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Chapter 8

Bridge Embankments – Seismic Risk Assessment and Ranking

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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 [1]. 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. The Seismic Retrofit Manual for Highway Bridges [3] stipulates techniques for assessing the seismic vulnerability of bridges with regard to technical and socio-economic issues. The seismic retrofit manual stipulates that for bridges near unstable slopes, detailed geotechnical investigations should be carried out to assess the potential for slope instability under seismic excitations. The required detailed investigations include material testing, borehole examination, and trenching to check for unstable layers and vertical fissures. However, for the preliminary evaluation of bridges on priority routes the use of detailed geo-technical investigations and sophisticated models are typically limited because of the associated cost and effort.

There is current interest in a careful assessment of the "most critical" embankments along priority routes. In order to achieve this goal, a means of assessing the embankments that qualify as "most critical" is required. Other than the work reported by the authors, almost no complete studies have been reported to identify and prioritize highway embankments that are susceptible to seismic failure. Data regarding soil types and depth of bedrock required for detailed seismic analysis and risk assessment are not available for the majority of bridge embankments. For instance, while the total number of bridges located on both I-24 and the Parkways in western Kentucky is 519 bridges, soil data is only available for few bridge sites. Therefore, the objective of this study is to provide a methodology to conduct seismic evaluations of bridge embankments in order to identify, rank, and prioritize the embankments that are susceptible to seismic failure and are in need of detailed analysis.

This Chapter addresses the technical component of embankment prioritization and is well-suited to a reliability-based model for seismic risk assessment.

In order to achieve the objective of this study, a flowchart is generated to assess the seismic vulnerability of multiple bridge embankments simultaneously. The embankment geometry, material, type of underlying soil, elevation of natural ground line, upper level of bedrock, and expected seismic event in accordance with associated seismic zone maps constitute the variables for each embankment. This methodology results in calculating the seismic slope stability capacity/demand (*C/D*) ratio, estimated displacement, and liquefaction potential of each bridge embankment for the respective expected seismic event. Seismic vulnerability ranking and prioritization of embankments are conducted by using the "Kentucky Embankment Stability Rating" (*KESR*) model. Three categories are identified in the KESR model to represent the failure risk of the embankments. A priority list of the embankments with the highest seismic risk can be generated for any set of embankments.

2. Seismic vulnerability and ranking of bridge embankments

In general, data regarding soil types and depth of bedrock are not available for many existing bridge embankments to allow for detailed seismic analysis and risk assessment. This Chapter provides a methodology that enables identifying the embankments that are susceptible to failure during a seismic event. Having categorized the embankments in a designated region according to the respective failure risk, a priority list that includes the most critical bridge embankments can then be highlighted. When site-specific data for a bridge embankment is available, it can be used to obtain the list of seismically deficient embankments. When site-specific data for a bridge embankment is not available, the proposed methodology outlines an approach to estimate the information that is required to obtain the priority list. It is understood that the resulting seismic risk of a specific embankment may not be very accurate due to limited available data or lack thereof. However, the estimated data and strength parameters that are available for utilization shall be assessed by a qualified geo-technical engineer in order to ensure valid results. In order to facilitate the application of the proposed ranking methodology, assumptions, calculations, and required checks are presented along with the parameters of each embankment. The parameters of each embankment include the respective geometry, material, seismic event, upper level of bedrock, level of natural ground line, soil type, and anticipated failure types. The following sections: input variables, embankment vulnerability analysis, ranking parameters, category identification, and ranking and prioritization are provided to outline all of the necessary steps to achieve the study objective.

3. Input variables

The geometry, material, level of natural ground line, soil type, seismic event, and upper level of bedrock constitute the required input for each embankment and are addressed in the following sub-sections.

Geometry: The ideal case for obtaining the geometry of a given embankment is to carry out an on-site inspection. Should there be difficulties encountered in gathering such on-site information, however, the embankment geometry may be taken from the bridge plans. It is assumed that utilizing data from a finalized set of bridge plans will not affect the accuracy of the final seismic ranking and priority list for a given embankment case. Embankment slopes are assumed to be free of 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 might have been used for erosion protection of the slope will not greatly influence 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 the idealized inclination (*b*) (Figure 1). The water table is assumed to be located below the embankment base in order to obtain the most critical seismic stability conditions. Analysis shall be 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, shall be considered in the ranking analysis and priority list.

Materials, Natural Ground Line and Soil Properties: 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 existing bridge embankment sites. Therefore, another approach is employed herein to specify the soil types and properties of applicable sites. It is assumed that any soil outside the embankment zone at a bridge site has uniform un-drained shear strength. The soil is considered to be in continuous contact with the bedrock layer, where the bedrock acts as a layer possessing high strength at some depth below the embankment.

Soil data is dependent on the level of the Natural Ground Line (*NGL*), shown in Figure 1. Both the "*Geologic Quadrant Maps of the United States"* that are provided in "*United States Geologic Survey* (*USGS*)" maps [2] and the "*Soil Conservation Service, Soil Survey"* maps that are reported by "*United States Department of Agriculture* (*USDA*)" [3] are used 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 is solely based on the soil data obtained from the "*Geologic Quadrangle Maps of the United States"*, provided by *USGS* [2]. Otherwise, the soil data is derived from the "*Soil Conservation Service, Soil Survey"*, provided by *USDA* [3].

The dependency on the *USDA* maps in this case can be attributed to the fact that the top 1.5 m. (5 ft) soil can be accurately obtained from these maps. Shear strengths are assigned as done so by [4] for non-cohesive soil materials, which were derived from analysis of standard penetration tests (Table 1). When a range of values is given for the shear strength of a given soil, the lowest value is assigned to accommodate for the anticipated liquefaction potential at many bridge sites [5]. The shear strength assigned for cohesive soils in Table 1 is chosen after examining commensurately accurate un-confined compression data. Shear strengths assigned to the embankment fill are adjusted to reflect the cyclic loading effects between un-drained failure for both cohesive and saturated cohesion-less soils, in addition to 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 marginal compaction may have occurred during construction. Should there be more accurate soil properties, they may replace those provided in Table 1.

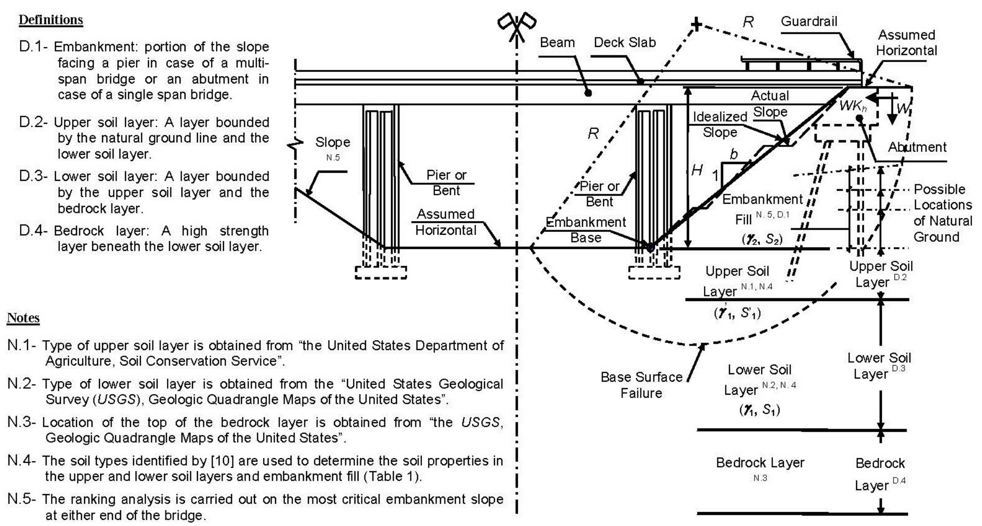


Figure 1. Bridge embankment representation for seismic ranking

Upper Level of Bedrock Layer: Data regarding the level under which a hard stratum, stiff bedrock layer, exists is not available for the majority of existing embankment sites; especially for small bridges. An initial assumption of the upper level of this hard stratum is estimated from the "*Geologic Quadrant Maps of the United States*" [2]. 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 seismic risk assessment of a bridge embankment, few 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 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 those assumptions yield a lower seismic slope stability *C/D* ratio. The top level of the bedrock layer, adopted in the ranking analysis, is the assumed elevation that results in the lowest seismic slope stability *C/D* ratio.

