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4501 South 2700 West, Box 148410 - Salt Lake City, Utah 84114-8410

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SEISMIC RETROFIT GUIDELINES FOR UTAH HIGHWAY BRIDGES

Prepared For:

Utah Department of Transportation
Research Division

Submitted by:

Utah State University,
Department of Civil & Environmental Engineering

Authored by:

Nash Wilson
Keri L. Ryan

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16. Abstract

Much of Utah's population dwells in a seismically active region, and many of the bridges connecting transportation lifelines predate the rigorous seismic design standards that have been developed in the past 10-20 years. Seismic retrofitting methods used in the United States have been investigated for applicability in the state of Utah. The guidelines developed here are based on the Federal Highway Administration's *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges*, which present the most current advances in retrofitting and earthquake engineering techniques. As a key difference from FHWA's guidelines, criteria for selecting earthquake ground motions are based on bridge anticipated service life. Bridges with 30 or more remaining years are analyzed for the 2500 and 500-year return periods, and bridges with less than 30 remaining years are analyzed for the 1000 and 100-year return periods. Flowcharts are provided to assist decision making in retrofit evaluation. The FHWA's indices method is recommended for screening and prioritizing bridges.

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Mark Mahan	Earthquake Engr. – Design
Cynthia MacLeay	Earthquake Engr. – Prioritization
Gary Goff	Structures Local Assistance

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EXECUTIVE SUMMARY

The objectives of this project were to develop seismic retrofit guidelines and design examples for Utah highway bridges. Near the beginning of the project, comprehensive retrofit guidelines for highway structures were published by Multidisciplinary Center for Earthquake Engineering Research (MCEER) and FHWA. As such, the decision was made to use the newly published guidelines as a basis for further investigation, prioritization of procedures, and adaptation as necessary for the state of Utah. Therefore, this report outlines much of the material in the Retrofit Manual, with appropriate modifications and expanded commentary. Presented in these guidelines are: (1) a comprehensive evaluation procedure to determine seismic deficiencies in existing bridges, (2) suggested retrofit measures to address the most common deficiencies in Utah highway bridges, and (3) a screening and prioritization algorithm to determine which bridges should be prioritized for comprehensive evaluation.

An important first step of a comprehensive retrofit evaluation is to determine the Seismic Retrofit Category (SRC). The SRC depends on the Seismic Hazard Level (SHL) and the bridge Performance Level (PL). SHL is a function of spectral accelerations and site dependent soil factors. PL is a function of bridge importance – bridges are classified as either *standard* or *essential* – and Anticipated Service Life (ASL) – generally computed as 75 years minus the age of the bridge. SRC is determined for both an upper level (UL) ground motion, representative of a rare event, and a lower level (LL) ground motion, representative of a frequent event. The FHWA manual recommends selecting the UL ground motion to be a 1000 year return period event for and the LL ground motion to be a 100 year return period event. However, in collaboration with UDOT personnel, we have modified this criteria so that the probability of exceedance of the event over the remaining life of the bridge is consistent with that of a new bridge (which are designed for UL = 2500 year and LL = 500 year events). Thus, if the bridge has ASL greater than about 30 years, the UL and LL ground motions are recommended to be chosen as 2500 and 500 year events, respectively.

The SRC determines the applicable analysis methods to obtain an accurate assessment of the bridge. Bridges in SRC C or D are generally evaluated by one of several elastic methods for demand analysis and one of several nonlinear methods for capacity assessment. The demand analysis methods include the Uniform Load Method, the Multi-Mode Spectral Method, and the Time History Method. The Uniform Load Method is based on the assumption that the bridge responds essentially as a single degree of freedom system. The capacity assessment procedures include the Component Capacity/Demand Method, the Capacity Spectrum Method, and the Structure Capacity/Demand Method. In the Component Capacity/Demand Method, the elastic demands are evaluated against capacities on a component by component basis. This method is intended for bridges that remain essentially elastic. The Capacity Spectrum Method is used in conjunction with the Uniform Load Method. A bilinear capacity curve of the entire structure is developed, and the intersection of the bridge capacity and demand, modified through equivalent damping to account for the effect of nonlinearity, is determined through iteration. In the Structure Capacity/Demand Method, a static pushover analysis of a detailed bridge model is performed to determine its capacity. The Component Capacity/Demand Method is generally recommended for the LL ground motion, wherein it is desired to keep the bridge response elastic. Because tracking the response of an existing bridge is inherently different than designing a new bridge, nonlinear methods are emphasized for evaluation to the UL ground motion. Guidance is given, but the engineer has some discretion in the selection of the analysis procedure. Nonlinear dynamic analysis is recommended for irregular and complex bridges.

Among many retrofit measures are listed, the following are expected to be most commonly and economically applicable. Excessive plastic rotation demands in columns can be alleviated by column jacketing, which both strengthens the column and enhances the ductility capacity through extra confinement. Seat lengths at expansion joints are often insufficient, which could cause spans to drop during strong shaking. This can be alleviated relatively easily through a combination of longitudinal cable restrainers and concrete seat extenders. Structurally deficient bearings can be addressed by strengthening the existing bearings or replacing the bearings with seismic isolation bearings. In general, seismic isolation, which reduces the demands through modification of the dynamic properties of the system, is a viable alternative to increasing the capacity of weak or non-ductile bridges.

The Seismic Rating Method Using Indices is recommended for prioritizing the bridge inventory. The indices method assigns each bridge a rating from 0 (least deficient) to 100 (most deficient). Specifically, a bridge vulnerability rating (from 0 to 10) is multiplied by a seismic hazard rating (from 1 to 10). Several factors are considered for developing the vulnerability rating, including bearing vulnerability, liquefaction vulnerability, column vulnerability, and abutment vulnerability. Bearing vulnerability is based on characteristics such as bearing type and support length. Liquefaction vulnerability is based on the soil characteristics. Column vulnerability is based on characteristics such as column length, amount of reinforcing steel, and framing factor. Abutment vulnerability is based on predictions of abutment settlement. Some factors needed to determine the vulnerability rating could be obtained directly from the National Bridge Inventory (NBI) database, while others would require more detailed evaluation of bridge plans and engineering judgment. The seismic hazard rating is simply the 1.0 second design spectral acceleration for the site (capped at 1g) multiplied by 10. The overall bridge rating can be combined with internal measures of criticality to arrive at a list of highest priority bridges for detailed retrofit evaluation.

Recommendations

Launching a comprehensive retrofit program for the entire bridge inventory would require a tremendous amount of resources that may detract from the state's other priorities. Alternatively, we recommend that the state consider a modest program focused on critical bridges that can be seismically upgraded with modest resources. Two types of retrofits can be performed relatively economically: (1) concrete seat extenders and cable restrainers for bridge superstructures at risk of loss of support of one or more spans, and (2) jacketing of columns with low strength and limited ductility capacity at risk of major substructure failures. Critical bridges can also be conveniently retrofitted seismically as they are upgraded for various other reasons, such as to increase capacity, extend the service life, or make major repairs.

1.0 Introduction

This document provides evaluation and retrofit criteria for bridges under consideration for retrofit in the State of Utah. The procedures are designed to assist bridge engineers using criteria based on the Federal Highway Administration's (FHWA) *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges* (FHWA, 2006), which is hereafter referred to as the Retrofit Manual. Where applicable, sections of the Retrofit Manual will be referenced for further explanation.

The Retrofit Manual presents a dual level process for evaluating bridges. This process is illustrated in Figure 1. In the dual level process, bridges are evaluated for the lower level ground motion, representative of a frequent, moderate magnitude event. Bridges are then evaluated for the upper level ground motion, representative of a rare, large magnitude event. These are separate processes that cannot be combined due to the difference in expected performance. For a lower level event, bridges are expected to respond elastically. For an upper level event, inelastic response is acceptable providing that collapse does not occur (Retrofit Manual, Section 1.4.6). The breakdown of the stages illustrated in Figure 1 are discussed in Sections 3 and 4 of this document.

Determination of the Seismic Retrofit Category (SRC) is the primary step in the seismic evaluation of a bridge. It is convenient to obtain the SRC for both upper and lower level events simultaneously. The SRC may help determine whether a bridge is exempt from further evaluation, or which evaluation procedure should be employed.

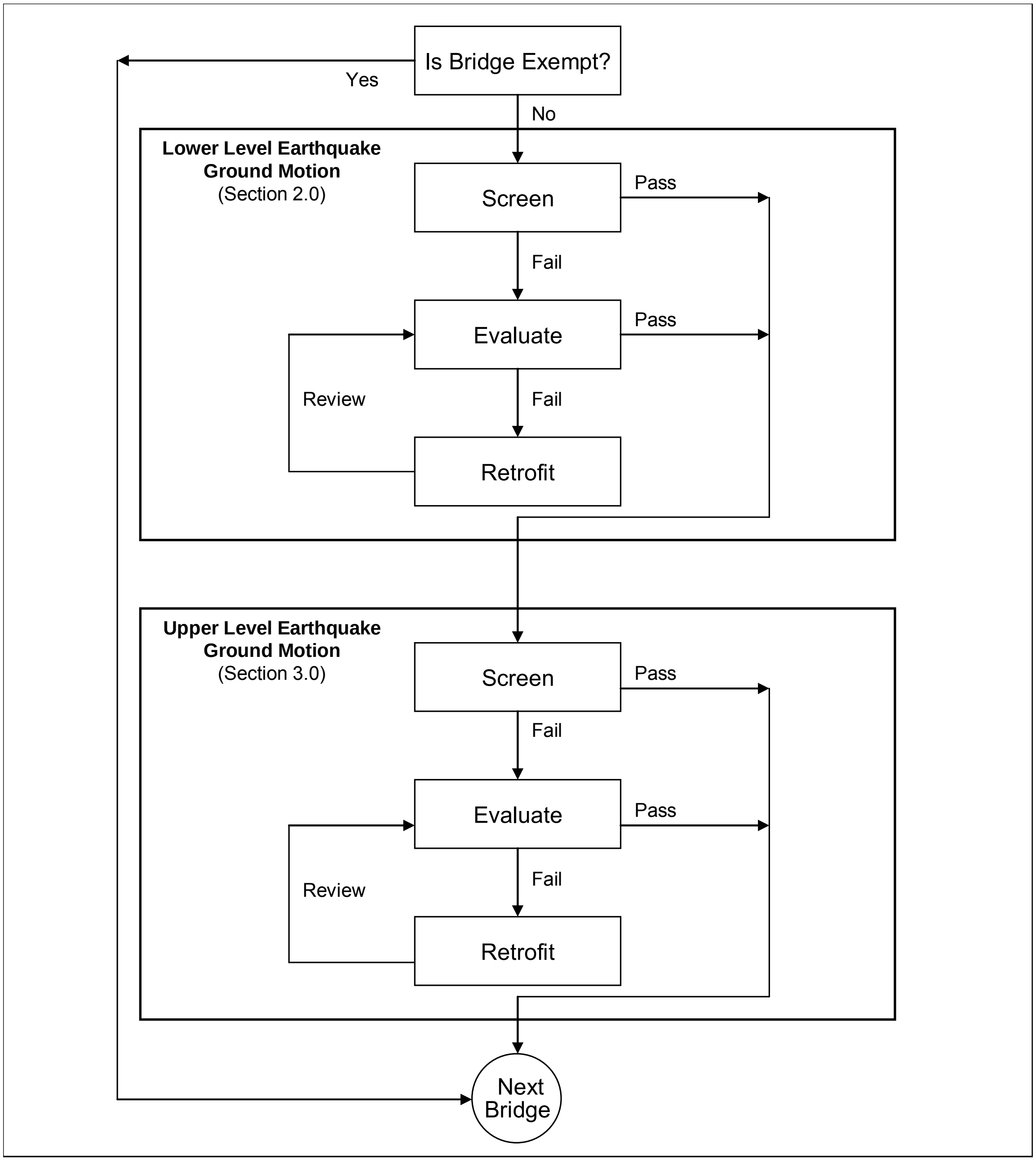

Figure 1. Flowchart for the dual level ground motion. (Retrofit Manual, Figure 1-7)

2.0 Determination of Seismic Retrofit Category

The Seismic Retrofit Category (SRC) is a function of the desired Performance Level (PL) and the anticipated Seismic Hazard Level (SHL). Factors affecting Performance Level (PL) include Bridge Importance and the Anticipated Service Life (ASL) of the bridge. Factors affecting the Seismic Hazard Level (SHL) include Spectral Accelerations (S_s and S_1), and Soil Factors (F_a and F_v). The process to determine the SRC is illustrated in Figure 2.

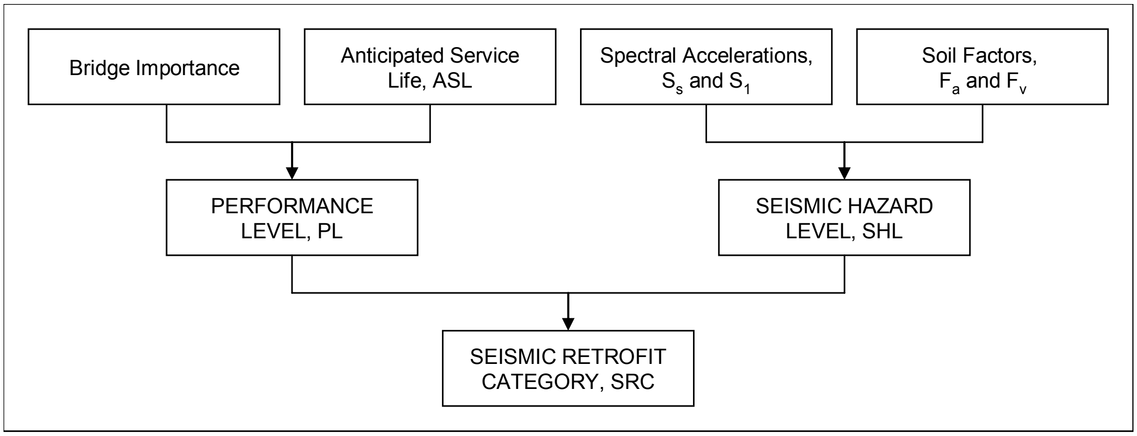

Figure 2. Determination of the SRC. (Retrofit Manual, Figure 1-9)

2.1 Bridge Importance

Bridge Importance is classified as either *standard* or *essential*. Engineering judgment is used to determine the category of bridge importance. *Essential* bridges are expected to remain functional after an earthquake. Essential bridges may be part of critical lifeline routes or critical links in the security and/or defense roadway network that must remain open immediately following an earthquake. All other bridges are classified as *standard* (Retrofit Manual, Section 1.4.3).

2.2 Anticipated Service Life, ASL

Anticipated Service Life (ASL) is determined by calculating the number of years a bridge is expected to remain in use. The Retrofit Manual suggests that ASL can be calculated by subtracting the bridge's age from the assumed service life of 75 years for new bridges. In all cases Utah bridge engineers should consider if a bridge has been rehabilitated during its service life and take into account the history of the bridge when selecting the Service Life Category. Table 1 is used to determine the proper Service Life Category (Retrofit Manual, Section 1.4.4).

Table 1. Selection of service life category. (Retrofit Manual, Table 1-1)

SERVICE LIFE CATEGORY	ANTICIPATED SERVICE LIFE
ASL 1	0 – 15 yrs
ASL 2	16 – 50 yrs
ASL 3	>50 yrs

2.3 Performance Level

The Retrofit Manual suggests different performance levels based on Bridge Importance and Anticipated Service Life (ASL). Minimum Performance Levels are given for both the upper and lower level events in Table 2. For the lower level ground motion, bridges are expected to respond elastically and therefore have the highest expected performance level (PL3). For the upper level ground motion, bridges are expected to sustain damage but prevent collapse and therefore have a lower performance level (PL1). Newer bridges (ASL > 50 years) should sustain minimal damage, and be repairable immediately following an upper level event. Because of the longer ASL, newer bridges have a higher performance level for the upper level motion. Utah bridge engineers may use their own judgment to raise or lower expected bridge performance based on factors such as economics or historical value (Retrofit Manual, Section 1.4.5).

2.4 Spectral Accelerations, S_s and S_1

The Retrofit Manual suggests using spectral accelerations to characterize earthquake ground motion (Retrofit Manual, Section 1.5.1). Both the short-period (S_s) and long period (S_1) spectral accelerations have been mapped by The US Geological Survey (USGS) for various exceedance probabilities (return periods.) The ground motion criteria adopted here differs from the recommendations of the Retrofit Manual which suggests using 1000 and 100-year return periods for upper and lower level motions, respectively, for all bridges. For Utah bridges, the upper level ground motion shall be selected to correspond to a 3% probability of exceedance (PE) or less over the remaining life of the bridge. A 3% PE corresponds to a 2500-year return period for bridges with an ASL ≥ 30 years, or a 1000-year return period for bridges with an ASL < 30 years (See Table 3). The design ground motions for the lower level event have been modified accordingly.

The guidelines in this report are intended to be consistent with a 3% PE over the life of the bridge, which is used in new design. This criterion allows for a design ground motion representative of the expected event regardless of the design goal (new design or retrofit.) Besides, the expected event in the Wasatch Fault region has an approximately 1400-year return period. Therefore, retrofitting bridges for a 1000-year event would be unconservative.

