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Analysis of Bridges for Seismic Hazard Mitigation in Kentucky

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Analysis of Bridges for Seismic Hazard Mitigation in Kentucky

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SYNOPSIS: The priority routes have been selected for Western Kentucky which shares the most hazardous New Madrid seismic zone. As the vital links on the priority routes, bridges need to be protected from collapse during earthquakes in order to maintain the access to the route for subsequent emergency traffic. In this paper, a support-loss type of bridge collapse due to earthquake induced abutment sliding is analyzed and corresponding criteria to this type of collapse is established. The analysis methods for existing bridge abutment are advanced. A computer program based on the methods is developed and applied to evaluate the potential earthquake induced damage of 276 bridges on the priority routes.

INTRODUCTION

Concern has grown in recent years over possible seismic activity of the New Madrid rift zone in the central United States, as shown in Figure 1. The New Madrid seismic zone is regarded by seismologists and disaster-response planners as the most hazardous earthquake zone east of the Rocky Mountains. According to seismologists' calculations, the probabilities of recurrence of sizable earthquakes in the New Madrid seismic zone will increase over the next 15 to 50 years. Western Kentucky is located in this high-risk seismic region.

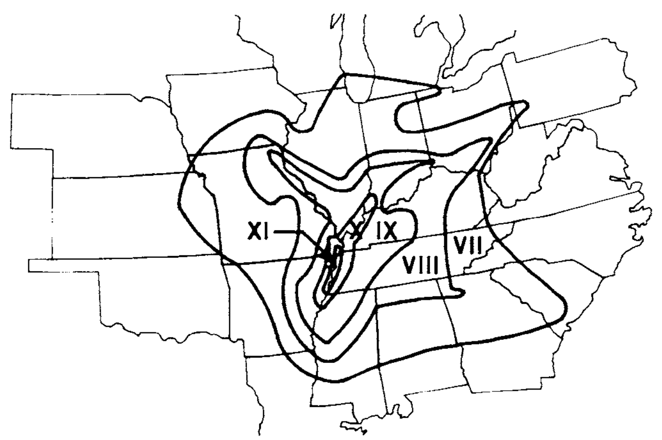

Figure 1. The New Madrid Seismic Zone

In 1987, the Kentucky Transportation Center began investigating the effect of earthquakes on Kentucky transportation system. The study area encompassed the 26 western-most counties in Kentucky. That area is most susceptible to earthquake damage since it is located in the New Madrid seismic zone. To permit transportation of post-quake emergency supplies and service into the affected area, some routes must remain passable. As one of the results of this investigation (Allen, Drnevich, Sayyedesadr and Fleckenstein, 1988), over 1,000 miles of highways in the region have been recommended as emergency or "priority" routes, as shown in Figure 2. These priority routes have been chosen to remain open and to provide vital transportation access after an earthquake has occurred. Also, it is anticipated that these routes would be the first routes repaired after an earthquake. All seismically significant features along the priority route have been surveyed and cataloged. Those features deemed seismically significant and cataloged are as follows: bridges, dams, pipelines, powerlines, high fills, cut slopes, buildings, faults, tanks, mines, trees, and traffic signs. Further studies have been extended to the seismic rating and seismic analysis of 276 bridges on the priority routes.

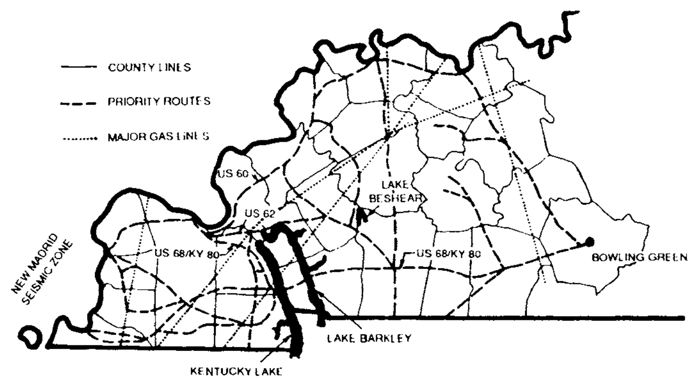

Figure 2. Study Area and Priority Routes

Bridges are by far the most significant and important features on the priority routes. Bridges form the critical links in the highway network and are most susceptible to earthquakes. If any bridge suffers major damage, such as span collapse, during an earthquake, the access to the route will be severed. On the other hand, bridges also represent the greatest economic risk if destroyed or damaged. Therefore, bridges on the priority routes must be protected from collapse to ensure the safety of motorists and vehicles on the bridges and maintain access over the bridges for subsequent emergency traffic. In order to prevent the collapse and minimize the hazard, some bridges require some form of seismic retrofitting. This paper discusses the criteria and the analysis procedures for estimating support-loss type of bridge collapse due to earthquake induced abutment sliding. If a bridge has the potential of collapse according to this analysis, it has priority of retrofitting.

Abutments support the ends of bridge spans and provide the lateral support for the soil or rock upon which the roadway rests immediately adjacent to the bridge. They are the most critical elements of a bridge during an earthquake. As the numerous cases of damage or failure to bridges induced by abutment displacement or failure have clearly demonstrated in prior earthquakes, the damage of an abutment is mainly associated with the movement and failure induced by the strong earthquake ground motion and high seismic lateral earth pressure. Severe abutment damage or movement may cause loss of bridge spans and hence cut the access to the route. In this paper, a support-loss type of bridge collapse due to earthquake induced abutment sliding is analyzed and corresponding criteria to this type of collapse is established. The forces involved in the movement of abutments during an earthquake are discussed and the analyses methods for existing bridge abutments are advanced. A spread sheet program based upon these methods has been developed and used to estimate potential earthquake damages of 276 bridges on the priority routes.

