Research Report
KTC-06-26/SPR-206-00-7F

KENTUCKY TRANSPORTATION CENTER

SEISMIC EVALUATION AND RANKING OF BRIDGE EMBANKMENTS ALONG I-24 IN WESTERN KENTUCKY

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SEISMIC EVALUATION AND RANKING OF BRIDGE EMBANKMENTS ALONG I-24 IN WESTERN KENTUCKY

by

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16. Abstract

This study represents one of the Seismic Evaluation of I-24 Bridges investigative series. The focus is on slope or embankment stability and liquefaction potential of embankments of bridges along I-24 in Western Kentucky. A rating system is derived to assist in identifying and prioritizing bridge embankments that are susceptible to failure during to projected seismic events.

Embankments of one hundred and twenty seven (127) bridges along I-24 in Western Kentucky are evaluated for 50-year and 250-year seismic events. Fifty-two (52) of the 127 embankments were rated as ‘critical’ for the 50-year event, and 60 were rated as ‘critical’ for the 250-year event. Based on this preliminary evaluation, it is recommended that the bridge embankments classified as ‘critical’ be further investigated by performing more detailed analysis.

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EXECUTIVE SUMMARY

BACKGROUND

The seismic evaluation of bridge stability is an important aspect of structural and earthquake engineering practice. To date, several codified specifications dealing with seismic design of bridge structures exist; most notably the seismic provisions by the American Association of State Highway and Transportation Officials (AASHTO 2002 and 2004). In 1995, the Federal Highway Administration (FHWA) published a guide entitled *"Seismic Retrofitting Manual for Highway Bridges"* (Buckle and Friedland, 1995), and known hereafter as the Manual. The Manual provided for bridge owners nationwide a roadmap for the evaluation and retrofit of bridges in seismic zones. The Manual discusses in details the following aspects: (1) a ranking procedure for preliminary seismic evaluation of highway bridges; (2) analytical techniques for detailed seismic evaluation, when such a need arises; and (3) retrofit guidelines for seismically deficient bridge components. Much of the evaluation effort focused on the stability and strength of a bridge's superstructure and substructure.

OBJECTIVE

The objective of this report is to provide a methodology for carrying out a preliminary seismic evaluation and ranking of embankments for bridges along I-24 in Western Kentucky. The methodology focuses on the slope or embankment stability assessment and the liquefaction potential.

SLOPE STABILITY AND LIQUEFACTION POTENTIAL OF BRIDGE EMBANKMENTS

Methodologies assessing the stability of bridge embankments and the potential of soil liquefaction are presented in this report. The methodologies focus on the following aspects: (1) slope stability capacity/demand (C/D) ratio; (2) embankment horizontal displacement (u); and (3) liquefaction potential of the foundation soil underneath the bridge embankment. Detailed discussions of these different aspects are presented in this report.

RANKING OF BRIDGE EMBANKMENTS

In order to facilitate the identification of the critical embankments for bridges along I-24 in Western Kentucky, a ranking system that is based on slope stability, liquefaction potential, and/or a combination of the two, has been established, and is presented in this report. The resulting ranking will assist in prioritizing the bridge embankments that are in need of highest attention or in demand of other course of action. Tables E1 and E2 list the bridge embankments that are considered 'critical' (designated as Class A) based on the 50-year and 250-year projected seismic events. Fifty two (52) of the one-hundred and twenty seven (127) embankments were rated as 'critical' for the 50-year event, and 60 were rated as 'critical' for the 250-year event.

SUMMARY AND RECOMMENDATION

A step-by-step procedure is presented for the ranking of the bridge embankments along I-24 in Western Kentucky. The ranking assists in identifying and prioritizing the bridge embankments that are susceptible to failure due to projected seismic events.

Based on this preliminary evaluation, it is recommended that the bridge embankments classified as 'critical' (Tables E1 and E2) be further investigated through carrying out more detailed analysis.

NOTE: This report is the seventh (7th) 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

Table E.1: Ranking of Critical Bridge Embankments along I-24 for the 50-Year Seismic Event
(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U[6] in (cm)		
Christian	24-0024-B00125 & 24-0024-B00125P	9	0.81	13.5 (34.2)	High	A1
	24-0024-B00090 & 24-0024-B00090P	9	0.78	1.5 (3.7)	High	A2
	24-0024-B00132 & 24-0024-B00132P	9	0.65	0.8 (2.0)	High	A3
Lyon	72-0024-B00035 & 72-0024-B00035P	15	0.96	0.2 (0.4)	High	A1
	72-5229-B00034	15	0.99	0.1 (0.2)	High	A2
	72-0024-B00044 & 72-0024-B00044P	15	1.14	0.0 (0.0)	High	A3
	72-0024-B00048 & 72-0024-B00048P	15	1.19	0.0 (0.0)	High	A4
	72-0024-B00039 & 72-0024-B00039P	15	1.29	0.0 (0.0)	High	A5
Trigg	111-0024-B00048 & 111-0024-B00048P	9	1.01	0.0 (0.0)	High	A1
Marshall	79-0024-B00117 & 79-0024-B00117P	15	0.77	35.4 (89.8)	High	A1
	79-0024-B00116 & 79-0024-B00116P	15	0.69	2.3 (5.8)	High	A2
	79-0024-B00113 & 79-0024-B00113P	15	0.83	0.8 (2.1)	High	A3
	79-0024-B00115 & 79-0024-B00115P	15	0.83	0.8 (2.1)	High	A4
	79-0095-B00112	15	0.87	0.4 (1.1)	High	A5
	79-0024-B00118 & 79-0024-B00118P	15	0.54	0.2 (0.4)	High	A6
	79-0024-B00114 & 79-0024-B00114P	15	0.96	0.1 (0.3)	High	A7
Caldwell	None of the bridges are 'critical'.					

1 As defined in the Kentucky Transportation Cabinet (KyTC) Bridge Inventory
2 The letter 'P' stands for parallel bridges.
3 PGA is the peak ground acceleration defined Street et al. (1996).
4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
6 Horizontal displacement (*u*) is calculated when C/D ratio is less than 1.0, or else *u* is equal zero.
7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table E.1 (Cont'): Ranking of Critical Bridge Embankments along I-24 for the 50-Year Seismic Event
(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Livingston	70-0024-B00063 & 70-0024-B00063P	15	0.60	2.0 (5.1)	High	A1
	70-0024-B00062 & 70-0024-B00062P	15	0.85	0.6 (1.5)	High	A2
McCracken	73-0024-B00104 & 73-0024-B00104P	15	0.79	5.6 (14.3)	High	A1
	73-0024-B00103 & 73-0024-B00103P	15	0.81	2.7 (6.9)	High	A2
	73-0068-B00060 & 73-0068-B00060P	15	0.83	1.7 (4.4)	High	A3
	73-0787-B00064	15	0.83	1.7 (4.3)	High	A4
	73-0024-B00107 & 73-0024-B00107P	15	0.83	1.0 (2.4)	High	A5
	73-0024-B00105 & 73-0024-B00105P	15	0.86	0.9 (2.2)	High	A6
	73-0024-B00112 & 73-0024-B00112P	15	0.86	0.5 (1.3)	High	A7
	73-0024-B00102 & 73-0024-B00102P	15	0.90	0.4 (1.0)	High	A8
	73-0131-B00009	15	0.90	0.3 (0.8)	High	A9
	73-0024-B00111 & 73-0024-B00111P	15	0.92	0.3 (0.7)	High	A10
	73-0024-B00100	Bridge over the Ohio River and is beyond the scope of this study				

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6 Horizontal displacement (*u*) is calculated when C/D ratio is less than 1.0, or else *u* is equal zero.
7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table E.2: Ranking of Critical Bridge Embankments along I-24 for the 250-Year Seismic Event
(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Christian	24-0024-B00125 & 24-0024-B00125P	9	0.81	54.2 (137.7)	High	A1
	24-0024-B00090 & 24-0024-B00090P	9	0.78	5.7 (14.5)	High	A2
	24-0024-B00132 & 24-0024-B00132P	9	0.65	3.1 (7.8)	High	A3
Lyon	72-0024-B00035 & 72-0024-B00035P	15	0.83	3.2 (8.1)	High	A1
	72-5229-B00034	15	0.86	2.1 (5.4)	High	A2
	72-0024-B00044 & 72-0024-B00044P	15	0.96	0.4 (1.1)	High	A3
	72-0024-B00048 & 72-0024-B00048P	15	0.99	0.3 (0.8)	High	A4
	72-0024-B00039 & 72-0024-B00039P	15	1.05	0.0 (0.0)	High	A5
Trigg	111-0024-B00048 & 111-0024-B00048P	9	1.01	0.0 (0.0)	High	A1
	111-6051-B00049	9	2.35	0.0 (0.0)	High	A2
Marshall	79-0024-B00117 & 79-0024-B00117P	15	0.77	145.3 (369.1)	High	A1
	79-0024-B00116 & 79-0024-B00116P	15	0.69	8.9 (22.7)	High	A2
	79-0024-B00113 & 79-0024-B00113P	15	0.83	3.2 (8.1)	High	A3
	79-0024-B00115 & 79-0024-B00115P	15	0.83	3.2 (8.1)	High	A4
	79-0095-B00112	15	0.87	1.7 (4.3)	High	A5
	79-0024-B00118 & 79-0024-B00118P	15	0.54	0.7 (1.7)	High	A6
	79-0024-B00114 & 79-0024-B00114P	15	0.96	0.4 (1.1)	High	A7
	79-0024-B00109	15	2.22	0.0 (0.0)	High	A8
Caldwell	None of the bridges are 'critical'.					

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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table E.2 (Cont'): Ranking of Critical Bridge Embankments along I-24 for the 250-Year Seismic Event

(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U[6] in (cm)		
Livingston	70-0024-B00063 & 70-0024-B00063P	15	0.60	7.8 (19.9)	High	A1
Livingston	70-0024-B00062 & 70-0024-B00062P	15	0.85	2.3 (5.9)	High	A2
McCracken	73-0024-B00104 & 73-0024-B00104P	19	0.75	31.4 (79.8)	High	A1
McCracken	73-0024-B00103 & 73-0024-B00103P	19	0.76	15.6 (39.5)	High	A2
McCracken	73-0024-B00120 & 73-0024-B00120P	19	0.67	11.3 (28.7)	High	A3
McCracken	73-0024-B00118 & 73-0024-B00118P	19	0.77	10.7 (27.3)	High	A4
McCracken	73-0068-B00060 & 73-0068-B00060P	19	0.77	10.4 (26.3)	High	A5
McCracken	73-0787-B00064	19	0.78	10.1 (25.8)	High	A6
McCracken	73-0024-B00115 & 73-0024-B00115P	19	0.79	6.6 (16.8)	High	A7
McCracken	73-0024-B00107 & 73-0024-B00107P	19	0.76	6.1 (15.5)	High	A8
McCracken	73-0024-B00105 & 73-0024-B00105P	19	0.80	5.7 (14.5)	High	A9
McCracken	73-0024-B00112 & 73-0024-B00112P	19	0.79	3.5 (8.9)	High	A10
McCracken	73-0024-B00102 & 73-0024-B00102P	19	0.83	2.9 (7.3)	High	A11
McCracken	73-0131-B00009	19	0.84	2.5 (6.4)	High	A12
McCracken	73-0024-B00111 & 73-0024-B00111P	19	0.85	2.2 (5.5)	High	A13
McCracken	73-0024-B00100	Bridge over the Ohio River and is beyond the scope of this study				

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8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

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1. INTRODUCTION

1.1 Background

The New Madrid and Wabash Valley Seismic Zones (Fig. 1.1) can cause considerable vibrations in Western Kentucky if a sizable earthquake were to occur in that region. The New Madrid Seismic Zone (NMSZ) is potentially one of the most destructive fault zones in the United States. In 1811-1812, four of the most severe earthquakes in the American history occurred in the New Madrid Seismic Zone. The instrumental observations indicate that the New Madrid Seismic Zone is still the most hazardous zone in the east of the Rocky Mountains (Johnston 1985; and Johnston and Nava 1985).