Table 2. Minimum performance levels for retrofitted bridges. (Retrofit Manual, Table 1-2)

EARTHQUAKE GROUND MOTION	BRIDGE IMPORTANCE and SERVICE LIFE CATEGORY					
	Standard			**Essential**		
	ASL 1	**ASL 2**	**ASL 3**	**ASL 1**	**ASL 2**	**ASL 3**
Lower Level Ground Motion 50 percent probability of exceedance in 75 years; return period is about 100 years.	PL0[4]	PL3	PL3	PL0[4]	PL3	PL3
Upper Level Ground Motion 7 percent probability of exceedance in 75 years; return period is about 1,000 years.	PL0[4]	PL1	PL1	PL0[4]	PL1	PL2

Notes:

1. Anticipated Service Life categories are:
    - ASL 1: 0 – 15 years
    - ASL 2: 16 – 50 years
    - ASL 3: > 50 years
2. Performance Levels are:
    - **PL0: No minimum** level of performance is recommended
    - **PL1: Life safety.** Significant damage is sustained and service is significantly disrupted, but life safety is preserved. The bridge may need to be replaced after a large earthquake.
    - **PL2: Operational.** Damage sustained is minimal and service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be reparable with or without restrictions on traffic flow.
    - **PL3: Fully Operational.** Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Damage is repairable without interruption to traffic.
3. Spectral ordinates, S_s & S_1, may be found for the Upper & Lower Level earthquake ground motions from http://earthquake.usgs.gov/research/hazmaps/design by using the Java Ground Motion Parameter Calculator and selecting the Probabilistic Hazard Curves analysis option.
4. Bridges assigned a Performance Level of PL0 have 15 years, or less, anticipated service life (ASL) and are candidates for replacement or rehabilitation.

Table 3. Selection of return period (years).

EARTHQUAKE GROUND MOTION	ANTICIPATED SERVICE LIFE			
	ASL < 30		**ASL ≥ 30**	
Lower Level Ground Motion	***100-Year Return Period***		***500-Year Return Period***	
	PE	Time (years)	PE	Time (years)
	50%	75	15%	75
	40%	50	10%	50
	26%	30	6%	30
Upper Level Ground Motion	***1000-Year Return Period***		***2500-Year Return Period***	
	PE	Time (years)	PE	Time (years)
	7%	75	3%	75
	5%	50	2%	50
	3%	30	1.2%	30

The USGS provides a Java Ground Motion Parameter Calculator which calculates spectral accelerations for 0.2, and 1.0-second periods based on inputting latitude and longitude coordinates, and return period. These spectral accelerations can be obtained at: *http://earthquake.usgs.gov/research/hazmaps/design*.

2.5 Soil Factors, F_a and F_v

Earthquake ground motions can be amplified by the overlying soils at a bridge site (Retrofit Manual, Section 1.5.2). To account for this behavior soil factors are determined based on site class and the spectral accelerations calculated in Section 2.4. Site classes are determined by their soil stiffness which is measured by either shear wave velocity in the upper 30 m (100 ft) or Standard Penetration Test (SPT) blowcounts and undrained shear strength. Site classes are shown in Table 4. Once the proper site class has been selected, soil factors F_a and F_v can be selected from Table 5.

2.6 Seismic Hazard Level, SHL

Using the products of the spectral ordinates S_s and S_1 and the soil factors F_a and F_v, the two points on the design spectrum S_{DS} and S_{D1} can be obtained. These points are used to determine the Seismic Hazard Level (SHL) of a bridge from Table 6 (Retrofit Manual, Section 1.5).

Table 4. Site classes. (Retrofit Manual, Table 1-3)

Site Class	**Description**
A	Hard rock with measured shear wave velocity, v_s >1500 m/sec (5,000 ft/sec)
B	Rock with 760 m/sec < $v_s \leq$ 1500 m/sec (2,500 ft/sec < $v_s \leq$ 5,000 ft/sec)
C	Very dense soil and soil rock with 360 m/sec < $v_s \leq$ 760 m/sec (1,200 ft/sec < $v_s \leq$ 2,500 ft/sec) **or** with either N > 50 blows/0.30m (50 blows/ft) or s_u > 100 kPa (2,000 psf)
D	Stiff soil with 180 m/sec $\leq v_s \leq$ 360 m/sec (600 ft/sec $\leq v_s \leq$ 1,200 ft/sec) **or** with either 15 $\leq$ N $\leq$ 50 blows/0.30m (15 $\leq$ N $\leq$ 50 blows/ft) or 50 kPa $\leq s_u \leq$ 100 kPa (1,000 $\leq s_u \leq$ 2,000 psf)
E	Soil profile with v_s < 180 m/sec (600 ft/sec) **or** with either N < 15 blows/0.30m (N < 50 blows/ft) or s_u < 150 kPa (1000 psf), **or** any profile with more than 3 m (10 ft) of soft clay defined as soil with PI > 20, w $\geq$ 40% and C < 25 kPa (500 psf)
F	Soils requiring site-specific evaluations 1. Peats or highly organic clays (H > 3 m [10 ft] of peat or highly organic clay where H = thickness of soil) 2. Very high plasticity clays (H > 8 m [25 ft] with PI > 75) 3. Very thick soft/medium stiff clays (H > 36 m [120 ft])

Exception: When the soil properties are not known in sufficient detail to determine the site class, site class D may be used. Site classes E or F need not to be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

Notes:

1. v_s is average shear wave velocity for the upper 30 m (100 ft) of the soil profile
   N is the average Standard Penetration Test (SPT) blowcount (blows/0.30m or blows/ft) (ASTM D1586) for the upper 30 m (100 ft) of the soil profile
   s_u is the average undrained shear strength in kPa (psf) (ASTM D2166 or D2850) for the upper 30 m (100 ft) of the soil profile
   PI is plasticity index (ASTM D4218)
   w is moisture content (ASTM D2216)

2. The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

3. The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known t be continuous to a depth of 30 m (100 ft)m surficial shear wave velocity measurements may be extrapolated to assess v_s.

4. Site classes A and B should not be used when there is more than 3 m (10 ft) of soil between the rock surface and the bottom of a spread footing.

Table 5. Site factors F_a and F_v. (Retrofit Manual, Table 1-4)

(a) Values of F_a as a function of site class and short period (0.2-sec) spectral acceleration, S_s

Site Class	**Spectral Acceleration at Short-Period (0.2 sec), S_s**[1]				
	$S_s \leq 0.25$	**$S_s = 0.50$**	**$S_s = 0.75$**	**$S_s = 1.00$**	**$S_s \geq 1.25$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]					

Notes:
1. Use straight-line interpolation for intermediate values of S_s.
2. Site-specific geotechnical investigation and dynamic sit response analysis should be performed for class F soils.

(b) Values of F_v as a function of site class and long-period (1.0-sec) spectral acceleration, S_1

Site Class	**Spectral Acceleration at Long-Period (1.0 sec), S_1**[1]				
	$S_1 \leq 0.1$	**$S_1 = 0.2$**	**$S_1 = 0.3$**	**$S_1 = 0.4$**	**$S_1 \geq 0.5$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]					

Notes:
1. Use straight-line interpolation for intermediate values of S_1.
2. Site-specific geotechnical investigation and dynamic sit response analysis should be performed for class F soils.

Table 6. Seismic hazard level. (Retrofit Manual, Table 1-5)

HAZARD LEVEL	**Using $S_{D1} = F_vS_1$**	**Using $S_{DS} = F_aS_s$**
I	$S_{D1} \leq 0.15$	$S_{DS} \leq 0.15$
II	$0.15 < S_{D1} \leq 0.25$	$0.15 < S_{DS} \leq 0.35$
III	$0.25 < S_{D1} \leq 0.40$	$0.35 < S_{DS} \leq 0.60$
IV	$0.40 < S_{D1}$	$0.60\ S_{DS}$

Notes:
1. For the purposes of determining the Seismic Hazard Level for Site Class E soils, the value of F_v and F_a need not be taken larger than 2.4 and 1.6 respectively, when S_1 is less than or equal to 0.10 and S_s is less than 0.25.
2. For the purposes of determining the Seismic Hazard Level for Site Class F soils, F_v and F_a values for Site Class E soils may be used with the adjustment described in Note 1 above.

2.7 Seismic Retrofit Category, SRC

The seismic retrofit category (SRC) is determined by the performance level (PL) and seismic hazard level (SHL) at the site. There are four categories, A through D, indicating increasing order of complexity in screening, evaluation, and retrofitting measures. Bridges in SRC A do not need further evaluation and are exempt from retrofit. The SRC determined for the lower level event may differ from the SRC determined for the upper level. Table 7 lists the seismic retrofit categories (Retrofit Manual, Section 1.6).

Table 7. Seismic retrofit categories. (Retrofit Manual, Table 1-6)

HAZARD LEVEL	PERFORMANCE LEVEL				
	During Upper Level Earthquake			During Lower Level Earthquake	
	PL0: No Minimum Level	**PL1: Life Safety**	**PL2: Operational**	**PL0: No Minimum Level**	**PL3: Fully Operational**
I	A	A	B	A	C
II	A	B	B	A	C
III	A	B	C	A	C
IV	A	C	D	A	D

3.0 Retrofitting Process for the Lower Level Ground Motion

The retrofitting process for the lower level ground motion involves a screening, evaluation, and retrofitting process as illustrated in Figure 1. Prior to screening, bridges may be considered exempt if they meet any of the following criteria (Retrofit Manual, Section 1.4.7):

- The bridge has 15 years or less of anticipated service life (Section 2.2).
- The bridge is 'temporary'.
- The bridge is closed to traffic and does not cross an active highway, rail or waterway.

Bridges in SRC A are exempt from screening and evaluation because they meet the criteria listed above. According to Table 7, the remaining bridges fall into SRC C or D for the lower level motion, and a single evaluation procedure is recommended for bridges in both categories. These bridges are screened according to Section 3.1 and may require further evaluation (Section 3.2) depending on the results of the screening. A flowchart describing the evaluation and retrofit process for the lower level ground motion is illustrated in Figure 3.

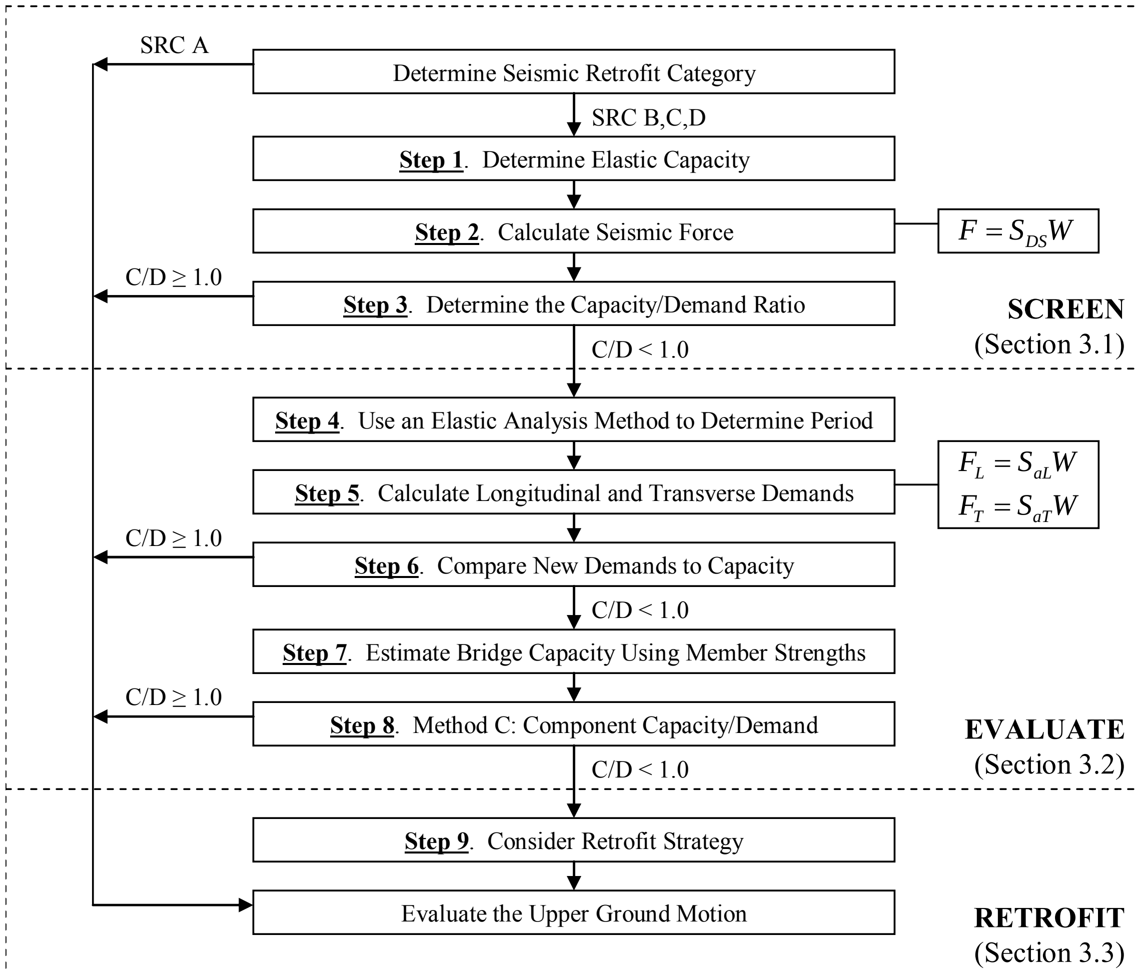

Figure 3. Flowchart for the Lower Level Ground Motion

3.1 Capacity/Demand Ratio (Simple)

The lower level earthquake ground motion has a return period of either 100 or 500 years depending on the bridge's anticipated service life. Bridges are designed to respond elastically to wind or braking loads, and are expected to perform similarly when subjected to the lower level earthquake event. The evaluation for this level of ground motion is a force-based approach because displacements are assumed to be small and within the default capacity of the bridge (Retrofit Manual, Section 1.7.2).

Step 1. To determine whether a bridge should be retrofitted for the lower level earthquake it is necessary to first calculate the elastic capacity of the bridge members in both the transverse and longitudinal directions. Elastic capacity in the transverse direction can be estimated by applying the factored wind load used in the design of the bridge. For the longitudinal direction, the factored braking load may be applied. The factored wind and braking loads should be taken from the current AASHTO LRFD Bridge Design Specifications. These capacities are greatly conservative, but are known to be within the elastic capacity of the bridge.

Step 2. To estimate the seismic force on the bridge, the weight of the superstructure must be obtained from the bridge design. Using superstructure weight and S_{DS} from the design spectrum calculated in Section 2.6, seismic force can be estimated by applying equation 3-1. This estimate should not be divided by a reduction factor (commonly 2.5) because the bridge is expected to respond elastically to the lower level event, and equation 3-1 represents the elastic demand for this ground motion.

$$F = S_{DS} W \tag{3-1}$$

Step 3. If the seismic demand is greater than the elastic capacity in either the transverse or longitudinal direction then a more detailed evaluation is necessary. If demand is less than capacity then proceed to evaluation for the upper ground motion.

3.2 Capacity/Demand Ratio (Detailed)

Step 4. Equation (3-1) is considered a conservative estimate of seismic demand because it assumes the period of the bridge is very short in both the transverse and longitudinal directions. A more accurate calculation of the period can be determined using an elastic analysis method such as the uniform load method (ULM), or the multi-mode response spectrum method (MM) described in Section 4.6.

Step 5. Once the spectral accelerations in the longitudinal and transverse directions (S_{aL} and S_{aT}) have been obtained from an elastic analysis method (ULM or MM), calculate the seismic demands from:

$$F_L = S_{aL} W \tag{3-2}$$

$$F_T = S_{aT} W \tag{3-3}$$

Step 6. Compare F_L and F_T to the elastic capacity determined in step 1. If the demand is less than the capacity then proceed to evaluation for the upper ground motion. However, if demand is still greater than capacity then proceed to step 7.

Step 7. Reevaluate capacity by calculating individual member strengths.

Step 8. Use Method C (section 4.4) to determine capacity/demand ratios for each member and component. For members and components with capacity/demand ratios greater than unity, proceed to evaluation for the upper ground motion; otherwise proceed to step 9.

3.3 Selection of a Retrofit Strategy

Step 9. Consider a retrofit strategy for strengthening the deficient component. Since bridges are expected to behave elastically when subjected to the lower level ground motion a force-based strategy for strengthening the deficient member is necessary. Displacements are expected to be relatively small and within the default capacity of the structure. Table 8 lists recommended retrofit measures depending on the deficient components and gives Retrofit Manual section numbers corresponding to each measure. For more information about the retrofit approaches, refer to Section 5 of this document.

The Retrofit Manual suggests that it may be useful to consider the upper ground motion event before pursuing a retrofit strategy for the lower ground motion because addressing deficiencies at the upper level event may also address deficiencies for the lower level event, and make retrofitting for the lower event unnecessary (Retrofit Manual, Section 1.7.3.) However, evaluation for the upper level event involves displacement-based approaches that may recommend retrofit measures that will not satisfy the strength requirements to achieve elastic performance in the lower level event.

4.0 Retrofitting Process for the Upper Level Ground Motion

To begin evaluation for the upper level ground motion, first determine the Seismic Retrofit Category (SRC) for the upper level event (See Section 2.0). The SRC for the upper level event is based on a larger earthquake ground motion (1000 or 2500 year event) and a lower performance level than the lower level event (100 or 500 year event). Bridges that have an SRC of A are exempt from further evaluation and do not require retrofit. Bridges with an SRC of B, C, or D require further evaluation. Figure 4 summarizes the retrofitting process for the upper level ground motion (Retrofit Manual, Section 1.8).