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

Table 1. Density and strength of soils and embankments

Seismic Event: The input Peak Ground Acceleration (*PGA*), which is the maximum bedrock acceleration at a designated embankment site, is obtained from 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 maps. The seismic maps to define the acceleration coefficient based on a uniform risk method of seismic hazard can be used. 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 is of 475 years [6]. Alternatively, seismic maps that may have been generated by State Departments of Transportation can be used. For the Commonwealth of Kentucky 50-year, 250-year, and 500-year seismic events were developed [7]. These events have a 90% probability of not being exceeded in 50 years, 250 years, and 500 years, respectively. All but four of the bridges and their embankments on priority routes in western Kentucky are required to withstand the 50-year and 250-year seismic events. The four other bridges are required to resist the 500-year seismic event.

4. Embankment vulnerability analysis

The potential for slope displacement to occur during an earthquake is assessed using a two-dimensional limit equilibrium stability analysis. Sutterer et al. [8] summarized the stability analysis using numerical formulation of both critical circular and wedge–shaped failures (Figure 2). Sutterer et al. [4] 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 (Figure 2). Regardless, 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. This Chapter defines a process to assign the seismic risk, rank and priority of a set of bridge embankments rather than providing only 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 causes little if any significant displacement. A horizontal earthquake acceleration (K_h) equals to two-thirds of the *PGA* is selected in the proposed methodology. This assumption accounts for those embankments in which the seismic acceleration either never exceeds the yield acceleration or very briefly exceeds the yield acceleration, and results in little or no displacement.

5. Ranking parameters

The embankment ranking and prioritization procedures in the seismic vulnerability methodology are based on three parameters that have to be derived for each embankment. They are the seismic slope stability *C/D* ratio, anticipated embankment displacement, and liquefaction potential at the embankment site. The way by which each parameter is calculated is described in the following sub-sections. After calculating the three ranking parameters, categorization of the embankment behavior during a specified seismic event is carried out using Table 2.

Capacity/Demand (C/D) Ratio: 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 Figure 2a, the factor of safety (FS_{cb}) is calculated from Eq. 1.

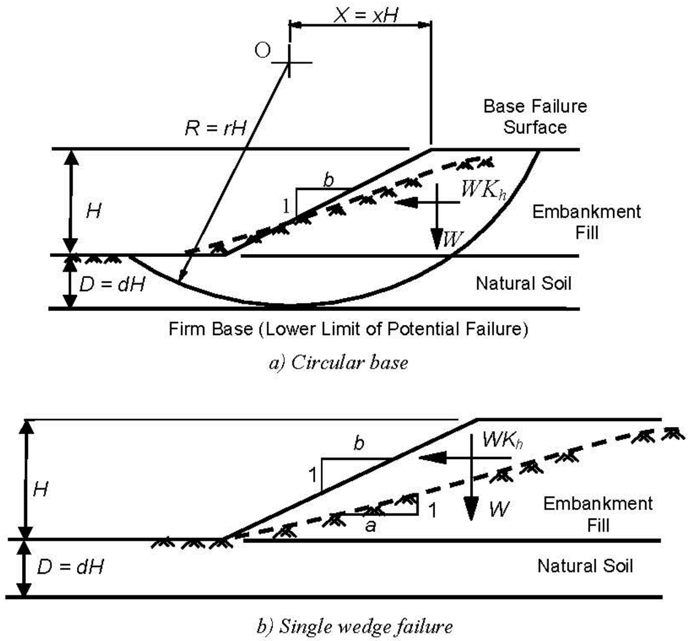


Figure 2. Failure types of bridge embankments *(a) circular base failure and (b) single wedge failure*

Category	Category Description	Ranking Parameter	Embankment Displacement	Seismic Risk
A	1) High liquefaction potential, or 2) Displacement exceeds 10 centimeters	Displacement & liquefaction potential	Loss of embankment	High risk
B	1) Moderate liquefaction potential, or 2) 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

Table 2. Categories of bridge embankment behavior during a seismic event

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

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 (Figure 1), and γ_1 is the density of the soil layer (Table 1). The parameters R_1, R_2, D_1, and D_2 are obtained from Equations (2), (3), (4), and (5), respectively.

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

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

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

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

where λ is the ratio of S_2/S_1, S_2 is the embankment soil un-drained shear strength and γ_2 is the embankment soil density (Table 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. 1 has to be calculated and is called the stability number for the designated slope. The use of Eq. 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 XP®" can be utilized to find r_c and x_c in order to minimize the factor of safety. By using pseudo-static analysis, assuming FS_{cb} = 1.0 in Eq. 1, and optimizing for r_c and x_c the horizontal earthquake acceleration factor (k_{hf}) is 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. 6.

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

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 Figure 2b. For a wedge type failure, the factor of safety (FS_w) is obtained from Eq. 7.

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

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 Figure 2b, is the parameter to be optimized. 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. 7, and optimizing by changing the parameter a. The critical K_{hfw} causing a wedge type failure of the embankment is obtained from Eq. 8.

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

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 of the failure cases.

Embankment Displacement: 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 is identified for a specified seismic event; this parameter is also known as the maximum acceleration ($A_{max.}$). 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

$A_{max.}$ equals to the *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 Y decreases, u increases correspondingly. For Y<1.0, embankment displacement is likely to occur. The displacement (u) can be estimated by the use of Eq. 9 [6].

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

where u is the displacement, in centimeters; α, β_1, β_2 are the bedrock coefficients that are required to calculate the embankment displacement. Dodds [8] reported the way by which the bedrock coefficients are calculated for both bedrock and soil sites based on the potential earthquake magnitude at the geographic location of the bridge site. The value of α for both bedrock and soil can be obtained by use of Eq. 10a and Eq. 10b. The parameter, β_1, can be calculated for both the bedrock and soil by the use of Eq. 11a and Eq. 11b, while β_2 can be calculated by the use of Eq. 12a and Eq. 12b.

$$\begin{aligned} (\alpha)_{bedrock} &= 0.735 \cdot M_{b,Lg} - 4.41 && \text{(a)} \\ (\alpha)_{soil} &= 1.025 \cdot M_{b,Lg} - 6.292 && \text{(b)} \end{aligned} \tag{10}$$

$$\begin{aligned} (\beta_1)_{bedrock} &= 0.35 \cdot M_{b,Lg} + 1.94 && \text{(a)} \\ (\beta_1)_{soil} &= 3.58 - 0.174 \cdot M_{b,Lg} && \text{(b)} \end{aligned} \tag{11}$$

$$\begin{aligned} (\beta_2)_{bedrock} &= 0.21 - 0.15 \cdot M_{b,Lg} && \text{(a)} \\ (\beta_2)_{soil} &= -0.794 - 0.056 \cdot M_{b,Lg} && \text{(b)} \end{aligned} \tag{12}$$

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 obtained from the $(C/D)_{min.}$ ratio. The analysis using this method eliminates the misleading condition of how to assess an embankment with $(C/D)_{min.}$ ratio<1.0. Instead, this method forces a consideration of the possible displacement that may be observed, a better prediction of the actual behavior of a given embankment during a seismic event.

Liquefaction Potential: The mechanical behavior, which includes the liquefaction potential during the seismic event, is another important parameter in the seismic vulnerability assessment and prioritization of bridge embankments. Cohesion-less soils, such as alluvium and sandy/gravelly continental deposits are susceptible to liquefaction, and alluvium is the most likely to experience liquefaction. Where the boring logs data is available, straight-forward steps are followed to define the liquefaction potential as reported by [9]. In order to overcome the difficulties encountered when such data is not available, an alternate approach to define the liquefaction potential is followed. The liquefaction potential has to be assessed in accordance with the following two sub-sections

Boring Logs Are Not Available: Where the 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 [1]. The susceptibility of the embankment soil to liquefaction is classified as one of three possible types (Table 3).

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.

Table 3. Liquefaction Susceptibility at a bridge embankment site

The three liquefaction possibilities are: high susceptibility, moderate susceptibility, 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 long periods of time by flowing water is often constructed on loose saturated cohesion-less deposits that are the most susceptible to liquefaction. Moderate susceptibility is associated with medium dense soils such as compacted sand soils. Low susceptibility is associated with dense soils.