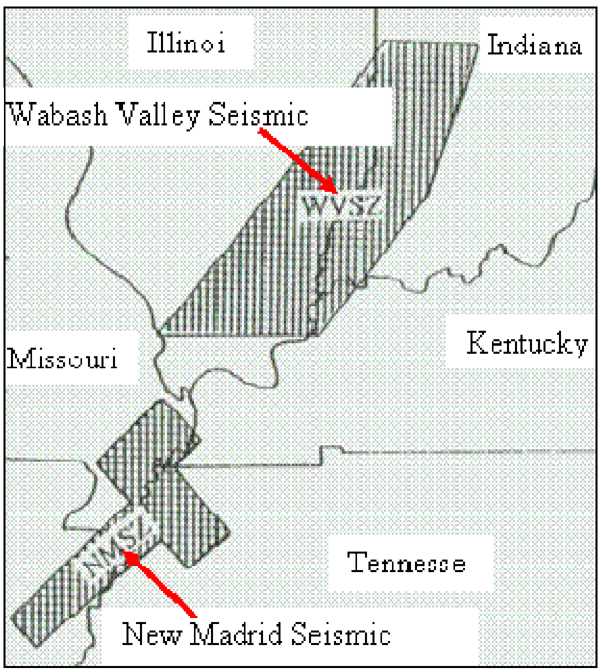


Fig. 1.1 – Seismic zones affecting Kentucky.

Interstate 24 (I-24) is located in close proximity to the NMSZ. The Federal Highway Administration has designated I-24 as a high-priority route and an emergency route for the city of Memphis, Tennessee. Due to its close proximity to the NMSZ, Memphis is at a high risk of structural damage both for the bridges and the buildings, which were built before the use of seismic building codes. It is for these reasons that emergency personnel and equipment from surrounding states must utilize clear and safe routes in the event that a major earthquake strikes.

1.2 Objective and Scope

The preliminary seismic evaluation and ranking of the bridges along I-24 in Western Kentucky and detailed seismic evaluation of selected I-24 bridges have been conducted, and they are presented in separate reports (KTC-06-22/SPR 206-99-3F and KTC-06-23/SPR 206-99-4F, respectively).

As a part of the *Seismic Evaluation of I-24 Bridges* series, this particular report deals with the evaluation and ranking of the bridge embankments along I-24 in Western Kentucky. The scope includes the slope stability of the embankments and liquefaction potential of the foundation soil. Methodologies for slope stability and liquefaction potential assessments will be presented herein. A ranking system derived based on the combination of the afore-mentioned aspects is used to rank the 127 bridge embankments along I-24 in Western Kentucky.

1.3 Report Organization

This report consists of the following chapters:

Chapter 1 provides introductory information on the seismic analysis of bridges. Objectives and tasks of this study are also presented.

Chapter 2 provides a discussion on the seismic slope stability analysis of the bridge embankments.

Chapter 3 provides a discussion on the liquefaction potential of the foundation soil at the embankment site.

Chapter 4 displays a ranking system that is derived based on the slope stability and liquefaction potential of the bridge embankments.

Chapter 5 provides a summary, conclusion, and recommendation of this study.

2. SLOPE STABILITY ASSESSMENT

2.1 Introduction

Seismic stability analysis and retrofit of earth embankments, including site remediation, has been, to date primarily, focused on embankment dams and earth retaining structures (Buckle and Friedland 1995). If a bridge embankment on a priority route is at a high failure risk, soil stabilization may be required depending on the importance of the bridge.

In this chapter, a methodology used to estimate the slope stability capacity/demand (C/D) ratio is discussed. In cases where the capacity/demand (C/D) ratio is less than 1.0, the potential mass displacement is estimated, and the method to predict such horizontal displacement is presented. It should be noted that the methodologies presented herein are for preliminary seismic evaluation and ranking purposes. For detailed assessment of an embankment's vulnerability during a seismic event, a far more sophisticated approach should be employed, which is beyond the scope of the current investigation.

Prior to the determination of the C/D ratio, and possible embankment displacement, the following input variables related the slope stability computation must first be addressed. They include the embankment geometry, material, level of natural ground line, soil type, seismic event, and upper level of bedrock.

2.2. Input Variables

The embankment geometry, material, level of natural ground line, soil type, seismic event, and upper level of bedrock, will be discussed in the following sub-sections. Most of the input variables will be explained with the aid of Fig. 2.1.

2.2.1 Embankment Geometry

The ideal case is to carry out an on-site inspection to obtain the actual geometry of each bridge embankment. However, should there be difficulties encountered in gathering the on-site information, the embankment geometry may be taken from the bridge plans. It is anticipated that utilizing the data that are obtained from the bridge plans will not affect the final seismic ranking and priority list since similar assumptions and approximations are used for all the embankments. Embankment slopes are assumed to be free from any evidence of impending failure, swampy conditions, or other terrain conditions that might be relevant to their stability. For a typically irregular slope, an idealization of the slope has to be performed in such a way that results in the lowest seismic slope stability *C/D* ratio. It is assumed that the material that was used or erosion protection of the slope will not have significant influence on the resulting seismic slope stability, and therefore is not considered as an input parameter. The embankment slope geometry is identified by its height (*H*) and idealized inclination (*b*) (Fig. 2.1). The water table is assumed to be located below the embankment base in order to obtain the most critical seismic stability conditions. The analysis is carried out on both ends of each bridge and the most critical embankment slope at either end, which results in the lower seismic slope stability *C/D* ratio, is considered in the ranking analysis and priority list.

2.2.2 Material, Natural Ground Line, and Soil Type

The soil profile at a bridge site is often composed of naturally deposited soils rather than controlled fill. The profile usually consists of multiple layers of different soils and the contact between softer foundations and stiffer bedrock soils is typically irregular. Defining the soil conditions at a site requires detailed site-specific sub-surface exploration that is not available at the majority of the existing bridge embankment sites as in the case of I-24 bridges. Therefore, it is assumed that the soil at a bridge site has a uniform un-drained shear strength, which is different from the embankment soil. The soil is considered to be in a continuous contact with the bedrock layer, which is a layer of high strength at some depth below the embankment.

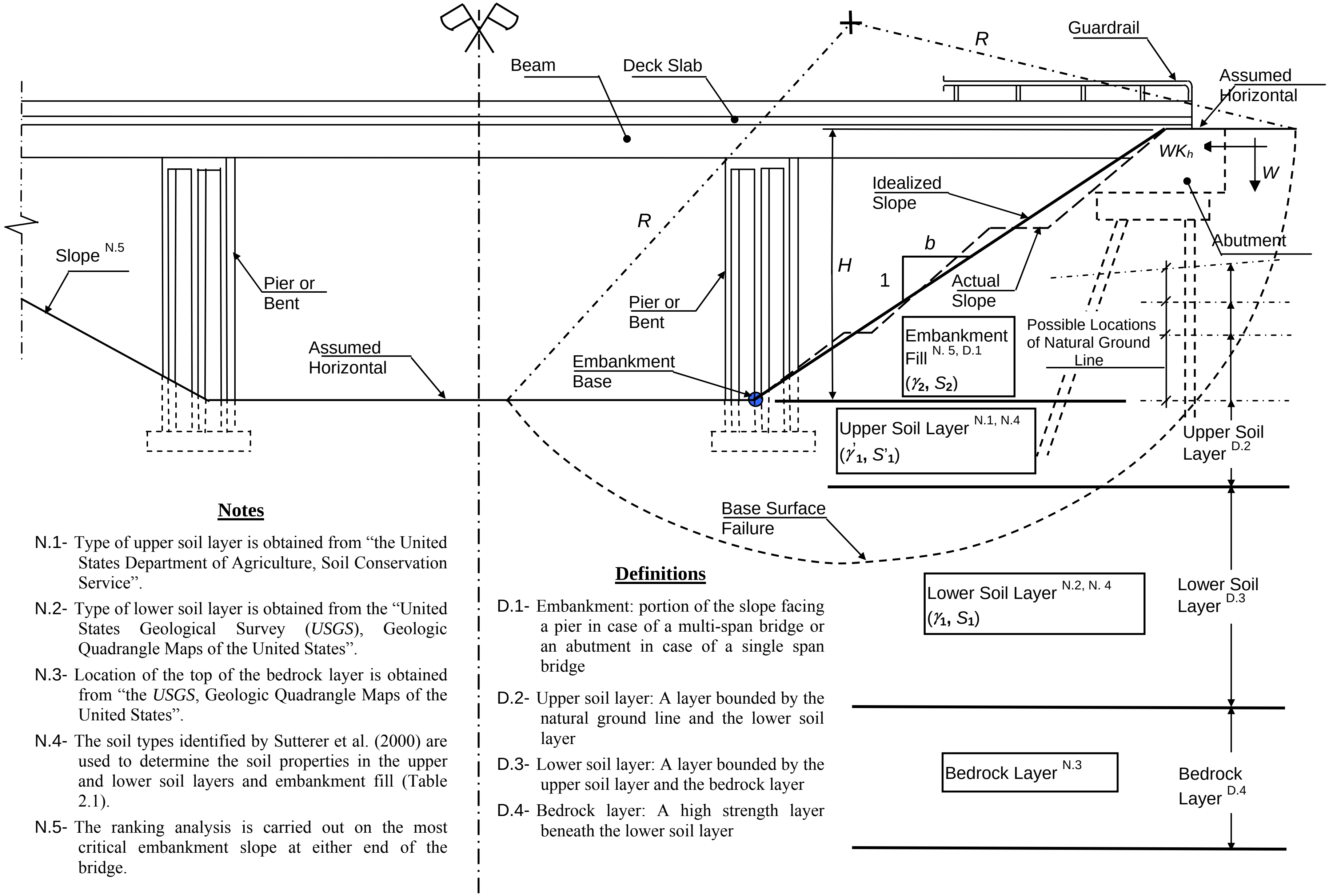
The source of the soil data is dependent on the level of the Natural Ground Line (*NGL*) shown in Fig. 2.1. Both the "*Geologic Quadrant Maps of the United States*" that are provided in "*United States Geologic Survey* (*USGS*)" maps and the "*Soil Conservation Service, Soil Survey*" maps that are reported by the "*United States Department of Agriculture* (*USDA*)" are used in this investigation to identify the soil type underneath an embankment. The way by which either map is chosen is based on the level of the *NGL* as compared to the embankment base. Whenever the level of the *NGL* is above the level of the embankment base by more than 1.5 m. (5 ft), the analysis will be solely based on the soil data obtained from the "*Geologic Quadrangle Maps of the United States*", that are provided by the *USGS*. Otherwise, an additional case in which the soil data is derived from the "*Soil Conservation Service, Soil Survey*", maps that are provided by the *USDA*, is considered. The dependency on the *USDA* maps can be attributed to the fact that the top 1.5 m. (5 ft) soil can be accurately obtained from such maps. The soil types and their respective strengths in the current investigation are presented in Table 2.1. Shear strengths are assigned by Sutterer et al. (2000) for cohesion-less soil materials, and are based on standard penetration tests (Table 2.1). Lower shear strengths are assigned to accommodate for the anticipated liquefaction potential at many bridge sites. The shear strength that is assigned for cohesive soils in Table 2.1 is chosen after examining comparable un-confined compression data. The shear strength that is assigned to the embankment fill is adjusted to reflect the cyclic loading effects between un-drained failure for both cohesive and saturated cohesion-less soils, and the intermediate behavior between drained and un-drained for dry and partially saturated soils. The density and shear strength of the embankment soils are conservatively estimated by assuming that a marginal compactive effort may have been applied during construction. Should there be more accurate soil properties, they may replace those provided in Table 2.1.