Table 8. Lower level earthquake retrofit measures (Retrofit Manual, Table 1-11, modified)

SEISMIC DEFICIENCY	RETROFIT MEASURES
Superstructure deficiencies	**8.2.1.1** Strengthening of Deck to Girder Connection **8.2.1.4** Girder Strengthening **8.2.4** Strengthening of Continuous Superstructures
Structurally deficient diaphragms	**8.2.1.2** Diaphragm Strengthening or Stiffening
Structurally deficient bearings/ connections	**8.3.1** Strengthening of Existing Bearings **8.4.2.2** Transverse Restrainers
Insufficient seat length	**8.2.2.1** Web and Flange Plates **8.4.2.1** Longitudinal Joint Restrainers
Flexurally deficient columns or piers	**9.2.1.2** Column Flexural Strengthening
Shear deficient columns or piers	**9.2.1.3** Column Ductility Improvement and Shear Strengthening
Structurally deficient pier caps	**9.3.2** Pier Cap Strengthening
Structurally deficient column-to-cap joints	**9.3.4** Strengthening of Column and Beam Joints
Unstable abutments	**10.2.2** Anchor Slabs **10.2.5** Transverse Abutment Anchors **10.2.6** Soil and Gravity Anchors
Structurally deficient abutments	**10.2.3** Diaphragm Walls **10.2.4** Transverse Abutment Shear Keys

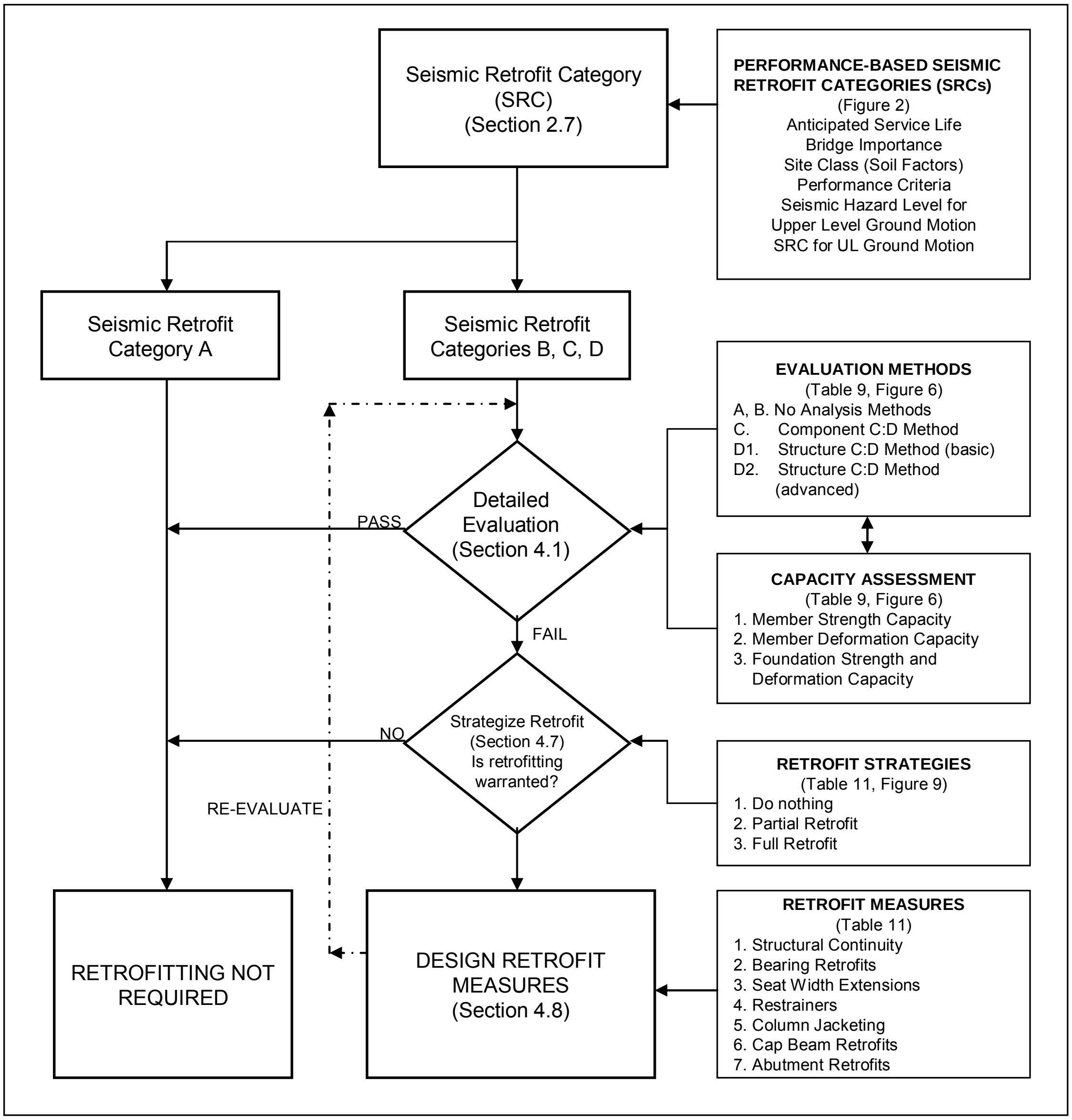

Figure 4. Retrofitting process for upper level ground motion (Retrofit Manual, Figure 1-10.)

4.1 Evaluation for the Upper Level Earthquake

There are several methods available to evaluate bridges for seismic retrofitting. The methods recommended by the Retrofit Manual include (Table 9):

- Method A1/A2: Connection forces and seat width checks.
- Method B: Component capacity checks.
- Method C: Component capacity/demand method.
- Method D1: Capacity spectrum method.
- Method D2: Structure capacity/demand method (pushover method).
- Method E: Nonlinear dynamic procedure (time history analysis).

Selecting the appropriate method depends on the SRC of the bridge in question. Table 9 illustrates the Retrofit Manual's recommendations for selecting an analysis method. Bridges in Utah that are candidates for retrofit are expected to be in SRC C or D and should be evaluated using methods C, D1, or D2 due to the moderate to large expected ground motions. Further information about method B (Retrofit Manual, Section 5.3), used to evaluate SRC C bridges in areas of low seismicity, and method E (Retrofit Manual, Section 5.7), recommended for irregular, complex bridges in hazardous soil conditions, may be found in the retrofit manual. These methods may be used at the designer's discretion. Method A1/A2, which uses calculations to perform checks of the connection forces and seat width capacities, is presented as it applies to initial screening for methods C, D1 and D2.

The *Bridge Type* column of Table 9 provides guidance relating to the applicability of evaluation methods for a bridge. Bridges meeting the criteria below are considered *regular*, otherwise they are considered *irregular*:

- Span length should not exceed 200 ft
- The ratio of the longest to shortest span lengths in frame should not exceed 1.5.
- The maximum skew angle should not exceed 30°, and the skew of adjacent piers or bents should not differ by more than 5°.
- For horizontally curved bridges, the subtended angle of the frame should not exceed 20°.
- The ratio of maximum to minimum pier stiffness should not exceed 2.0, including the effect of foundation stiffness.
- The ratio of maximum to minimum pier lateral strength should not exceed 1.5.

Table 9. Evaluation methods for existing bridges (Retrofit Manual, Table 1-9.)

METHOD		CAPACITY ASSESSMENT	DEMAND ANALYSIS	APPLICABILITY		COMMENTS
				SRC[1]	Bridge Type	
A1/ A2	**Connection and Seat Width Checks**	Uses default capacity due to non-seismic loads for connections and seat widths.	Not required	A – D	All single span bridges.	Hand method, spreadsheet useful.
				B	Bridges in low hazard zones.	
B	**Component Capacity Checks**	Uses default capacity due to non-seismic loads for connections, seats, columns and foundations.	Not required	C	Regular bridges, but subject to limitations on F_vS_1.	Hand method, spreadsheet useful.
C	**Component Capacity/Demand Method**	Uses component capacities for connections, seat widths, column details, footings, and liquefaction susceptibility.	Elastic Methods[2]: • ULM • MM • TH	C & D	Regular and irregular bridges that respond almost elastically, such as those in low-to-moderate seismic zones and those with stringent performance criteria.	Calculates C/D ratios for individual components. This is the C:D Method of previous FHWA highway bridge retrofitting manuals. Software required for demand analysis.
D1	**Capacity Spectrum Method**	Uses bilinear representation of structure capacity for lateral load, subject to restrictions on bridge regularity.	Elastic Methods[2]: • ULM	C & D	Regular bridges that behave as single-degree-of-freedom systems and have 'rigid' in-plane superstructures.	Calculates C/D ratios for complete bridge, for specified limit states. Spreadsheet useful.
D2	**Structure Capacity/Demand Method**	Uses pushover curve from detailed analysis of superstructure, individual piers and foundation limit states.	Elastic Methods[2]: • ULM • MM • TH	C & D	Regular and irregular bridges.	Calculates C/D ratios for bridge superstructure, individual piers, and foundations. Also known as *Nonlinear Static Procedure* or *Displacement Capacity Evaluation Method*. Software required for demand and capacity analysis.
E	**Nonlinear Dynamic Method**	Uses component capacities for connections, seat widths columns and footings.	Inelastic Methods[2]: • TH	D	Irregular complex bridges, or when site specific ground motions are to be used such as for bridges of major importance.	Most rigorous method, expert skill required. Software essential.

Notes: 1. SRC = Seismic Retrofit Category for upper level earthquake.
2. ULM = Uniform Load Method; MM = Multi-Mode Spectral Method; TH = Time History Method

Performance objectives also govern the applicability of an evaluation method. For example, bridges that are expected to respond elastically should be evaluated using method C: component capacity/demand method. This approach may apply to upgrades of major bridges in lower seismicity regions. The flowchart of Figure 5 illustrates the process of selecting an appropriate analysis method for bridges in Utah.

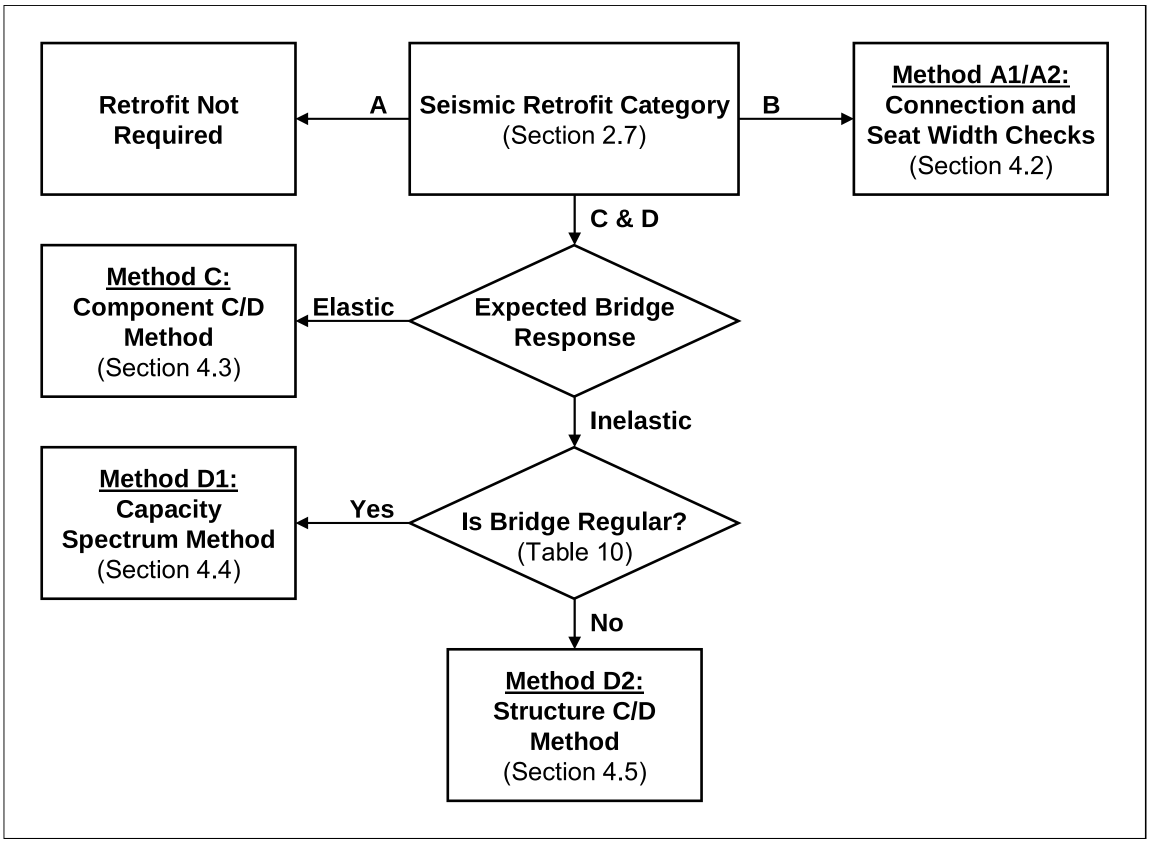

Figure 5. Flowchart for selecting evaluation method.

The evaluation methods listed in Table 9 specify a procedure for capacity analysis and provide one or more options for demand analysis. The evaluation methods emphasize the calculation of capacity on a member or component of a bridge, (Retrofit Manual, Section 1.11). Methods for determining the demand of a member or component are typically determined using modeling and elastic spectral analysis techniques. A clearer illustration of the relationship between demand and capacity assessment for each method is shown in Figure 6.

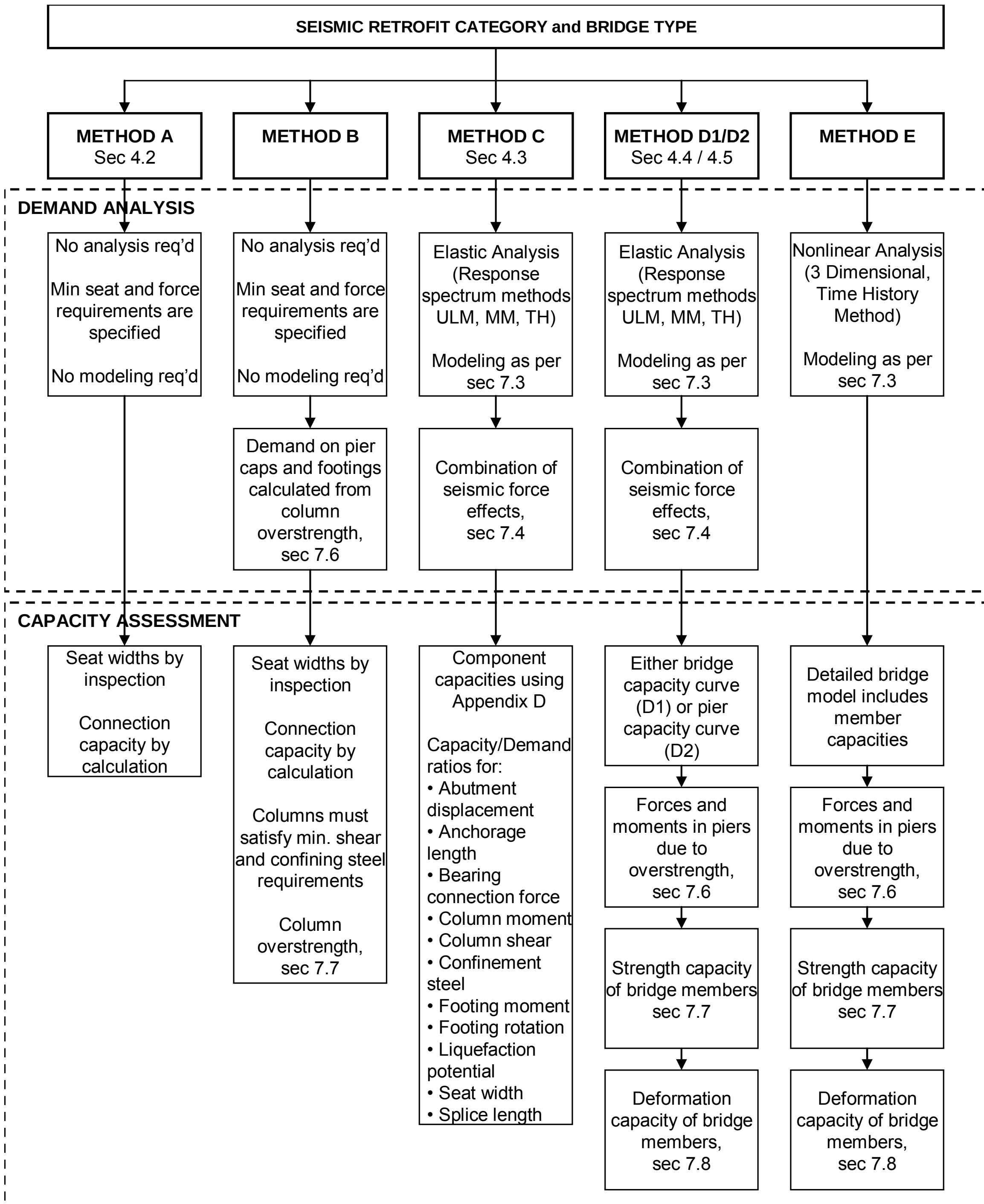

Figure 6. Evaluation methods highlighting capacity/demand (Retrofit Manual, Figure 1-13.)

4.2 Method A1/A2: Connection Forces and Seat Width Checks

This method checks the capacity of the bridge connections against a percentage of the loads applied to the bridge. Also, minimum seat widths are checked. Method A1 is a three-step process. (Retrofit Manual, Section 5.2)

Step 1. Determine the 0.2-second period spectral acceleration, S_s at the bridge site. If S_s is greater than or equal to 0.10 proceed to method A2, if less than 0.10 proceed to step 2.

Step 2. Calculate the tributary dead load of each segment of the bridge and take 10% of this load to compare to the capacity of the corresponding connection. The connection capacity must exceed this load.

Step 3. Check the seat width of the bridge against the equation:

$$N = \left[4.0 + 0.02L + 0.08H + 1.1\sqrt{H}\sqrt{1+\left(2\frac{B}{L}\right)^2}\right]\frac{(1+1.25F_v S_1)}{\cos\alpha} \qquad (4\text{-}1)$$

Where: N = minimum seat width (in)

L = distance between joints (ft)

H = tallest pier between joints (ft)

B = width of the superstructure (ft)

α = angle of skew (degrees)

Method A2:

Step 1. Check that the 0.2-second period spectral acceleration, $S_{s,}$ is greater than or equal to 0.10.