Boring Logs Are Available: Where the boring log data is available, the liquefaction potential at the bridge site is determined by the method reported by [9]. 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 [10] is used. The average cyclic shear induced by the seismic event is obtained from Eq. 13.

$$\frac{\tau_{h,avg}}{\sigma_0^{'}} \cong 0.65 \frac{A_{\max.}}{g} \cdot \frac{\sigma_0}{\sigma_0^{'}} \cdot r_d \tag{13}$$

where $\tau_{h,avg}$ is the average cyclic shear stress during the time history of interest, σ'_o 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'_o and σ'_o) are replaced with the effective and total vertical stresses. The stress reduction factor (r_d), defined by [10], is computed using the depth (z) in meters as shown in Eq. 14.

$$r_d = (1 - \frac{z}{91}) \tag{14}$$

The soil penetration resistance is the corrected normalized standard penetration resistance, $N_{1,60}$, which is defined by [10] and [5] in Eq. 15.

$$N_{1,60} = C_N \cdot \frac{ER_m}{60} \cdot N_m \tag{15}$$

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. 16.

$$FS_l = \frac{[\tau_{avg} / \sigma_0^{'}]_{l,M=M}}{[\tau_{h'avg} / \sigma_0^{'}]} \tag{16}$$

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.

6. Category identification

Ranking and prioritization of embankments is based on the input parameters including geometry, material, seismic event, upper level of bedrock layer, level of natural ground line and soil type. Seismic vulnerability ranking and prioritization is conducted using the 'Kentucky Embankment Stability Ranking' (*KESR*) model in which three categories are incorporated to specify the failure risk of each embankment [4]. Application of the proposed methodology results in obtaining the three aforementioned ranking parameters known as the $(C/D)_{min.}$ ratio, embankment displacement, and liquefaction potential. The *KESR* model assumes one of the following three possibilities (*A*, *B*, or *C*) of embankment behavior during a seismic event, as described in Table 2: (*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 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.

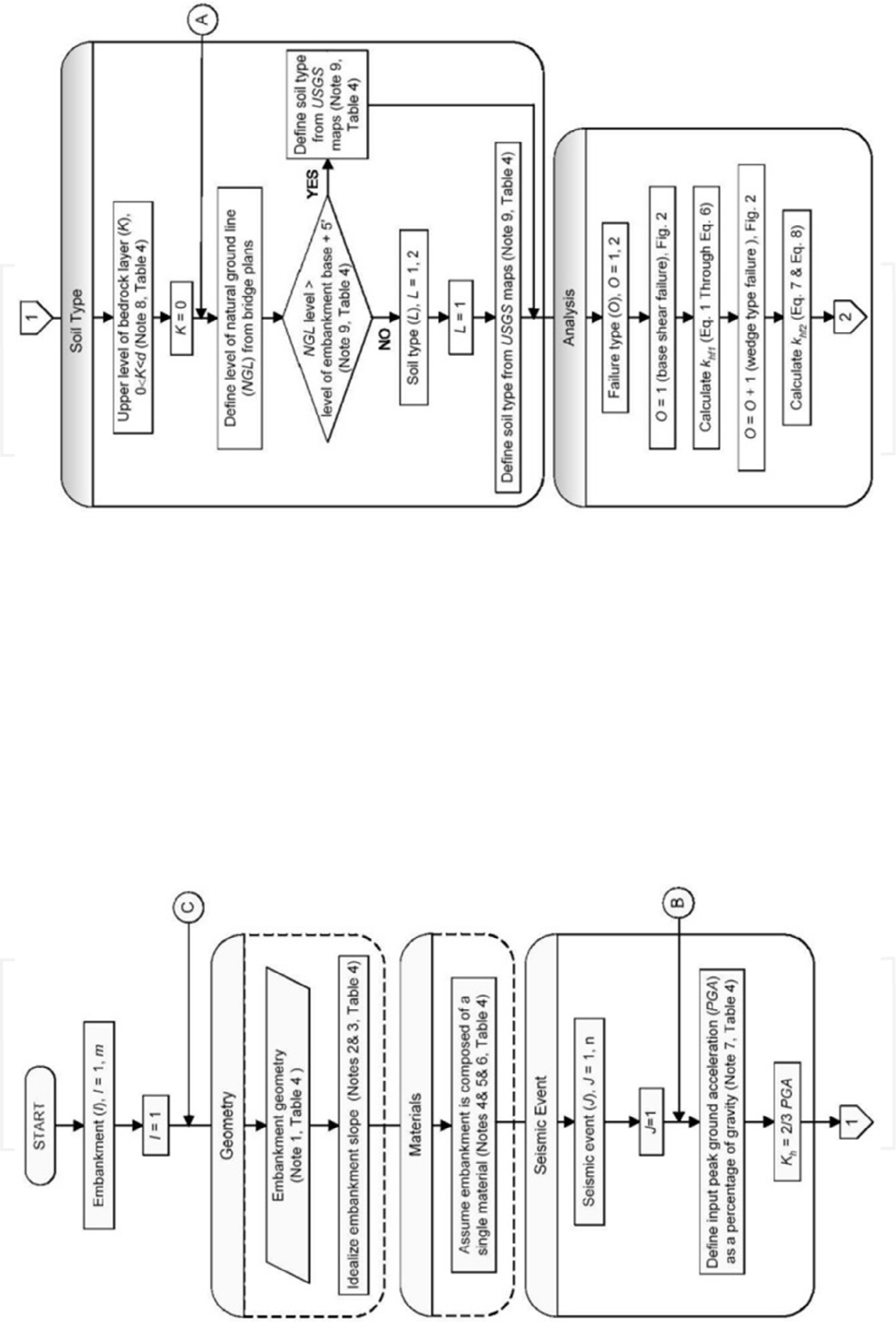
7. Ranking and prioritization

After classifying the bridge embankments to category *A*, category *B*, or category *C* in accordance with the criteria listed in Table 2, 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 embankment prioritization in each category becomes a feasible task. This proposed ranking model is useful 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 provided ranking model is to compare the seismic risk of the several embankments, regardless of the existence of highly accurate input data in the ranking model, it is the authors' recommendation to further conduct detailed assessments of the behavior of those at-risk embankments. In such detailed assessments, accurate data from sub-soil explorations is 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.

8. Step-by-step seismic risk identification of bridge embankments

In order to facilitate the application of the proposed ranking methodology to prioritize bridge embankments, a complete flowchart has been generated. The flowchart provides a useful tool that promotes achieving the final goal of the study. The flowchart in its current form and sequences ensures a minimal effort from the engineer/researcher to apply the specified ranking methodology. Parameters of each embankment including its geometry, material, seismic event, upper level of bedrock, level of natural ground line, soil type, and anticipated failure types are taken into consideration during the development of the flowchart. All considerations, assumptions, calculations and required checks are arranged in a defined order in the flowchart. The loops of the flowchart, shown in Figure 3, allow relative ranking of bridge embankments. Titles are provided to identify the different sections of the flowchart including geometry, materials, seismic event, soil type, analysis, ranking parameters, category identification, and final ranking/prioritization. Notes to explain the steps of the methodology are numbered consecutively, listed in Table 4, and need to be considered along with the flowchart during the seismic risk prioritization of bridge embankments in a designated region.


START
Embankment (I), I = 1, m
I = 1
C
Geometry
Embankment geometry (Note 1, Table 4)
Idealize embankment slope (Notes 2& 3, Table 4)
Materials
Assume embankment is composed of a single material (Notes 4& 5& 6, Table 4)
Seismic Event
Seismic event (J), J = 1, n
J=1
B
Define input peak ground acceleration (PGA) as a percentage of gravity (Note 7, Table 4)
K_h = 2/3 PGA
1
1
Soil Type
Upper level of bedrock layer (K), $0<K<d$ (Note 8, Table 4)
K = 0
A
Define level of natural ground line (NGL) from bridge plans
NGL level > level of embankment base + 5' (Note 9, Table 4)
YES
Define soil type from USGS maps (Note 9, Table 4)
NO
Soil type (L), L = 1, 2
L = 1
Define soil type from USGS maps (Note 9, Table 4)
Analysis
Failure type (O), O = 1, 2
O = 1 (base shear failure), Fig. 2
Calculate k_{hf1} (Eq. 1 Through Eq. 6)
O = O + 1 (wedge type failure), Fig. 2
Calculate k_{hf2} (Eq. 7 & Eq. 8)
2