Table 2.1: Density and Strength of Soils and Embankments

Geologic Formation	Mass Density γ		Shear Strength S	
	(g/cm^3)	(lb/ft^3)	(kg/cm^2)	(lb/ft^2)
Alluvium	1.92	120	0.20	410
Weathered loess	1.84	115	0.35	717
Continental deposits	2.00	125	0.75	1536
Residuum	2.08	130	1.00	2048
Embankment	2.00	125	0.50	1024

2.2.3 Upper Level of Bedrock Layer

Data regarding the level under which a hard stratum (stiff bedrock layer) exists is not always available for the majority of the existing embankment sites; especially for smaller bridges. An initial assumption of the upper level of this hard stratum is estimated from the "*Geologic Quadrant Maps of the United States*", maps that are provided by the USGS. The actual upper level of the stiff bedrock layer specifically falls within the range from the level of the embankment base down to the top level of the hard stratum. For the sake of conducting seismic risk assessment of a bridge embankment, different upper levels of the bedrock layer within that range are considered. Wherever the upper level of the bedrock layer is not known at a bridge site, the following three assumptions of this level are made, and the most critical case is considered in the ranking analysis: (1) at the same level of the embankment base; (2) at the same level of the bottom level of the lower soil layer, which is also the upper level of the hard stratum; and (3) at mid-height of the lower soil layer. Other assumptions of the top level of the bedrock layer may be considered if they yield a lower seismic slope stability C/D ratio. The top level of the bedrock layer that is adopted in the ranking analysis is the one that results in the worst scenario (i.e. the lowest seismic slope stability C/D ratio).



Notes

N.1- Type of upper soil layer is obtained from "the United States Department of Agriculture, Soil Conservation Service".

N.2- Type of lower soil layer is obtained from the "United States Geological Survey (*USGS*), Geologic Quadrangle Maps of the United States".

N.3- Location of the top of the bedrock layer is obtained from "the *USGS*, Geologic Quadrangle Maps of the United States".

N.4- The soil types identified by Sutterer et al. (2000) are used to determine the soil properties in the upper and lower soil layers and embankment fill (Table 2.1).

N.5- The ranking analysis is carried out on the most critical embankment slope at either end of the bridge.

Definitions

D.1- Embankment: portion of the slope facing a pier in case of a multi-span bridge or an abutment in case of a single span bridge

D.2- Upper soil layer: A layer bounded by the natural ground line and the lower soil layer

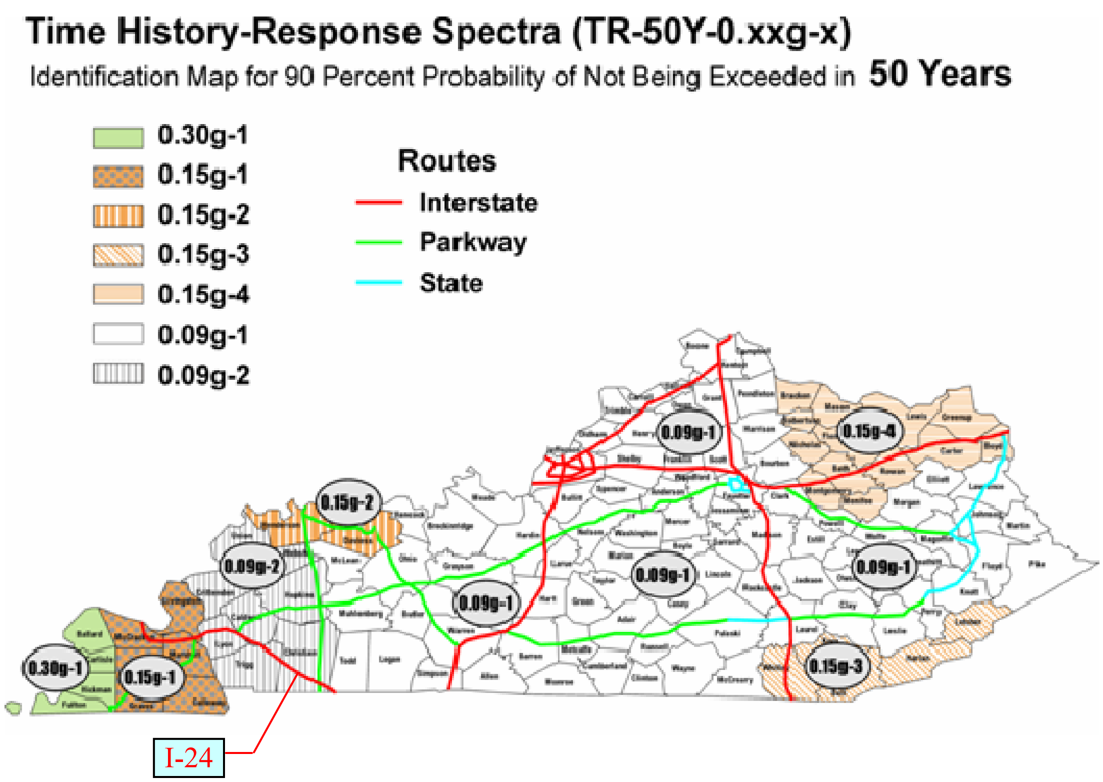
D.3- Lower soil layer: A layer bounded by the upper soil layer and the bedrock layer

D.4- Bedrock layer: A high strength layer beneath the lower soil layer

Fig. 2.1 – Bridge Embankment's Representation for Seismic Ranking.

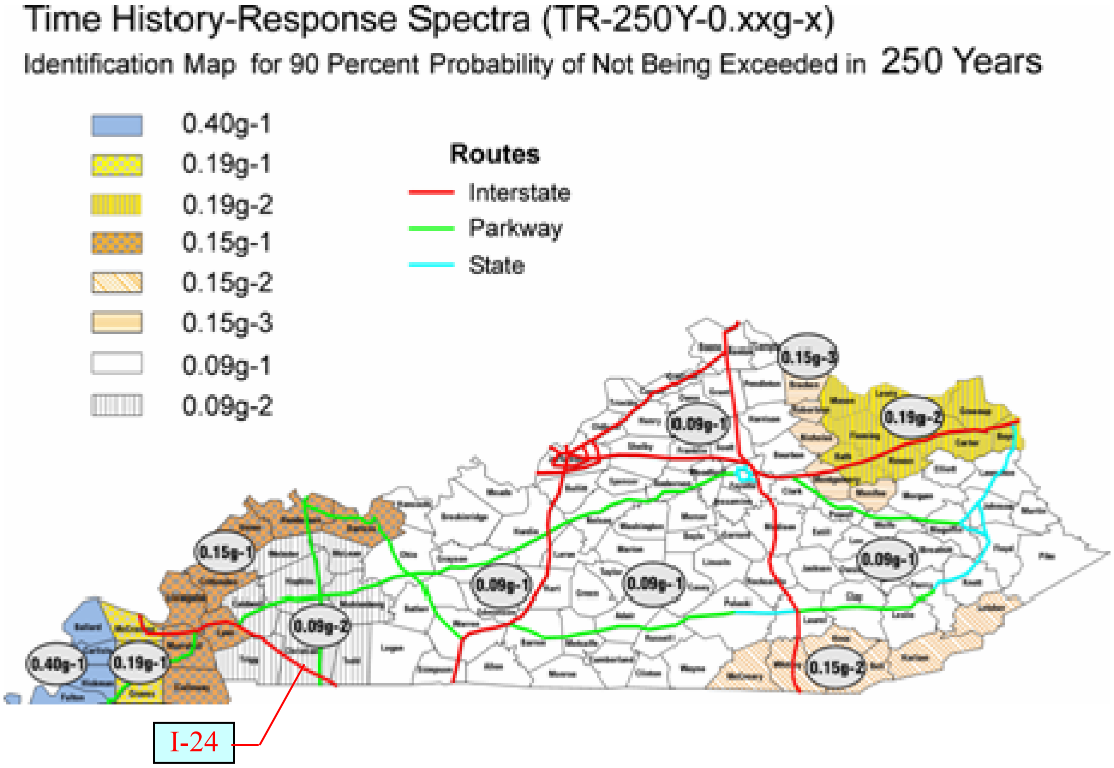
2.2.4 Seismic Events

The input Peak Ground Acceleration (PGA), which is the maximum bedrock acceleration at a designated embankment site, is obtained from the seismic maps that are generated for specific seismic events. The choice of the seismic event is based on the importance and anticipated performance of the bridge as well as its geographic location on the seismic map. The seismic maps provided by AASHTO (2002) define the acceleration coefficient based on a uniform risk method for seismic hazard. The probability that the acceleration coefficient will not be exceeded for a 50-year event is estimated to be 90%, with an expected return period of 475 years (AASHTO 2002). Alternatively, seismic maps that may have been generated by the State Department of Transportation can be used. For the Commonwealth of Kentucky, for instance, 50-year, 250-year, and 500-year seismic events are developed by Street et al. (1996). These events have a 90% probability of not being exceeded in 50 years, 250 years, and 500 years, respectively. For instance, almost all of the bridges and their embankments on priority routes in western Kentucky shall withstand the 50-year and 250 year seismic events. The 50-year and 250-year seismic maps, depicted in Fig. 2.2, are used in the current investigation.



(a) 50-year seismic event

Fig. 2.2 – Seismic maps of Kentucky (Street et. al. 1996).



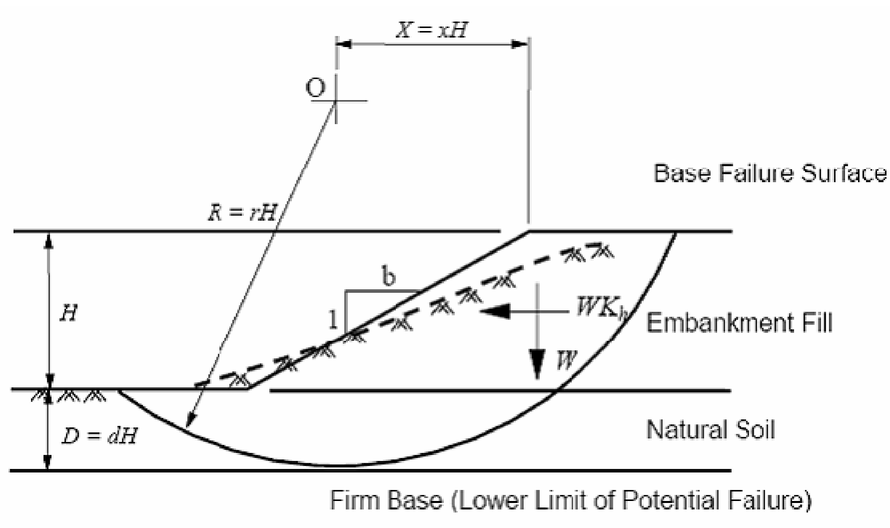
(b) 250-year seismic event

Fig. 2.2 (Cont') – Seismic maps of Kentucky (Street et. al. 1996).