Step 2. Calculate the tributary dead load of each segment of the bridge and take 25% of this load to compare to the capacity of the corresponding connection. The connection capacity must exceed this load.

Step 3. Check the seat width of the bridge against equation 4-1.

4.3 Method C: Component Capacity/Demand Checks

This method is limited to bridges that will behave in a mostly elastic fashion. Demand is calculated using an elastic analysis method such as the uniform load method, or the multi mode spectral analysis method (see Section 4.6). Method C may overestimate the seismic vulnerability because it focuses on individual component behavior rather than the response of a bridge as a complete structure (Retrofit Manual, Section 5.4.1.) The steps for this method are presented in the flowchart in Figure 7.

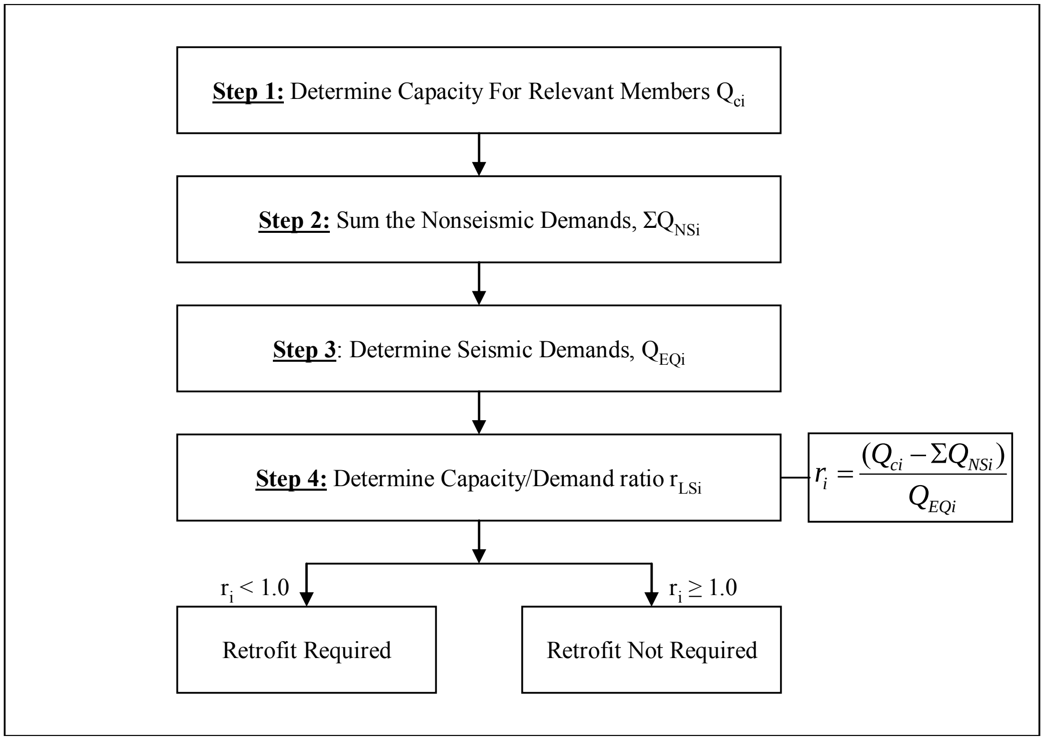

Figure 7. Flowchart for method C.

Step 1. Determine the capacity, Q_{ci}, for each of the relevant members in the structure.

Step 2. Determine the sum of the non-seismic force and displacement demands, ΣQ_{NSi}, for each of the members in the structure, for each load combination in equations 6-4, 6-2, 7-1, and 7-2 of the Standard Specifications (AASHTO, 2002) or Table 3.4.1-1 of the LRFD Specifications (AASHTO, 2007).

Step 3. Using the elastic demand spectrum characterized by F_a, S_s, F_v and S_1 (Section 2.4 & 2.5), perform an elastic dynamic analysis (see Section 4.6) to determine the seismic demand, Q_{EQi}, on each of the members. The analysis should reflect the anticipated condition of the structure and the foundation during this earthquake.

Step 4. For each member or component (i), determine the capacity/demand ratio from:

$$r_i = \frac{Q_{ci} - \sum Q_{NSi}}{Q_{EQi}} \quad (4\text{-}2)$$

If $r_i \geq 1.0$, the corresponding member has adequate capacity for the level of demand. Otherwise, devise retrofit measures that increase the displacement, strength, or ductility capacity, of the specific member or component.

Further information on computing component capacity/demand ratios is given in Appendix D of the Retrofit Manual. A list of the relevant components is given below, (Retrofit Manual, Appendix D.1):

1. Support length and restrainer C/D ratios:
   r_{ad} displacement C/D ratio for abutment
   r_{bd} displacement C/D ratio for bearing seat or expansion joint
   r_{bf} force C/D ratio for bearing or expansion joint restrainer
2. Column C/D ratios:
   r_{ca} anchorage length C/D ratio for column longitudinal reinforcement
   r_{cc} confinement C/D ratio for column transverse reinforcement
   r_{cs} splice length C/D ratio for column longitudinal reinforcement
   r_{cv} shear force C/D ratio for column
   r_{ec} bending moment C/D ratio for column
3. Footing C/D ratios:
   r_{ef} bending moment C/D ratio for footing
   r_{fr} rotation C/D ratio for footing
4. Soil C/D ratio:
   r_{sl} acceleration C/D ratio for liquefaction potential

4.4 Method D1: Capacity Spectrum Method

This method is limited to bridges that behave as single degree of freedom structures in both the transverse and longitudinal directions. Also, this method assumes equal displacements at the tops of the columns in the transverse and longitudinal direction. Because of these assumptions, bridges must have regular geometry (refer to Table 9) and uniform distribution of weight and stiffness. In general, to meet the equal displacement criteria, the bridge must satisfy the requirements of a *regular* bridge described in Section 4.1, (Retrofit Manual, Section 5.5.6).

The Capacity Spectrum method estimates the bridge response by finding the intersection of the pushover curve with a demand spectrum. A simplified bilinear pushover curve is generated assuming the bridge behaves as a single degree-of-freedom system. A demand spectrum represents an elastic spectrum as force or acceleration demand versus spectral displacement, which is the same form as the pushover curve. The effects of inelasticity on the demand are accounted for through equivalent damping.

The Retrofit Manual gives further background information (Retrofit Manual, Section 5.5) and outlines method D1 as a series of steps (Retrofit Manual, Section 5.5.5), which are repeated here in Figure 8. The major components to Method D1 are listed below, and appropriate commentary is provided:

- Part A: Initialization and Calculation of Bridge Capacity. Part A determines the initial elastic demand using the Uniform Load Method (Section 4.6) and the capacity curve. The capacity curve is uniquely determined by the initial stiffness k_1, yield force F_y, and postyield stiffness k_2. The initial stiffness k_1 is identical to the stiffness from the demand procedure. The yield force F_y is a sum of the lateral capacities of participating bridge columns, and is computed as:

$$F_y = \begin{cases} \sum_i \left(\frac{M_n}{H}\right)_i & \text{for cantilever columns} \\ \sum_i \left(\frac{2M_n}{H}\right)_i & \text{for fixed-fixed columns} \end{cases}$$

  where M_n is the nominal moment capacity of the column under gravity loads, computed from the moment-axial interaction curve, and H is the clear height of the column. The Retrofit Manual recommends taking the postyield stiffness as 5% of the initial stiffness. The procedure should be applied to both the longitudinal and transverse direction.
- Part B: Capacity/Demand Ratio Checks (r). Part B evaluates the capacity/demand ratio of the bridge at several identified displacement limit states. The displacement limit states (Step B1) might include displacement based on allowable plastic hinge rotation, displacement to avoid P-Δ instability, and maximum seat width. For each limit state, the corresponding capacity is evaluated (Step B2), and the demand spectrum that intersects the pushover curve at that location is identified (Steps B3 and B4). The demand spectrum modifies the 5% damped spectrum by damping factors B_L and B_S, which are functions of ductility (Step B3; see Table 5-4 of the Retrofit Manual). If this demand (in terms of spectral accelerations F_aS_s or F_vS_1) exceeds the elastic demand spectral coefficients by a factor of 1.5, then the displacement limit state is unlikely to be exceeded. Otherwise, retrofit measures might be undertaken to increase the displacement limit state.
- Part C: Bridge Response (F, Δ). Part C is an iterative procedure to find the intersection point of the pushover curve and the damping modified demand spectrum. The iteration starts by assuming the displacement equals the elastic demand displacement Δ= Δel, computes the point on the pushover curve and hence the capacity Cc corresponding to Δ (Step C3), and computes the spectral displacement Sd corresponding to Cc using the damping modified

demand spectrum (Steps C2 and C4). The procedure is repeated starting with Sd until convergence ($Sd = \Delta$). This procedure gives the estimated displacement and force demands in the bridge and they can be compared with pre-determined limit states.

Note: Parts B and C are two different approaches to determining if the bridge has sufficient capacity to resist the demands, and either approach is acceptable.

4.5 Method D2: Structure Capacity/Demand Method

This method is commonly referred to as the nonlinear static procedure or pushover analysis. A pushover analysis is used to determine displacement capacity, and an elastic analysis (Section 4.6) assesses displacement demands on the bridge. This method is used to analyze bridges with irregular geometry that expect inelastic performance and do not meet the equal displacement criteria discussed in Section 4.4 (Retrofit Manual, Section 5.6).

Step 1. Determine the strength and deformation capacity for each pier of the bridge.

Step 2. For each pier, carry out a nonlinear static pushover analysis until the structural displacement reaches the collapse limit state (limit state 5). Note the structural displacements, Δ_{ci}, at each of the limit states (i), namely at:

1. First yield,
2. Slight damage with cracking,
3. Moderate damage that is reparable,
4. Irreparable damage at the limit of life safety, and
5. Structural collapse.

Step 3. Determine the sum of the nonseismic displacement demands $\Sigma\Delta_{NSd}$ for each of the load combinations given in equations 6-1, 6-2, 7-1, and 7-2 of the *Standard Specifications* (AASHTO, 2002), or Table 3.4.1-1 of the *LRFD Specifications* (AASHTO, 2007).

Step 4. Using the elastic demand spectrum determined by S_s and S_1 and site factors, F_a and F_v (Table 5), perform an elastic dynamic analysis (see Section 4.6) to determine the seismic displacement demands, Δ_{EQd}, on each pier of the bridge. The analysis should reflect the anticipated condition of the structure and the foundation during this earthquake.

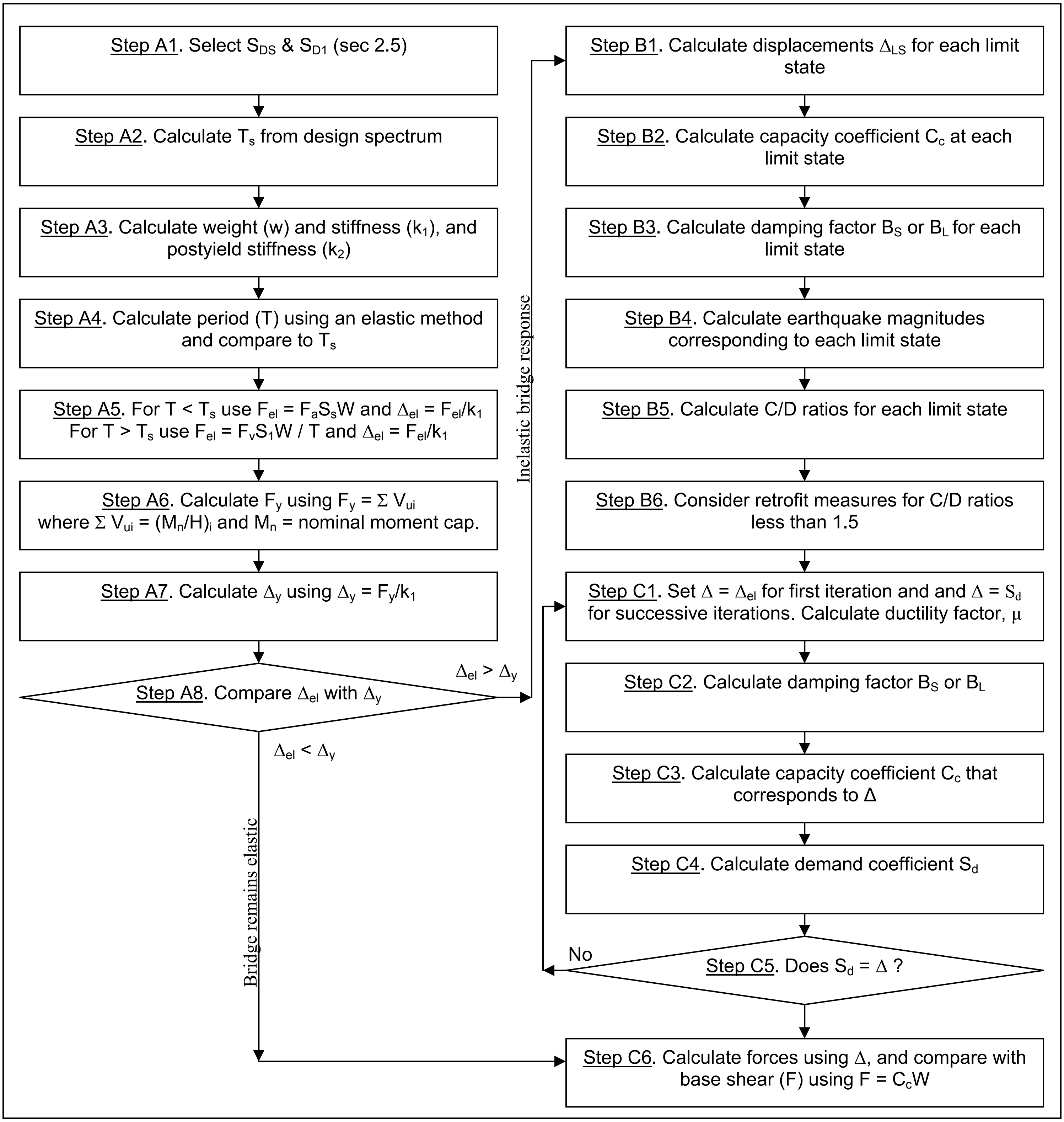

Figure 8. Flowchart for Method D1.

Step 5. Select an ultimate limit state for acceptable bridge performance from Step 1 based on performance goal. Guidance for calculating displacement limit states for the pushover analysis procedure is given in Section 7.8 of the Retrofit Manual. These guidelines are based on ultimate curvature or hinge rotations to corresponding to various types of failure, and the controlling (minimum displacement) limit state can be found. Determine the capacity/demand ratio (r_{LS}) for this limit state from the following:

$$r_{LS} = \frac{\left(\Delta_c - \sum \Delta_{NSd}\right)}{\Delta_{EQd}} \tag{4-3}$$

If $r_{LS} \geq 1.5$, the limit state is not likely to be reached and no remedial action is required.

If $1.0 \leq r_{LS} < 1.5$, the limit state may be reached and some remedial action may be required.

If $r_{LS} < 1.0$, the limit state is likely to be reached and retrofit measures which increase deformability or ductility capacity of the bridge should be considered. These measures might include extending the seat widths at pier caps and/or abutments, adding restrainers, jacketing columns, and strengthening joints and foundations as needed.

4.6 Elastic Analysis Methods

The Retrofit Manual describes three elastic analysis methods in Section 5.4.2. They are listed below in increasing order of rigor and complexity:

- Uniform Load Method (ULM)
- Multi-Mode Spectral Analysis Method (MM)
- Elastic Time History Method (TH)

The Uniform Load Method and Multi-Mode Spectral Analysis Methods are discussed below. The Time History Method is not discussed in this document, but is described in Section 5.4.2.3 of the Retrofit Manual. Bridges must satisfy the criteria of Table 10 to be a candidate for use of the ULM; otherwise the MM is recommended.

Table 10. Restrictions on the uniform load method (Retrofit Manual, Table 5-3)

Parameter	Value				
Number of spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	20°	20°	30°	30°	30°
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum pier stiffness ratio from span-to-span, excluding abutments	-	4	4	3	2

The ULM is used to determine the seismic demand on bridge members and components. The ULM idealizes the bridge as a single degree-of-freedom system, but a detailed model of the bridge must be obtained to determine the idealized properties and to track component force demands. The stiffness is approximated as F/u_{max}, where u_{max} is the peak displacement in the bridge model due to a uniformly distributed load of magnitude F. Once stiffness and weight are known, the period of the bridge can be determined, and the equivalent earthquake load is obtained from the design spectrum created in Section 2.6. The force and displacement demands are scaled according to the earthquake loading. The steps in the ULM are described in the Retrofit Manual (Section 5.4.2.1).

The MM method is typically used when coupling occurs in one or more directions, and involves creating a three-dimensional model for the bridge. An eigenvalue analysis is used to determine the modes and frequencies. A reduced number of modes of vibration may be used for modal analysis, where the cumulative modal mass is at least 90%. The design spectrum is used to determine the spectral demand for each mode. The Retrofit Manual provides guidelines for damping when performing modal analysis in Section 7.3.4. Modal combination should be performed using the Complete Quadratic Combination (CQC) method, but if the modal periods are well separated then the square-root-of-the-sum-squares (SRSS) method may be appropriate. The Retrofit Manual provides further guidelines for the use of this method in Section 5.4.2.2. The LARSA software contains a built in response spectrum analysis procedure.