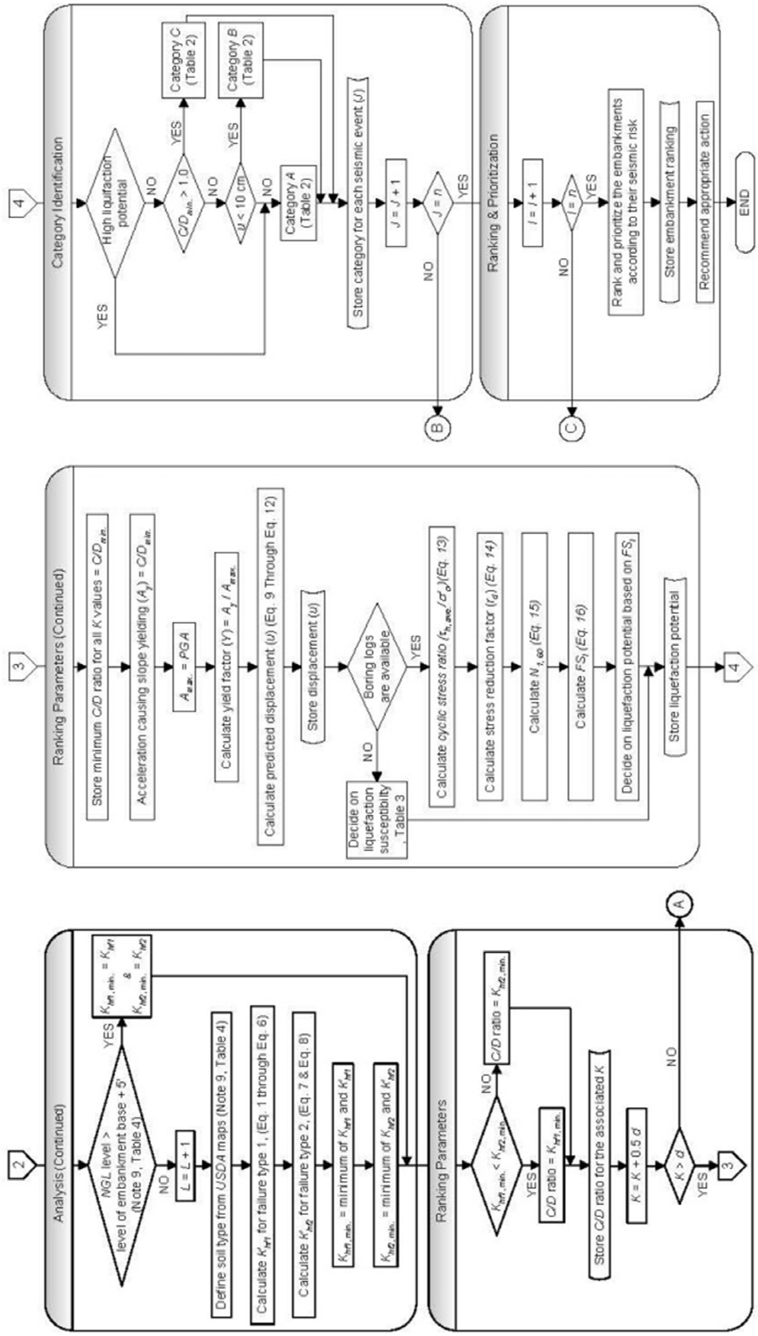


Figure 3. Flowchart for seismic risk assessment and ranking of bridge embankments

Note #	Description
Note 1	Embankment slope and geometry are taken from the bridge plans.
Note 2	For a typically irregular embankment slope, the highest idealized slope is used and is chosen in a way that results in the least $(C/D)_{min.}$ ratio.
Note 3	*H* = height of the embankment, *b* = slope inclination (Figure 1).
Note 4	Density and shear strengths of the embankment fill and soils underneath are identified in Table 1. A soil layer beneath the embankment fill has uniform un-drained shear strength different from that of the embankment fill.
Note 5	The analysis is carried out on the most critical embankment slope at either end of the bridge.
Note 6	The water table is assumed to be located below the embankment base.
Note 7	The input surface acceleration for a designated embankment site is obtained in accordance with the earthquake magnitude.
Note 8	The upper level of the bedrock layer is based on the "*Geologic Quadrangle Maps of the United States*" provided by "*United States Geological Survey* (*USGS*)", where the bedrock layer provides the lower bound for the soil layer (Figure 1).
Note 9	The choice whether to only consider soil data from the *USGS* maps or to consider an additional case from the *USDA* maps is dependent upon the level of the "Natural Ground Line" (*NGL*) as compared to the level of the embankment base.

Table 4. Complimentary notes to Figure 3 "Flowchart for seismic risk assessment and ranking of multiple bridge embankments"

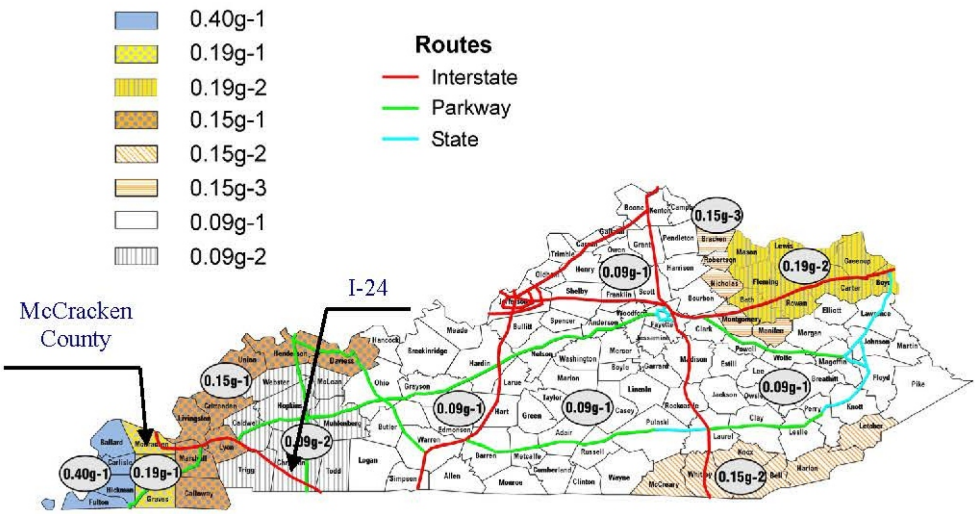


Figure 4. Predicted "Peak Ground Acceleration" (PGA) of all counties in the Commonwealth of Kentucky during a 250-year seismic event

9. Bridges in the commonwealth of Kentucky

Bridges in the western region of the Commonwealth of Kentucky are located near the New Madrid seismic zone, which is potentially one of the most destructive fault zones in the United States. It extends through the Mississippi River Valley and encompasses 26 counties in western Kentucky in the area of its strongest influence. Studies have shown that the probability of an earthquake with a 6.3 magnitude on the Richter scale to hit this area within the next 50 years exceeds 80%.

Passing through seven counties in western Kentucky, I-24 is considered a vital transportation link for the commonwealth of Kentucky. I-24 passes through McCracken, Livingston, Marshall, Lyon, Trigg, Caldwell, and Christian counties in western Kentucky (Figure 4). The objective of this part of the Chapter is to investigate the seismic risk of all bridge embankments on or over I-24 in western Kentucky.

In order to achieve the study objective, a means of accessing which embankments qualify as "most critical" is required. The methodology presented earlier in this Chapter is applied to assess the seismic vulnerability of I-24 bridge embankments. The embankment geometry, materials, type and properties of underlying soil, elevation of the natural ground line, and upper level of bedrock are estimated for each embankment. The minimum seismic slope stability capacity/demand, $(C/D)_{min}$ ratio, embankment displacement, and liquefaction potential of each bridge embankment are calculated. Bridge embankments along I-24 in western Kentucky are assigned one of three possible categories to represent their seismic failure risk. A final priority list of the embankments with the highest seismic risk is generated for the 127 bridges on or over I-24 in western Kentucky.

On-Site Inspection of I-24 Bridges in Western Kentucky: On-site inspection of the bridges, including photographing different structural components of each bridge, was carried out. The on-site inspection records form an invaluable source that assists in pre-earthquake evaluation studies as well as post-earthquake inspection.