SUPPORT-LOSS TYPE OF BRIDGE COLLAPSE

In this paper, a bridge span failing due to lack of support is defined as a support-loss type of collapse. Piers may vibrate and abutments may slide when subjected to earthquake induced ground motion. Either pier vibration or abutment sliding can cause the loss of support length, and trigger the collapse of bridge superstructure.

Most of the subject bridges in this study are multiple span, simply beam structures which are vulnerable to earthquakes. Usually, the bearings at two abutments are fixed ones. And, at least one of the piers adjacent to one of the abutments will have two expansion bearings, which allow for relatively free horizontal movement. The displacement at the abutment will be transmitted totally to the superstructure of the end span if the superstructure is assumed to be rigid. Because of the expansion bearings, the superstructure of the end span will move freely in the direction of the abutment sliding and push the superstructure of the adjacent span with the same displacement in the same direction. If the total sliding displacement of an abutment during an earthquake is greater than the support length of this adjacent span , the superstructure of this span will consequently be pushed off the top of pier and span will collapse completely. Since the abutment and pier will respond to the earthquake motion simultaneously, the most critical case occurs when the direction of the earthquake induced vibration of the pier is just opposite to the direction of abutment sliding as shown in Figure 3.

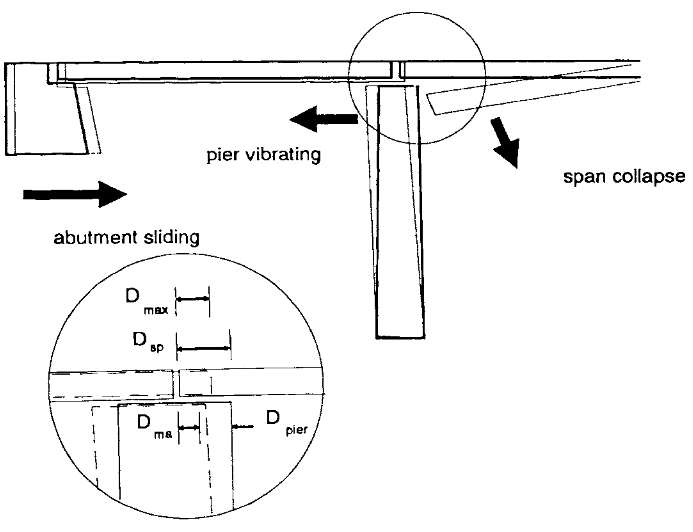

Figure 3. Support-loss Type of Collapse

The criterion to this critical condition for support-loss type of collapse may be expressed as:

$$D_{max} < D_{ma} = D_{sp} - D_{pier} \qquad \text{no collapse}$$
$$D_{max} \geq D_{ma} = D_{sp} - D_{pier} \qquad \text{collapse}$$

where:

D_{max} = maximum relative sliding displacement of abutment
D_{ma} = maximum allowable sliding displacement of abutment
D_{sp} = support length of superstructure on the pier top
D_{pier} = maximum displacement at pier top during vibration

ABUTMENT SLIDING DURING EARTHQUAKES

ABUTMENT SLIDING

For an existing abutment, the static resistance against sliding has a minimum factor of safety of 1 to keep the statically stable condition. During earthquakes, however, abutment's pseudostatic factor of safety against sliding could be less than 1 because of the earthquake induced change of forces acting on the abutment. As a consequence, those abutments which did not slide under the static condition might have the potential of sliding during earthquakes. This potential sliding might cause collapse of the bridge span according to aforementioned criteria. It is important to know whether sliding will occur during an earthquake and what the magnitude of the sliding would be. A criterion is established and a pseudo-static method is developed to determine the pseudo-static resistance against earthquake induced sliding.

Figure 4 shows the force diagrams of free body abutment under both static and seismic condition. Following section discusses the forces acting on abutments during earthquakes.

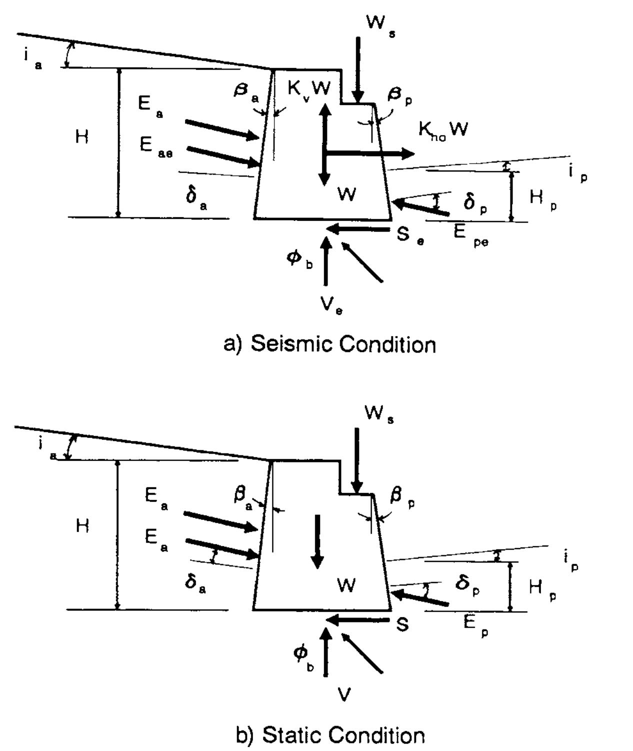

Figure 4. Force Diagrams under Seismic and Static Condition

where:

- W_s = superstructure transmitted vertical load
- W = weight of abutment
- E_s = resultant of equivalent earth pressure due to wheel load on the backfill adjacent to the abutment
- E_{ae} = resultant of active seismic earth pressure
- E_a = resultant of active static earth pressure
- E_{pe} = resultant of passive seismic earth pressure
- E_p = resultant of passive static earth pressure
- K_hW = horizontal earthquake inertia force
- K_vW = vertical earthquake inertia force
- V_e = seismic total vertical resultant at abutment base
- V = static total vertical resultant at abutment base
- S_e = seismic total horizontal resultant at abutment base
- S = static total horizontal resultant at abutment base
- ϕ_b = friction angle at abutment base

* All the forces are per unit abutment length.