4.7 Selecting a Retrofit Strategy

The Retrofit Manual describes a *Retrofit Strategy* as the overall plan for the seismic retrofit of a bridge. A *Retrofit Approach* is the philosophy of seismic enhancement adopted for a bridge. For example, strengthening is a common approach to retrofitting for the lower level earthquake and enhancing displacement capacity is a common approach to retrofitting for the upper level earthquake. A *Retrofit Measure* is the physical modification of a component in a bridge for the purpose of upgrading overall seismic performance. For example, longitudinal joint restrainers would be an appropriate retrofit measure for bridges with insufficient seat lengths.

The Retrofit Manual suggests using a five-step process to select a retrofit strategy. This process is illustrated in Figure 9.

<u>Step 1.</u> Conduct a detailed as-built evaluation. This step has probably been completed in Section 4. If a retrofit strategy is being considered, then evaluation methods C, D1, or D2 have probably been used to determine that a retrofit is warranted. It is also recommended that a field review of the bridge be performed to identify constraints that may eliminate some retrofit measures.

<u>Step 2.</u> Identify alternative retrofit strategies. The as-built evaluation will help identify what retrofit approaches need to be considered. After determining a retrofit approach, Table 11 may be helpful in selecting an appropriate retrofit measure. At this stage it is important to consider that some measures may be eliminated due to excessive cost, constructability, aesthetics, or other similar problems.

<u>Step 3.</u> Evaluate alternative retrofit strategies. When a strategy has been selected it should be analyzed using the same evaluation procedure used in the as-built evaluation. At this point, cost estimates should be generated for each retrofit alternative.

<u>Step 4.</u> Conduct a strategy meeting. At this meeting, recommendations and cost estimates should be presented to representatives of agencies that have an interest in the project. These may include Federal, state, and local government agencies, structural and geotechnical engineering specialists, and environmental and citizen groups.

<u>Step 5.</u> Document the strategy selection. This report should include all calculations of the as-built and as-retrofitted evaluations, preliminary plans and sketches showing the proposed retrofit, a summary of conclusions and recommendations, preliminary cost estimates, and a summary of the discussions from the strategy meeting.

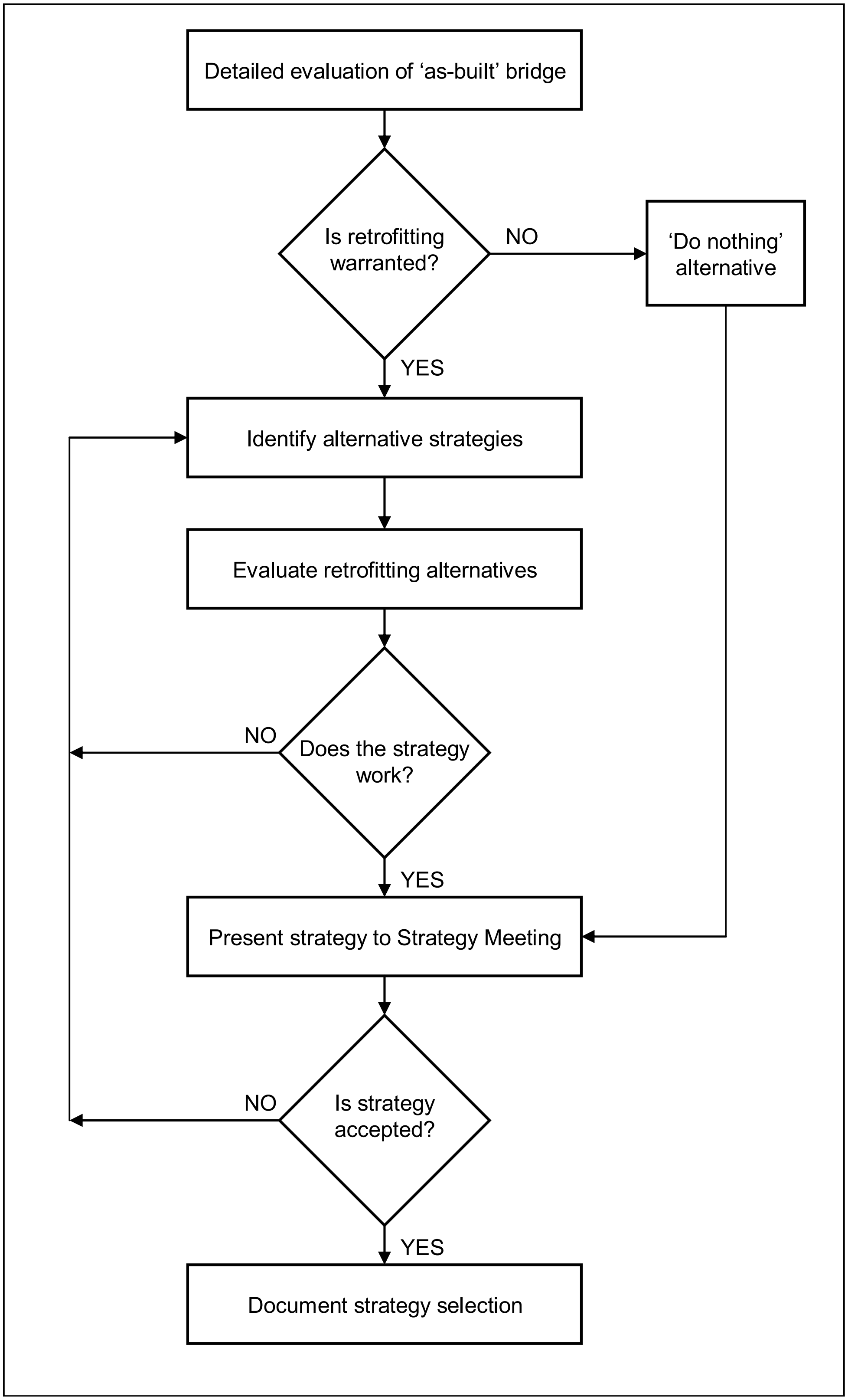

Figure 9. Selection of retrofit strategy (Retrofit Manual, Figure 1-14.)

4.8 Seismic Retrofit Measures

Table 11 shows possible retrofit measures for corresponding seismic deficiencies. For more information about a retrofit measure refer to the corresponding section number in the Retrofit Manual. Areas of possible seismic deficiencies include the superstructure, bearings, bearing seats, pier caps, columns, and abutments. Seismic evaluation may determine deficiencies in the footings or soil conditions, and measures have been established to mitigate these problems. Footing and soil retrofits are not expected retrofit measures for Utah bridges; therefore, these measures are not discussed in this document but are referred to in the Retrofit Manual. (Retrofit Manual, Section 1.13)

Research in earthquake engineering continually provides new and innovative solutions to seismic hazards allowing the availability of many retrofit measure options. It may be helpful to consider the latest state-of-the-art practices when considering retrofit measures.

5.0 Retrofit Measures

The goal of this chapter is to provide an overview of the retrofit measures expected to be implemented in the State of Utah. Selection of the appropriate retrofit measure depends greatly upon the expected performance of the bridge. Life safety is generally a minimum performance level (PL1). Retrofit measures that remedy deficiencies in the bridge's superstructure, bearings, and bearing seats achieve this goal and require the least effort, (Retrofit Manual, Section 8.1). As a bridge's deficiencies move from the superstructure to the substructure and into the foundation the retrofit costs increase considerably, (Retrofit Manual, Section 1.12.2.2). Because of these costs and relative effectiveness of retrofit measures, it is recommended that measures for the superstructure be considered first before moving onto the substructure. This chapter will provide information on retrofit measures in that order. For greater detail, refer to the corresponding sections in the Retrofit Manual.

5.1 Retrofit Measures for the Superstructure

The bridge's superstructure consists of the bridge deck, girders, beams, bearings, and seats. Retrofit measures employed at this level are meant to prevent bridge collapse and effectively distribute inertial forces to the substructure.

Table 11. Seismic retrofit matrix (Retrofit Manual, Table 1-11)

SEISMIC DEFICIENCY	RETROFIT APPROACH			
	Strengthening	Displacement Capacity Enhancement	Force Limitation	Response Modification
Superstructure deficiencies	8.2.1.1 Strengthening of Deck to Girder Connection 8.2.1.4 Girder Strengthening 8.2.4 Strengthening of Continuous Superstructures			
Structurally deficient diaphragms	8.2.1.2 Diaphragm Strengthening or Stiffening		8.2.1.3 Energy Dissipating Ductile Diaphragms	
Structurally deficient bearings/ connections	8.3.1 Strengthening of Existing Bearings 8.4.2.2 Transverse Restrainers			8.3.2.2 Replacement with Seismic Isolation Bearings 8.4.3 Energy Dissipation Devices 8.4.4 Shock Transmission Units
Insufficient seat length	8.2.2.1 Web and Flange Plates 8.4.2.1 Longitudinal Joint Restrainers	8.4.1.1 Concrete Seat Extensions and Catcher Blocks		8.2.2.2 Superstructure Joint Strengthening 8.2.3 Reduction of Dead Load
Flexurally deficient columns or piers	9.2.1.2 Column Flexural Strengthening	9.2.1.3 Column Ductility Improvement and Shear Strengthening	9.2.1.6 Limitation of Column Forces	
Shear deficient columns or piers	9.2.1.3 Column Ductility Improvement and Shear Strengthening			
Structurally deficient pier caps	9.3.2 Pier Cap Strengthening		9.3.3 Reduction of Pier Cap Forces	9.3.5 Supergirders
Structurally deficient column-to-cap joints	9.3.4 Strengthening of Column and Beam Joints			9.3.5 Supergirders
Abutment fill settlement		10.2.1 Approach Slabs		
Unstable abutments	10.2.2 Anchor Slabs 10.2.5 Transverse Abutment Anchors 10.2.6 Soil and Gravity Anchors			10.2.2 Anchor Slabs
Structurally deficient abutments	10.2.3 Diaphragm Walls 10.2.4 Transverse Abutment Shear Keys			

5.1.1 Structural Continuity

As a bridge ages, factors such as creep, shrinkage and settlement have already occurred and temperature effects still need consideration (MDT, 2004). The Retrofit Manual suggests that a structure can be made continuous without affecting its ability to accommodate changes in temperature if the bridge is relatively short or if the joint is fixed-fixed (Retrofit Manual, Section 8.2.2.2). To make the bridge continuous, a portion of the deck must be removed on either side of the existing joint and the girders must be spliced together with spice plates at the flanges. The joint is then replaced with reinforced concrete. This technique is illustrated in Figure 10, which is taken from the Retrofit Manual. By making the bridge continuous, seismically induced forces are better shared between the supports.

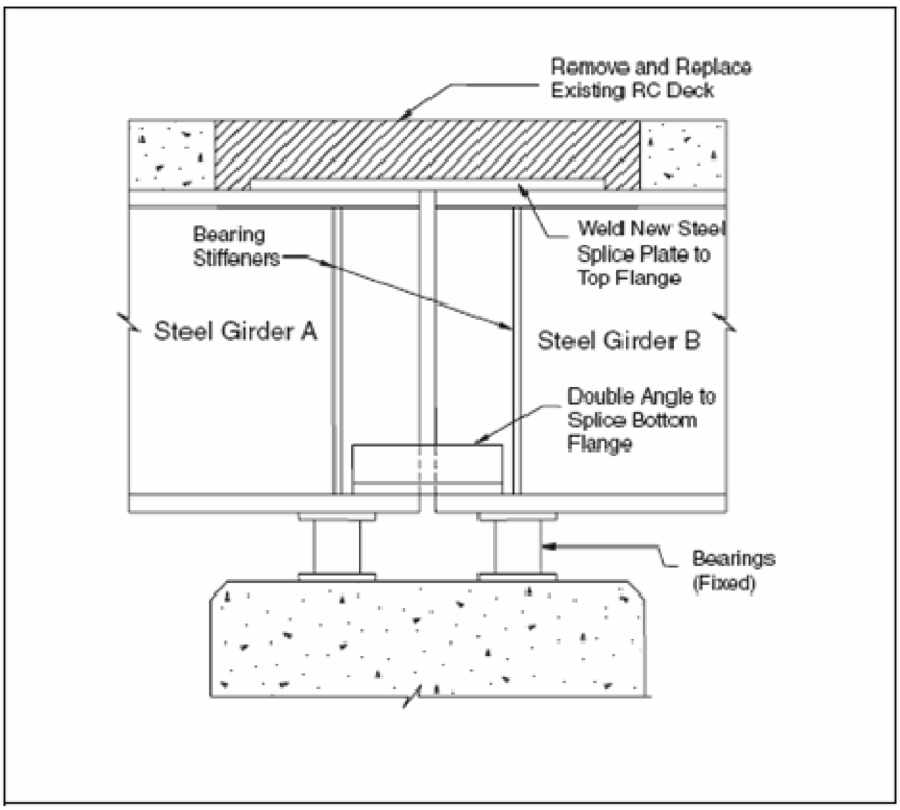

Figure 10. Bridge made continuous (Retrofit Manual, Figure 8-8)

5.1.2 Bearing Retrofits

If modeling shows dislodging of the superstructure from the bearings but the seats are wide enough and drop distance is relatively short to prevent collapse then the performance level of the bridge may be acceptable (Retrofit Manual, Section 8.3). Tall steel rocker bearings and fixed bearings have been known to perform poorly during seismic events and are strong candidates for retrofitting. Rocker bearings should be replaced due to the possibility that strengthening the bearings may result in failure of the columns. Newer bearings typically have smaller vertical dimensions than what they are replacing and it will be necessary to compensate for the difference in height as part of the retrofit (Retrofit Manual, Section 8.3.2.1).

Recommended retrofits include replacing the tall existing bearings with elastomeric bearing pads. This retrofit will require the implementation of a steel or concrete pedestal (see Figure 11). Another option is the use of a seismic isolation bearing. Bridges that already have bearings at every pier and abutment are good candidates for seismic isolation (Retrofit Manual, Section 8.3.2.2). Isolation bearings provide the benefit of protecting other structural components from damage by lengthening the natural period of vibration of the bridge and dissipating energy. Because isolators have physical properties that vary with temperature, it may be important to test the performance of isolators being considered for retrofit (AASHTO, 1999). If seismic isolation is being considered as a retrofit approach, it is recommended that designers refer to Section 8.3.2.2 of the Retrofit Manual.

5.1.3 Seat Width Extension

Seat width extensions are intended to provide increased capacity for displacements between the superstructure and the substructure, hence avoiding collapse. Minimum seat widths should be calculated using equation 4-1 in Section 4.2 of this document. The Retrofit Manual recommends using the following load cases to design the seat extensions:

1. Vertical load equal to twice the dead load reaction.
2. Vertical load equal to the dead load reaction plus a horizontal load due to the earthquake.

The horizontal earthquake loading should be equal to the lesser of the dead load reaction times the spectral acceleration coefficient, or the dead load times the maximum feasible coefficient of friction between the girder and seat extension.

5.1.4 Restrainers

Just as extending the seat width of a bridge will increase the displacement capacity, using a restrainer will decrease the displacement demand. These retrofit measures can be used in conjunction to remedy a deficiency in the expansion joint. Restrainers are typically cables or bars that prevent longitudinal movements at expansion joints. Due to the added flexibility of cables, they are generally preferred over bars (Retrofit Manual, Section 8.4.2.1).

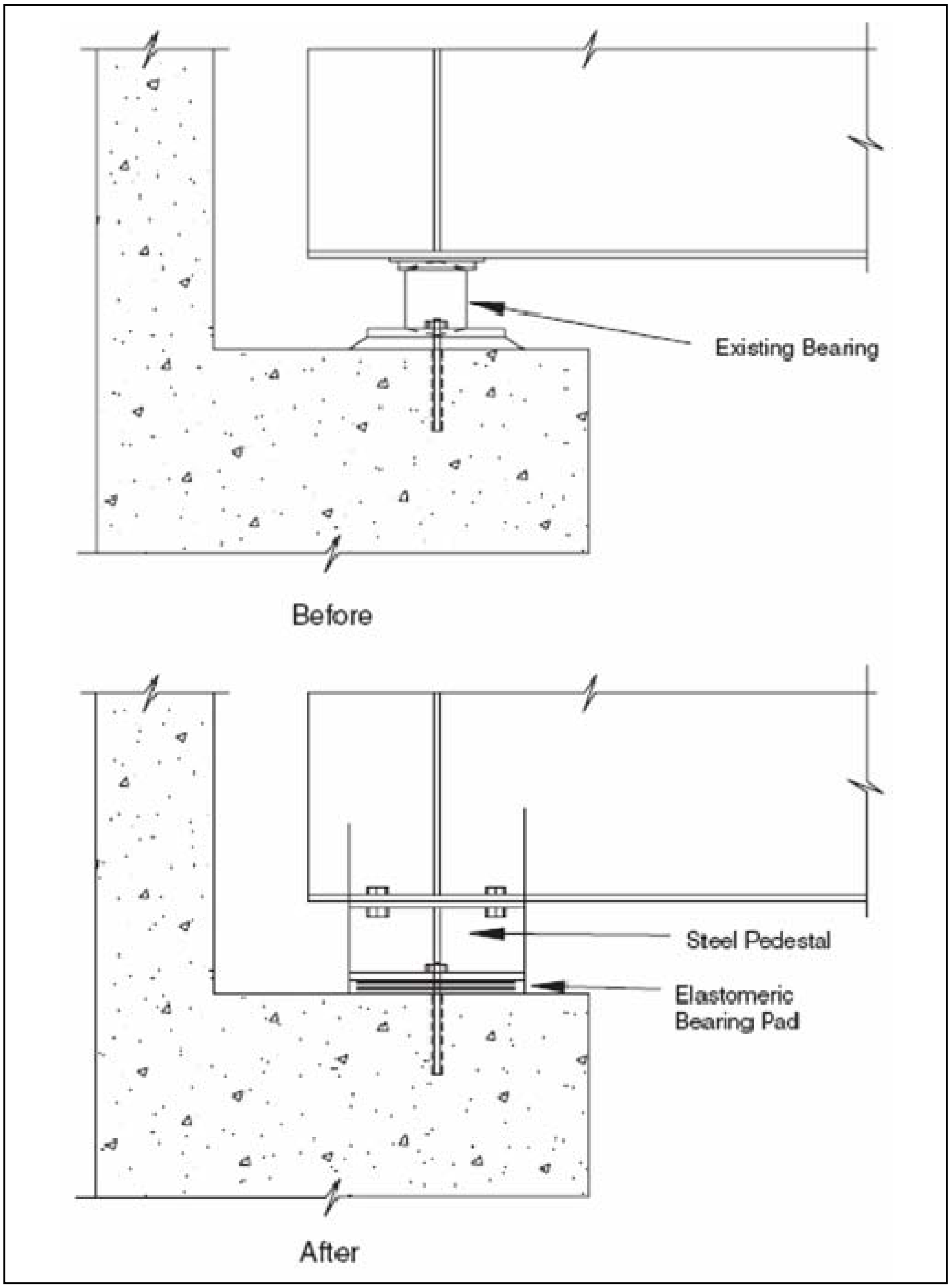

Figure 11. Installation of an elastomeric bearing pad (Retrofit Manual, Figure 8-19.)

