Improved Screening Procedure for Seismic Retrofitting of Highway Bridges

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Recent damaging earthquakes in California and elsewhere have demonstrated once again the seismic vulnerability of highway bridges in the United States. Retrofitting programs for correcting deficiencies in existing bridges have been proposed since the 1970s, but until very recently only California has been active in the field. In 1983 FHWA published a set of retrofitting guidelines for bridges; recently they were revised to reflect advances made in the state of the art during the past decade. The improved screening procedure, which has been recommended in the revised FHWA manual, is presented. Differences between the old and new procedures include a new priority-ranking process, revised seismic performance categories, expanded definitions for bridge importance, and new flow charts to illustrate and clarify the assignment of structure vulnerabilities.

The recent occurrence of damaging earthquakes in California, Costa Rica, and the Philippines has demonstrated, once again, the vulnerability of highway bridges that have not been designed adequately to resist seismic loads. Although seismic design codes have been in place in the United States for a number of years, more than 75 percent of the U.S. bridge inventory was constructed before these codes became effective. There is therefore a pressing need to develop and implement appropriate seismic retrofit programs throughout the United States.

The practice of seismic retrofitting is, however, limited to a few states, primarily California, Illinois, Nevada, and Washington. This situation exists partly because bridge owners are faced with many competing demands on their limited resources. But it is also due to the limited availability of tools and technologies for retrofitting. It is therefore of importance that bridge owners have rational methodologies for screening bridges for their seismic risk and ranking those that are deficient according to their vulnerability, importance, cost, and other societal factors.

One of the first attempts to develop a rational prioritization methodology was undertaken by the Applied Technology Council (ATC) in the early 1980s. The ATC-6-2 project was conducted for FHWA and resulted in the publication of the 1983 FHWA report *Seismic Retrofitting Guidelines for Highway Bridges* (*1*). The guidelines introduced a preliminary screening procedure and a method for evaluating an existing bridge. They also described potential retrofitting measures for the most common seismic deficiencies.

Since 1983 several states have developed their own screening and priority-ranking procedures. Usually these have been based on the ATC-6-2 methodology, augmented by additional parameters and, in some cases, nontechnical considerations. Two papers by Buckle have summarized the preliminary screening and prioritization procedures in use by five states as of 1992 (*2,3*).

Since then, several other states have begun seismic retrofit programs and rapid screening procedures have also been proposed for bridges in Canada (*4*).

In the 10 years since publication of the FHWA guidelines, the state of the art in seismic retrofit has advanced substantially. As a consequence, the FHWA publication has been revised and reissued as a manual for the seismic retrofitting of highway bridges (*5*). The revision reflects experience gained with the use of the 1983 guidelines as well as new knowledge acquired through research and earthquake reconnaissance studies. It also reflects recent changes in seismic design philosophy that have been proposed for the design of new highway bridges under projects sponsored by AASHTO through NCHRP (Project 20-7, Task 45, for the revision of current seismic design criteria and Project 12-33 for limit state design specifications), and by the California Department of Transportation (Caltrans) through the ATC (Project ATC-32, for the review of Caltrans seismic bridge design specifications).

As part of this review, the screening procedures in the 1983 guidelines were examined and modified as appropriate. This paper summarizes these modified procedures, which are based on the previous methodology but refined as necessary to include a new priority-ranking process, revised seismic performance categories, expanded definitions for bridge importance, and new flow charts to illustrate and clarify the assignment of structure vulnerabilities. For completeness of presentation, some of the material in this paper is taken directly from the 1983 guidelines; the pioneering work by the authors of these earlier guidelines is again recognized.

SEISMIC RETROFITTING OF HIGHWAY BRIDGES

Not all bridges in a highway system can be retrofitted simultaneously; instead, those bridges with the highest priority should be retrofitted first. The screening and ranking of bridges for retrofitting requires not only consideration of the engineering factors but also an appreciation for the economic, social, administrative, and practical aspects of the problem. But it should always be remembered that seismic retrofitting is only one of several possible courses of action. Others are closing the bridge, replacing the bridge, and taking no action at all and accepting the risk of seismic damage.

Bridge closure (or replacement) usually is not justified by seismic deficiency alone and generally will be considered only when other deficiencies exist. Therefore, for all practical purposes, a choice must be made between retrofitting and accepting the seismic risk. This choice will depend largely on the importance of the bridge and on the cost and effectiveness of the various retrofitting alternatives. If the cost is high and the bridge is critically important, retrofit (or even replacement) may be the best strategy. If the cost is high and importance is not an issue, accepting the risk may be the most attractive option.

Regardless of the outcome, bridges must first be screened and those found to be deficient subjected to a second, more detailed evaluation. If a bridge is still considered vulnerable, retrofit measures are designed and cost data obtained. At this point the decision to proceed with retrofitting must be made considering cost, remaining useful life, importance, and other socioeconomic factors. In general, therefore, the seismic retrofitting process can be divided into the following three major steps:

1. Preliminary screening,
2. Detailed evaluation, and
3. Design of retrofit measures.

This paper describes only the first step: the preliminary screening procedure as recommended in the revised FHWA manual for seismic retrofitting (*5*).

PRELIMINARY SCREENING METHODOLOGIES

The intent of a preliminary screening methodology is to develop a prioritization scheme on which to base a retrofit program. The methodology requires access to, or a compilation of, a seismic inventory of all bridges to be screened, followed by the execution of one of several possible numerical rating (or ranking) schemes. Since not all the issues can be reduced to a numerical factor, a critical review of the results is usually undertaken and other factors, such as redundancy and economic constraints, are taken into account when a prioritized list is finally assembled.

