > 1.0	*Capacity is adequate.*
CAPACITY/DEMAND RATIOS FOR COLUMNS AND FOOTING		
		Comment:
4. Force Capacity/Demand Ratio for Column r_{ec}	1.20 > 1.0	*Capacity is adequate.*
5. Force Capacity/Demand Ratio for Footing r_{ef}	1.74 > 1.0	*Capacity is adequate.*
6. Anchorage Capacity/Demand Ratio at Bent Cap $r_{ca(Cap)}$	1.0	*Capacity is adequate.*
7. Anchorage Capacity/Demand Ratio at Footing $r_{ca(Footing)}$	1.0	*Capacity is adequate.*
8. Splice Capacity/Demand Ratio at Bent Cap $r_{cs(Cap)}$	N/A	
9. Splice Capacity/Demand Ratio at Footing $r_{cs(Footing)}$	N/A	
10. Transverse Confinement Capacity/Demand Ratio r_{cc}	> 1.0	*Capacity is adequate.* [a]
11. Column Shear Capacity/Demand Ratio r_{cv}	2.59 > 1.0	*Capacity is adequate.*
12. Footing Rotation and/or Yielding Ratio r_{fr}	-	*Not applicable.*

[a] $r_{cc} = \mu r_{ec}$, where $2 \le \mu \le 4$.

6 SUMMARY AND RECOMMENDATIONS

I-24, in Western Kentucky lies just east of the New Madrid Seismic Zone (NMSZ). The zone remains active with an average of nearly 200 seismic events recorded annually. Due to its locality and socioeconomics factors, I-24 is designated by the Federal Highway Administration (FHWA) as one of the high priority and emergency routes, which must remain functional and operational after an earthquake event. Therefore, the objective of this study is to perform seismic evaluation on selected highway bridges on/over I-24 which are deemed susceptible to severe damage during a major earthquake event.

Prior to performing detailed seismic evaluation of the selected bridges, 127 highway bridges (82 on and 45 over I-24) were ranked using a seismic rating system. The rating system ranked these bridges based on several factors: structural vulnerability, seismic and geotechnical hazards, and socioeconomic factors. All in all, 14 bridges (parallel bridges included) were selected based on the ranking procedure for subsequent seismic evaluation. All 14 bridges are in McCracken County with a peak ground acceleration of 0.19g (i.e. highest amongst counties in the proximity of NMSZ). These bridges constructed in a similar time frame (i.e. late 1960s) are of reinforced, prestressed, and steel-composites types – representative of typical bridge construction types in Kentucky.

A capacity/demand (C/D) ratio method outlined in the *Seismic Retrofitting Manual for Highway Bridges* (Buckle, I.G. and Friedland, I.M., 1995) was used to evaluate four main bridge components; namely the expansion joints, bearings, columns, and footing. Two other aspects of a bridge are embankment and foundation stability, and these were not evaluated in this study (seismic performance of the embankment and foundation stability of selected bridges on/over I-24 is studied and presented in another report in this series). The C/D ratio method required the determination of various structural responses (i.e. displacements, forces, etc.) under a prescribed dynamic event. In this study, SAP 2000 was used to achieve that task by first creating a finite element model of all 14 bridges, and followed subsequently by dynamic analysis, based on a given time-history spectra response of a 250-year event. A summary of C/D ratios of all 14 bridges is presented in Table 5.1. Seismic deficiencies of these bridges are shown in Table 5.2.

The results indicate that the rating system is an effective means in terms of identifying and prioritizing highway bridges for seismic evaluation and retrofit.

TABLE 6.1. C/D ratios of selected Interstate-24 bridges

C/D Ratios / BIN	Joints and/or Bearings		Columns and/or Footings									Bridge ranks
	r_{bd}	r_{bf}	r_{ec}	r_{ef}	r_{ca} (cap)	r_{ca} (footing)	r_{sc} (cap)	r_{sc} (footing)	r_{cv}	r_{cc}	r_{fr}	
73-0024-00112 73-0024-00112 P	**0.61**	4.42	1.30	1.03	1.0	1.0	-	-	2.60	1.97	-	14
73-0068-00060 73-0068-00060 P	1.50	1.23	**0.56**	10.2	1.0	1.0	-	-	1.12	1.57	-	24
73-0024-00102 73-0024-00102 P	1.63	1.90	-	-	-	-	-	-	-	-	-	29
73-0024-00120 73-0024-00120 P	1.07	2.50	1.20	1.74	1.0	1.0	-	-	2.40	2.59	-	29
73-0024-00107 73-0024-00107 P	1.78	8.24	**0.69**	-	-	-	-	-	1.38	3.44	-	36
73-0024-00115 73-0024-00115 P	**0.61**	4.64	**0.69**	**0.96**	1.0	1.0	-	-	1.38	1.41	-	36
73-3075-00065	**0.74**	3.81	**0.81**	1.05	1.0	1.0	-	-	1.62	1.92	-	48
73-0024-00113	**0.67**	1.0	**0.35**	1.13	1.0	1.0	-	-	**0.7**	**0.92**	-	48

Note: When C/D ratio is less than 1.0, retrofitting measure must be performed

Table 6.2: Summary of seismic deficiencies of selected bridges along I-24.

Bridge Number (BIN)	Ranking	Seismic Deficiencies
73-0024-00112 73-0024-00112 P	14	- Bearing seat capacity
73-0068-00060 73-0068-00060 P	24	- Column flexural capacity
73-0024-00107 73-0024-00107 P	36	- Column flexural capacity
73-0024-00115 73-0024-00115 P	36	- Bearing seat capacity - Column flexural capacity - Footing flexural capacity
73-3075-00065	48	- Bearing seat capacity - Column flexural capacity
73-0024-00113	48	- Bearing seat capacity - Column flexural capacity - Column shear capacity - Column transverse confinement

Note that two pairs of bridges [73-0024-00102 (P) and 73-0024-00120 (P)] with a rank of 29 possess no seismic deficiency (See tables 5.1 and 5.2). All 14 bridges in this investigation contain one or more forms of seismic deficiencies, as illustrated in Table 5.1. This indicates that the rating system is an effective means in determining and prioritizing highway bridges for seismic evaluation and retrofit processes.

It is recommended that the following measures in one form or another be taken to overcome these deficiencies:

- Bearing seat deficiency – Bearing seat width or length be extended, and/or restrainer be provided to avoid loss of support due to excessive lateral movement;
- Column flexural deficiency – Columns be redesigned, resized, and/or strengthened. Isolated bearing seat may also be considered to reduced lateral forces;
- Footing flexural deficiency – see Column flexural deficiency;
- Column shear deficiency – see Column flexural deficiency; and
- Column transverse confinement – see Column flexural deficiency.

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