5.2 Retrofit Measures for the Substructure

Caltrans witnessed bridge failures in the 1989 Loma Prieta and 1994 Northridge earthquakes despite their prior implementation of retrofit measures such as restrainers and seat width extensions (Retrofit Manual, Section 9.1). Many of these failures were due to insufficient column ductility and flexural and shear strength. Retrofit measures for bridge substructures focus on increasing ductility and shear strength.

5.2.1 Column Jacketing

Employing a technique known as jacketing can increase the ductility of columns. Typically, steel jackets and composite fiber/epoxy jackets are the most popular types of column jackets used (Retrofit Manual, Section 9.2). Steel jackets are basically a solid steel shell that is placed around the column. The gap between the existing column and the steel jacket is pressure grouted with a pure cement grout [Retrofit Manual, Section 9.2.1.3(a)]. Fiber composite jacketing uses high strength glass, carbon, or aramid fibers oriented in the circumferential direction of the column, and bound in polyester, vinyl ester, or epoxy resin matrix [Retrofit Manual, Section 9.2.1.3(b)]. The Retrofit Manual warns of potential freeze-thaw problems in regions of cold temperatures. The moisture that collects between the jacket and the column can freeze and expand creating a weakened and damaged wrap. Also, glass and carbon fibers are weakened by moisture absorption under high temperatures.

5.2.2 Cap Beam Retrofits

General deficiencies in pier caps include inadequate positive and negative reinforcement. This causes plastic hinging in the pier cap and will lead to failure if ductility is not sufficient. The Retrofit Manual recommends that the preferred location for plastic hinging be at the end of the columns. Retrofit design must ensure that the cap beams are either capable of accommodating the ductility demands placed on them, or are capable of elastically resisting the forces that will result from plastic hinging in the columns (Retrofit Manual, Section 9.3).

5.3 Retrofit Measures for the Abutments

Abutments are classified as either seat-type or integral abutments as illustrated in Figure 12. Abutment retrofits are not expected and therefore not discussed in this document.

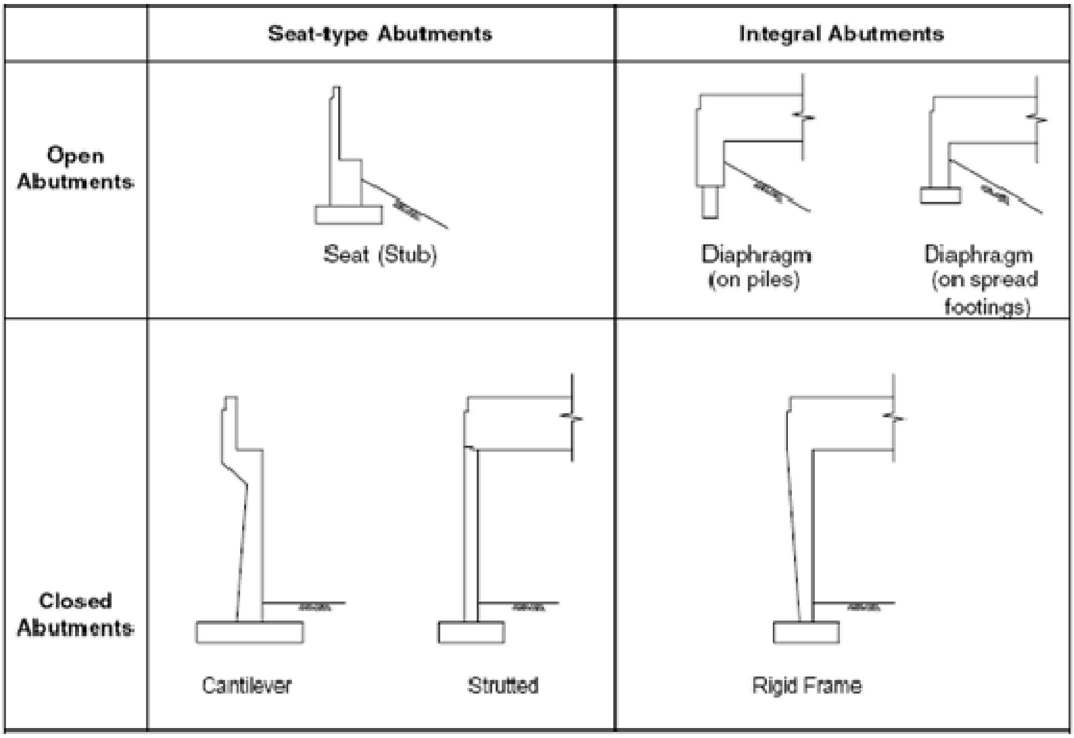

Figure 12. Types of abutments (Retrofit Manual, Figure 10-1.)

6.0 Prioritization Process for the Upper Level Motion

Many methods are available to screen bridges for seismic deficiency and prioritize them based on need for retrofit. These methods typically involve determining the seismic hazard at the bridge site, and the estimating the vulnerability of the bridge to a seismic event. Some methods consider factors such as bridge importance, age, criticality to the transportation infrastructure, and other factors that affect prioritization. Several methods were reviewed including:

- FHWA Seismic Rating Method Using Indices (Retrofit Manual, Section 4.2)
- FHWA Seismic Rating Method Using Expected Damage (Retrofit Manual, Section 4.3)
- Montana Department of Transportation Screening Procedure (MDT, 2004)
- Caltrans Bridge Prioritization Algorithm (Maroney and Gates, 1992; Gilbert, 1994)
- Comparison of Risk Algorithms Used in California, Washington, and Missouri (Caltrans, 2006).

Ultimately, the FHWA's Indices method was selected because it follows current FHWA retrofit guidelines and is easily incorporated in the process. The MDT, WSDOT, and Caltrans algorithms quantify bridge importance by implementing global utility functions. This was initially considered as an advantage; however, the Utah Department of Transportation is developing criticality factors which will be used to quantify the importance factor suggested in the indices method.

The Seismic Rating Method Using Indices is presented in Chapter 4 of the Retrofit Manual. Bridges are ranked based on quantitative assessments of structural vulnerability and seismic hazard. A qualitative assessment of bridge importance, network redundancy, and other factors is then applied to modify the ranking of bridges for retrofit prioritization. This method was originally presented in the 1995 FHWA Retrofit Manual to screen and prioritize bridges (FHWA, 1995). The process is illustrated in Figure 13, and represented by the equation (Retrofit Manual, Section 4.2):

$$P = f(\text{R, importance, non-seismic, and other factors...}) \tag{6-1}$$

where:

P = Priority index

R = bridge Rank based on structural vulnerability (V), and seismic hazard (E)

6.1 Calculation of Bridge Rank

Calculation of bridge rank is a quantitative process where vulnerability (V) and hazard (E) are given a rating from 0 to 10 and rank is determined by multiplying these ratings (Retrofit Manual, Section 4.2.1):

$$R = VE \tag{6-2}$$

The product will result in a value ranging from 0 to 100 where a higher score represents a greater need for retrofit.

6.1.1 Vulnerability Rating (V)

The SRC must be determined to identify which components of the bridge need to be screened for vulnerability. Bridges in SRC A are exempt from seismic retrofit evaluation and do not need to be screened or prioritized. Bridges in SRC B will need to be screened for seismic deficiencies in the bearings, transverse restraints, support lengths and liquefaction. Bridges in SRC C and D require screening for deficiencies in the columns, foundations, and abutments in addition to the requirements for bridges in SRC B. This process is outlined in Figure 13.

The Retrofit Manual suggests that retrofitting the superstructure components (i.e. bearings, connections, seat widths) is an economic approach due to experience with these retrofits and effectiveness of meeting the desired performance level. Retrofitting the substructures (i.e. columns, footings, abutments) is considered more difficult due to limited experience and the high costs associated with these retrofits. Therefore, screening of the substructure components is limited to SRC C and D where higher levels of seismic hazard and bridge performance are expected.

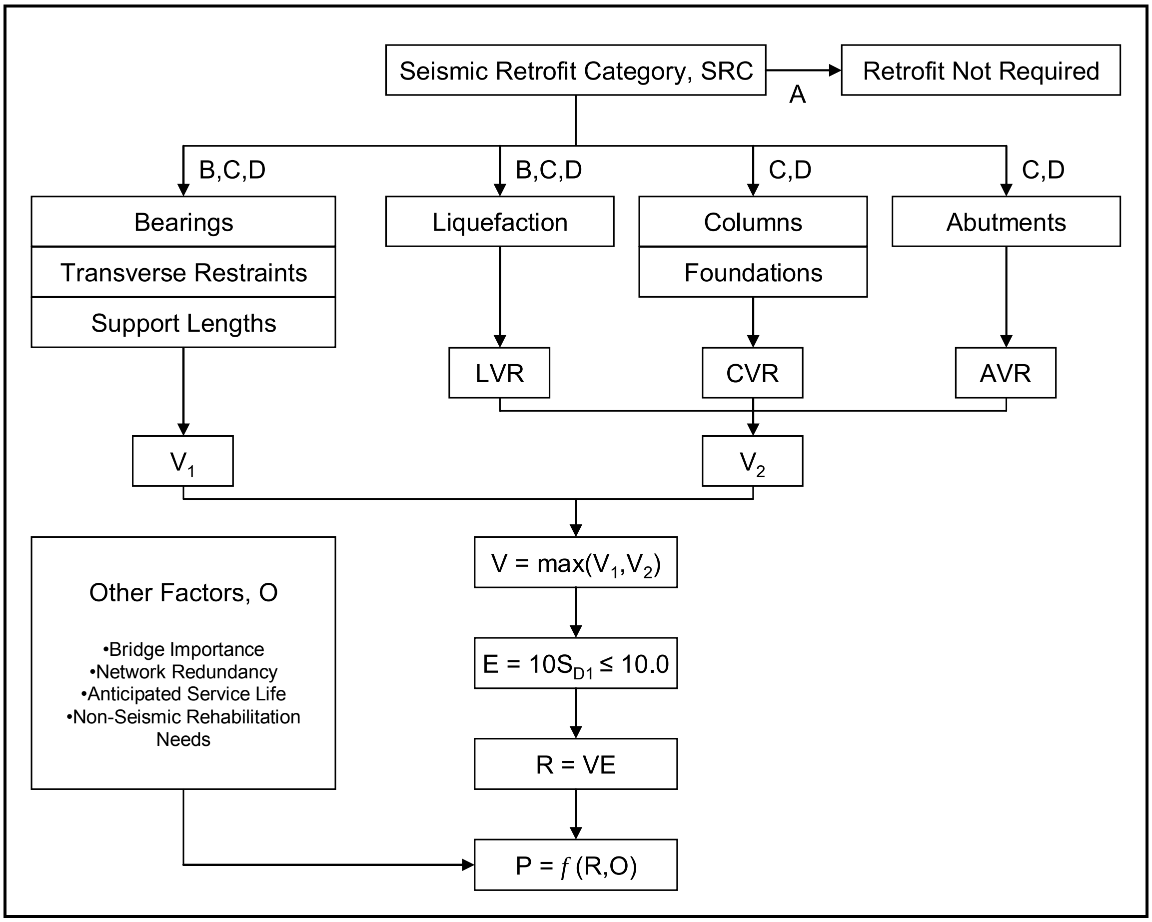

Figure 13. Screening and Prioritization for the Upper Level Motion

The Retrofit Manual provides a step-by-step procedure to determine the vulnerability rating for bearings. This process is illustrated by the flowchart in Figure 14 and the procedure is outlined in the steps that follow (Retrofit Manual, Section 4.2.1.1):

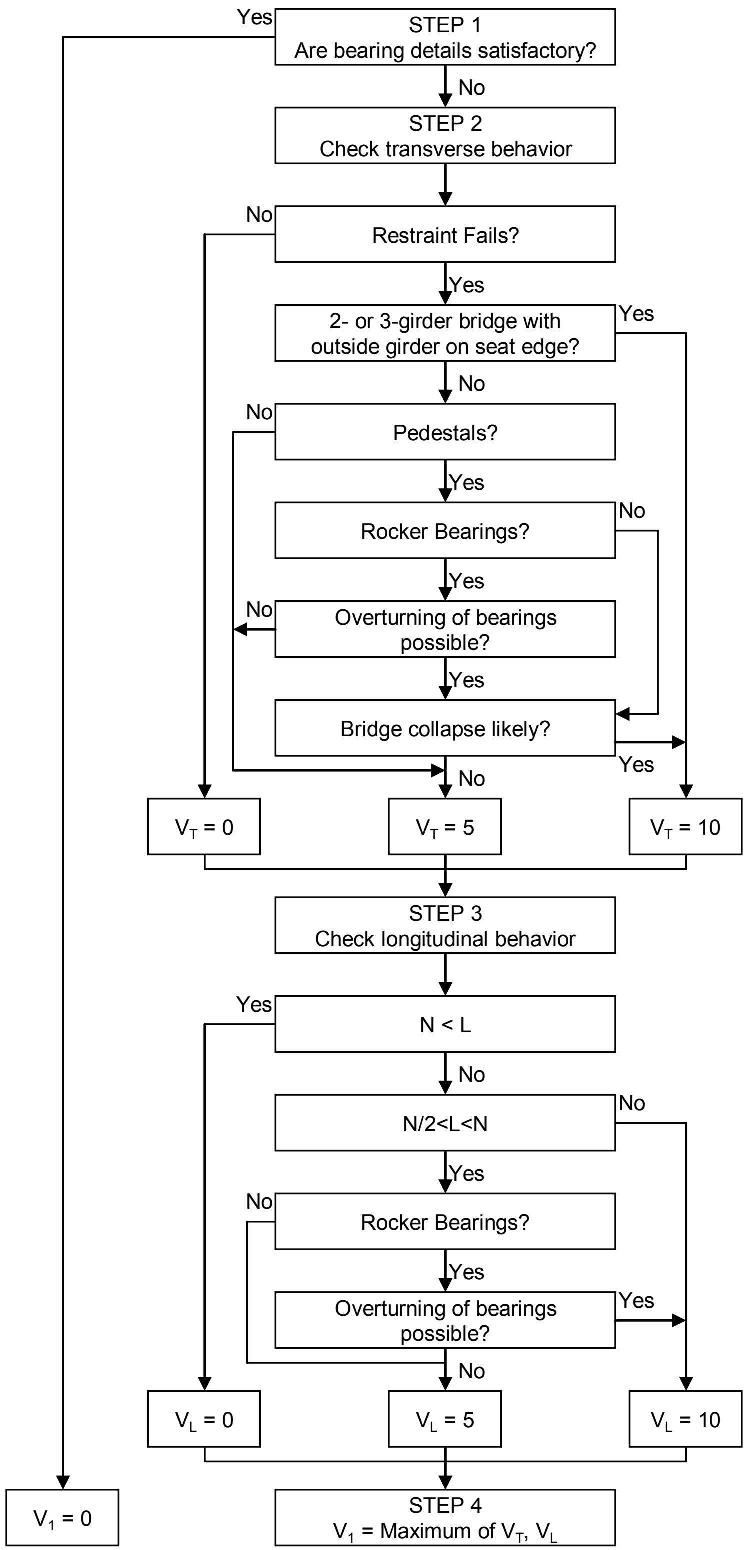

Figure 14. Flowchart for Determining Bearing Vulnerability (Retrofit Manual, Figure 4-4.)

Step1. Determine if the bridge has satisfactory bearing details. Such bridges include:

a. Continuous structures with integral abutments.

b. Continuous structures with seat-type abutments where all of the following conditions are met:

   1. Either (a) the skew is less than 20° (0.35 rad), or (b) the skew is greater than 20° (0.35 rad) but less than 40° (0.70 rad) and the length-to-width ratio of the bridge deck is greater than 1.5.
   2. Rocker bearings are not used.
   3. The abutment's bearing seat under the end diaphragm is continuous in the transverse direction and the bridge has more than three beams.
   4. The support length is equal to, or greater than, the minimum required length (N) given in equation 4-3.

If the bearing details are determined to be satisfactory, a vulnerability rating, V_1, of 0 may be assigned and the remaining steps for bearings omitted.

Step 2. Determine the vulnerability to structure collapse or loss of access to the bridge due to transverse movement, V_T.

Before significant transverse movement can occur, the transverse restraint must fail. In the absence of calculations showing otherwise, assume that the bearing keeper bars and/or the anchor bolts in bridges in SRC C and D will fail. Also assume that nominally reinforced, nonductile concrete shear keys will fail in bridges in SRC D.

When the transverse restraint is subject to failure, beams are vulnerable to loss of support if either of the following conditions exist:

a. Individual beams are supported on individual pedestals or columns.

b. The exterior beam in a 2- or 3-beam bridge is supported near the edge of a bearing seat regardless of whether the bearings are on individual pedestals or not.