I-24 Bridge Inventory in Western Kentucky: One objective of the on-site inspection is to have an informative source of accurate and updated bridge records, which are required for most assessment studies including the current study of seismic ranking and prioritization of I-24 bridge embankments in western Kentucky. Another objective of the on-site inspection is to provide engineers and transportation officials with information delineating the current bridges' conditions in order to facilitate future comparisons with post-earthquake conditions immediately after future earthquakes. Through these comparisons, significant changes can be reported and further studies can be carried out. All the bridges and embankments along I-24 in western Kentucky were visually inspected, photographed and the records were stored in a database. The on-site inspection represents a significant supplement to the "as-built" bridge plans. A comprehensive inventory of the bridges was compiled by review of the "as-built" bridge plans, construction and maintenance records, and on-site inspection forms. The inventory provides an essential data record, which is utilized for risk assessment of I-24 bridges and embankments in western Kentucky. A one-page sample of the I-24 bridge inventory for McCracken County is presented in Table 5. Similar inventories for Livingston, Marshall, Lyon, Trigg, Caldwell, and Christian counties are shown elsewhere [11].

Characteristics of I-24 Bridge Inventory in Western Kentucky: Eighty-one bridges are located on I-24 and 45 bridges are constructed over I-24, resulting in a total of 127 bridges either on or over the interstate in western Kentucky. Of the 127 bridges, many bridges were designed without following stringent seismic design guidelines, and may not withstand severe seismic events. Lyon and Marshall Counties are located approximately 115 Kilometers (72 miles) and 96 kilometers (60 miles) northeast of the center of the New Madrid seismic zone, respectively. McCracken County, located approximately 72 kilometers (45 miles) northeast of the center of the New Madrid seismic zone, has the largest number of bridges among all other counties with an average of two bridges per mile. The 127 bridges are categorized based on several characteristics, including: structural type, number of spans, maximum span length, skew angle, construction materials, and bearing types. Eighty three percent of the bridges are skewed, of which, 13% have a skew angle exceeding 40 degrees. McCracken County includes the largest number of bridges (38 bridges), followed by Lyon County (27 bridges), Marshall County (21 bridges), Christian County (20 bridges), Trigg County (11 bridges), Livingston County (seven bridges), and Caldwell County (three bridges).

10. Embankment properties

The geometry of each bridge embankment on or over I-24 in western Kentucky is taken from the bridge plans. The geometry of the 127 studied embankments is classified into five types (Figure 5*a*-5*e*). An embankment has either a single slope or double slopes separated by a perm. The inventory of I-24 bridge embankments in western Kentucky shows that a given slope has one of three possible inclinations (1:1, 2:1, or 3:1), where the first number of the ratio represents the horizontal unit and the second number represents the vertical unit. The drawings shown in Figure 5*a*-5*d* are for cases where the feature crossed by the bridge is either a highway or a railway. The drawing shown in Figure 5*e* is found when the bridge crosses a waterway. The embankment slope geometry is identified by its height (H) and the idealized inclination (b) (Figure 6). The analysis is carried out on both ends of each bridge and the most critical embankment slope at either end; whichever analysis results in a lower seismic slope stability C/D ratio is considered in the seismic vulnerability ranking.

Accurate identification of the soil characteristics requires detailed site-specific subsurface exploration. This approach is expensive, and such data is not available for the majority of the bridge embankments along I-24 in western Kentucky. Pflazer [14] reported on the use of existing geo-technical data to supplement site investigations. Another approach to specify the soil type and its properties is to use existing geological and agricultural maps. The source of soil data is dependent on the *NGL* (Figure 5*f*-5*g*). The *USGS* and the *USDA* are used to identify the soil type underneath an embankment. The way by which either

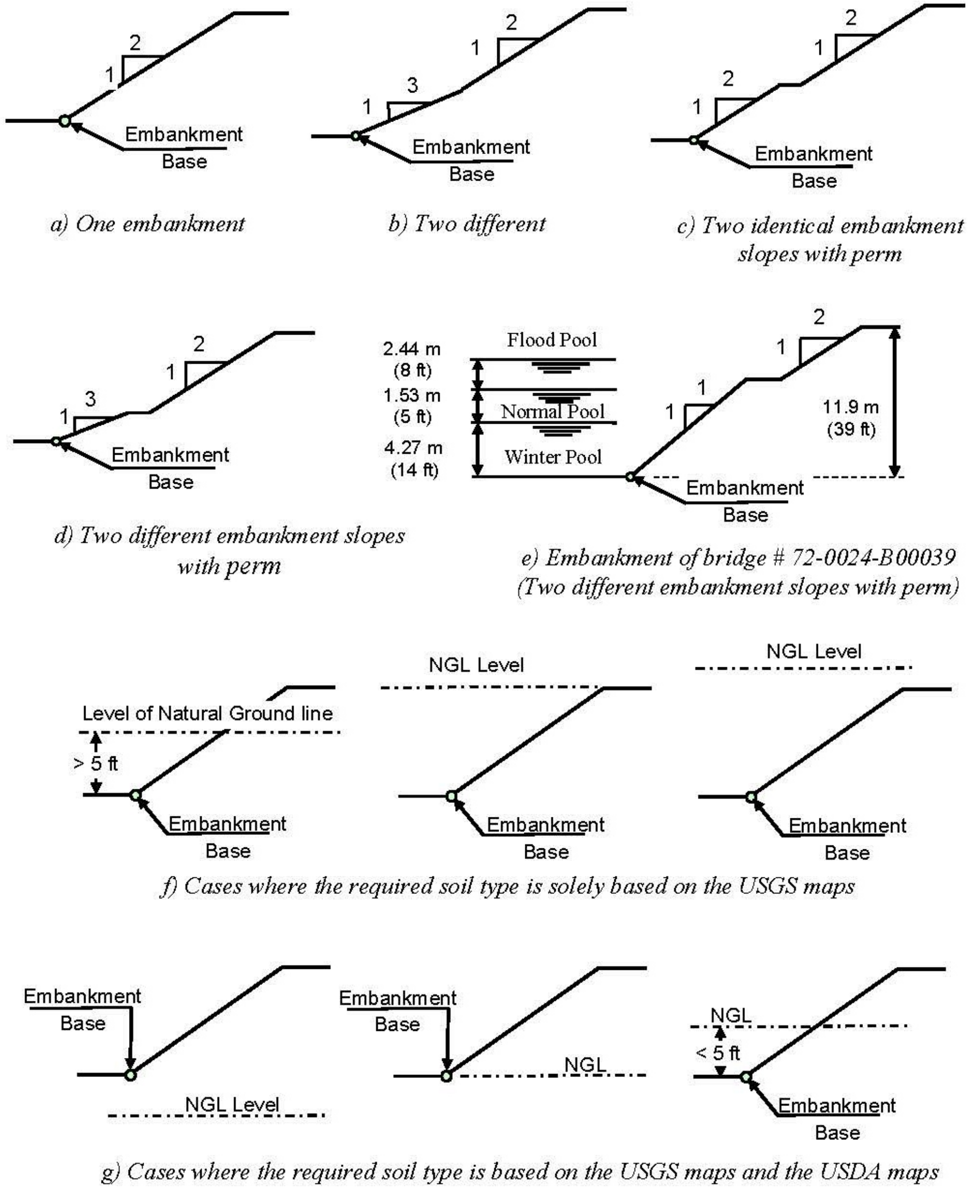


Figure 5. Embankments along I-24 in western Kentucky: geometry classification (Figs. a, b, c, d, e), level of "Natural Ground Line" (NGL) and source of the soil data (Figs. f, g)