Buckle (*2,3*) notes that many screening and prioritization schemes in use today include three important components:

- Seismicity of the bridge site,
- Vulnerability of the structural system, and
- Importance of the bridge.

These schemes usually address each of these variables separately by requiring that an importance, seismicity, and vulnerability rating be calculated for each bridge. These individual ratings are then combined to arrive at an overall seismic rating.

More recently, Basöz et al. have proposed a methodology to priority rank highway bridges for seismic retrofitting on the basis of risk (*6*). Risk, in this approach, is expressed primarily as a function of the expected dollar loss. The vulnerability and importance of a bridge are the two main criteria used for the overall ranking purposes. A lifeline network analysis is then used to integrate the vulnerability and importance criteria.

The revised FHWA manual described earlier recommends a modified screening and prioritization scheme in which the quantitative variables (seismic, geotechnical, and structural vulnerabilities) are separated from the qualitative factors (importance and other socioeconomic issues) in a two-step process. To do so, the ranking system requires first the calculation of a bridge rank that is based on engineering factors and then the assignment of a priority index based on this rank and socioeconomic (e.g., importance) and nonseismic issues. Figure 1 illustrates this screening procedure as it might apply to bridges in different seismic performance categories (SPCs).

Bridge Classification

Before seismic retrofitting can be undertaken for a group of bridges, they should first be classified according to their SPC. This classification is determined by a combination of seismic hazard and structure importance.

Seismic hazard may be quantified by the acceleration coefficient (A); when multiplied by the acceleration due to gravity (g), the product, Ag, represents the likely peak horizontal ground acceleration that will occur due to an earthquake sometime within a 475-year period. More rigorously, this acceleration has a 10 percent probability of being exceeded within a 50-year time frame (*7*). Maps showing the distribution of A throughout the United States are given elsewhere (*5,7*).

Bridge importance is not so readily quantified. Two importance classifications are specified in the FHWA manual: essential and standard. *Essential* bridges are those that should continue to function after an earthquake or that cross routes that should continue to operate immediately after an earthquake. All other bridges are classified as *standard*. The determination of the importance classification of a bridge is subjective, and consideration should be given to societal/survival and security/defense requirements.

The societal/survival evaluation addresses a number of socioeconomic needs and includes, for example, the need for access for emergency relief and recovery operations just after an earthquake. Security/defense requirements may be evaluated using the 1973 Federal-Aid Highway Act, which requires that each state develop a plan for defense highways. The defense highway network provides connecting routes to military installations, industries, and resources not covered by the Federal-aid primary routes.

An essential bridge, then, satisfies one or more of the following conditions:

- It is required to provide secondary life safety; for example, it provides access to local emergency services such as hospitals. This category also includes bridges that cross routes that provide secondary life safety and bridges that carry lifelines such as electric power and water supply pipelines.
- Its loss would create a major economic impact; for example, such a bridge serves as a major link in a transportation system.
- It is formally defined by a local emergency plan as critical; say, it enables civil defense, fire departments, and public health agencies to respond immediately to disaster situations. This category also includes bridges that cross routes that are defined as critical in a local emergency response plan and bridges that are located on identified evacuation routes.
- It serves as a critical link in the security/defense roadway network.

From these considerations for seismic hazard and importance, four SPCs are defined as given in Table 1. As in the 1983 FHWA guidelines (*1*), these SPCs are used to set minimum retrofit requirements. For example, a bridge in SPC A need not be retrofitted at all, whereas an SPC B bridge need be evaluated only for connections and seat widths.

Note that these SPCs are assigned differently than those in the AASHTO specifications for new design, where no allowance for structure importance is made in seismic zones with acceleration coefficients of less than 0.29 (*7*). In view of the high cost of retrofitting, it is important to distinguish between essential and standard structures, especially so in low to moderate seismic zones. Such a distinction also enables a more rational allowance for the nature of the seismic hazard in the central and eastern United States, where the maximum credible earthquake is expected to be much larger than the "design" earthquake (475-year event). This implies that if an essential bridge in the East is to survive a large earthquake, it may need to be retrofitted to a standard higher than that required by the previous guidelines, which did not distinguish between essential and standard bridges in low to moderate seismic zones. This observation is reflected in the assignment of SPCs for essential bridges in Table 1.

Seismic Inventory of Bridges

The first step in implementing a seismic rating system is to compile an inventory with the objective of establishing the following basic information: (a) the struc-