In either of these cases, the vulnerability rating, V_T, should be 10.

Steel rocker bearings have been known to overturn transversely, resulting in a permanent superstructure displacement. All bridges in SRC D are vulnerable to this type of failure. Bridges in SRC C are vulnerable only when the skew is greater than 40º (0.70 rad).

When bearings are vulnerable to a toppling failure but structure collapse is unlikely, the vulnerability rating, V_T, should be 5. Otherwise V_T=0.

Step 3. Determine the vulnerability of the structure to collapse or loss of access due to excessive longitudinal moment, V_L.

V_L is determined according to the available support length (L) measured in a direction perpendicular to the centerline of the support. This is done by comparing L with the minimum required length N, (equation 4-1), as follows:

If $L \geq N$ then $V_L = 0$ regardless of bearing type.

If $N > L \geq 0.5N$ and rocker bearings are not used, then $V_L = 5$.

If $N > L \geq 0.5N$ and rocker bearings are used, then $V_L = 10$.

If $0.5N > L$ then $V_L = 10$ regardless of bearing type.

Step 4. Calculate vulnerability rating for connections, V_1, from values V_T and V_L, with

V_1 = greater of V_T and V_L.

The vulnerability rating for the other components in the bridge that are susceptible to failure, V_2, is calculated from the individual component ratings as follows (Retrofit Manual, Section 4.2.1.1(b)):

$$V_2 = CVR + AVR + LVR \leq 10 \qquad (6\text{-}3)$$

The variables CVR, AVR, and LVR are column vulnerability rating, abutment vulnerability rating, and liquefaction vulnerability rating respectively. The Retrofit Manual provides guidelines for calculating each of these ratings. This process is illustrated in the flowchart in Figure 15, and explained in the commentary below.

The liquefaction vulnerability rating, LVR, should be calculated for bridges in SRC B, C, and D. This rating is based on soil susceptibility and seismic hazard. The step-by-step process is taken directly from the Retrofit Manual.

Step 1. Determine the susceptibility of foundation soils to liquefaction.

High susceptibility is associated with the following conditions:

a. Where the foundation soil, that is providing lateral support to piles or vertical support to footings, consists generally of saturated loose sands, saturated silty sands, or non-plastic silts.
b. Where soils similar to a. (above) underlie the abutments fills or are present as continuous seams, which could lead to abutment slope failures.

Moderate susceptibility is associated with foundation soils that are generally medium dense soils, e.g., compact sands.

Low susceptibility is associated with foundation soils that are generally dense soils.

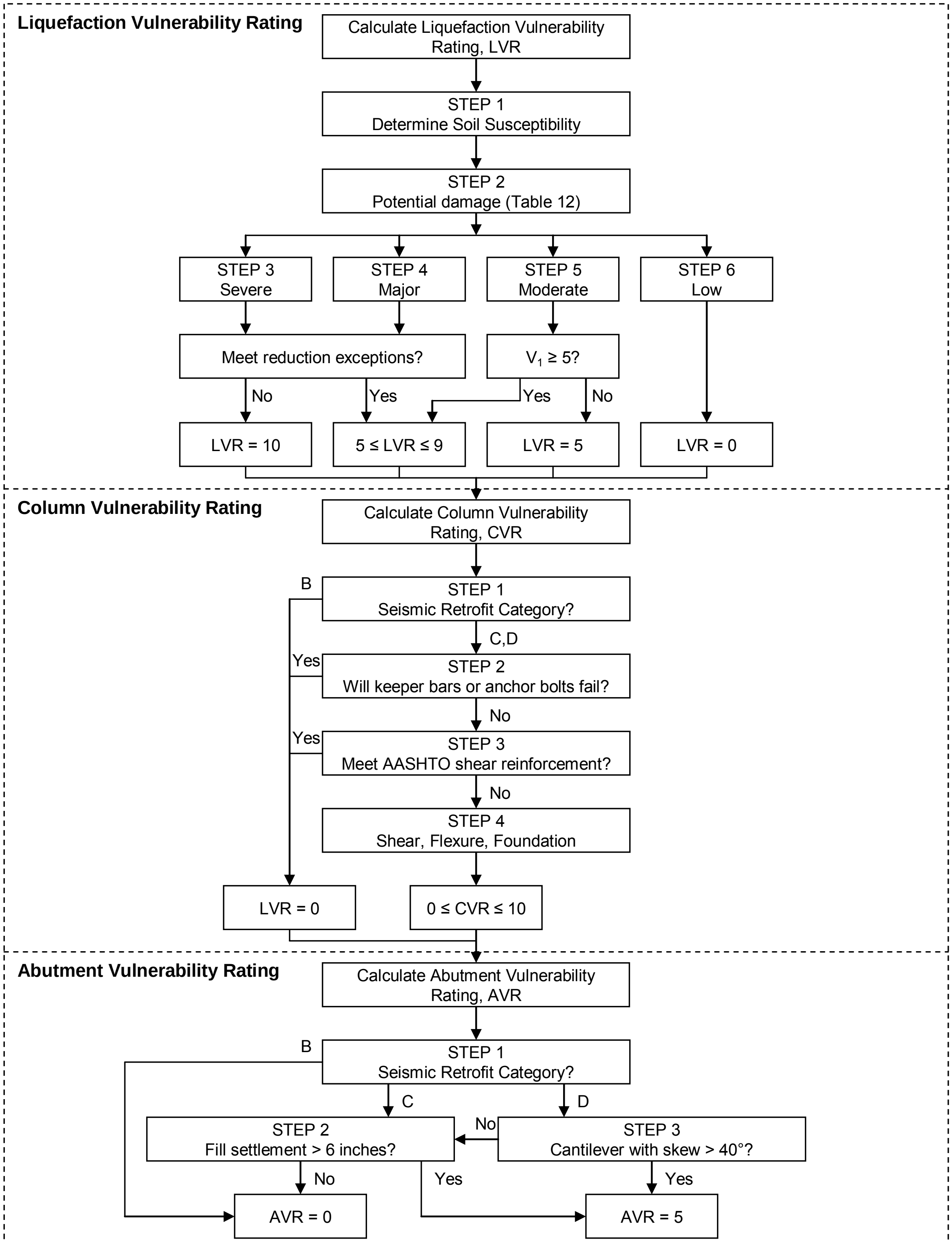

Figure 15. Flowchart for determining V_2.

Step 2. Use Table 12 to determine the potential for liquefaction-related damage where susceptible soil conditions exist.

For all sites where $S_{D1} > 0.49$, engineering judgment should be applied to determine the possibility of greater damage.

Table 12. Potential for liquefaction-related damage (Retrofit Manual, Table 4-2.)

Soil Susceptibility To Liquefaction	**Seismic Coefficient, S_{D1}**				
	$S_{D1} \leq 0.14$	**$0.14 < S_{D1} \leq 0.24$**	**$0.24 < S_{D1} \leq 0.39$**	**$0.39 < S_{D1} \leq 0.49$**	**$S_{D1} > 0.49$**
Low	Low	Low	Low	Low	Low
Moderate	Low	Low	Moderate	Major	Severe
High	Low	Moderate	Major	Severe	Severe

Step 3. In general, bridges subject for sever liquefaction-related damage should be assigned a vulnerability rating, LVR, of 10. This rating may be reduced to 5 for single-span bridges with skews less than 20^o (0.35 rad) or for rigid box culverts.

Step 4. Bridges subject to major liquefaction-related damage should be assigned a vulnerability rating, LVR, of 10. This rating may be reduced to between 5 and 9 for single-span bridges with skews less than 40^o (0.70 rad), and for rigid box culverts and continuous bridges with skews less than 20^o (0.35 rad), provided one of the following conditions exists:

a. Reinforced concrete columns that are integral with the superstructure and have a CVR less than 5 and a height in excess of 8 m (25 ft).
b. Steel columns (except those constructed of non-ductile material) that are in excess of 8 m (25 ft) high.
c. Columns that are not integral with the superstructure, provided that large movements of the substructure will not result in instability.

Step 5. Bridges subjected to moderate liquefaction-related damage should be assigned a vulnerability rating, LVR, of 5. This rating should be increased to between 6 and 10 if the vulnerability rating for the bearings, V1, is greater than or equal to 5.

Step 6. Bridges subjected to low liquefaction-related damage shall be assigned a vulnerability rating, LVR, of 0.

The column vulnerability rating, CVR, is based on shear, flexural, and foundation failure. The Retrofit Manual provides a step-by-step procedure to determine CVR that is provided below.

Step 1. Assign a column vulnerability rating, CVR, of 0 to bridges classified as SRC B.

Step 2. Assign a vulnerability rating, CVR, of 0 if keeper bars or anchor bolts can be relied upon to fail, thereby preventing the transfer of load to the columns, piers, or footings.

Step 3. If columns and footings have adequate transverse steel as required by the current AASHTO Specifications, assign a column vulnerability rating, CVR, of 0.

Step 4. If none of the above apply (steps 1 through 3), check the column for shear, splice details, and foundation deficiencies, and give CVR the highest value calculated from the following steps:

Step 4a. Column vulnerability due to shear failure.

$$CVR = Q - P_R \tag{6-4a}$$

where:

$$Q = 13 - 6\left(\frac{L_c}{P_s F b_{max}}\right) \tag{6-4b}$$

Lc = effective column length,

Ps = amount of main reinforcing steel expressed as a percent of the column cross-sectional area,

F = framing factor:

2.0 for multi-column piers fixed top and bottom,

1.0 for multi-column piers fixed at one end,

1.5 for box girder superstructure with a single-column pier, fixed at top and bottom, and

1.25 for superstructures other than box girders with a single-column pier, fixed at top and bottom.

b_{max} = maximum transverse column dimension, and

P_R = the total number of points to be deducted from Q for factors known to reduce susceptibility to shear failure, as listed in Table 13.

Table 13. Values for P_R.

Factor	P_R
Seismic coefficient, $S_{D1} < 0.5$	3
Skew $\leq 20°$ (0.35 rad)	2
Continuous superstructure, integral abutments of equal stiffness and length-to-width radio < 4	1
Grade 40 (or below) reinforcement	1

Step 4b. Column vulnerability due to flexural failure at splices.

To account for flexural failure when the column longitudinal reinforcement is spliced in a plastic hinge location, the following CVR should be used for columns supporting superstructures longer than 90 m (300 ft), or for superstructures with expansion joints:

$$CVR = 7 \text{ for } S_{D1} < 0.5 \quad (6\text{-}5a)$$

$$CVR = 10 \text{ for } S_{D1} \geq 0.5 \quad (6\text{-}5b)$$

where S_{D1} is the seismic coefficient defined in Section 2.6.

CVR need not be taken greater than seven unless microzoning confirms S_{D1} is greater than or equal to 0.5.

Step 4c. Column vulnerability due to foundation deficiencies.

The following CVR should be used for columns supported on pile footings that are not reinforced for uplift, or for poorly confined foundation shafts. This step is only applicable if microzoning yields values of SD1 greater than or equal to 0.5:

$$CVR = 5 \text{ for } 0.5 \leq S_{D1} \leq 0.6 \quad (6\text{-}6a)$$

$$CVR = 10 \text{ for } S_{D1} > 0.6 \quad (6\text{-}6b)$$

Step 4d. Assign overall column vulnerability rating, CVR.

Set the column vulnerability rating, CVR, to the highest value calculated for CVR in steps 4a, 4b, and 4c.

The abutment vulnerability rating, AVR, is based on estimated abutment settlement during an earthquake. The Retrofit Manual provides a step-by-step procedure for assigning a vulnerability rating based on the seismic design coefficient, S_{D1}:

<u>Step 1</u>. If bridges are classified as SRC B, assign a vulnerability rating, AVR of 0.

Step 2. Determine the vulnerability of the structure to abutment fill settlement. The fill settlement in normally compacted approach fills may be estimated as follows:

a. One percent of the fill height when $0.24 < S_{D1} \leq 0.39$.

b. Two percent of the fill height when $0.39 < S_{D1} \leq 0.49$.

c. Three percent of the fill height when $S_{D1} > 0.49$.

The above settlements should be doubled if the bridge is a water crossing. When fill settlements are estimated to be greater than 150 mm (6 in), assign a vulnerability rating, AVR, for the abutment of 5. Otherwise assign a value of 0 for AVR.

Step 3. Bridges classified as SRC D should be assigned a vulnerability rating, AVR, of 5 regardless of the estimated fill settlement, if all of the following conditions are present:

- Cantilever abutments.
- Skews greater than 40 deg (0.70 rad).
- Distance between the abutment seat and the underside of the footing exceeds 3m (10 ft).

Otherwise, assign a value of 0 for AVR unless the fill settlement calculated in step 2 is greater than 150 mm (6 in), in which case AVR should be 5.

6.1.2 Seismic Hazard Rating (E)

Seismic hazard is determined by applying the design spectral acceleration, S_{D1}, for the 1 second period. The hazard rating is calculated using the equation:

$$E = 10\ S_{D1} \leq 10.0 \tag{6-5}$$

6.2 Calculation of Priority Index

A list showing deficient bridges can be compiled using bridge rank in order of decreasing rank. This list can be reorganized by considering criticality factors such as functional class, economic/political impacts, traffic volume, time to restore use, and emergency response. The Retrofit Manual presents this as a qualitative process that will require a combination of engineering and societal judgment.

Appendix A. Spectral Accelerations for Different Locations and Hazard Levels

This appendix has been provided to illustrate relative seismic hazard throughout the state of Utah. Figures A1 – A4 show spectral coefficients (S_s & S_1 in %g) for the 500 and 2500-year return periods. These maps highlight areas of high seismic risk in relation to areas of lower seismic risk. Spectral coefficients are provided in Table A1 in tabular format for the 100, 500, 1000, and 2500-year return periods for the following urban areas:

- Cedar City
- Logan
- Moab
- Ogden
- Price
- Provo
- Richfield
- Salt Lake City
- St. George
- Wendover

Spectral accelerations represent ground motions that structures are expected to experience during a seismic event. These coefficients are determined from factors such as earthquake magnitude, distance from earthquake source to bridge site, and percent contribution of probable sources. Data based on mean values for these factors is presented in Tables A2 and A3 in tabular format for the 100, 500, 1000, and 2500-year return periods. These factors correspond with the spectral coefficients represented in Table A1. The data was obtained from the USGS website (USGS, 2002).

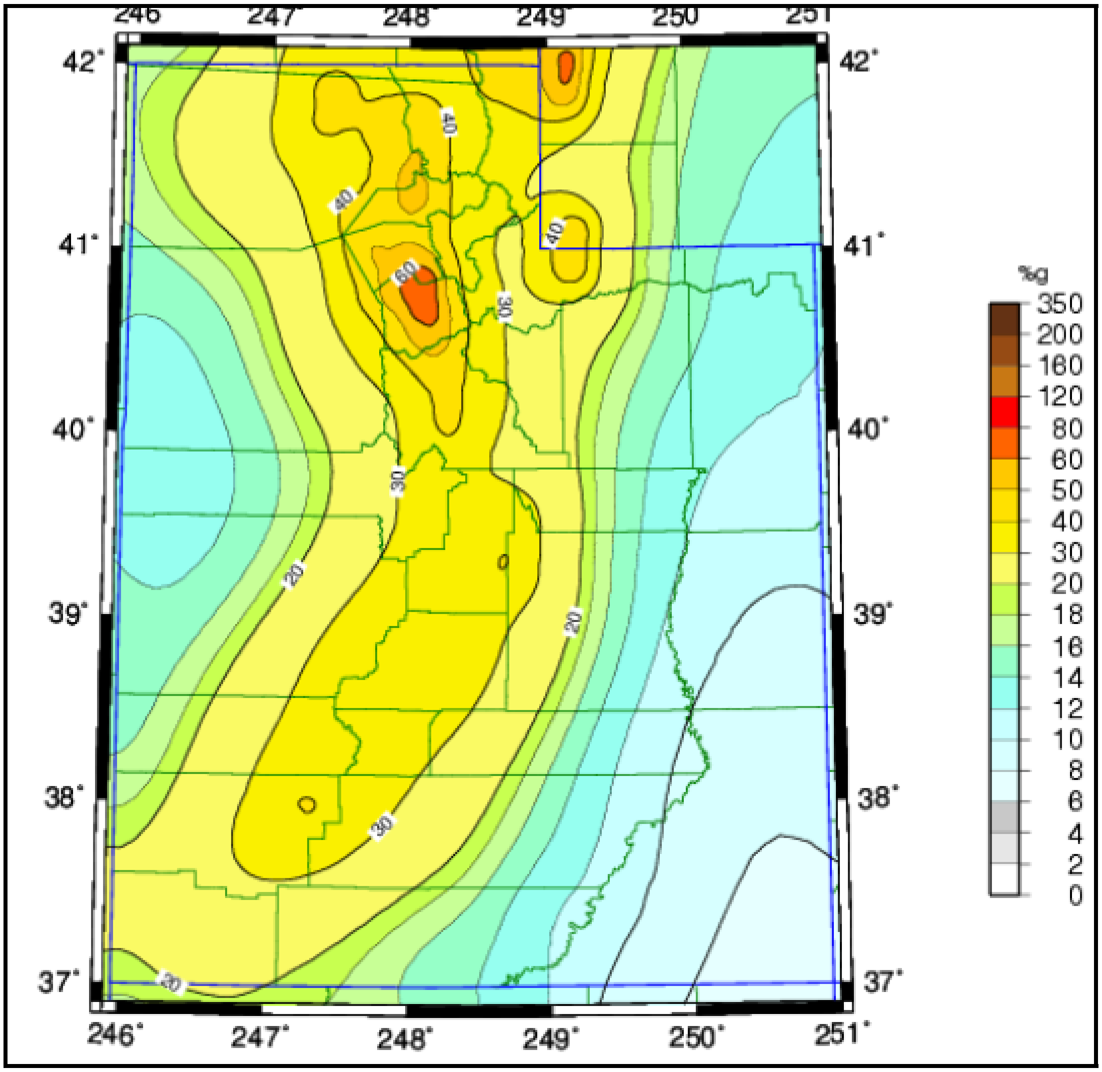

Figure A1. S_s (500-year return period)