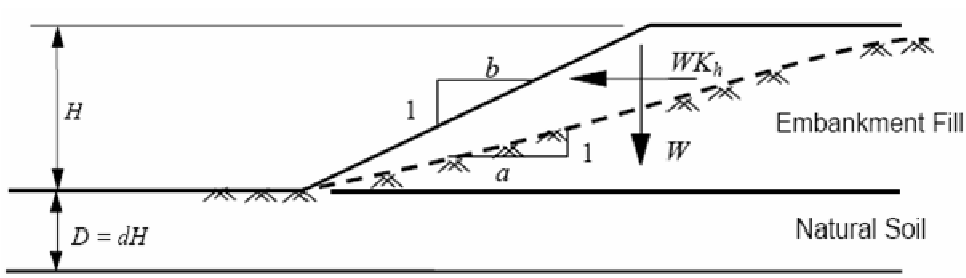
2.3 Slope (Embankment) Stability Analysis

A two-dimensional limit equilibrium slope stability analysis is employed herein to estimate the C/D ratio (Sutterer et al. 2000). Sutterer et al. (2000) summarized the stability analysis using numerical formulation of both critical circular and wedge–shaped failures (Fig. 2.3). Sutterer et al. (2000) reported that pseudo-static analysis of homogeneous slopes showed that seismically loaded embankments with uniform foundation soils, and slope inclinations flatter than 1 horizontal to 1 vertical and steeper than 4 horizontal to 1 vertical, most probably fail in a base failure mode. Steeper slopes may be subjected to a toe circle failure type in the embankment alone (Fig. 2.3). Accordingly, most highway bridge embankments fall within the range dominated by base failures. In assessing the seismic vulnerability of each embankment, both failure types are considered in the proposed methodology, and the one that results in a lower C/D ratio is considered. The importance of this study is in defining the process that is followed to assign the seismic risk, rank, and priority of the bridge embankments rather than providing the required derivations and equations.

The horizontal earthquake acceleration in the seismic slope stability analysis often ranges from 50% to 100% of the PGA assigned for the embankment site. The PGA is often a single spike of motion of a very brief duration and thus causes little if any significant displacement. Therefore, a reasonable value of the horizontal earthquake acceleration (K_h) equals to two-thirds of the PGA is selected in the seismic ranking methodology of the embankments. This assumption accounts for those embankments in which the seismic acceleration either never or briefly exceeds the yield acceleration, and results in little or no displacement.



(a) Circular base failure



(b) Wedge failure

Fig. 2.3 – Slope (embankment) failure modes (Sutterer et. al. 2002).

2.3.1 Capacity/Demand (C/D) Ratio of Slope (Embankment) Stability

The seismic slope stability C/D ratio of a bridge embankment is calculated for two possible failure types, known as circular base failure and wedge type failure. For a circular base failure that is shown in Fig. 2.3a, the factor of safety (FS_{cb}) is calculated from Eq. 2-1.

$$FS_{cb} = \left[\frac{R_1 - R_2}{D_1 + K_h \cdot D_2} \right] \cdot \frac{S_1}{\gamma_1 \cdot H} \tag{2-1}$$

where FS_{cb} is the factor of safety against circular base failure, S_1 is the un-drained shear strength of the soil beneath the embankment, H is the embankment height (Fig. 2.1), and γ_1 is the density of the soil layer (Table 2.1). The parameters R_1, R_2, D_1, and D_2 are obtained from Equations (2-2), (2-3), (2-4), and (2-5), respectively.

$$R_1 = 40 \cdot \sqrt{\frac{d}{r}} \cdot r \cdot (d + 12 \cdot r) \cdot (\lambda - 2) \tag{2-2}$$

$$R_2 = \sqrt{\frac{1+d}{r}} \cdot (9 \cdot (1+d)^2 + 40 \cdot (1+d) \cdot r + 480 \cdot r^2) \cdot \lambda \tag{2-3}$$

$$D_1 = 40 \cdot \sqrt{2} \cdot \left(1 + b^2 + 3d + 3d^2 - 3r - 6dr - 3bx + 3x^2\right) \tag{2-4}$$

$$D_2 = 40 \cdot \sqrt{2} \cdot (b + 3bd - 2 \cdot (-d \cdot (d - 2r))^{3/2} - 2 \cdot ((-1-d) \cdot (1 + d - 2r))^{3/2} - 3br - 3x - 6dx + 6rx) \tag{2-5}$$

where λ is the ratio of S_2/S_1, S_2 is the un-drained shear strength of the embankment soil and γ_2 is the density of the embankment soil (Table 2.1). For the values of x and r that result in the lowest factor of safety, designated x_c and r_c, the term in brackets of Eq. 2-1 has to be calculated and is called the stability number for the designated slope. The use of Eq. 2-1 in a spreadsheet with an optimization function provides reliable estimates of these parameters over the designated slope inclinations. Specifically, the "Solver®" function in "Microsoft Excel®" can be utilized to find r_c and x_c in order to minimize the factor of safety. Microsoft Excel Solver® uses the Generalized Reduced Gradient (*GRG2*) nonlinear optimization code (Lasdon et al., 1978, Waren et al., 1987, and Lasdon and Waren, 1989). By using pseudo-static analysis, assuming $FS_{cb} = 1.0$ in Eq. 2-1, and optimizing for r_c and x_c the horizontal earthquake acceleration factor (K_{hf}) shall be obtained for different assumed elevations of the upper level of the bedrock layer. The critical K_{hf} causing a circular base failure is obtained from Eq. 2-6.

$$K_{hf} = \frac{(R_1 - R_2)\cdot\dfrac{S_1}{\gamma_1 \cdot H} - D_1}{D_2} \tag{2-6}$$

Although a base failure predominates for the slope geometry typically encountered in highway embankments, a wedge failure extending upward from the toe of the embankment may be more critical for steeper slopes. The wedge type failure geometry is depicted in Fig. 2.3b. For a wedge type failure, the factor of safety (FS_w) is obtained from Eq. 2-7.

$$FS_w = \frac{2\cdot(1+a^2)}{(a-b)\cdot(1+a\cdot K_h)}\cdot\frac{S}{\gamma\cdot H} \tag{2-7}$$

Where FS_w is the factor of safety against embankment wedge failure, S is selected as the estimated shear strength along the base of the failure wedge and the parameter a, shown in Fig. 2.3b, is the parameter to be optimized and $\gamma = \gamma_1$, the horizontal earthquake acceleration factor (K_{hfw}) shall be obtained for different assumed elevations of the upper level of the bedrock layer by using pseudo-static analysis, assuming FS_w = 1.0 in Eq. 2-7, and optimizing for the parameter a. The critical K_{hfw} causing a wedge type failure of the embankment is obtained from Eq. 2-8.

$$K_{kfw} = \frac{1}{a}\cdot\left[\frac{2\cdot(1+a^2)}{(a-b)}\cdot\frac{S}{\gamma_1\cdot H} - 1\right] \tag{2-8}$$

The lesser factor of safety for a circular base failure (FS_{cb}) and for a wedge type failure (FS_w) is then called the capacity/demand (C/D) ratio for the designated elevation of the upper level of the bedrock layer. Similar processes are followed for other elevations of the upper level of the bedrock layer in order to obtain the overall least C/D ratio, which is called the minimum capacity/demand ratio, $(C/D)_{min}$. The considered horizontal earthquake acceleration (K_{hf}) is the one that corresponds to the $(C/D)_{min}$ from all the failure cases.

2.3.2 Embankment Displacements

For an embankment with C/D_{min}. < 1.0, it is important to estimate how far the mass actually displaces during the seismic event. This is carried out by calculating the anticipated embankment displacement (u). For a designated embankment, the PGA, also known as the maximum acceleration (A_{max}), for a specified seismic event is identified. For the embankment to displace, the maximum acceleration has to exceed the acceleration causing embankment yielding. Assuming that the yield acceleration is equal to the K_{hf}, that corresponds to the $(C/D)_{\min}$ from all the failure cases, the yield factor (Y) is estimated as the ratio of A_y/A_{max}, where A_y is the yield acceleration, and A_{max} = PGA. By utilizing the site geometry and the specified sub-surface conditions, it is possible to use a simple model to determine the approximate yield acceleration of a bridge embankment. A sliding block solution can then be applied to estimate the displacement of the slope for a specified PGA exceeding A_y. As the yield factor decreases, the displacements increase correspondingly. For a yield factor < 1.0, an embankment displacement is likely to occur. The displacement (u) can be estimated by the use of Eq. 2-9, (Ambraseys and Menu, 1988).

$$\log_{10}(u) = \alpha + \beta_1 \log_{10}\left(1 - \frac{A_y}{A_{\max}}\right) + \beta_2 \log_{10}\left(\frac{A_y}{A_{\max}}\right) \tag{2-9}$$

where u is the displacement, in centimeters. α, β_1, and β_2 are the bedrock coefficients that are required to calculate the embankment displacement. Dodds (1997) reported the way by which the bedrock coefficients are calculated for both the bedrock and soil sites based on the potential earthquake magnitude at the geographic location of the bridge site. The values of α for both the bedrock and the soil can be calculated by Eq. 2-10a and Eq. 2-10b. The values of β_1 can be calculated for both the bedrock and the soil by the use of Eq. 2-11a and Eq. 2-11b, while the values of β_2 can be calculated by the use of Eq. 2-12a and Eq. 2-12b.

$$(\alpha)_{bedrock} = 0.735 \cdot M_{b,Lg} - 4.41 \tag{2-10a}$$

$$(\alpha)_{soil} = 1.025 \cdot M_{b,Lg} - 6.292 \tag{2-10b}$$

$$(\beta_1)_{bedrock} = 0.35 \cdot M_{b,Lg} + 1.94 \tag{2-11a}$$

$$(\beta_1)_{soil} = 3.58 - 0.174 \cdot M_{b,Lg} \tag{2-11b}$$

$$(\beta_2)_{bedrock} = 0.21 - 0.15 \cdot M_{b,Lg} \tag{2-12a}$$

$$(\beta_2)_{soil} = -0.794 - 0.056 \cdot M_{b,Lg} \tag{2-12b}$$

Where $M_{b,Lg}$ is the body-wave magnitude of the anticipated earthquake. As the seismic slope stability of an embankment decreases, a larger displacement is expected, providing a stronger indication of an at-risk embankment than that is obtained from the $(C/D)_{\min}$ ratio. The analysis using this method eliminates the misleading condition of how to assess an embankment that has $(C/D)_{\min}$ ratio < 1.0. Instead, it forces a consideration of the possible displacement that may be observed, a better prediction of the actual behavior.

3. LIQUEFACTION POTENTIAL ASSESSMENT

3.1 General

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Increased pore-water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs, the soil stratum softens, allowing large cyclic deformations to occur. In loose materials, the softening is also accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense granular materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressures dissipate.

3.2 Methodology for Liquefaction Potential

Many procedures have been developed over the last forty years to evaluate the liquefaction potential. Of these procedures, the most popular one is provided by Seed and Idriss (1971). This method, known also as the simplified procedure, has been modified and refined since its first inception, through Seed (1979), and Seed and Idriss (1982). Soil types at the embankment or bridge sites can typically be obtained from soil boring logs, and subsequent information can then be used for liquefaction assessment.

3.2.1 Soil Boring Logs Are Not Available

Where the soil boring log data of each embankment site is not available, the liquefaction potential can be addressed based on the Seismic Retrofit Manual for Highway Bridges (Buckle and Friedland 1995). The susceptibility of the embankment soil to liquefaction is classified in one of three possible types (Table 3.1). The three liquefaction possibilities are: high, moderate, and low susceptibility. High susceptibility is associated with saturated loose sands, saturated silty sands, or non-plastic sands. A bridge that crosses a waterway where soils have been deposited over the years by flowing water is often constructed on loose saturated cohesion-less deposits that are most susceptible to liquefaction. Moderate susceptibility is associated with medium dense soils such as compacted sand soils. Low susceptibility is associated with dense soils.

Table 3.1: Liquefaction Susceptibility at a Bridge Embankment Site

Liquefaction Type	**Liquefaction Susceptibility**	**Parameters and Signs**
A	High	1) Associated with saturated loose sands, saturated silty sands, or non-plastic sands. 2) A bridge that crosses a waterway is often constructed on loose saturated cohesion-less deposits that are most susceptible to liquefaction.
B	Moderate	Associated with medium dense soils such as compacted sand soils.
C	Low	Associated with dense soils.