FORCES ACTING ON ABUTMENTS

Earth Pressure

Commonly known as the Mononobe-Okabe analysis, the seismic earth pressure on a retaining wall type of abutment was derived based upon the following assumptions:

a. Only plane failure surfaces are considered;
b. The abutment is free to move sufficiently to produce minimum active seismic earth pressure;
c. The backfill is cohesionless soil and fully drained;
d. The soil behind the abutment behaves as a rigid body.

The M-O analysis was described in detail by (Seed and Whitman, 1970) and (Davies, Richards, and Chen 1986). Referring to Figure 4, the total seismic thrust due to static and earthquake active earth pressures is

$$E_{ae} = \frac{1}{2}\gamma_a H^2 (1 - K_v) K_{ae} \tag{1}$$

in which

$$K_{ae} = \frac{\cos^2(\phi_a - \theta - \beta_a)}{\cos\theta \cos^2\beta_a \cos(\delta_a+\beta_a+\theta)\left[1+\left(\dfrac{\sin(\phi_a+\delta_a)\sin(\phi_a-\theta-i_a)}{\cos(\delta_a+\beta_a+\theta)\cos(i_a-\beta_a)}\right)^{1/2}\right]^2}$$

and

$$\theta = \tan^{-1}\left(\frac{K_h}{1-K_v}\right)$$

where:

- E_{ae} = resultant of active seismic pressure
- γ_a = unit weight of soil behind the abutment
- H = height of abutment
- K_{ae} = active seismic earth pressure coefficient
- K_h = horizontal earthquake acceleration coefficient
- K_v = vertical earthquake acceleration coefficient
- ϕ_a = friction angle of back fill soil
- δ_a = friction angle between abutment back wall and soil
- β_a = vertically inclined angle of abutment back wall
- i_a = backfill slope angle

When two abutments slide toward each other, the passive resistance by berms or slope protections can be expressed as follows:

$$E_{pe} = \frac{1}{2}\gamma_p H_p^2(1 - K_v) K_{pe} \tag{2}$$

in which

$$K_{pe} = \frac{\cos^2(\phi_p - \theta + \beta_p)}{\cos\theta \cos^2\beta_p \cos(\delta_p-\beta_p+\theta)\left[1-\left(\dfrac{\sin(\phi_p+\delta_p)\sin(\phi_p-\theta+i_p)}{\cos(\delta_p-\beta_p+\theta)\cos(i_p-\beta_p)}\right)^{1/2}\right]^2}$$

and

$$\theta = \tan^{-1}\left(\frac{K_h}{1-K_v}\right)$$

where:

- E_{ap} = resultant of passive seismic pressure
- γ_p = unit weight of soil in berm or slop protection
- H_p = height of berm or slope protection
- K_{pe} = passive seismic earth pressure coefficient
- ϕ_p = friction angle of soil in berm or slope protection
- δ_p = friction angle between abutment front wall and soil
- β_p = vertically inclined angle of abutment front wall
- i_p = slope angle of berm or slope protection

In order to see more easily the effect of earthquakes on active and passive earth pressures, K_{ae} and K_{pe} can be normalized by dividing their static values K_a and K_p to give magnification ratios F_a and F_p. The static earth pressures coefficients K_a and K_p can be obtained approximately from eq.(1) and eq.(2) by assigning $K_h=K_v=0$ (static condition).

$$F_a = \frac{K_{ae}}{K_a} \tag{3}$$

$$F_p = \frac{K_{pe}}{K_p} \tag{4}$$

where:

- F_a = magnification ratio for active earth pressure
- K_a = active static earth pressure coefficient
- F_p = magnification ratio for passive earth pressure
- K_p = passive static earth pressure coefficient

Figure 5 and Figure 6 show the influences of soil friction angles and earthquake acceleration coefficients on active and passive magnification ratios.

Earthquake acceleration coefficient has a large effect on the earth pressures. While the passive earth pressure decreases with increasing earthquake acceleration coefficient, the active earth pressure increases. As compared to the static condition, active seismic earth pressure is greater than the static one, and passive seismic earth pressure is less than the static one. On the other hand, the value of soil friction angles has little effect on the magnification ratios until quite suddenly, over a short range of ϕ, F_a and F_p change rapidly and become infinite for specific critical value of ϕ_{cr}. This condition may be presented as

$$\phi \geq \phi_{cr} = i + \theta = i + \tan^{-1}\left(\frac{K_h}{1-K_v}\right) \tag{5}$$

This is also the necessary condition under which M-O analysis could have a real solution. If the stated condition is not satisfied, this implies that an equilibrium condition