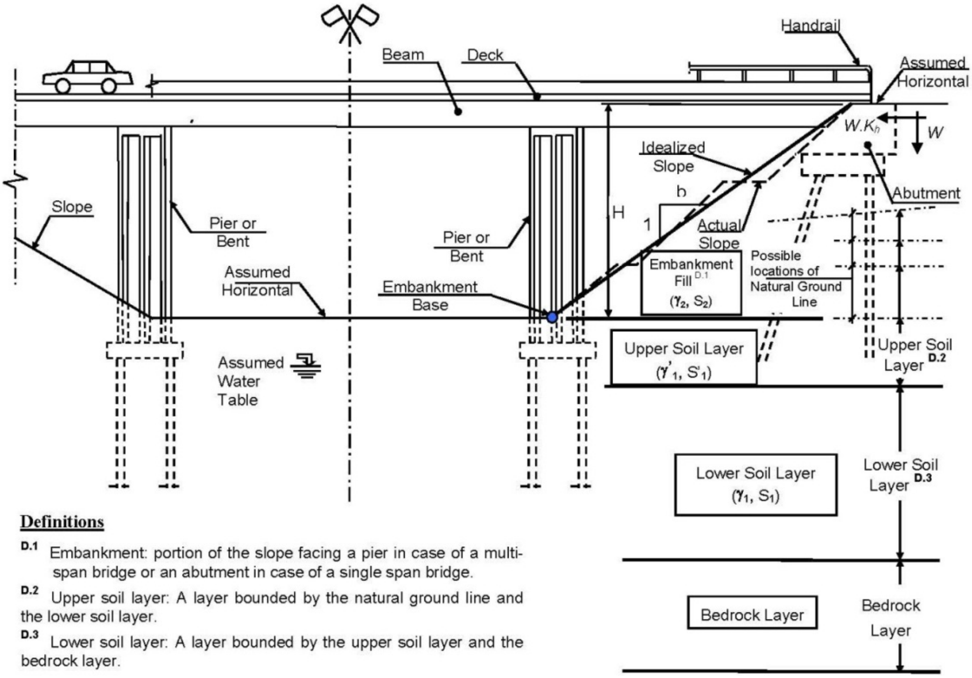


Figure 6. Example of bridge embankment geometry and materials

map is chosen for a given bridge site is based on the level of the "*Natural Ground Line*" (*NGL*) as compared to the respective embankment base (Figures 2*f*, 2*g*, and 3). Whenever the level of the *NGL* is above the level of the embankment base by more than 1.50 m (5 ft), the soil type is solely identified in accordance with the *USGS* maps. Whenever the level of the *NGL* is either above the level of the embankment base by less than 1.50 m (5 ft) or below the level of the embankment base, the soil type is based on both the *USGS* maps, and the *USDA* maps. After specifying the soil type, conservative soil characteristics including shear strength and mass density are estimated. The upper and lower soil layers' types (Figure 5) for embankments in McCracken County are provided in Table 6. Shear strength and mass density for bridge embankments are derived following the guidelines presented earlier. Data regarding the level below which a hard stratum (stiff bedrock layer) exists is not available for the majority of bridge embankment sites along I-24 in western Kentucky. The upper level of the stiff bedrock layer, which falls within the range from the embankment base down to the upper level of the hard stratum, is initially estimated from the *USGS maps.*

District	Status [1]	County [2]	Route	Bridge Bin # [3]	P [4]	Year Built	Main Spans [5]	Appr-oach Spans [6]	Max. Span Length (ft)	Structure Length [7] (ft)	MP [8]	On / Over I-24	Feature Crossed
1	SM	73	131	B00009		1968	2	0	118	294	15.79	Over	I-24
1	SM	73	68	B00060		1968	2	0	92	233	16.16	Over	I-24
1	SM	73	68	B00060	P	1968	2	0	92	233	16.16	Over	I-24
1	SM	73	787	B00064		1966	2	0	94	228	16.88	Over	I-24
1	SM	73	3075	B00065		1966	2	0	92	242	14.09	Over	I-24 @ 14.09
1	SM	73	24	B00100 [9]			2	16	730	5634	0.10	On	Ohio River
1	SM	73	24	B00101	P	1968	3	0	54	133	2.21	On	KY 1420
1	SM	73	24	B00101		1968	3	0	54	133	2.21	On	KY 1420
1	SM	73	24	B00102	P	1969	1	0	110	142	2.96	On	KY 305
1	SM	73	24	B00102		1969	1	0	110	142	2.96	On	KY 305
1	SM	73	24	B00103	P	1969	3	0	74	181	3.46	On	P&L Railway
1	SM	73	24	B00103		1969	3	0	74	181	3.46	On	P&L Railway
1	SM	73	24	B00104	P	1968	3	0	69	170	3.69	On	P&L Railway
1	SM	73	24	B00104		1968	3	0	69	170	3.69	On	P&L Railway
1	SM	73	24	B00105		1969	2	0	80	224	4.33	On	US 60
1	SM	73	24	B00105	P	1969	2	0	80	224	4.33	On	US 60
1	SM	73	24	B00107	P	1967	3	0	50	115	4.59	On	Perkins Creek
1	SM	73	24	B00107		1967	3	0	50	115	4.59	On	Perkins Creek
1	SM	73	24	B00111		1971	3	0	51	121	5.60	On	Buchner Lane
1	SM	73	24	B00111	P	1971	3	0	51	121	5.60	On	Buchner Lane
1	SM	73	24	B00112		1971	2	2	85	196	6.87	On	US 45
1	SM	73	24	B00112	P	1971	2	2	85	196	6.87	On	US 45
1	SM	73	24	B00113		1974	4	0	105	337	7.36	Over	I-24 @ Elmdale Rd
1	SM	73	24	B00114		1963	5	0	98	458	9.77	On	P&L Railroad
1	SM	73	24	B00114	P	1963	5	0	98	458	9.77	On	P&L Railroad
1	SM	73	24	B00115	P	1971	3	0	53	143	10.32	On	Island Creek
1	SM	73	24	B00115		1971	3	0	53	143	10.32	On	Island Creek
1	SM	73	24	B00116	P	1975	2	0	96	197	11.04	On	KY 1954
1	SM	73	24	B00116		1975	2	0	96	197	11.04	On	KY 1954
1	SM	73	24	B00117 [9]		1972	2	0	15	34	11.44	On	Bee Bridge
1	SM	73	24	B00118	P	1975	3	0	71	191	11.98	On	Old L & N Railroad
1	SM	73	24	B00118		1975	3	0	71	191	11.98	On	Old L & N Railroad
1	SM	73	24	B00119	P	1971	1	2	101	172	12.60	On	KY 450 (Oaks Rd)
1	SM	73	24	B00119		1971	1	2	101	172	12.63	On	KY 450 (Oaks Rd)
1	SM	73	24	B00120	P	1975	3	0	200	486	13.30	On	Clarks River
1	SM	73	24	B00120		1975	3	0	200	486	13.30	On	Clarks River
1	SM	73	62	B00121		1971	2	0	105	260	6.39	Over	I-24
1	SM	73	994	B00122		1971	2	2	108	256	8.61	Over	I-24

[1] Status is defined as SM (State Maintained), RS (Rural Secondary), or County (Locally Maintained). SM bridges are the only bridges to appear in this sample page.

[2] County # 73 stands for McCracken County in western Kentucky.

[3] Bridge Bin # is as appears in the Kentucky Transportation Cabinet bridge inventory.

[4] The letter P, as defined in the Kentucky Transportation Cabinet bridge inventory, stands for a parallel bridge which is located westbound on I-24

[5] Main spans stands for the number of main spans of the designated bridge.

[6] Approach spans stands for the number of approach spans of the designated bridge.

[7] Structure length is the total length of bridge including the approaches.

[8] MP stands for the mile point to which the bridge is logged.

[9] The designated bridge under consideration is out of the scope of the study.

Table 5. Inventory of I-24 bridges in McCracken County, western Kentucky

County	Bridge Number [1, 2]	Upper Soil Layer [3, 4]		Lower Soil Layer [3, 5]	
		Depth [6] (ft)	Soil Type [6]	Depth [7] (ft)	Soil Type [7]
McCracken	73-0024-B00101 & 73-0024-B00101P [2]	0'- 5'	Heavy Silt Loam	5'- 30'	Lacustrine Deposits
	73-0024-B00102 & 73-0024-B00102P [2]	0'- 5.5'	Gravely Sandy Clay Loam	5.5'- 90'	Loess layer followed by Continental Deposits
	73-0024-B00103 & 73-0024-B00103P [2]	0'- 5.5'	Silt Loam	5.5'- 90'	Loess layer followed by Continental Deposits
	73-0024-B00104 & 73-0024-B00104P [2]	0'- 5.5'	Silt Loam	5.5'- 90'	Loess followed by Continental Deposits
	73-0024-B00105 & 73-0024-B00105P [2]	0'- 5.8'	Silty Clay Loam	5.8'- 90'	Loess layer followed by Continental Deposits
	73-0024-B00107 & 73-0024-B00107P [2]	0'- 5'	Silt Loam	5'- 95'	Alluvium
	73-0024-B00111 & 73-0024-B00111P [2]	0'- 5'	Silt Loam	5'- 90'	Loess layer followed by Continental Deposits
	73-0024-B00112 & 73-0024-B00112P [2]	0'- 5.8'	Silt Loam	5.8'- 95'	Alluvium
	73-0024-B00114 & 73-0024-B00114P [2]	0'- 5'	Heavy Silt Loam	5'- 200'	Lacustrine Deposits
	73-0024-B00115 & 73-0024-B00115P [2]	0'- 5.5'	Heavy Silt Loam	5.5'- 200'	Lacustrine Deposits
	73-0024-B00116 & 73-0024-B00116P [2]	0'- 5'	Heavy Silt Loam	5'- 200'	Lacustrine Deposits
	73-0024-B00118 & 73-0024-B00118P [2]	0'- 5.8'	Silty Clay Loam	5.8'- 200'	Lacustrine Deposits
	73-0024-B00119 & 73-0024-B00119P [2]	0'- 5.8'	Silt Loam	5.8'- 200'	Lacustrine Deposits
	73-0024-B00120 & 73-0024-B00120P [2]	0'- 5.5'	Heavy Silt Loam	5.5'- 200'	Lacustrine Deposits
	73-0131-B00009	0'- 5.8'	Silty Clay Loam	5.8'- 90'	Loess layer followed by Continental Deposits
	73-0068-B00060 & 73-0068-B00060P [2]	0'- 6.3'	Silty Clay Loam	6.3'- 90'	Loess layer followed by Continental Deposits
	73-0787-B00064	0'- 5'	Heavy Silt Loam	5'- 90'	Loess layer followed by Continental Deposits
	73-3075-B00065	0'- 5'	Heavy Silt Loam	5'- 93'	Continental Deposits
	73-0024-B00113	0'- 4.5'	Silty Loam	4.5'- 93'	Continental Deposits
	73-0062-B00121	0'- 5.5'	Silt Loam	5.5'- 93'	Continental Deposits
	73-0994-B00122	0'- 5.8'	Silty Clay Loam	5.8'- 93'	Continental Deposits