3.2.2 Soil Boring Logs Are Available

Where the soil boring log data is available, the liquefaction potential at the bridge site is accurately determined by the method reported by Seed et al. (1983). To determine a reasonably accurate value of the cyclic stress ratio causing liquefaction and induced by the earthquake motion, a correlation between the liquefaction characteristics and standard penetration test (SPT) blow-count values (*N* values), described by Seed et al (1985) is used. The average cyclic shear induced by the seismic event is obtained from Eq. 3.1.

$$\frac{\tau_{h,avg}}{\sigma_0^{'}} \cong 0.65\frac{A_{\max.}}{g}\cdot\frac{\sigma_0}{\sigma_0^{'}}\cdot r_d \tag{3.1}$$

where $\tau_{h,avg}$ is the average cyclic shear stress during the time history of interest, σ'_e is the effective overburden stress at any depth, A_{max} is the maximum earthquake ground surface acceleration, and r_d is a stress reduction correction factor. The mean effective and total stresses (σ'_e and σ'_o) are replaced with the effective and total vertical stresses. The stress reduction factor (r_d), defined by Seed et al (1985) is computed using the depth (z) in meters as shown in Eq. 3.2.

$$r_d = (1-\frac{z}{91}) \tag{3.2}$$

The soil penetration resistance is the corrected normalized standard penetration resistance, $N_{1,60}$, which is defined by Seed et al. (1985) and Seed and Harder (1990) in Eq. 3.3.

$$N_{1,60} = C_N \cdot \frac{ER_m}{60} \cdot N_m \tag{3.3}$$

Where C_N is the correction coefficient, ER_m is rod energy ratio, and N_m is the measured SPT blow-count per foot. With the determination of both the cyclic stress ratio induced during the earthquake and the cyclic stress ratio required to cause liquefaction, the factor of safety against liquefaction (FS_l) is calculated as shown in Eq.3.4.

$$FS_l = \frac{[\tau_{avg} / \sigma_0']_{l,M=M}}{[\tau_{h,avg} / \sigma_0']} \tag{3.4}$$

Where $[\tau_{avg} / \sigma_0']_{l,M=M}$ is the cyclic stress ratio required to cause liquefaction at any magnitude M, and $[\tau_{h,avg} / \sigma_0']$ is the cyclic stress ratio induced during an earthquake of the same magnitude. No liquefaction is predicted to occur for $FS_l > 1.0$.

3.3 Liquefaction Potential Index

The severity of liquefaction is quantified by Iwasaki et al (1982) the liquefaction potential index, P_L. The liquefaction potential index is defined as:

$$P_L = \int_0^{20} F(z) \cdot w(z) dz \tag{3.5}$$

where

$F(z)$ = 1-F_L for $F_L \leq 1.0$,
$F(z)$ = 0 for $F_L > 1.0$, and
$w(z)$ = 10-0.5z,

z is the depth in meters. The liquefiable overall soil depth is limited to 20 m. $w(z)$ is calculated for the critical soil layer across the profile. A summary of how the liquefaction potential is classified for an embankment, when *FS* is less than 1.0, is presented in Table 3.2.

Table 3.2: Liquefaction Potential classification

Liquefaction potential Index*	Classification
0 < LPI < 5	Low
5 ≤ LPI < 15	Moderate
15 ≤ LPI	High

* when *FS* (Section 3.2.2) is less than 1.0. Embankments with *FS* ≥ 1.0 are non-liquefiable.

4. EMBANKMENT RANKING

4.1 Ranking Parameters

Slope stability and liquefaction of the embankment constitute an important aspect of grade investigation. Therefore, it is only logical that the ranking of a given bridge embankment be a function of both. In this study, the ranking system is devised based on these two parameters. To be consistent with a prior KTC study, the *Kentucky Embankment Stability Ranking* (KESR) (Sutterer et al. 2000) defined the following three categories. A flow chart of such ranking system is presented in Fig. 4.1.

The KESR model assumes one of the following three possibilities (A, B, and C) of embankment behavior during a seismic event as described in Table 4.1: (A) loss of embankment, (B) significant movement, and (C) no significant movement. High seismic risk is assigned to category A. Significant seismic risk without loss of the embankment is assigned to category B, while low seismic risk is assigned to category C. The embankment displacement and the liquefaction potential are the ranking parameters for category A and category B. Conversely, the ranking of the embankments within category C is solely based on the anticipated $(C/D)_{min}$ ratio. For an embankment to be assigned category A, either the displacement shall exceed 10 centimeters (4 inches) or a high liquefaction potential is probable during the specified seismic event.

An embankment in category B meets one of the following two criteria: 1) moderate liquefaction potential; or 2) an anticipated $(C/D)_{min}$ ratio less than 1.0, along with a displacement of less than 10 centimeters (4 inches). An embankment in category C shall have $(C/D)_{min.}$ ratio greater than or equal to 1.0.

Table 4.1: Categories of Bridge Embankment Behavior during a Seismic Event

Category	Category Description	Ranking Parameter	Embankment Displacement	Seismic Risk
A	High liquefaction potential, or Displacement exceeds 10 centimeters	Displacement & liquefaction potential	Loss of embankment	High risk
B	Moderate liquefaction potential, or Capacity/Demand $(C/D)_{min}$ ratio is less than 1.0, and displacement is less than 10 centimeters	Displacement & liquefaction potential	Significant movement	Significant risk without loss of embankment
C	$(C/D)_{min}$ ratio is greater than or equal to 1.0	$(C/D)_{min}$ ratio	No significant movement	Low risk

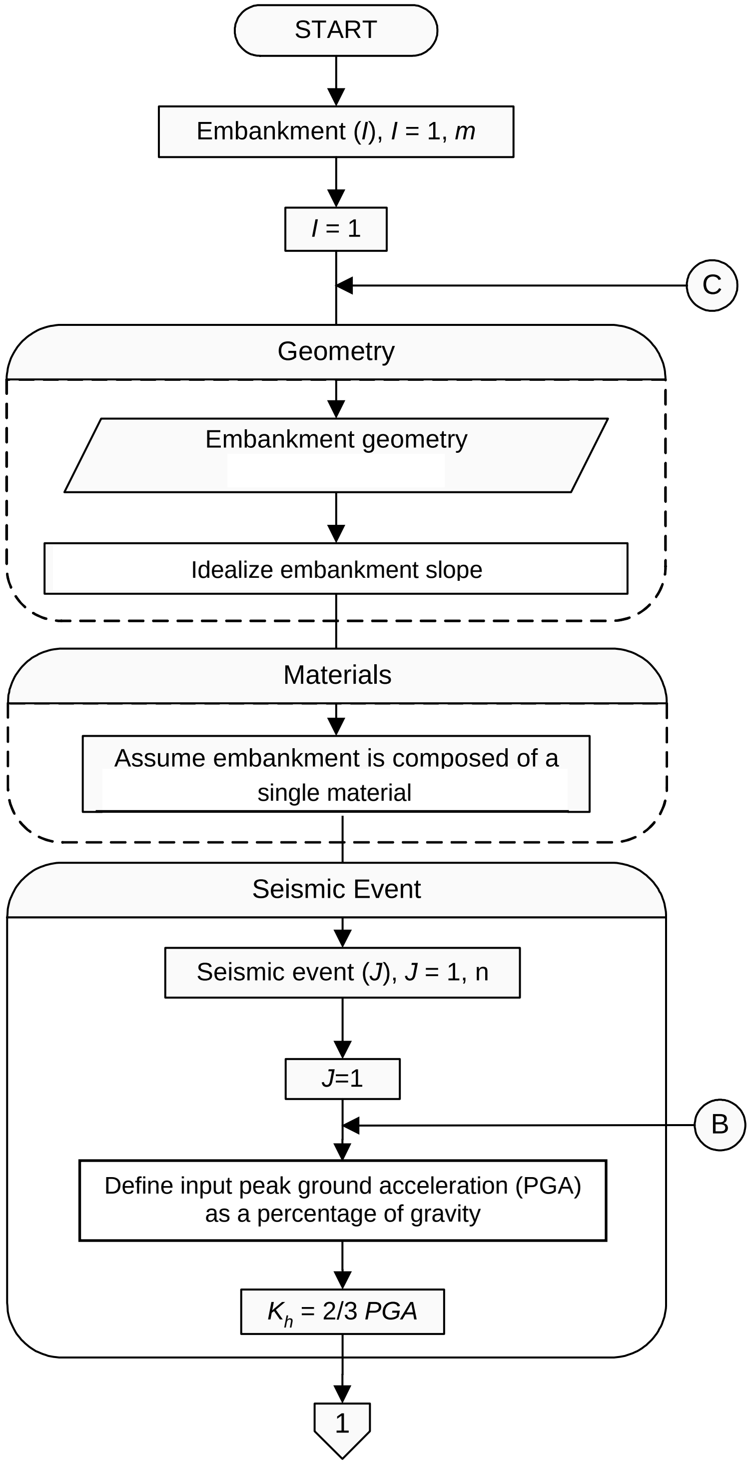


Fig. 4.1 – Flowchart for Seismic Risk Assessment and Ranking of Bridge Embankments

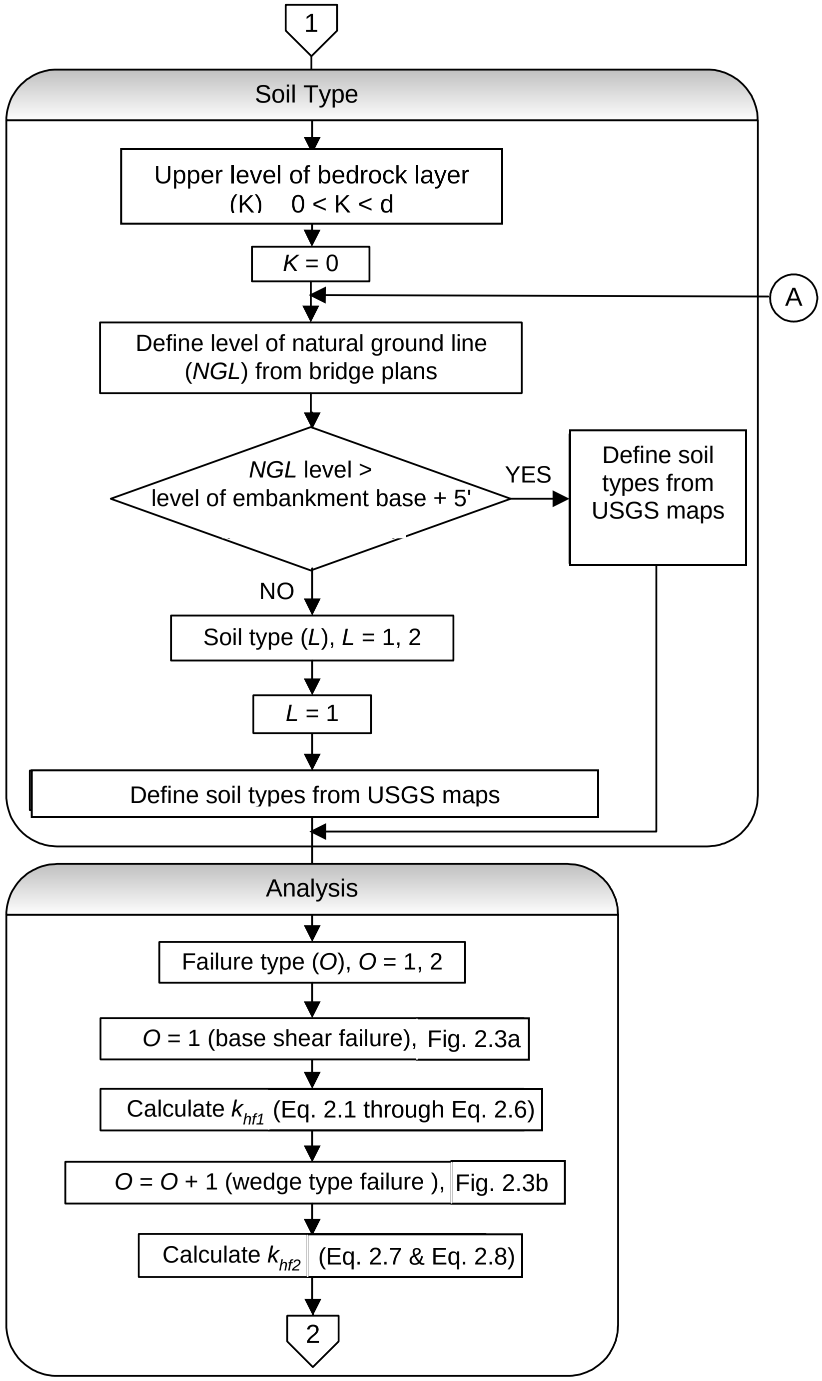


Fig. 4.1 (Cont') – Flowchart for Seismic Risk Assessment and Ranking of Bridge Embankments