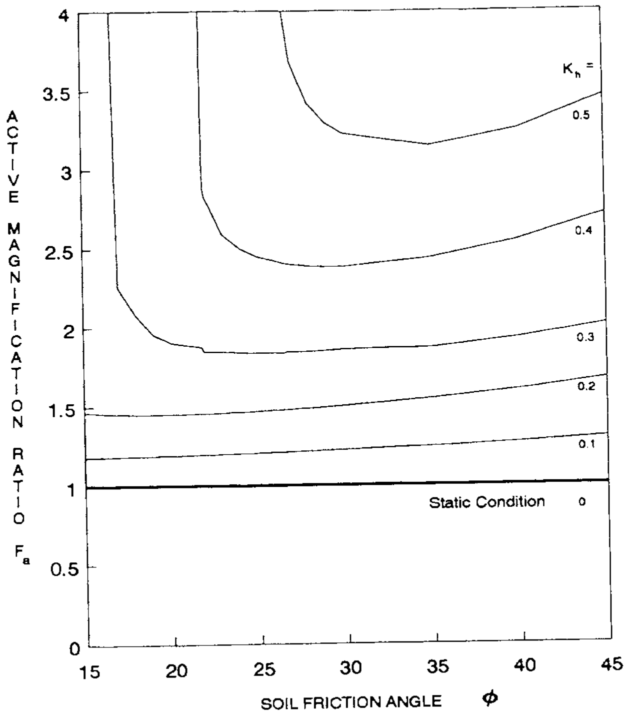

Figure 5. Active Magnification Ratios

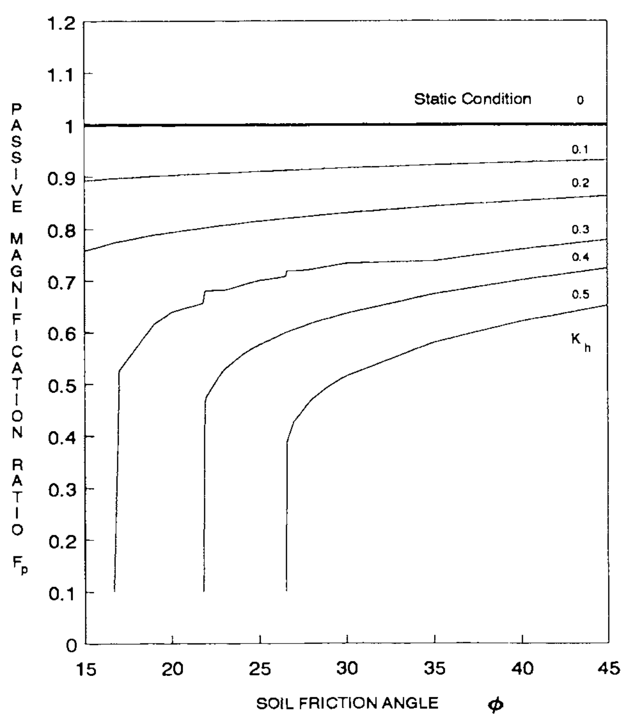

Figure 6. Passive Magnification Ratios

will not exist. The limiting value of K_{hcr} which provides an absolute upper bound for the seismic acceleration which may be transmitted to any structure whatsoever that is constructed on a soil having given strength characteristics can be given by

$$K_{hcr} = (1 - K_v) \tan (\phi - i) \tag{6}$$

where:
K_{hcr} = critical earthquake acceleration coefficient

For the cases involved in this study, ϕ ranges from approximately 25 to 35 degrees and maximum earthquake acceleration coefficient is 0.2. Calculating K_{hcr} from eq. (6) by assuming K_v=0 and i=0, the minimum K_{hcr} is 0.47. It is greater than K_h of 0.2 for the study area. Therefore, the Mononobe-Okabe analysis is valid for this study. Some of the values of K_{hcr} are shown in Figure 8.

Gravity Force

The weight of an abutment acting at its center of gravity is the major force in maintaining its stability against sliding. In this study, the weight per unit project length of abutment is used in the force equilibrium analysis. The abutment weight per unit project length is defined as:

$$W = \frac{\text{Total abutment weight}}{\text{Total project length}} \tag{7}$$

The Load Transmitted from the Superstructure

The reactions from the superstructure may be transmitted to the bridge seat of an abutment through the bearings in several ways. Roller and rocker bearings providing for expansion and contraction are assumed to transmit only vertical forces to the abutment. On the other hand, fixed bearings at the end of the bridge subject the abutment to vertical as well as horizontal reactions. The loads from the superstructure are assumed to be distributed over the entire length of the front wall of an abutment. Only the vertical reaction is taken into account in this analysis. This vertical force transmitted from the superstructure per length of abutment is

$$W_s = \frac{\text{Total superstructure transmitted vertical load}}{\text{Total project length of abutment}} \tag{8}$$

Additional Earth Pressure due to Wheel Loads

The active earth pressure against the back of the abutment is increased whenever wheel loads are transmitted to the backfill immediately behind the abutment. The magnitude of this additional active earth pressure depends upon the properties of soil, position of the wheel and magnitude of the wheel load. This earth pressure increase should be considered in the analysis since it will increase the tendency for sliding of the abutment. Usually, wheel loads are assumed to be equivalent to a uniformly distributed load, q, often taken as 240 psf for H-10 highway loading (Peck, Hanson, and Thornburn, 1974). This uniform surcharge is commonly considered as an additional backfill layer, as shown in Figure 7, having a height H_s = q/γ, where γ is the unit weight of backfill material.