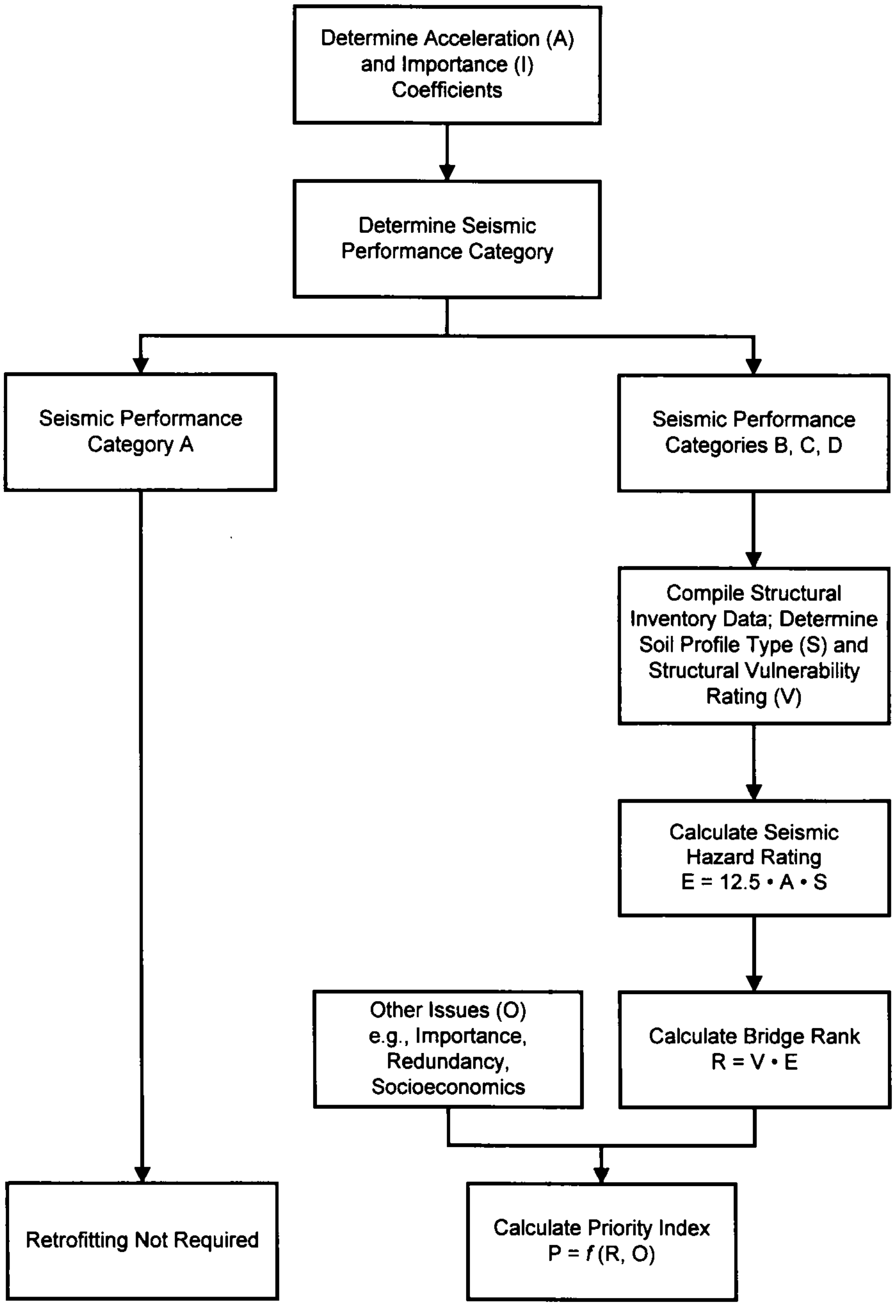


FIGURE 1 Preliminary screening procedure (5).

TABLE 1 Seismic Performance Categories

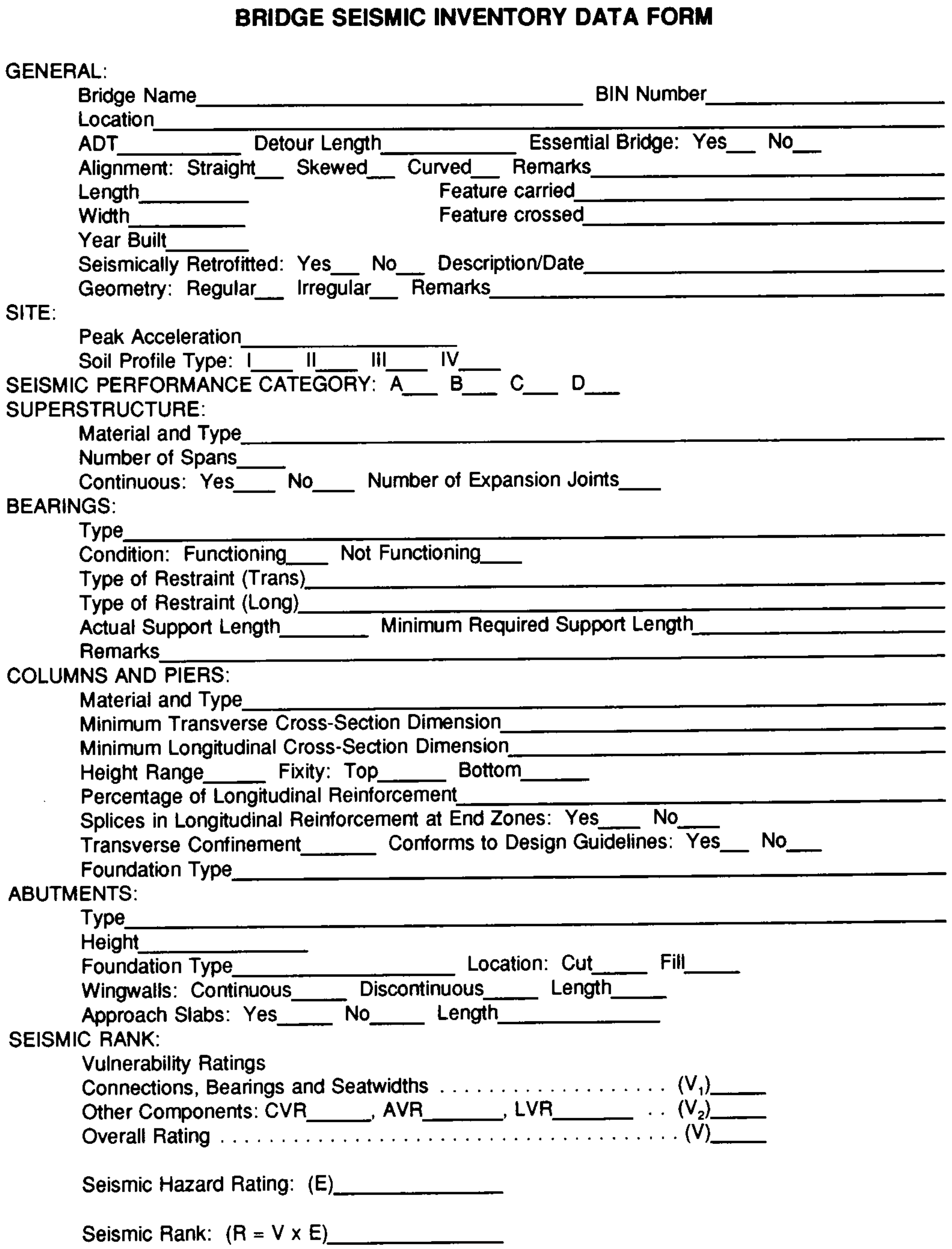
Acceleration Coefficient	Importance Classification	
	Essential	Standard
$A \le 0.09$	B	A
$0.09 < A \le 0.19$	C	B
$0.19 < A \le 0.29$	C	C
$0.29 < A$	D	C