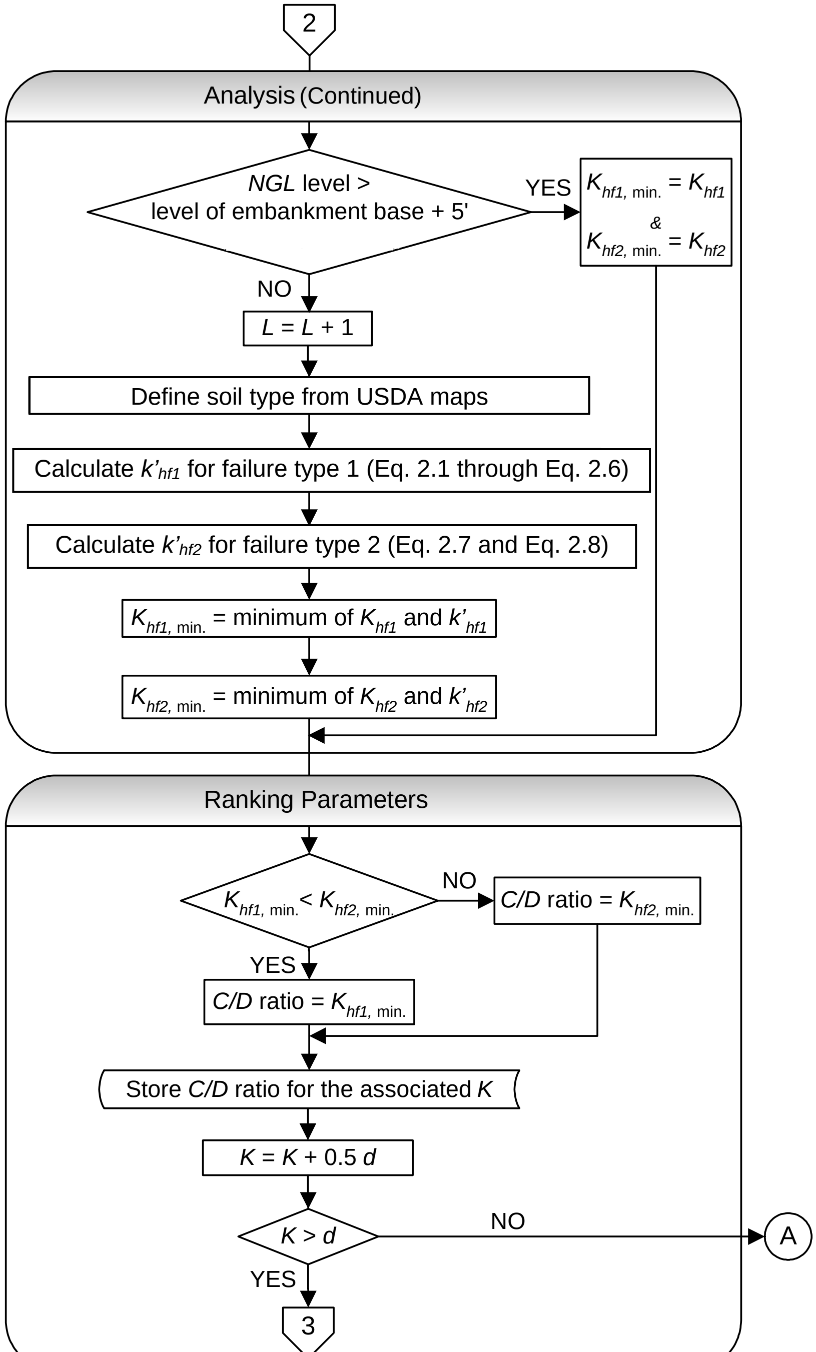


Fig. 4.1 (Cont') – Flowchart for Seismic Risk Assessment and Ranking of Bridge Embankments

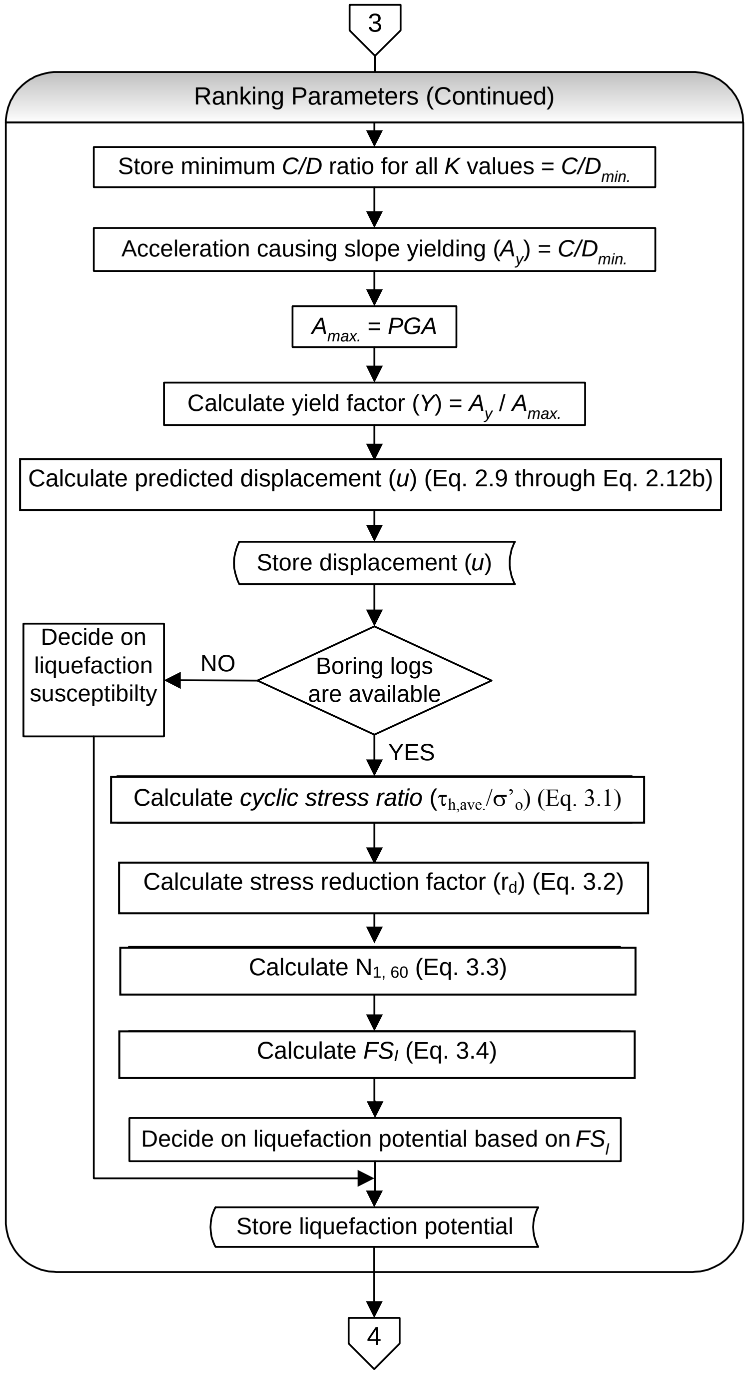


Fig. 4.1 (Cont') – Flowchart for Seismic Risk Assessment and Ranking of Bridge Embankments

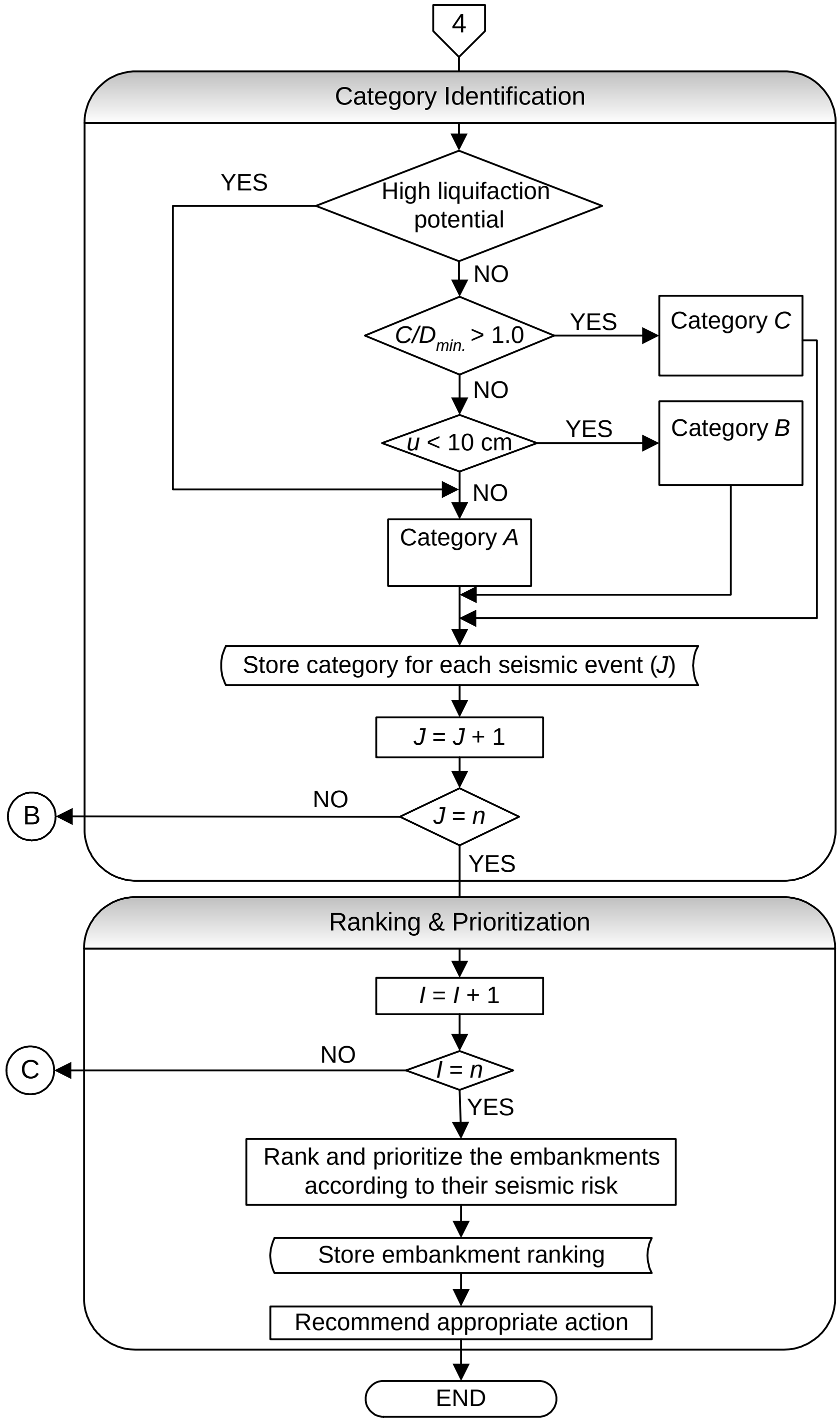


Fig. 4.1 (Cont') – Flowchart for Seismic Risk Assessment and Ranking of Bridge Embankments

4.2 Ranking and Prioritization

After classifying the bridge embankments into category A, category B, or category C in accordance with the criteria listed in Table 4.1, a prioritization within each category is carried out based on the significance of the three ranking parameters. For instance, the higher the displacement of an embankment in category A, the higher its seismic risk, and thus it is assigned a higher priority or ranking. The same applies for the prioritization of the embankments in category B. On the other hand, the lower the $(C/D)_{min}$ ratio of an embankment in category C, the higher its seismic risk, and thus it is assigned a higher priority or ranking.