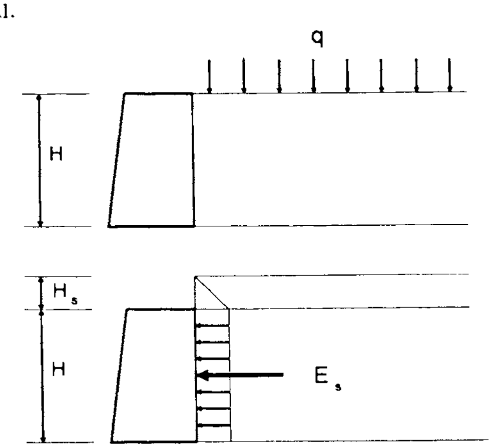

Figure 7. Wheel Load Induced Equivalent Earth Pressure

The corresponding additional horizontal earth pressure is assumed to be uniformly distributed across the height of the abutment with a magnitude of $K_a H_s$, where K_a is the static active earth pressure coefficient. The resultant of this additional earth pressure E_s may be assumed to act at the mid height of the abutment and may be calculated by

$$E_s = K_a \gamma_a H_s H = K_a q H \tag{9}$$

where:

q = equivalent uniformly distributed wheel loading

Earthquake Inertia Forces

The earthquake inertia forces are induced by the ground motion due to earthquakes. Both horizontal earthquake inertia force ($K_h W$) and vertical earthquake inertial force ($K_v W$) may contribute to the potential sliding of abutment during earthquakes.

Static and Seismic Conditions

The comparison of the forces related to the sliding of an abutment under static conditions and under seismic conditions is summarized hereinafter. In most cases, total resisting forces under seismic conditions are less than those under the static conditions while total driving forces under seismic condition are greater than those under static condition. As a consequence, the factor of safety for sliding in seismic condition will be less than that in static condition. The abutment, therefore, is more likely to slide during earthquakes.

	Seismic Conditions		Static Conditions
Driving Forces	$E_s \cos(\delta_a+\beta_a)$	=	$E_s \cos(\delta_a+\beta_a)$
	$E_{ae} \cos(\delta_a+\beta_a)$	>	$E_a \cos(\delta_a+\beta_a)$
	$E_{pe} \sin(\delta_p-\beta_p)$	<	$E_p \sin(\delta_p-\beta_p)$
	$K_h W$	>	0
	$K_v W$	>	0
Resisting Forces	$E_s \sin(\delta_a+\beta_a)$	=	$E_s \sin(\delta_a+\beta_a)$
	$E_{ae} \sin(\delta_a+\beta_a)$	>	$E_a \sin(\delta_a+\beta_a)$
	$E_{pe} \cos(\delta_p-\beta_p)$	<	$E_p \cos(\delta_p-\beta_p)$
	W	=	W
	W_s	=	W_s
Factor of Safety	FS_e	<	FS

MAXIMUM RESISTANCE AGAINST SLIDING

Define a maximum resistance coefficient K_{ho} corresponding to a steady acceleration $K_{ho}g$, where g is the acceleration of gravity, acting in the proper direction which would just overcome the resistance to the sliding of abutment. For a given value of horizontal earthquake acceleration, $K_h g$, the following criterion is established.

If $K_h g \geq K_{ho} g$, sliding will take place.

If $K_h g < K_{ho} g$, sliding will not occur.

The value of K_{ho} for a given abutment may be calculated through the force equilibrium shown in the Figure 4 with K_h substituted by K_{ho}.

$$V_e = W_s + (1-K_v)W + (E_s+E_{ae})\sin(\delta_a+\beta_a) - E_{pe}\sin(\delta_p-\beta_p) \tag{10}$$

$$S_e = K_{ho}W + (E_s+E_{ae})\cos(\delta_a+\beta_a) - E_{pe}\cos(\delta_p-\beta_p) \tag{11}$$

$$S_e = V_e \tan\phi_b \tag{12}$$

Note that E_{ae} and E_{pe} are functions of K_{ho} (See eq.(1) and eq. (2) with $K_h=K_{ho}$), it is very difficult to derive an explicit expression for the direct calculation of K_{ho} from the equations. A rough but conservative estimate of K_{ho} is given in Figure 8. A dimensionless abutment weight is employed as follows,

$$\omega = \frac{W}{1/2\ \gamma_a H^2} \tag{13}$$

where:

ω = dimensionless abutment weight

In Figure 8, following assumptions have been adopted:

a) $W_s=0$; b) $i_a=i_p=\beta_a=\beta_p=0$; c) $\phi_b=\phi_a=\phi_p$; d) $E_s=0$; e) $\delta_a=\delta_p=\phi_a/2=\phi_p/2$; f) $E_{pe}=0$; and g) $K_v=0$.

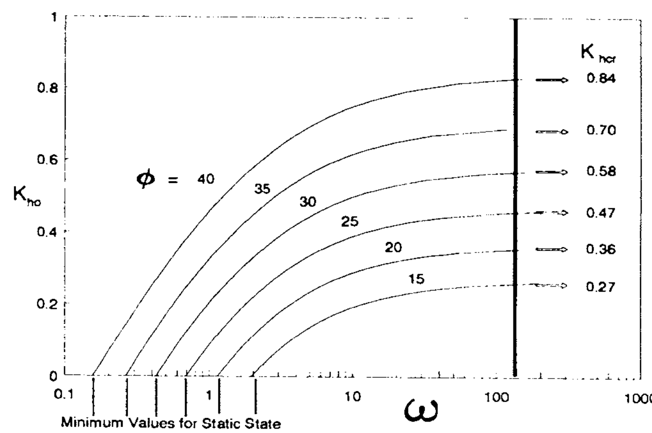

Figure 8. Maximum Resistance Coefficient Against Sliding

If the value of K_h is less than K_{ho} shown in Figure 8 for a given abutment having a known ω and ϕ, the abutment will not slide due to an earthquake. However, if K_h is greater than K_{ho}, it does not necessarily mean that the abutment will slide during an earthquake since the K_{ho} is conservative without considering the positive effects of W_s and E_{pe}. The sliding might occur in some of the abutments and might not in some others. Therefore, K_{ho} should be used only for a rough estimate and may not be used for further calculations such as the magnitude of the sliding, etc. A more accurate and simple method is presented in the following sections.