[1] As defined in the Kentucky Transportation Cabinet bridge inventory.

[2] The letter P, as defined in the Kentucky Transportation Cabinet inventory, stands for a parallel bridge that is located in the westbound lane along the I-24 in western Kentucky.

[3] Upper soil layer and lower soil layer are shown in Fig. 3.

[4] Upper soil layer's data is based on the '*United States Department of Agriculture*' (*USDA*) maps, '*Soil Conservation Service*'.

[5] Lower soil layer's data is based on the '*United States Geological Survey*' (*USGS*), '*Geologic Quadrant Maps of the United States*'.

[6] Data is obtained from the 'Soil Survey of Ballard and McCracken Counties, Kentucky', the *United States Department of Agriculture*' (*USDA*) maps, '*Soil Conservation Service*', 1976.

[7] Data is obtained from the 'Geology of the Puducah East Quadrangle, Kentucky', Map GQ-531, 'United States Geological Survey' (*USGS*), '*Geologic Quadrant Maps of the United States*'.

Table 6. Types of upper and lower soil layers for embankment sites in McCracken County, western Kentucky

Other upper levels of the bedrock layer within that range are also considered, and the controlling case is the one that results in the lowest seismic slope stability *C/D* ratio. The input *PGA* at a designated embankment site is obtained from seismic maps generated by [7] for 50-year, 250-year, and 500-year events. The 50-year, 250-year, and 500-year events are seismic events with a 90% probability of not being exceeded in 50 years, 250 years, and 500 years, respectively. Figure 4 illustrates an example of the anticipated *PGA* of all counties in the Commonwealth of Kentucky during the 250-year seismic event. The peak ground acceleration for McCracken County during the 250-year event is 0.19 *g*, where *g* is the gravitational acceleration. Other anticipated *PGAs* of all counties in the Commonwealth of Kentucky during the 50-year and 500-year seismic events can be found in the Kentucky Transportation Center report [11]. With the exception of the parallel bridges at the Cumberland River crossing, and at the Tennessee River crossing, each bridge and their embankments along I-24 in western Kentucky is evaluated for the 50-year and 250 year seismic events, for which valuable input data is taken from a study conducted by Street et al. [7]. During the 50-year seismic event, the bridges are expected to behave elastically without any disruption to traffic. During the 250-year seismic event, partial damage to the bridges is permitted, and the bridges are expected to remain accessible to emergency traffic. I-24 parallel bridges at the Cumberland River and at Tennessee River crossings are evaluated for the 250-year seismic event and the maximum credible 500-year seismic event. Detailed evaluation of these bridges and their embankments are presented elsewhere [11].

11. Vulnerability analysis of I-24 bridge embankment in Kentucky

For a bridge on or over I-24 in western Kentucky, the potential of an embankment slope to displace during a designated earthquake event is assessed using the two-dimensional limit equilibrium stability analysis. During the seismic vulnerability evaluation of each embankment, the possibility of occurrence of either circular or wedge–shaped slope failure [11] is investigated and the one that results in the lesser *C/D* ratio is considered in the ranking process. K_h equals to 2/3 of the *PGA*. The ranking and prioritization procedure of the embankments is based on three main parameters: (1) seismic slope stability $(C/D)_{min.}$ ratio, (2) embankment displacement, and (3) liquefaction potential at the embankment site. For embankments with $(C/D)_{min.}$ ratio against sliding<1.0, estimation of how far the embankment actually displaces during the ground excitation is necessary. Hence, the displacement of the embankment is calculated. The maximum acceleration ($A_{max.}$) for a specified seismic event is identified for a designated embankment. For slope displacement to occur, the maximum acceleration must exceed the acceleration causing yielding in the embankment slope (A_y). The $(C/D)_{min.}$ ratio is calculated for each embankment, and is used to assign a rank for each embankment relative to the other embankments along I-24 in western Kentucky.

Assuming that the yield displacement is equal to K_{hf}, which corresponds to the $(C/D)_{min.}$ ratios for all the possible failure cases, the resulting 'Yield Factor' (*Y*) is estimated as the ratio of A_y/A_{max}, where A_y is the acceleration causing yielding in the embankment slope and $A_{max.}$ is equal to the *PGA*. The displacement of the slope with a specified *PGA* exceeding the A_y is estimated. At intervals for which the *PGA* exceeds A_y (*Y* is less than 1.0), the occurrence of slope displacement is expected. Decreasing A_y results in increasing the magnitude of the embankment displacement, correspondingly. 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 obtained from the $(C/D)_{min.}$ ratio analysis. One advantage of this methodology is that the analysis eliminates the misleading condition of how to assess an embankment that has $(C/D)_{min.}$ ratio<1.0, and instead forces a consideration of the possible embankment displacement. The vulnerability rating for a designated soil is based on quantitative assessment of liquefaction susceptibility and the anticipated magnitude of the acceleration coefficient [1]. Bridges subjected to low liquefaction potential shall be assigned a low vulnerability rating.

It is stipulated that it is not necessary to calculate liquefaction potential for the bridge sites, which are required to resist a seismic acceleration of less than 0.09 *g* [1]. The majority of the area surrounding the fault in the New Madrid Seismic Zone lies on fluvial and alluvial deposits and sandy soils. Defining the liquefaction potential is a matter of considerable concern during the seismic assessment of bridges and their embankments in this region. Western Kentucky encompasses several major bodies of water, including the Ohio River, Mississippi River, Barkley Lake, and Kentucky Lake. These bodies of water cause the saturated soils within the area to be highly susceptible to liquefaction potential. The proximity to these four bodies of water necessitates particular concern when examining the liquefaction potential for bridge sites along I-24 in western Kentucky.

The method to calculate the liquefaction potential is dependent on the availability of the soil boring logs. Whenever the boring logs of an embankment site along I-24 in western Kentucky are not available, the susceptibility of an embankment soil to liquefaction is classified in one of three ways. High susceptibility is associated with saturated loose sands, saturated silty sands, or non-plastic sands. A bridge that crosses a waterway is often constructed on loose saturated cohesionless 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.

Whenever the boring logs of an embankment site along I-24 in western Kentucky are available, the liquefaction potential of the bridge site is accurately determined by the method developed by Seed et al. [9, 10] and reported earlier in this Chapter. This method includes the following four steps: (1) determination of time history of shear stresses induced by the earthquake ground motion; (2) converting the time history to an equivalent number of stress cycles; (3) calculation of the cyclic shear stresses required to cause liquefaction in the same number of stress cycles; and (4) judging the liquefaction potential by comparing the shear stress induced during the earthquake with that required to cause liquefaction.

Liquefaction potential of few embankment sites along I-24 in western Kentucky is estimated using standard penetration tests (*SPT*) provided by the 'Kentucky Transportation Cabinet, Department of Materials and Geotechnical Testing.' For the rest of the bridge embankments along I-24 in western Kentucky, any judgment of the liquefaction potential is solely based on the surrounding soil type. The soil type is obtained from the *USGS* and *USDA* maps. A detailed method to predict the liquefaction potential is shown in Zatar et al. [12, 13].