tural characteristics needed to determine the vulnerability rating for each bridge; and (b) the seismic and geotechnical hazard at each bridge site. This information may be obtained from a bridge owner's records, FHWA's National Bridge Inventory, as-built plans, maintenance records, the regional disaster plan, on-site bridge inspection records, and other sources.

Also required at this time is the importance of each bridge so that an SPC may be assigned. An example of a form on which to record this inventory data is shown in Figure 2.

Seismic Rating System

To calculate the seismic rating of a bridge, consideration should be given to structural vulnerability, seismic and geotechnical hazards, and various socioeconomic fac-



BRIDGE SEISMIC INVENTORY DATA FORM

GENERAL:
Bridge Name________________________ BIN Number____________
Location____
ADT________ Detour Length________ Essential Bridge: Yes__ No__
Alignment: Straight__ Skewed__ Curved__ Remarks____________
Length________ Feature carried____________________
Width________ Feature crossed____________________
Year Built______
Seismically Retrofitted: Yes__ No__ Description/Date____________
Geometry: Regular__ Irregular__ Remarks____________________
SITE:
Peak Acceleration____________
Soil Profile Type: I___ II___ III___ IV___
SEISMIC PERFORMANCE CATEGORY: A__ B__ C__ D__
SUPERSTRUCTURE:
Material and Type____________________________________
Number of Spans____
Continuous: Yes___ No___ Number of Expansion Joints___
BEARINGS:
Type____
Condition: Functioning___ Not Functioning___
Type of Restraint (Trans)____________________________
Type of Restraint (Long)_____________________________
Actual Support Length______ Minimum Required Support Length____________
Remarks____
COLUMNS AND PIERS:
Material and Type____________________________________
Minimum Transverse Cross-Section Dimension________________
Minimum Longitudinal Cross-Section Dimension_______________
Height Range_____ Fixity: Top_____ Bottom_____
Percentage of Longitudinal Reinforcement___________________
Splices in Longitudinal Reinforcement at End Zones: Yes___ No___
Transverse Confinement_____ Conforms to Design Guidelines: Yes__ No__
Foundation Type_____________________________________
ABUTMENTS:
Type____
Height__________
Foundation Type______________ Location: Cut____ Fill____
Wingwalls: Continuous____ Discontinuous____ Length____
Approach Slabs: Yes____ No____ Length__________
SEISMIC RANK:
Vulnerability Ratings
Connections, Bearings and Seatwidths . . . . (V_1)_____
Other Components: CVR_____, AVR______, LVR______ . . (V_2)_____
Overall Rating . . . . (V)_____

Seismic Hazard Rating: (E)____________

Seismic Rank: (R = V x E)____________

FIGURE 2 Sample bridge seismic inventory form (5).

tors including importance. This is accomplished first by making independent ratings of each bridge in the areas of vulnerability and seismic hazard and second by considering importance and other issues (redundancy, nonseismic structural issues, and various societal and economic issues) to obtain a final, ordered determination of bridge retrofit priorities.

The rating system is therefore composed of two parts: the first is quantitative, the second qualitative. The quantitative part produces a seismic rating (called the bridge rank) based on structural vulnerability and seismic hazard. The qualitative part modifies the rank in a subjective way that accounts for such factors as importance, network redundancy, nonseismic deficiencies, remaining useful life, and other similar issues for inclusion in an overall priority index. Engineering and societal judgments are thus the keys to the qualitative stage of the screening process. This leads to the definition of a priority index as follows:

$$P = f\,(R, \text{importance, nonseismic factors, societal and economic issues} \ldots) \tag{1}$$

where P is the priority index and R is a rank based on structural vulnerability and seismicity.

In summary, bridge rank is based on structural vulnerability and seismic hazard, whereas retrofit priority is based not only on bridge rank, but also on importance, nonseismic deficiencies, economic factors, network redundancy, and the like.

Calculation of Bridge Rank

As noted, the bridge rank, R, is based on a structural vulnerability rating, V, and a seismic hazard rating, E. Each rating lies in the range 0 to 10, and the rank is found by multiplying these two ratings:

$$R = V \times E \tag{2}$$

It follows that R can range from 0 to 100, and the higher the score, the greater the need for the bridge to be retrofitted (ignoring, at this time, all other factors). Recommendations for assigning values for V and E are described in the following sections.