REQUIRED MINIMUM WEIGHT OF ABUTMENT

THE MAXIMUM DISPLACEMENT OF ABUTMENT

The total relative displacement of a retaining wall depends on the earthquake acceleration, velocity time history, and maximum resistance coefficient of the wall, K_{ho}.

Newmark, Franklin and Chang computed the maximum displacement response of several natural and synthetic earthquake records by scaling all records at a normalized maximum acceleration of 0.5g (A=0.5) and a normalized maximum ground velocity of 30 in/sec. An upper bound envelope curve of all recorded maximum displacements in terms of the ratio of the maximum resistance coefficient, K_{ho}, to the maximum earthquake acceleration coefficient of A, (Newmark 1965) and (Franklin and Chang 1977). An approximation to the curve for relatively low displacement is expressed in the following relation for any consistent set of units.

$$D_{max} = 0.087 \frac{v^2}{A\,g} \left[\frac{K_{ho}}{A}\right]^{-4} \tag{14}$$

where:

D_{max} = maximum relative displacement of the wall subjected to an earthquake record with A and v

A = maximum acceleration coefficient of an earthquake

v = maximum ground velocity of an earthquake

Since this expression is obtained from the envelope curve based upon the data encompassed most of the big recorded earthquakes in California and other locations, it may reasonably be used directly to estimate the maximum displacement for an earthquake in many other areas where the possible acceleration coefficient, A, and ground velocity, v, are less than 0.5 and 30 in/sec, respectively.

CALCULATION OF REQUIRED MINIMUM WEIGHT

Corresponding to the criterion described previously, an abutment is not allowed to have a sliding displacement more than D_{ma} in order to prevent the span-loss type of collapse. In other words, a minimum weight of abutment is required to ensure that the possible sliding displacement is less than the maximum allowable displacement D_{ma}. For a given potential earthquake having a possible A and v, the reference resistance coefficient K_{href} corresponding to the allowable maximum displacement D_{ma} may be obtained by converting eq. (14).

$$K_{href} = 0.543\, A \sqrt[4]{\frac{v^2}{Ag\, D_{ma}}} \tag{15}$$

where:

K_{href} = reference resistance coefficient under which the abutment will have sliding displacement of D_{ma}

This indicates that an abutment subject to earthquake motion having a horizontal acceleration of K_{href} g will have a displacement of D_{ma}. Since any displacement greater than D_{ma} will lead to span collapse, the abutment must have a certain amount of weight which will prevent the abutment from having this much displacement. This certain weight of abutment is defined here as the required minimum weight, W_{req}. Therefore the criteria in terms of D_{ma} can be rewritten to a criterion in terms of W_{req}.

$D_{max} < D_{ma}$	$\Rightarrow$	$W > W_{req}$	no collapse
$D_{max} \geq D_{ma}$	$\Rightarrow$	$W \leq W_{req}$	collapse

where:

W = actual weight of abutment per unit length

W_{req} = required minimum weight of abutment per unit length

Assuming that an abutment will slide during an earthquake, it should have sufficient weight to limit the resulting displacement within an allowable value of D_{ma}, thus preventing support-loss type of collapse as defined in this paper. If the actual weight of abutment is less than the required minimum weight, the abutment will have a sliding displacement sufficiently large to cause a support-loss type of collapse. The formula for calculating W_{req} may be derived from eq.(10), (11), and (12).

$$\begin{aligned} W_{req} = &+ \left[\frac{\cos(\delta_a+\beta_a) - \sin(\delta_a+\beta_a)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right] E_{ae} \\ &- \left[\frac{\cos(\delta_p-\beta_p) - \sin(\delta_p-\beta_p)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right] E_{pe} \\ &- \left[\frac{\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right] W_s \\ &+ \left[\frac{\cos(\delta_a+\beta_a) - \sin(\delta_a+\beta_a)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right] E_s \end{aligned} \tag{16}$$

in which

$$\theta_{ref} = \tan^{-1}\left[\frac{K_{href}}{1-K_v}\right]$$

From eq. (16), the effects of various types of force on the required minimum weight may be clearly seen. The seismic active earth pressure and wheel load induced equivalent active earth pressure are the forces leading to sliding and therefore increase the required weight as they increase. On the other hand, the seismic passive earth pressure and superstructure transmitted vertical load are the forces resisting sliding and therefore decrease the required weight as they increase. Separating the earthquake affected factors from the four terms in eq.(16), the equation may be rewritten as

$$W_{req} = C_{ae}\,\gamma\, H^2 - C_{pe}\,\gamma\, H_b^2 - C_{ws}\, W_s + C_s\, q\, H \tag{17}$$

in which

$$C_{ae} = \left[\frac{\cos(\delta_a+\beta_a) - \sin(\delta_a+\beta_a)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right]\frac{K_{ae}}{2}(1-K_v)$$

$$C_{pe} = \left[\frac{\cos(\delta_p-\beta_p) - \sin(\delta_p-\beta_p)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right]\frac{K_{pe}}{2}(1-K_v)$$

$$C_{ws} = \left[\frac{\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right]$$

$$C_s = \left[\frac{\cos(\delta_a+\beta_a) - \sin(\delta_a+\beta_a)\tan\phi_b}{(1-K_v)\tan\phi_b - \tan\phi_{ref}}\right] K_a$$

The values of the coefficients C_{ae}, C_{pe}, C_{ws}, C_s versus maximum allowable displacement D_{ma} for given soil friction angles ϕ are given in Figure 9 to Figure 12. For all the practical purposes, these charts are for the situations when:

a) A=0.2; b) V=30A; c) K_v=0; d) $\phi_b=\phi_a=\phi_p$;

e) $\delta_a=\delta_p=\phi_a/2=\phi_p/2$; and f) $i_a=i_p=\beta_a=\beta_p=0$.