Having completed the classification and categorization of all embankments in a certain region due to an anticipated seismic event, the prioritization of the embankments in each category becomes a feasible task. This proposed ranking model is useful, however, for a quick sensitivity assessment of the effect of various site conditions, earthquake magnitudes, and site geometry on possible movement of a designated embankment. Since the intent of the proposed ranking model is to compare the seismic risk of the embankments, regardless of having very accurate input data in the ranking model, it is the authors' recommendation to further conduct detailed assessments for the behavior of those at-risk embankments. In such detailed assessments, accurate data from sub-soil explorations are to be incorporated. Eventually, a priority list for the seismic risk of all the considered embankments can be prepared, which enables decision makers to take appropriate actions.

Embankments of the one-hundred and twenty seven (127) bridges along I-24 in Western Kentucky were rated based on the ranking system described, and the results are presented in Tables 4.2 and 4.3 for the 50-year and the 250-year seismic events, respectively.

Table 4.2: Ranking of Bridge Embankments along I-24 for the 50-Year Seismic Event
(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	*U*[6] in (cm)		
Christian	24-0024-B00125 & 24-0024-B00125P	9	0.81	13.5 (34.2)	High	A1
	24-0024-B00090 & 24-0024-B00090P	9	0.78	1.5 (3.7)	High	A2
	24-0024-B00132 & 24-0024-B00132P	9	0.65	0.8 (2.0)	High	A3
	24-0024-B00130 & 24-0024-B00130P	9	2.15	0.0 (0.0)	Low	C1
	24-0024-B00129 & 24-0024-B00129P	9	2.29	0.0 (0.0)	Low	C2
	24-0024-B00128	9	2.29	0.0 (0.0)	Low	C2
	24-0024-B00122 & 24-0024-B00122P	9	2.37	0.0 (0.0)	Low	C4
	24-0024-B00133	9	2.37	0.0 (0.0)	Low	C4
	24-0107-B00127	9	2.39	0.0 (0.0)	Low	C6
	24-0024-B00134	9	2.46	0.0 (0.0)	Low	C7
	24-0272-B00121	9	2.53	0.0 (0.0)	Low	C8
	24-0164-B00123	9	2.53	0.0 (0.0)	Low	C9
	24-0695-B00124	9	2.72	0.0 (0.0)	Low	C10
	24-0115-B00131	9	2.80	0.0 (0.0)	Low	C11
Lyon	72-0024-B00035 & 72-0024-B00035P	9	0.96	0.2 (0.4)	High	A1
	72-5229-B00034	9	0.99	0.1 (0.2)	High	A2
	72-0024-B00044 & 72-0024-B00044P	9	1.14	0.0 (0.0)	High	A3
	72-0024-B00048 & 72-0024-B00048P	9	1.19	0.0 (0.0)	High	A4
	72-0024-B00039 & 72-0024-B00039P	9	1.29	0.0 (0.0)	High	A5
	72-0024-B00041 & 72-0024-B00041P	9	2.21	0.0 (0.0)	Low	C1

1 As defined in the Kentucky Transportation Cabinet (KyTC) Bridge Inventory
2 The letter 'P' stands for parallel bridges.
3 PGA is the peak ground acceleration defined Street et al. (1996).
4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
6 Horizontal displacement (*u*) is calculated when C/D ratio is less than 1.0, or else *u* is equal zero.
7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.2 (Cont'): Ranking of Bridge Embankments along I-24 for the 50-Year Seismic Event
(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Lyon	72-5118-B00045	9	2.26	0.0 (0.0)	Low	C2
	72-0024-B00036 & 72-0024-B00036P	9	2.29	0.0 (0.0)	Low	C3
	72-0024-B00037 & 72-0024-B00037P	9	2.30	0.0 (0.0)	Low	C4
	72-5039-B00040	9	2.33	0.0 (0.0)	Low	C5
	72-5123-B00046 & 72-5123-B00046P	9	2.44	0.0 (0.0)	Low	C6
	72-0295-B00038	9	2.53	0.0 (0.0)	Low	C7
	72-9001-B00049 & 72-9001-B00049P	9	2.65	0.0 (0.0)	Low	C8
	72-0903-B00047	9	2.83	0.0 (0.0)	Low	C9
	72-0810-B00033	9	2.95	0.0 (0.0)	Low	C10
	72-5225-B00032	9	3.03	0.0 (0.0)	Low	C11
	72-0093-B00042	9	3.08	0.0 (0.0)	Low	C12
	72-0293-B00043	9	3.69	0.0 (0.0)	Low	C13
Trigg	111-0024-B00048 & 111-0024-B00048P	9	1.01	0.0 (0.0)	High	A1
	111-6051-B00049	9	2.16	0.0 (0.0)	Low	C1
	111-0024-B00027 & 111-0024-B00027P	9	2.35	0.0 (0.0)	Low	C2
	111-0024-B00050	9	2.39	0.0 (0.0)	Low	C3
	111-0024-B00043	9	2.48	0.0 (0.0)	Low	C4

1 As defined in the Kentucky Transportation Cabinet (KyTC) Bridge Inventory
2 The letter 'P' stands for parallel bridges.
3 PGA is the peak ground acceleration defined Street et al. (1996).
4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.2 (Cont'): Ranking of Bridge Embankments along I-24 for the 50-Year Seismic Event

(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	*U*[6] in (cm)		
Trigg	111-0024-B00044 & 111-0024-B00044P	9	2.53	0.0 (0.0)	Low	C5
Trigg	111-6049-B00047	9	2.70	0.0 (0.0)	Low	C6
Marshall	79-0024-B00117 & 79-0024-B00117P	15	0.77	35.4 (89.8)	High	A1
Marshall	79-0024-B00116 & 79-0024-B00116P	15	0.69	2.3 (5.8)	High	A2
Marshall	79-0024-B00113 & 79-0024-B00113P	15	0.83	0.8 (2.1)	High	A3
Marshall	79-0024-B00115 & 79-0024-B00115P	15	0.83	0.8 (2.1)	High	A4
Marshall	79-0095-B00112	15	0.87	0.4 (1.1)	High	A5
Marshall	79-0024-B00118 & 79-0024-B00118P	15	0.54	0.2 (0.4)	High	A6
Marshall	79-0024-B00114 & 79-0024-B00114P	15	0.96	0.1 (0.3)	High	A7
Marshall	79-1610-B00092	15	2.00	0.0 (0.0)	Moderate	B1
Marshall	79-0024-B00109	15	2.22	0.0 (0.0)	Moderate	B2
Marshall	79-0024-B00111	15	2.29	0.0 (0.0)	Moderate	B3
Marshall	79-0024-B00081 & 79-0024-B00081P	15	3.31	0.0 (0.0)	Moderate	B4
Marshall	79-0024-B00082 & 79-0024-B00082P	15	**24'x9'x75' RC Box Culvert and is excluded from this study**			
Marshall	79-0024-B00136	15	**DBL 12'x4'x203' RC Box Culvert and is excluded from this study**			

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5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.2 (Cont'): Ranking of Bridge Embankments along I-24 for the 50-Year Seismic Event

(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4] C/D[5] ratio	Slope Stability[4] U^6 in (cm)	Liquefaction Potential[7]	Embankment Ranking[8]
Livingston	70-0024-B00063 & 70-0024-B00063P	15	0.60	2.0 (5.1)	High	A1
	70-0024-B00062 & 70-0024-B00062P	15	0.85	0.6 (1.5)	High	A2
	70-0453-B00064 & 70-0453-B00064P	15	1.89	0.0 (0.0)	Moderate	B1
	70-0024-B00061	15	**24'x9'x75' RC Box Culvert and is excluded from this study.**			
Caldwell	17-0276-B00066 & 17-0276-B00066P	9	2.44	0.0 (0.0)	Low	C1
	17-0139-B00065	9	2.57	0.0 (0.0)	Low	C2
McCracken	73-0024-B00104 & 73-0024-B00104P	15	0.79	5.6 (14.3)	High	A1
	73-0024-B00103 & 73-0024-B00103P	15	0.81	2.7 (6.9)	High	A2
	73-0068-B00060 & 73-0068-B00060P	15	0.83	1.7 (4.4)	High	A3
	73-0787-B00064	15	0.83	1.7 (4.3)	High	A4
	73-0024-B00107 & 73-0024-B00107P	15	0.83	1.0 (2.4)	High	A5
	73-0024-B00105 & 73-0024-B00105P	15	0.86	0.9 (2.2)	High	A6
	73-0024-B00112 & 73-0024-B00112P	15	0.86	0.5 (1.3)	High	A7
	73-0024-B00102 & 73-0024-B00102P	15	0.90	0.4 (1.0)	High	A8
	73-0131-B00009	15	0.90	0.3 (0.8)	High	A9
	73-0024-B00111 & 73-0024-B00111P	15	0.92	0.3 (0.7)	High	A10
	73-0024-B00100	**A Bridge over the Ohio River and is excluded from this study.**				

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5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.2 (Cont'): Ranking of Bridge Embankments along I-24 for the 50-Year Seismic Event

(The 50-year event is a seismic event that has a 90% probability of not being exceeded in 50 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	*U*[6] in (cm)		
McCracken	73-0024-B00120 & 73-0024-B00120P	15	0.71	1.9 (4.8)	Moderate	B1
	73-0024-B00118 & 73-0024-B00118P	15	0.83	1.8 (4.6)	Moderate	B2
	73-0024-B00115 & 73-0024-B00115P	15	0.85	1.0 (2.7)	Moderate	B3
	73-0024-B00119 & 73-0024-B00119P	15	0.88	0.5 (1.4)	Moderate	B4
	73-0024-B00116 & 73-0024-B00116P	15	0.88	0.5 (1.4)	Moderate	B4
	73-0024-B00114 & 73-0024-B00114P	15	0.92	0.2 (0.6)	Moderate	B6
	73-0024-B00101 & 73-0024-B00101P	15	0.98	0.1 (0.2)	Moderate	B7
	73-0994-B00122	15	2.00	0.0 (0.0)	Moderate	B8
	73-0062-B00121	15	2.14	0.0 (0.0)	Moderate	B9
	73-3075-B00065	15	2.17	0.0 (0.0)	Moderate	B10
	73-0024-B00113	15	2.50	0.0 (0.0)	Moderate	B11
	73-0024-B00117	15	**DBL 14'x6'x230' RC Box Culvert and is excluded from this study.**			