Vulnerability Rating

Although the performance of a bridge is determined by the interaction of all its components, it has been observed in past earthquakes that certain bridge components are more vulnerable to damage than others: the connections, bearings, and seats; columns and foundations; abutments; and soils. Of these, bridge bearings appear to be the most economical to retrofit. For this reason, the vulnerability rating proposed in the calculation of bridge rank is determined by examining the connections, bearings, and seat details separately from the remainder of the structure. A separate rating, V_1, is calculated for these components. The vulnerability rating for the rest of the structure, V_2, is determined from the sum of the ratings for each of the other components that are susceptible to failure. The overall rating for the bridge is then given by the maximum of V_1 and V_2. A flow chart summarizing the process is shown in Figure 3.

The determination of these vulnerability ratings requires considerable engineering judgment. Ratings may assume any value between 0 and 10. A value of 0 means a very low vulnerability to unacceptable seismic damage, a value of 5 indicates a moderate vulnerability to collapse or a high vulnerability to loss of access, and a value of 10 means a high vulnerability to collapse. Intermediate values may, of course, be assigned.

For bridges classified as SPC B, it is usually sufficient to calculate only the vulnerability ratings for bearings, joint restrainers, and support lengths along with a rating for liquefaction effects for bridges on certain sites. Experience has shown that most connection, bearing, and seat deficiencies can be corrected economically.

For bridges classified as SPC C or D, vulnerability ratings are also generated for the columns, abutments, and foundations. Experience with retrofitting these components is much more limited than for bearings. They are generally more difficult to retrofit and doing so may not be as cost-effective.

The vulnerability ratings V_1 and V_2 can then be compared to indicate the type of retrofitting needed. If the rating for the bearings is equal to or less than the rating of other components, simple retrofitting of only the bearings may be of little value. Conversely, if the bearing rating is greater, then benefits may be obtained by retrofitting only the bearings. A comparison of these two ratings during the preliminary screening process may be helpful in planning the type of comprehensive retrofit program needed, but it should not serve as a substitute for a detailed evaluation of individual bridges.

Connections, Bearings, and Seatwidths

Transverse restraint of a bridge superstructure is almost always provided at the bearings. Common types of restraints include shear keys, keeper bars, and anchor bolts. Restraints are usually brittle by nature (i.e., nonductile) and may be subjected to large seismically induced forces resulting from the redistribution of force from ductile components such as the columns. In ad-

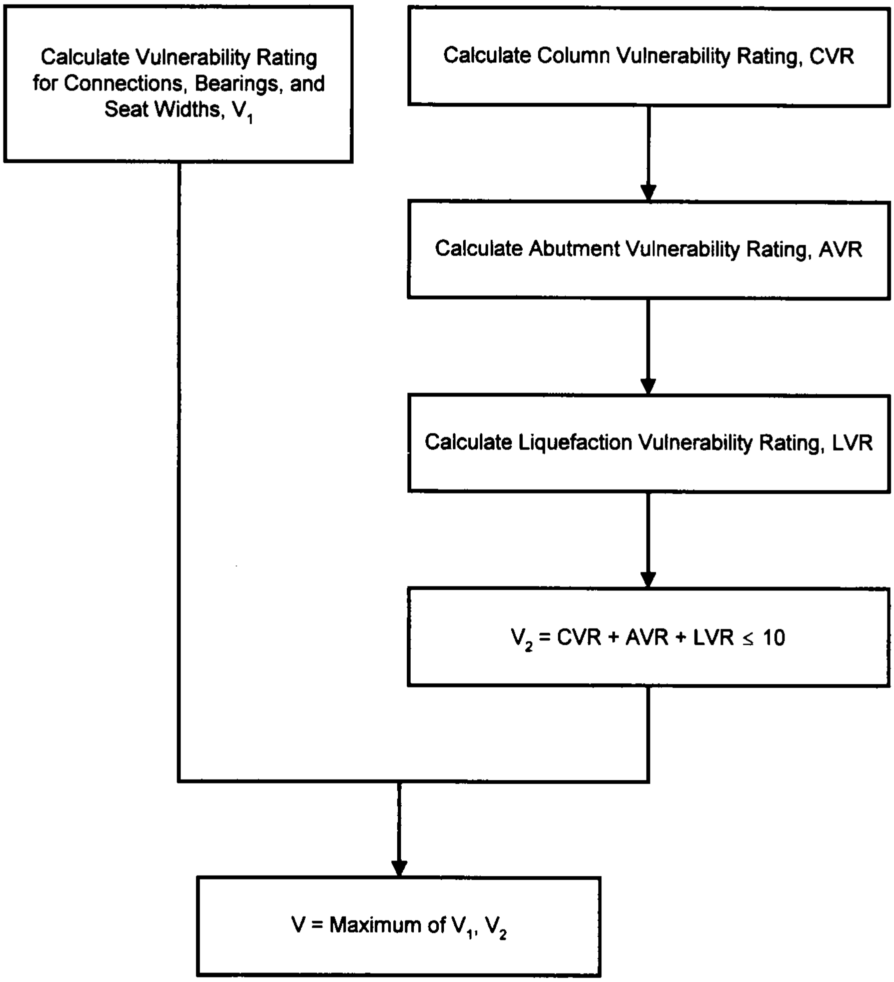


FIGURE 3 Procedure for calculating vulnerability rating, *V* (*5*).

dition, when several individual bearings with keeper bars are present at a support, the keeper bars do not resist load equally because of slight variations in clearances. Therefore, individual restraints may be subjected to very high forces. In some structures, collapse may occur because of loss of support resulting from large relative transverse or longitudinal movements at the bearings. The expected movement at a bearing is dependent on many factors and cannot be easily calculated. The AASHTO specifications require a minimum support length at all bearings in newly constructed bridges (*7*). Since it may be difficult to predict relative movement, the minimum support lengths, as required by the AASHTO specifications, may be used as the basis for checking the adequacy of longitudinal support lengths.