By these chart and eq. (17), the required minimum weight of the abutment corresponding to the maximum allowable displacement D_{ma} may be calculated. After comparing calculated W_{req} with the actual W, the possibility of span-loss type of collapse may be estimated.

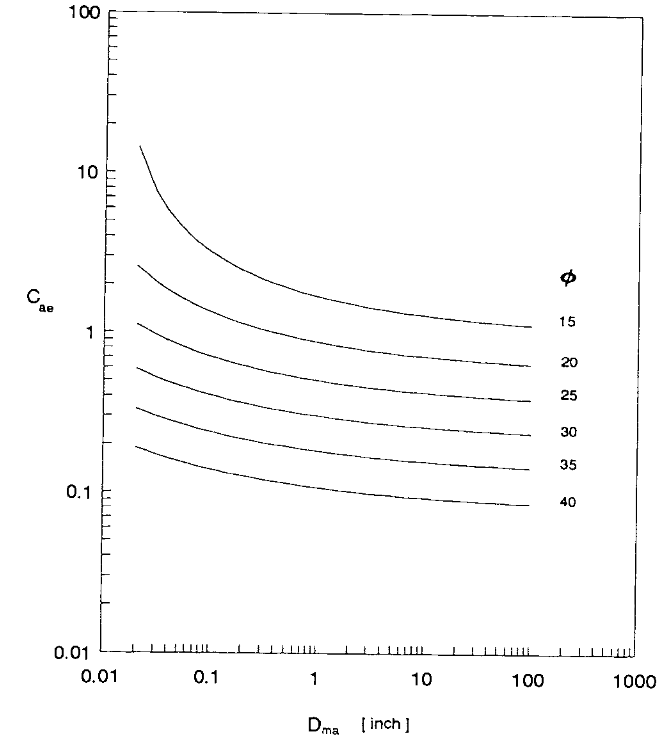

Figure 9. Coefficient C_{ae} Versus D_{ma}

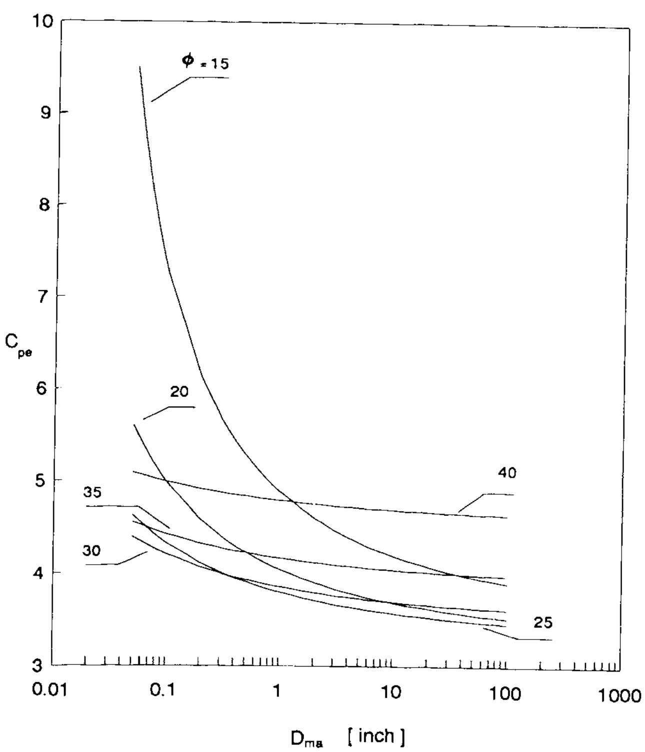

Figure 10. Coefficient C_{pe} Versus D_{ma}

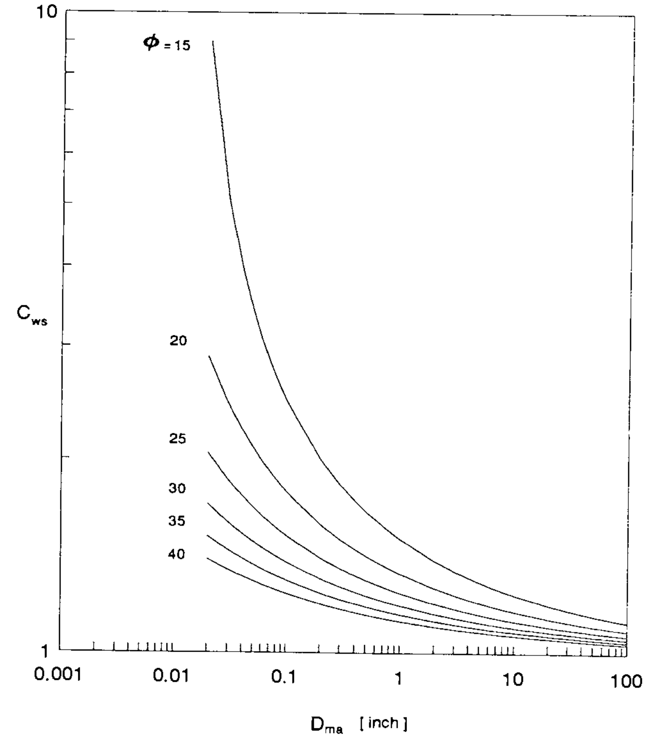

Figure 11. Coefficient C_{ws} versus D_{ma}

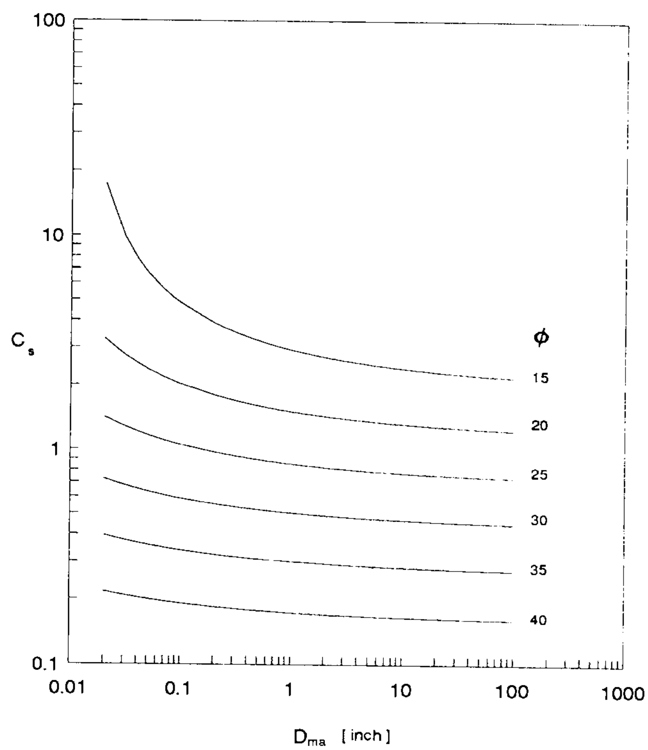

Figure 12. Coefficient C_s Versus D_{ma}

BRIDGE ANALYSES

MAXIMUM ALLOWABLE ABUTMENT SLIDING DISPLACEMENT

Piers in vibration due to earthquake induced ground motion can be simplified as a single degree of freedom system. According to theory of structure dynamics, the maximum earthquake deflection at the top of pier D_{pier} can be determined by response spectra developed from the synthetic seismograms of numerically modeled large New Madrid earthquake in the sites of Western Kentucky. The pier earthquake deflection analysis is described in detail by (Ouyang, Allen, Drnevich, and Fleckenstein, 1990). The length of support at the top of pier D_{sp} can be obtained from "as-built" bridge plans or from field investigations. The maximum allowable sliding displacement of abutment D_{ma} can be calculated by $D_{ma} = D_{sp} - D_{pier}$. Notice that D_{pier} might be greater than D_{sp} and makes D_{ma} less than 0. This means that the dynamic deflection of pier at top is sufficiently great to cause the suport-loss type of collapse regardless of the response of the abutment.

COMPUTER PROGRAM

A spread sheet computer program has been developed to perform the analyses described in this paper. Material properties, geometric properties, weight of abutment, superstructure transmitted load, and maximum allowable sliding displacement of abutment are required as input. The output of calculations include a) the required minimum weight of abutment to prevent support-loss collapse; b) capacity/demand ratio of abutment weight; and c) the conclusion of analyses, i.e. presumed safe or potentially unsafe.

RESULTS OF ANALYSES

139 bridges out of 276 bridges on the referenced priority routes have retaining wall type of abutments. Among those 139 bridges, 57 are single span bridges which will not have the support-loss collapse as defined in this paper. These single span bridges are presumed to be safe according to aforementioned criteria. 82 multiple span bridges with retaining wall type of abutment have been analyzed by using the computer program based on the analysis procedures and criteria provided in this paper. The analyses results indicate that 14 bridges (17% of 82 analyzed bridges) may have the potential possibility of support-loss type of collapse due to possible earthquake induced abutment sliding. The analysis is on the conservative side because some of the positive factors such as the strong lateral links between the superstructure and abutment have not been taken into account and also because the most critical conditions are always employed for the analysis when the exact behavior is not known or the necessary data are not available.