12. Category identification, ranking, and prioritization of the I-24 bridge embankments in Western Kentucky

In the KESR model, three categories are sought out to specify the failure risk of each embankment during a designated seismic event. A category for each bridge embankment along I-24 in western Kentucky is assigned. The assigned category is based on the three ranking parameters: the $(C/D)_{min.}$ ratio, the embankment displacement, and the liquefaction potential. Definition of the three categories (*A*, *B*, and *C*) is provided in Table 3. All 127 bridge embankments along I-24 in western Kentucky were analyzed using the procedures provided in the flowchart of Figure 6. The yield factor, $(C/D)_{min.}$ ratio, displacement, and liquefaction potential for each embankment are identified, and a seismic embankment category is assigned. Further prioritization within each category was carried out based on the significance of the three ranking parameters. The embankments are ranked starting from the one with the highest seismic risk. For instance, a bridge embankment in category *A* with a ranking of *A*1 is more susceptible to damage than a bridge embankment with a ranking of *A*2 or *A*3. The same also applies for categories *B* and *C*. The ranking comprises a priority list that will be provided to senior state engineers, who may utilize its information to take appropriate actions. Based on the priority list, accurate soil data for those embankments with the highest risk may be needed in order to accurately identify their risk.

Due to its immense size, the full listing of the 127-embankment ranking and prioritization is not presented. However, a sample ranking and prioritization list for all embankments in McCracken County is presented for the 250-year seismic event (Table 7). Some of the embankments, which are in Category *B* during the 50-year seismic event, fall in Category *A* during the 250-year seismic event. For instance, the analysis of Bridge # 73-0024-B00118 in McCracken County to resist the 50-year seismic event results in a displacement of 4.6 centimeters (1.8 inches), and thus falls in category *B*. The analysis for the same bridge to resist the 250-year seismic event results in a displacement of 27.3 centimeters (10.7 inches) and thus is considered to fall in Category *A*. None of the embankments in McCracken County fall within category *C* since the assigned *PGA* for McCracken County is the highest among all counties along I-24 in western Kentucky, in addition to the associated liquefaction potential. This is not the case for Christian, Lyon, Trigg, and Caldwell counties.

County	Bridge # [1,2]	Peak Ground Acceleration [3] (%g)	Slope Height [4]		Yield Factor [5] Y	Horizontal Displacement [6] U		Minimum C/D Ratio [7] $(C/D)_{min}$	Liquefaction Potential [8]	Seismic Embankment Category [9]	Seismic Embankment Ranking [10]
			(m)	(ft)		(cm)	(in)				
McCracken	730024B00104 & 730024B00104P	19	9.1	30	0.084	79.8	31.4	0.75	High	A	A1
	730024B00103 & 730024B00103P	19	8.7	28.5	0.136	39.5	15.6	0.76	High	A	A2
	730024B00120 & 730024B00120P	19	11.6	38	0.166	28.7	11.3	0.67	High	A	A3
	730024B00118 & 730024B00118P	19	8.4	27.5	0.172	27.3	10.7	0.77	Moderate	A	A4
	730068B00060 & 730068B00060P	19	8.4	27.4	0.175	26.3	10.4	0.77	High	A	A5
	730787B00064	19	8.1	26.6	0.177	25.8	10.1	0.78	High	A	A6
	730024B00115 & 730024B00115P	19	7.9	26	0.226	16.8	6.6	0.79	Moderate	A	A7
	730024B00107 & 730024B00107P	19	7.5	24.5	0.236	15.5	6.1	0.76	High	A	A8
	730024B00105 & 730024B00105P	19	7.8	25.5	0.244	14.5	5.7	0.8	High	A	A9
	730024B00112 & 730024B00112P	19	6.9	22.5	0.308	8.9	3.5	0.79	High	A	A10
	730024B00102 & 730024B00102P	19	7	23.1	0.335	7.3	2.9	0.83	High	A	A11
	730131B00009	19	6.9	22.6	0.355	6.4	2.5	0.84	High	A	A12
	730024B00111 & 730024B00111P	19	6.7	22	0.377	5.5	2.2	0.85	High	A	A13
	730024B00119 & 730024B00119P	19	7.3	24	0.3	9.4	3.7	0.82	Moderate	B	B1
	730024B00116 & 730024B00116P	19	7.3	24	0.3	9.4	3.7	0.82	Moderate	B	B1
	730024B00114 & 730024B00114P	19	6.6	21.7	0.388	5.1	2	0.85	Moderate	B	B3
	730024B00101 & 730024B00101P	19	5.8	19	0.495	2.4	0.9	0.9	Moderate	B	B4
	730994B00122	19	7.9	26	1.844	0	0	1.81	Moderate	B	B5
	730062B00121	19	7.3	24	1.997	0	0	1.93	Moderate	B	B6
	733075B00065	19	7.2	23.6	2.029	0	0	1.96	Moderate	B	B7
	730024B00113	19	6.1	20	2.351	0	0	2.24	Moderate	B	B8

[1] As defined in the Kentucky Transportation Cabinet bridge inventory.

[2] The letter P, as defined in the Kentucky Transportation Cabinet inventory, stands for a parallel bridge which is located westbound on I-24.

[3] The peak ground acceleration is the maximum bedrock acceleration at a designated embankment's site ($A_{maximum}$) during a seismic event.

[4] The slope height is the idealized height, at either ends of the embankment that results in the lowest C/D ratio. (Refer to Figure 6).

[5] The yield factor, Y, is the ratio of the minimum yield acceleration (A_y) to the peak ground acceleration ($A_{maximum}$).

A_y is the K_{hf} that corresponds to the obtained $(C/D)_{min.}$ ratio (based on Eq. (6) and Eq. (8).

[6] The displacement U of an embankment is calculated by Eq. (9).

[7] $(C/D)_{min.}$ ratio is derived from Eq. (1) and Eq. (7).

[8] The liquefaction potential is determined in accordance with the method shown in this chapter.

[9] The category of embankment behavior is defined in accordance with Table 6

[10] In general, a bridge embankment in category A with a ranking of A1 is more susceptible to damage than a bridge embankment with a ranking of A2 or A3. The same applies to categories B and C.

Table 7. Seismic ranking for I-24 bridge embankments in McCracken County for a 250-year event County, western Kentucky

One complete example of the calculation procedures to identify the seismic risk of a bridge embankment in McCracken County is provided in Zatar and Harik [16]. Similar procedures are followed in order to identify the seismic risk of all the 127 bridge embankments in all seven counties along I-24 in western Kentucky. Full details and results of the ranking and prioritization of the bridges along I-24 in western Kentucky are provided in the Kentucky Transportation report [11].

13. Summary and conclusions

This document describes the authors' efforts in addressing the technical component of embankment prioritization, and is well suited to a reliability-based model for seismic risk assessment. A methodology is presented to quickly conduct seismic assessment and ranking of bridge embankments in order to identify and prioritize those embankments that are highly susceptible to failure. The step-by-step methodology is provided in a flowchart that is specifically designed to ensure minimal effort on behalf of the engineer/researcher.

The proposed ranking model is useful for a quick sensitivity assessment of the effect of various site conditions, earthquake magnitudes, and site geometry on possible movement of a designated embankment. The methodology was applied on 127 bridge embankments on a priority route in western Kentucky in order to identify and prioritize the embankments, which are susceptible to failure. Data regarding soil types and depth of bedrock is not available for the majority of the 127 bridge embankments of I-24 in western Kentucky. However, obtaining detailed geo-technical investigations and sophisticated models are typically limited because of the associated cost and effort. The methodology outlines possible approaches to predict the unavailable information regarding a bridge embankment site. The embankment geometry, material, type of underlying soil, elevation of the natural ground line, and upper level of bedrock are the variables of each embankment. Seismic slope stability capacity/demand ratio, displacement, and liquefaction potential of each bridge embankment along I-24 in western Kentucky are estimated. Three categories are presented to identify the failure risk and provide a priority list of the embankments. The seismic vulnerability during projected 50-year, 250-year, and 500-year seismic events are obtained and the associated seismic performance criteria are examined. An example of seismic ranking and prioritization of bridge embankments along I-24 in McCracken County in western Kentucky is presented. The priority list enables decision makers to take appropriate actions.

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