Support skew has a major effect on the performance of bridge bearings. Rocker bearings have been the most vulnerable in past earthquakes, and, at highly skewed-supports, these bearings may overturn during even moderate seismic shaking. In such cases, it is necessary to consider the potential for collapse of the span, which will depend largely on the geometry of the bearing seat. In some cases, settlement and vertical misalignment of a span due to an overturned bearing may be a minor problem, resulting in an only temporary loss of access that may be restored by backfilling with asphalt or other similar material. The potential for total loss of support should be the primary criteria when rating the vulnerability of the bearings.

A suggested step-by-step method for determining the vulnerability rating for connections, bearings, and seat-widths (V_1) is illustrated in Figure 4. Details are described elsewhere (*5*).

Columns, Abutments, and Liquefaction Potential

The vulnerability rating for the other components in the bridge that are susceptible to failure, V_2, is calcu-

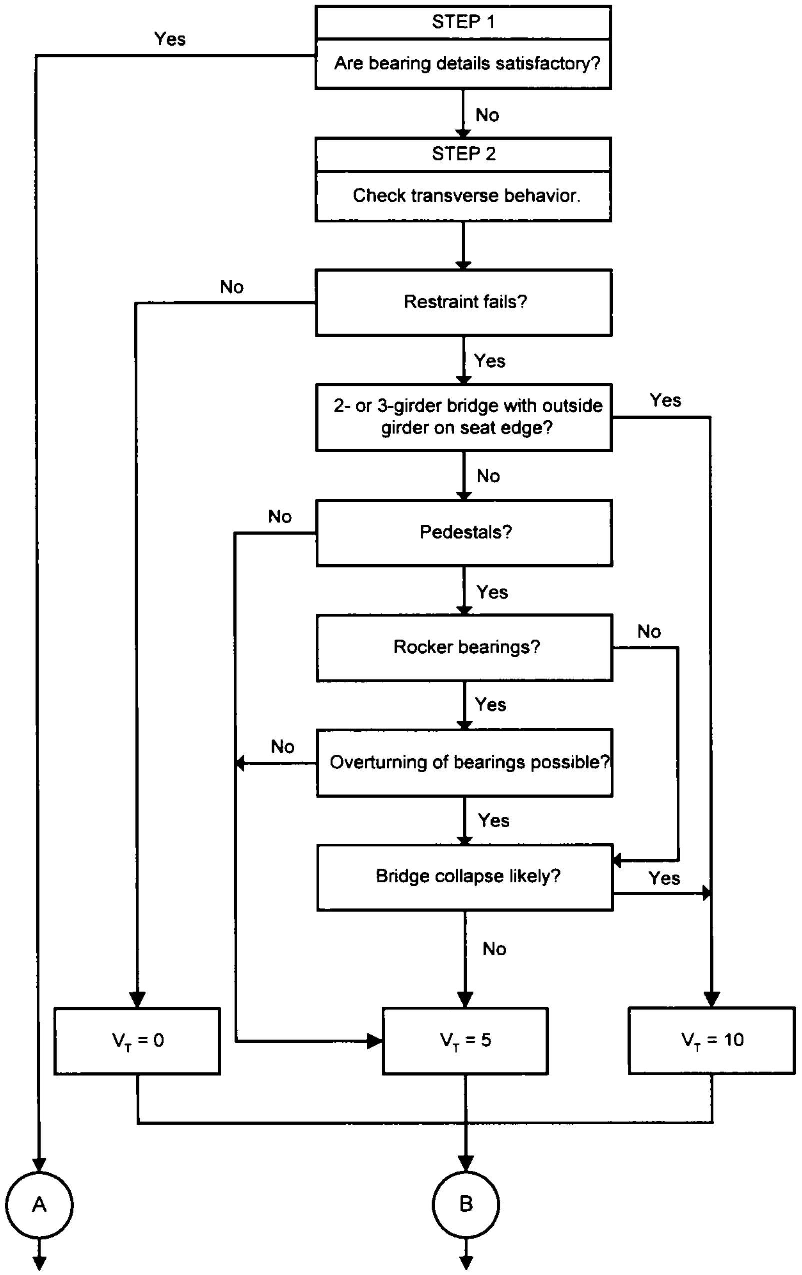


FIGURE 4 **Procedure for calculating vulnerability rating for connections, bearings, and seat widths, V_1; L = actual seat width and N = seat width required by AASHTO (5,7) (*continued on next page*).**

lated from the ratings for the individual components as follows:

$$V_2 = \text{CVR} + \text{AVR} + \text{LVR} \leq 10 \qquad (3)$$

where

CVR = column vulnerability rating,
AVR = abutment vulnerability rating, and
LVR = liquefaction vulnerability rating.