SUMMARY AND CONCLUSIONS

A system of priority routes for use after an earthquake has been selected by the Kentucky Transportation Center for the western part of Kentucky. It was necessary to analyze nearly 300 bridges to determine whether they might succumb to the possibility of support-loss type failures. This paper discussed the criteria and analysis procedures for estimating the support-loss type of bridge collapse due to earthquake induced abutment sliding.

A support-loss type of collapse has been formulated and the corresponding criterion has been established. If the sliding displacement of an abutment plus the dynamic deflection at the top of a pier adjacent to the abutment are greater than the support length of superstructure on the pier top, the span is likely to collapse and hence the route will be cut off.

The most important forces acting on an abutment during an earthquake are seismic active and passive earth pressures, superstructure transmitted load, gravity force of the abutment, wheel load induced equivalent active earth pressure, and horizontal and vertical inertia forces. The effects of different forces on and abutment sliding also have been shown.

The maximum dynamic resistance against the sliding of an abutment during an earthquake is analyzed and a conservative and approximate method for estimating the maximum dynamic resistance coefficient has been provided. If the potential earthquake horizontal acceleration coefficient is greater than the maximum dynamic resistance coefficient, the abutment is likely to slide during an earthquake.

The procedures for calculating the required minimum weight of an abutment is advanced and a formula along with several charts are presented for the practical use. The support-loss type of collapse is not likely to occur when the actual weight of an abutment is greater than the required minimum weight.

A spread sheet program has been developed and applied to analyze 83 multiple span bridges on the priority routes which have retaining wall type of abutments. The analyses results indicate that 14 bridges (17% of 83 bridges) may have potential support-loss type of collapse due to abutment sliding during earthquakes.

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DISCLAIMER

The contents of this paper reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the University of Kentucky, the Kentucky Transportation Cabinet, nor the Federal Highway Administration.

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