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4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.3: Ranking of Bridge Embankments along I-24 for the 250-Year Seismic Event
(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Christian	24-0024-B00125 & 24-0024-B00125P	9	0.81	54.2 (137.7)	High	A1
	24-0024-B00090 & 24-0024-B00090P	9	0.78	5.7 (14.5)	High	A2
	24-0024-B00132 & 24-0024-B00132P	9	0.65	3.1 (7.8)	High	A3
	24-0024-B00130 & 24-0024-B00130P	9	2.15	0.0 (0.0)	Low	C1
	24-0024-B00129 & 24-0024-B00129P	9	2.29	0.0 (0.0)	Low	C2
	24-0024-B00128	9	2.29	0.0 (0.0)	Low	C2
	24-0024-B00122 & 24-0024-B00122P	9	2.37	0.0 (0.0)	Low	C4
	24-0024-B00133	9	2.37	0.0 (0.0)	Low	C4
	24-0107-B00127	9	2.39	0.0 (0.0)	Low	C6
	24-0024-B00134	9	2.46	0.0 (0.0)	Low	C7
	24-0272-B00121	9	2.53	0.0 (0.0)	Low	C8
	24-0164-B00123	9	2.53	0.0 (0.0)	Low	C9
	24-0695-B00124	9	2.72	0.0 (0.0)	Low	C10
	24-0115-B00131	9	2.80	0.0 (0.0)	Low	C11
Lyon	72-0024-B00035 & 72-0024-B00035P	15	0.83	3.2 (8.1)	High	A1
	72-5229-B00034	15	0.86	2.1 (5.4)	High	A2
	72-0024-B00044 & 72-0024-B00044P	15	0.96	0.4 (1.1)	High	A3
	72-0024-B00048 & 72-0024-B00048P	15	0.99	0.3 (0.8)	High	A4
	72-0024-B00039 & 72-0024-B00039P	15	1.05	0.0 (0.0)	High	A5
	72-0024-B00041 & 72-0024-B00041P	15	1.87	0.0 (0.0)	Low	C1

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5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.3 (Cont'): Table 4.3: Ranking of Bridge Embankments along I-24 for the 250-Year Seismic Event

(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Lyon	72-5118-B00045	15	1.91	0.0 (0.0)	Low	C2
	72-0024-B00036 & 72-0024-B00036P	15	1.94	0.0 (0.0)	Low	C3
	72-0024-B00037 & 72-0024-B00037P	15	1.94	0.0 (0.0)	Low	C4
	72-5039-B00040	15	1.97	0.0 (0.0)	Low	C5
	72-5123-B00046 & 72-5123-B00046P	15	2.06	0.0 (0.0)	Low	C6
	72-0295-B00038	15	2.13	0.0 (0.0)	Low	C7
	72-9001-B00049 & 72-9001-B00049P	15	2.21	0.0 (0.0)	Low	C8
	72-0903-B00047	15	2.38	0.0 (0.0)	Low	C9
	72-0810-B00033	15	2.47	0.0 (0.0)	Low	C10
	72-5225-B00032	15	2.54	0.0 (0.0)	Low	C11
	72-0093-B00042	15	2.58	0.0 (0.0)	Low	C12
	72-0293-B00043	15	3.06	0.0 (0.0)	Low	C13
Trigg	111-0024-B00048 & 111-0024-B00048P	9	1.01	0.0 (0.0)	High	A1
	111-6051-B00049	9	2.35	0.0 (0.0)	High	A2
	111-0024-B00027 & 111-0024-B00027P	9	2.16	0.0 (0.0)	Low	C1
	111-0024-B00050	9	2.39	0.0 (0.0)	Low	C2
	111-0024-B00043	9	2.48	0.0 (0.0)	Low	C3

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Table 4.3 (Cont'): Table 4.3: Ranking of Bridge Embankments along I-24 for the 250-Year Seismic Event

(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
Trigg	111-0024-B00044 & 111-0024-B00044P	9	2.53	0.0 (0.0)	Low	C4
	111-6049-B00047	9	2.70	0.0 (0.0)	Low	C5
Marshall	79-0024-B00117 & 79-0024-B00117P	15	0.77	145.3 (369.1)	High	A1
	79-0024-B00116 & 79-0024-B00116P	15	0.69	8.9 (22.7)	High	A2
	79-0024-B00113 & 79-0024-B00113P	15	0.83	3.2 (8.1)	High	A3
	79-0024-B00115 & 79-0024-B00115P	15	0.83	3.2 (8.1)	High	A4
	79-0095-B00112	15	0.87	1.7 (4.3)	High	A5
	79-0024-B00118 & 79-0024-B00118P	15	0.54	0.7 (1.7)	High	A6
	79-0024-B00114 & 79-0024-B00114P	15	0.96	0.4 (1.1)	High	A7
	79-0024-B00109	15	2.22	0.0 (0.0)	High	A8
	79-1610-B00092	15	2.00	0.0 (0.0)	Moderate	B1
	79-0024-B00111	15	2.29	0.0 (0.0)	Moderate	B2
	79-0024-B00081 & 79-0024-B00081P	15	3.31	0.0 (0.0)	Moderate	B3
	79-0024-B00082 & 79-0024-B00082P	15	**24'x9'x75' RC Box Culvert and is excluded from this study.**			
	79-0024-B00136	15	**DBL 12'x4'x203' RC Box Culvert and is excluded from this study.**			

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2 The letter 'P' stands for parallel bridges.
3 PGA is the peak ground acceleration defined Street et al. (1996).
4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
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7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.3 (Cont'): Table 4.3: Ranking of Bridge Embankments along I-24 for the 250-Year Seismic Event

(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U[6] in (cm)		
Livingston	70-0024-B00063 & 70-0024-B00063P	15	0.60	7.8 (19.9)	High	A1
	70-0024-B00062 & 70-0024-B00062P	15	0.85	2.3 (5.9)	High	A2
	70-0453-B00064 & 70-0453-B00064P	15	1.89	0.0 (0.0)	Moderate	B1
	70-0024-B00061	15	**24'x9'x75' RC Box Culvert and is excluded from this study.**			
Caldwell	17-0276-B00066 & 17-0276-B00066P	9	2.44	0.0 (0.0)	Low	C1
	17-0139-B00065	9	2.57	0.0 (0.0)	Low	C2
McCracken	73-0024-B00104 & 73-0024-B00104P	19	0.75	31.4 (79.8)	High	A1
	73-0024-B00103 & 73-0024-B00103P	19	0.76	15.6 (39.5)	High	A2
	73-0024-B00120 & 73-0024-B00120P	19	0.67	11.3 (28.7)	High	A3
	73-0024-B00118 & 73-0024-B00118P	19	0.77	10.7 (27.3)	High	A4
	73-0068-B00060 & 73-0068-B00060P	19	0.77	10.4 (26.3)	High	A5
	73-0787-B00064	19	0.78	10.1 (25.8)	High	A6
	73-0024-B00115 & 73-0024-B00115P	19	0.79	6.6 (16.8)	High	A7
	73-0024-B00107 & 73-0024-B00107P	19	0.76	6.1 (15.5)	High	A8
	73-0024-B00105 & 73-0024-B00105P	19	0.80	5.7 (14.5)	High	A9

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4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
6 Horizontal displacement (u) is calculated when C/D ratio is less than 1.0, or else u is equal zero.
7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

Table 4.3 (Cont'): Table 4.3: Ranking of Bridge Embankments along I-24 for the 250-Year Seismic Event

(The 250-year event is a seismic event that has a 90% probability of not being exceeded in 250 years)

County	BIN[1,2]	PGA[3] (%g)	Slope Stability[4]		Liquefaction Potential[7]	Embankment Ranking[8]
			C/D[5] ratio	U^6 in (cm)		
McCracken	73-0024-B00112 & 73-0024-B00112P	19	0.79	3.5 (8.9)	High	A10
	73-0024-B00102 & 73-0024-B00102P	19	0.83	2.9 (7.3)	High	A11
	73-0131-B00009	19	0.84	2.5 (6.4)	High	A12
	73-0024-B00111 & 73-0024-B00111P	19	0.85	2.2 (5.5)	High	A13
	73-0024-B00100	**A Bridge over the Ohio River and is excluded from this study.**				
	73-0024-B00119 & 73-0024-B00119P	19	0.82	3.7 (9.4)	Moderate	B1
	73-0024-B00116 & 73-0024-B00116P	19	0.82	3.7 (9.4)	Moderate	B1
	73-0024-B00114 & 73-0024-B00114P	19	0.85	2.0 (5.1)	Moderate	B3
	73-0024-B00101 & 73-0024-B00101P	19	0.90	0.9 (2.4)	Moderate	B4
	73-0994-B00122	19	1.81	0.0 (0.0)	Moderate	B5
	73-0062-B00121	19	1.93	0.0 (0.0)	Moderate	B6
	73-3075-B00065	19	1.96	0.0 (0.0)	Moderate	B7
	73-0024-B00113	19	2.24	0.0 (0.0)	Moderate	B8
	73-0024-B00117	19	**DBL 14'x6'x230' RC Box Culvert and is excluded from this study.**			

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2 The letter 'P' stands for parallel bridges.
3 PGA is the peak ground acceleration defined Street et al. (1996).
4 Details for slope stability calculations are presented in Chapter 2.
5 Capacity/demand ratio is defined in Chapter 2.
6 Horizontal displacement (*u*) is calculated when C/D ratio is less than 1.0, or else *u* is equal zero.
7 Details for liquefaction potential calculations are presented in Chapter 3.
8 Only bridge embankments with a rank classification of A (critical) are listed herein. A bridge embankment with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 in that specific county, and so forth.

5. SUMMARY, CONCLUSION, AND RECOMMENDATION

The seismic evaluation of bridge stability is an important aspect of structural/earthquake engineering practice. To date, several codified specifications dealing with seismic design of bridge structures exist; most notably the seismic provisions by the American Association of State Highway and Transportation Officials (AASHTO 2002 and 2004). In 1995, the Federal Highway Administration (FHWA) published a guide titled *Seismic Retrofitting Manual for Highway Bridges* (Publication No. FHWA-RD-94-052) – known hereafter simply as the Manual. The Manual provided for bridge owners nationwide a roadmap for the evaluation and retrofit of bridges in seismic zones. The Manual discusses in details the following aspects: (1) a ranking procedure for a preliminary seismic evaluation of highway bridges; (2) analytical techniques for detailed seismic evaluation, when such a need arises; and (3) retrofit guidance for certain seismically deficient bridge components. Much of the evaluation effort concentrated on the stability and strength of a bridge's superstructure and substructure.

The objective of this report is to provide a methodology for and to conduct a preliminary seismic evaluation and ranking of embankments for bridges on and over I-24. The methodology focuses on the slope or embankment stability assessment and the liquefaction potential.

Methodologies assessing the stability of bridge embankments and the potential of soil liquefaction are presented in this report. The methodologies focus on the following aspects: (1) the slope stability capacity/demand (C/D) ratio; embankment horizontal displacement (u); and (3) liquefaction potential of foundation soil underneath a bridge embankment. Detailed discussions of these different aspects are presented in this report.

In order to facilitate the identification of critical embankments for bridges on and over I-24, a ranking system based on slope stability, liquefaction potential, and/or a combination of the two, has been established and is presented in this report. This ranking will assist in prioritizing bridge embankments that are in need of highest attention or in demand of other course of action. Tables E1 and E2 list the bridge embankments that are considered 'critical' (designated as Class A) based on 50-year and 250-year event earthquakes. Fifty two (52) of the one hundred and twenty seven (127) embankments were rated as 'critical' for the 50-year event, and 60 were rated as 'critical' for the 250-year event.

A step-by-step procedure is presented for the ranking of bridge embankments on and over I-24. The ranking assists in identifying and prioritizing bridge embankments that are susceptible to failure due to projected seismic events.

Based on this preliminary evaluation, it is recommended that bridge embankments classified as 'critical' (Tables E1 and E2) be further investigated by performing more detailed analysis.

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