Suggested methods for calculating of each of these component ratings are also given elsewhere (5). Brief notes on each rating are presented in the following:

- *Column Vulnerability Rating.* Columns have failed in past earthquakes because of lack of adequate transverse reinforcement and poor structural details. Excessive ductility demands have resulted in degradation of column strength in shear and flexure. In several col-

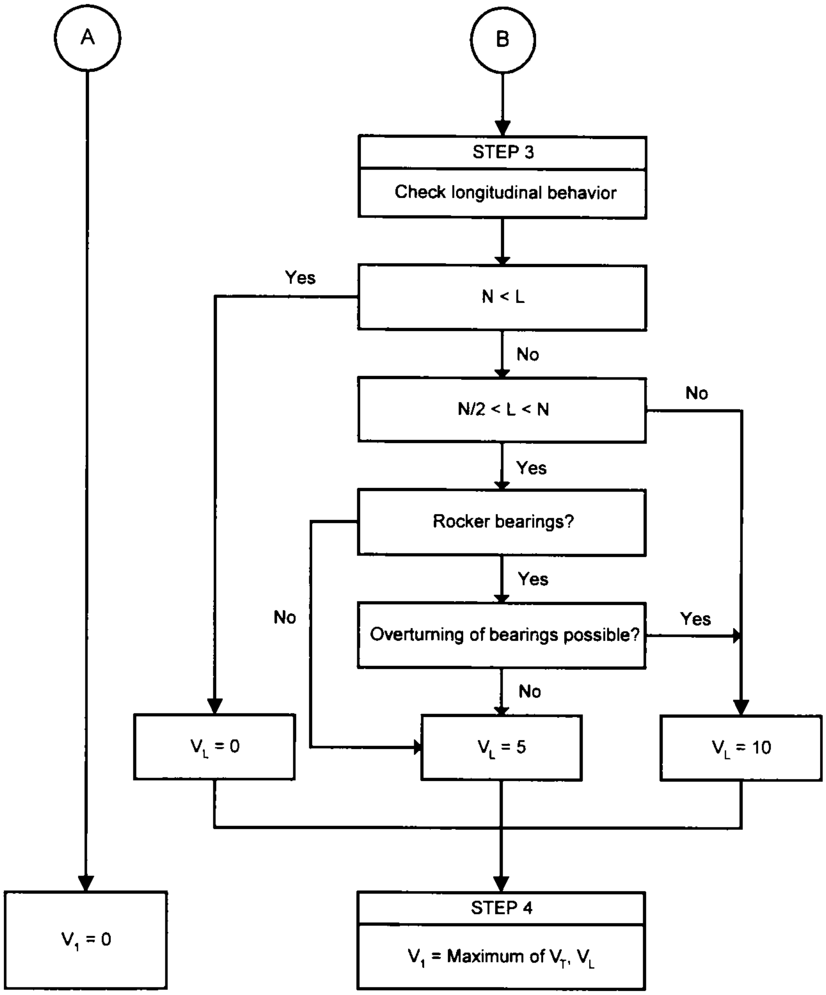


FIGURE 4 (*continued*).

lapses in past earthquakes, columns have failed in shear, resulting in column disintegration and substantial vertical settlement. Column failure may also be due to pullout of the longitudinal reinforcing steel, mainly at the footings. Fortunately, most bridge column failures occur during earthquakes with high ground accelerations of relatively long duration. Values for CVR range from 0 (negligible vulnerability) to 10 (maximum vulnerability). Details are given elsewhere (*5*).

- *Abutment Vulnerability Rating.* Abutment failures during earthquakes do not usually result in total collapse of the bridge. This is especially true for earthquakes of low to moderate intensity. Therefore, the AVR should be based on damage that may temporarily prevent access to the bridge.

One of the major problems observed in past earthquakes has been the settlement of approach fill at the abutment. Settlements ranging from 3 to 15 percent of the fill height have been observed in past earthquakes. This difference in behavior is assumed to be due to differences in abutment types (wall versus spill-through), construction of fills, and groundwater levels.

Additional fill settlements are possible in the event of structural failures at the abutments due to excessive seismic earth pressures or seismic forces transferred from the superstructure. Certain abutment types such as spill-through abutments and those without wing walls may be more vulnerable to this type of damage than others. Except in unusual cases, the maximum AVR should be 5. Details are given elsewhere (*5*).

- *Liquefaction Vulnerability Rating.* Although several types of ground failure can result in bridge damage during an earthquake, instability resulting from liquefaction is the most significant. The vulnerability rating for foundation soil is therefore based on

  –A quantitative assessment of liquefaction susceptibility,

  –The magnitude of the acceleration coefficient, and

–An assessment of the susceptibility of the bridge structure itself to damage resulting from liquefaction-induced ground movement.

The vulnerability of different types of bridge structures to liquefaction has been illustrated by failures during past earthquakes such as the 1964 Alaskan earthquake. The observed damage has demonstrated that bridges with continuous superstructures and supports can withstand large translational displacements and usually remain serviceable (with minor repairs). However, bridges with discontinuous superstructures or nonductile supporting members are usually severely damaged by liquefaction. These observations have been taken into account in developing the vulnerability rating procedure described elsewhere (*5*). The maximum value for LVR is 10, which should be assigned, for example, to multispan bridges in moderate to high seismic zones on soils with moderate to high susceptibility to liquefaction.

Seismic Hazard Rating

As a measure of seismic hazard, the peak ground acceleration in rock or competent soil is used, modified by the site coefficient to allow for soil amplification effects. The seismic hazard rating is defined as follows:

$$E = 12.5 \cdot A \cdot S \leq 10 \tag{4}$$

where A is the acceleration coefficient as given in the AASHTO specifications (*7*) and S is the site coefficient as given here: E ranges from 0.625 ($A = 0.05$, $S = 1$) to 10 ($A = 0.4$, $S = 2$).

Soil Profile Type	*Site Coefficient*
I	1.0
II	1.2
III	1.5
IV	2.0

In locations where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, or where the profile does not fit any of the four types, the site coefficient shall be based on engineering judgment. Soil profiles are defined as follows:

- Soil Profile Type I: A soil profile composed of rock of any description, either shale-like or crystalline in nature, or of stiff soils where the soil depth is less than 60 m (200 ft) and the soils overlying rock are stable deposits of sands, gravels, or stiff clays.
- Soil Profile Type II: A soil profile with stiff cohesive or deep cohesionless soil where the soil depth exceeds 60 m (200 ft) and the soils overlying the rock are stable deposits of sands, gravels, or stiff clays.
- Soil Profile Type III: A soil profile with soft to medium-stiff clays and sands, characterized by 9 m (30 ft) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.
- Soil Profile Type IV: A soil profile with soft clays or silts greater than 12 m (40 ft) in depth.

Calculation of Priority Index

Once a rank has been calculated for each bridge using Equation 2, the bridges in the inventory may be listed in numerical order of decreasing rank. This order should be modified to include such factors as bridge importance, network redundancy, nonseismic deficiencies, remaining useful life, and various societal and economic issues.

Some guidance on assigning importance has been given earlier in this paper. Network redundancy is generally beneficial, but if a bridge is part of a highly redundant highway network, the likelihood that alternative routes and structures will also be damaged in the same earthquake must be considered. If, for example, an overpass can be bypassed by using the on- and off-ramps, then a relatively convenient detour may be nearby, provided that these ramps are undamaged and remain open. If, on the other hand, the structure in question is a river crossing, the nearest detour may be several miles away; however, the possibility of its also being damaged may not be so great. As a consequence, it is not clear which bridge should receive the higher priority when considering redundancy alone.

In many cases, a judgment call will be necessary to decide these issues, so experience and common sense play a major role in assigning the priority index to individual bridges.

COMPARISON OF SCREENING PROCEDURES

Table 2 summarizes the most important differences between the screening procedure contained in the 1983 FHWA guidelines and that presented here. Major differences are apparent in the definition of the seismic performance categories and the calculation of a prioritized list of bridges for retrofitting.

The principal reason for upgrading the minimum requirements for essential bridges is to ensure that seatwidths and bearings will be checked for all bridges in this importance category, regardless of the seismic hazard. This is consistent with the requirements for all new bridges, and it appears reasonable to require similar standards for essential existing bridges. More rigorous requirements are also recommended for essential bridges in the moderate seismic zones, which reflects the expec-

TABLE 2 Comparison of Screening Procedures

	1983 FHWA Guidelines (Ref. 1)		**1995 FHWA Manual (Ref. 5)**	
Seismic Performance Categories	**Bridge Importance**		**Bridge Importance**	
	Essential	**Standard**	**Essential**	**Standard**
Acceleration Coefficient				
$A \leq 0.09$	A	A	B	A
$0.09 < A \leq 0.19$	B	B	C	B
$0.19 < A \leq 0.29$	C	C	C	C
$0.29 < A$	D	C	D	C
Seismic Rating Procedure				
Structure Vulnerability	$V = \max(V_1, V_2) \leq 10$ where V_1 = vulnerability of bearings and V_2 = vulnerability of other components		$V = \max(V_1, V_2) \leq 10$ where V_1 = vulnerability of bearings and V_2 = vulnerability of other components	
Seismic Hazard	$S = 25.A \leq 10$ where A = acceleration coefficient		$E = 12.5.A.S \leq 10$ where A = acceleration coefficient and S = site coefficient based on 4 soil profiles	
Importance	I = 6–10 essential bridges = 0–5 standard bridges			
Rank (max value = 100) and Priority Index	$R = w_1V + w_2S + w_3I$ where w_1, w_2, w_3 are weighting factors (sum = 10.0)		$R = V \cdot E$ P = priority index = f(R, importance, and socio-economic factors)	

tation that earthquakes will occur in the eastern and central United States that will be much larger than the design earthquake. If essential bridges are to remain operational in these circumstances, more extensive retrofitting may be required than previously recommended.

The two-step process for developing the prioritized list of bridges for retrofitting recognizes the difficulty (if not impossibility) of assigning numerical factors to such issues as importance and redundancy. Research in this area and the application of geographic information systems to highway networks may improve the situation and help quantify some of these subjective issues. In the meantime, the process described herein clearly separates the engineering from the societal factors and should improve the reliability and credibility of the results produced by this particular screening procedure.

Table 2 also indicates that the numerical expression for bridge rank is changed from an additive relationship to a multiplicative one. A disadvantage in this change is that the value for rank is now more sensitive to slight changes in the values assigned to the parameters *V* and *E*. On the other hand, a particular advantage is that in low seismic zones, the rank becomes a small number regardless of the vulnerability. This is a more reasonable result than that obtained under an additive rule. The balance of the argument appears to favor the multiplicative expression; indeed, this recommendation follows the trend already adopted by several state departments of transportation (*3*).

SUMMARY

Seismic retrofitting of highway bridges is a pressing need for many state departments of transportation. Up-to-date guidance concerning screening procedures, evaluation methods, and retrofit options is required and, in response to this need, FHWA recently revised its 1983 retrofitting guidelines. This paper has described a new screening procedure that is one of several improvements contained in the revised FHWA manual. This procedure separates the quantitative from the qualitative assessments in a two-step process. In addition to other modifications concerning importance, SPCs, and editorial changes, the improved screening procedure is expected to give more credible results while enhancing the safety of the U.S. highway bridge inventory—especially for those bridges that are essential to emergency response and recovery immediately after a major earthquake.

ACKNOWLEDGMENTS

This paper is based on the new FHWA *Seismic Retrofitting Manual for Highway Bridges*, which is a revision of the 1983 FHWA guidelines for seismic retrofitting. The earlier work was developed by the ATC, and the authors are indebted to those engineers and researchers who developed this pioneering work in the early 1980s.

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