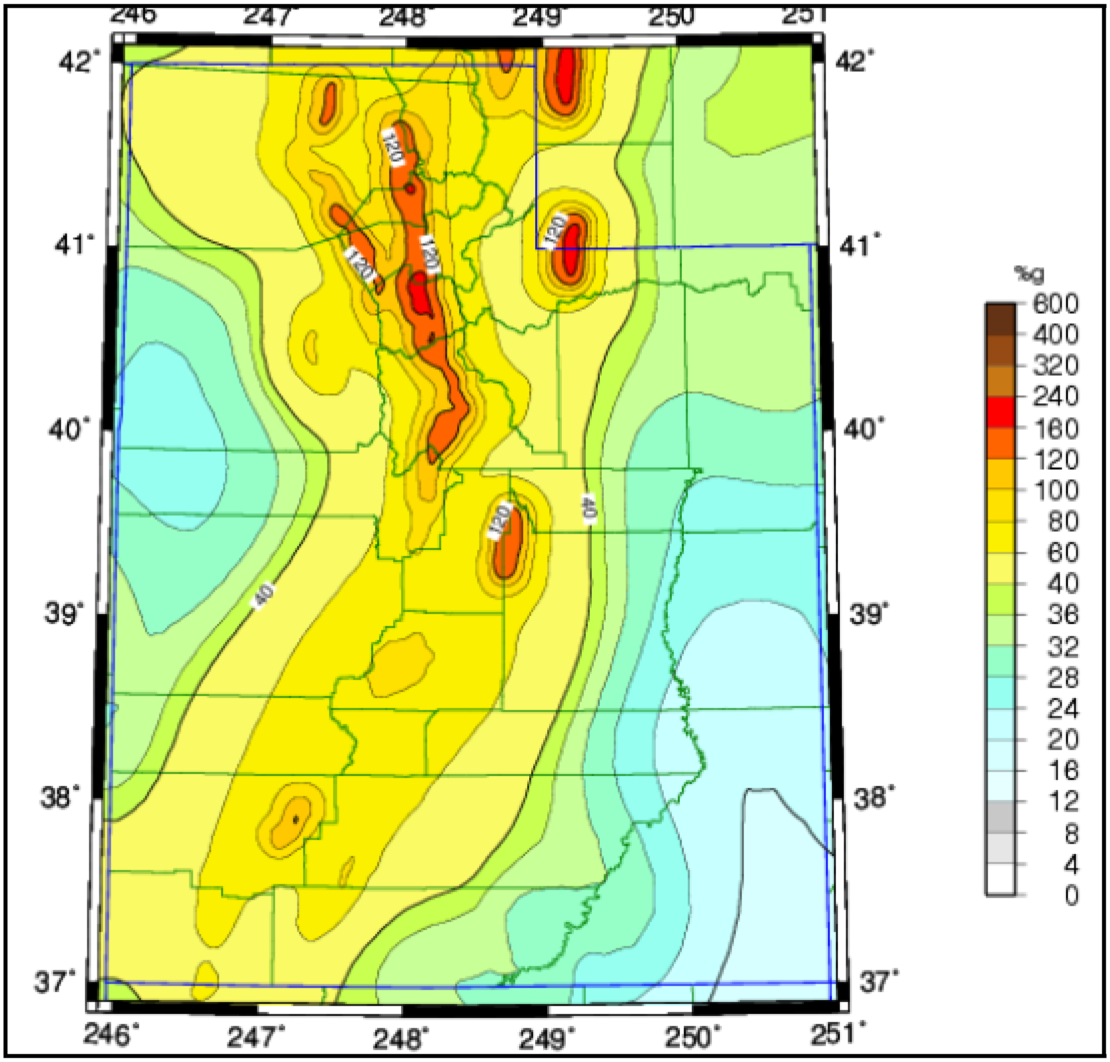

Figure A2. S_s (2500-year return period)

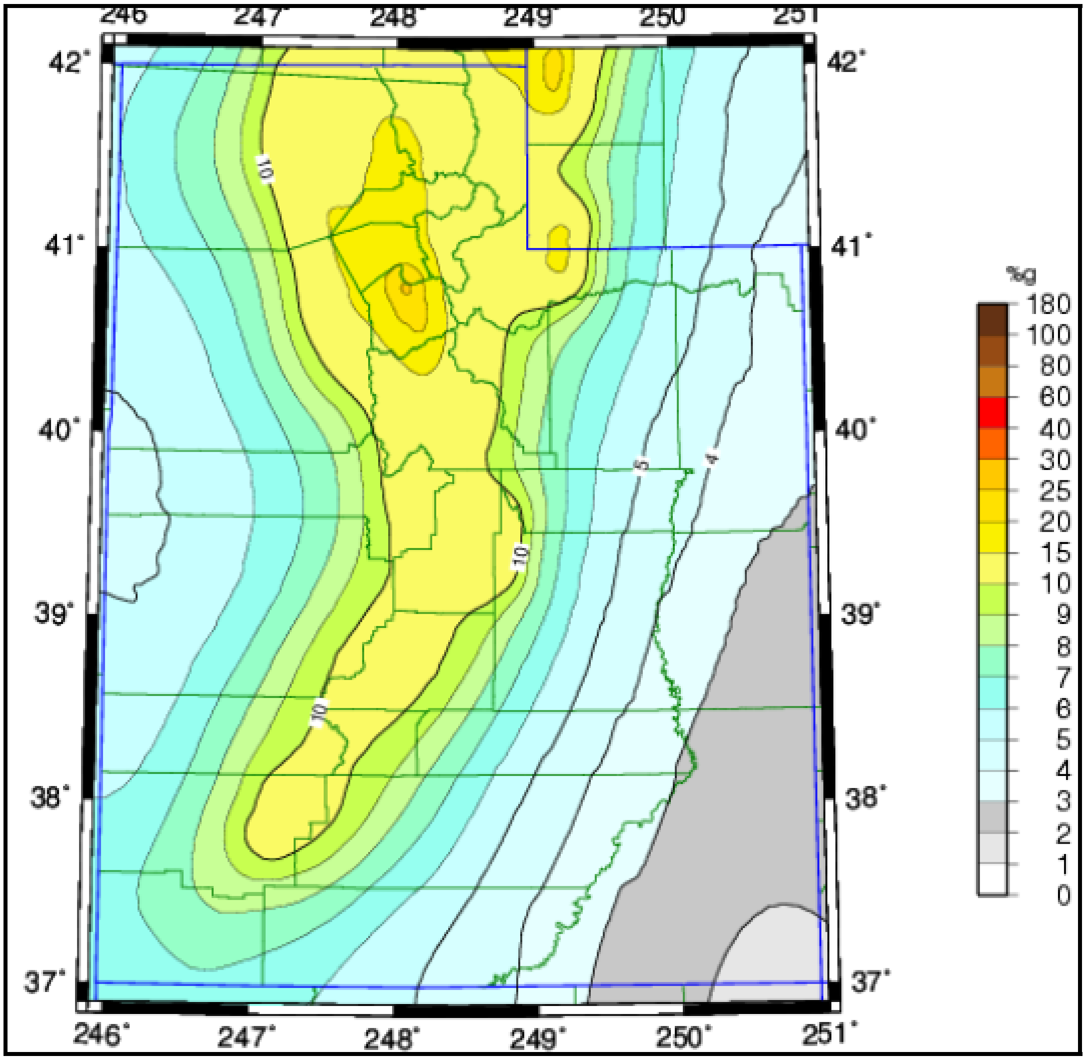

Figure A3. S_1 (500-year return period)

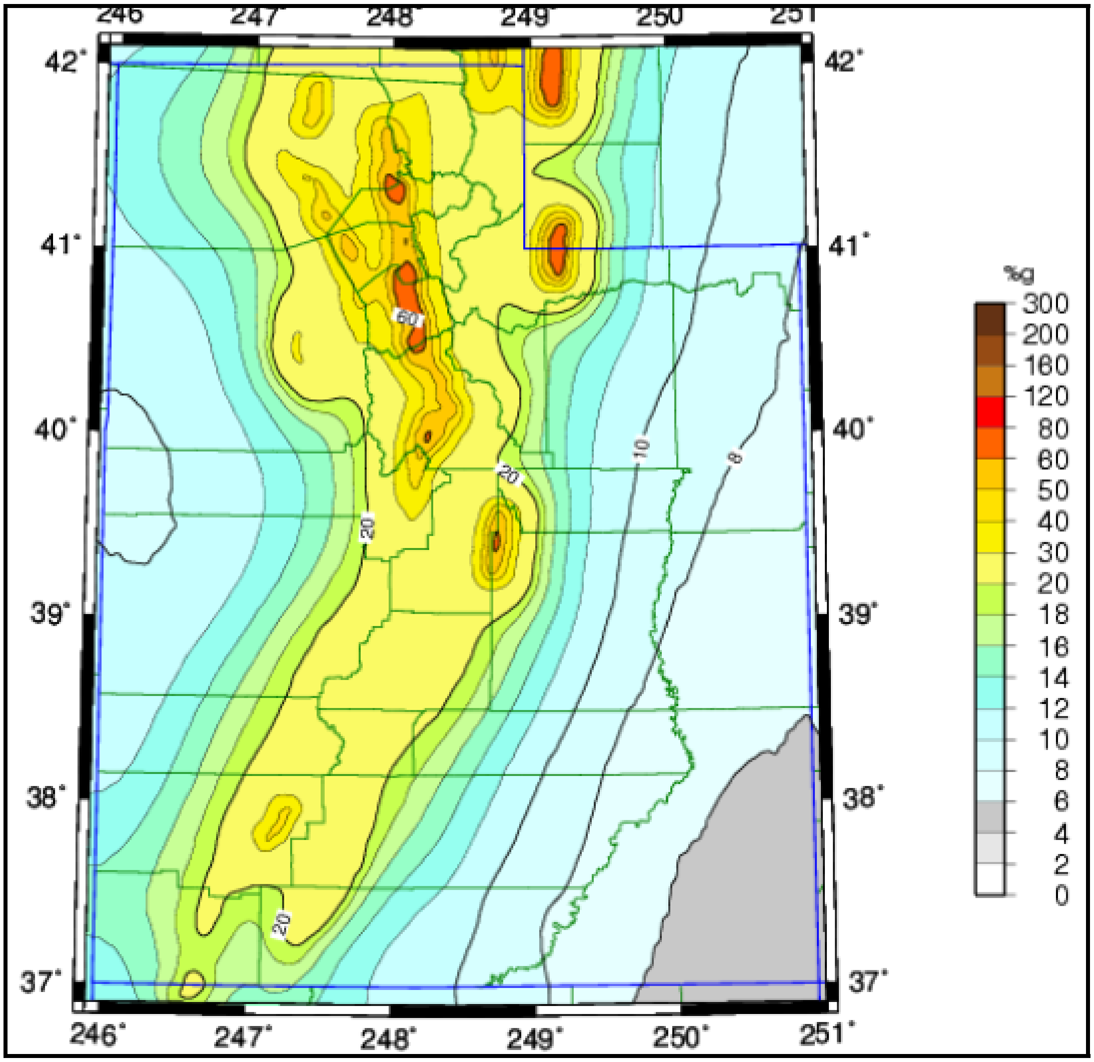

Figure A4. S_1 (2500-year return period)

Table A1. Spectral coefficients for urban areas in Utah

City	Zip Code	Lat (°N)	Long (°W)	100-year		500-year		1000-year		2500-year	
				S_s (%g)	S_1 (%g)	S_s (%g)	S_1 (%g)	S_s (%g)	S_1 (%g)	S_s (%g)	S_1 (%g)
Cedar City	84721	37.7	113.1	12.01	3.33	32.72	9.65	47.03	14.27	71.29	22.61
Logan	84321	41.7	111.7	16.9	5.38	43.08	14.28	61.34	20.74	93.28	33.07
Moab	84532	38.6	109.5	3.74	0.94	9.40	2.81	13.45	4.09	21.26	6.36
Ogden	84401	41.3	112.0	16.62	5.42	52.26	18.37	84.42	32.08	139.74	58.25
Price	84501	39.4	110.9	8.42	2.66	21.57	7.28	30.93	10.33	47.12	15.40
Provo	84603	40.2	111.7	13.64	4.37	42.44	14.36	70.17	25.69	125.09	53.03
Richfield	84701	38.7	112.0	16.22	4.32	40.24	11.03	56.03	15.74	82.51	24.19
Salt Lake City	84111	40.8	111.9	17.27	5.45	70.19	25.13	110.7	42.44	170.4	68.93
St. George	84770	37.1	113.6	8.80	2.68	22.10	7.00	32.27	10.17	50.33	16.08
Wendover	84083	40.7	114.0	6.87	2.40	14.78	5.64	20.22	7.62	30.77	10.98

Notes: 1. Spectral accelerations, S_s & S_1, were determined using zip codes and exact return periods instead of probabilities of exceedance over a specified amount of time.

Table A2. Earthquake data for urban areas in Utah (100 and 500 year return period)

City	100-year						500-year					
	S_s (%g)			S_1 (%g)			S_s (%g)			S_1 (%g)		
	R[1] (mi)	Mag[2]	ε_o[3]	R (mi)	Mag	ε_o	R (mi)	Mag	ε_o	R (mi)	Mag	ε_o
Cedar City	24.1	5.97	-0.22	38.8	6.24	-0.09	12.7	6.12	0.17	21.1	6.43	0.23
Logan	22.9	6.21	-0.03	34.6	6.53	0.06	12.5	6.31	0.31	20.6	6.66	0.42
Moab	102.1	5.77	0.16	129.7	6.12	-0.08	85.0	5.95	0.52	117.2	6.39	0.46
Ogden	20.6	6.36	-0.54	30.6	6.61	-0.45	5.0	6.72	-0.65	7.2	6.85	-0.67
Price	44.7	6.12	-0.10	62.0	6.47	-0.09	22.9	6.24	0.09	38.3	6.63	0.29
Provo	25.7	6.29	-0.47	37.8	6.57	-0.38	8.9	6.63	-0.47	13.4	6.82	-0.46
Richfield	19.1	5.87	-0.01	32.2	6.19	0.11	10.6	5.99	0.35	19.3	6.30	0.43
Salt Lake City	16.6	6.38	-0.88	25.2	6.60	-0.76	3.5	6.71	-0.45	4.2	6.81	-0.44
St. George	38.2	6.02	-0.03	55.5	6.29	0.06	18.4	6.11	0.08	34.4	6.46	0.30
Wendover	59.8	6.16	0.24	81.5	6.51	0.17	35.7	6.11	0.41	65.9	6.59	0.67

Notes: All distances, magnitude, and ε_o are represented by mean values.

1. R = Distance (in miles) from the earthquake hypocenter to the site.
2. Mag = Earthquake moment magnitude.
3. ε_o = The number of standard deviations that a given probabilistic ground motion is from the median motion for the site. A negative ε_o implies the probabilistic motion is less than the median motion expected from the site

Table A3. Earthquake data for urban areas in Utah (1000 and 2500 year return period)

City	1000-year						2500-year					
	S_s (%g)			S_1 (%g)			S_s (%g)			S_1 (%g)		
	R[1] (mi)	Mag[2]	ε_o[3]	R (mi)	Mag	ε_o	R (mi)	Mag	ε_o	R (mi)	Mag	ε_o
Cedar City	9.9	6.2	0.38	15.8	6.5	0.40	7.3	6.3	0.63	10.9	6.55	0.59
Logan	9.6	6.4	0.49	15.7	6.7	0.59	7.2	6.4	0.73	11.1	6.75	0.79
Moab	75.9	6.0	0.60	110.2	6.5	0.69	63.8	6.1	0.70	99.9	6.52	0.94
Ogden	2.8	6.9	-0.08	2.6	7.0	-0.17	2.3	6.9	0.60	1.7	7.02	0.54
Price	18.6	6.3	0.40	29.5	6.7	0.53	15.6	6.2	0.71	22.4	6.68	0.84
Provo	5.3	6.8	-0.18	6.1	7.0	-0.34	3.7	6.9	0.38	2.9	7.08	0.20
Richfield	8.5	6.1	0.55	14.1	6.4	0.55	6.8	6.1	0.82	9.6	6.43	0.73
Salt Lake City	2.2	6.8	0.12	2.4	6.9	0.18	1.7	6.8	0.67	1.7	6.90	0.81
St. George	13.4	6.2	0.23	25.8	6.5	0.40	9.9	6.2	0.55	17.3	6.60	0.59
Wendover	25.1	6.1	0.40	56.9	6.6	0.83	15.5	6.0	0.42	44.2	6.57	0.94

Notes: All distances, magnitude, and ε_o are represented by mean values.

1. R = Distance (in miles) from the earthquake hypocenter to the site.
2. Mag = Earthquake moment magnitude.
3. ε_o = The number of standard deviations that a given probabilistic ground motion is from the median motion for the site. A negative ε_o implies the probabilistic motion is less than the median motion expected from the site

Appendix B. Minutes of Sept. 18, 2006 Joint Meeting Between UDOT and Caltrans

1. Attendees

Utah Project TAC Attendees:

Hugh Boyle, Ray Cook, Daniel Hsiao, Keri Ryan

Caltrans Attendees

Mark Yashinsky	Earthquake Engr. – Post EQ Recovery
Rand Held	Structures Local Assistance
Craig Whitten	Earthquake Engr. - Design
Don Lee	Earthquake Engr. – Isolation
Mark Mahan	Earthquake Engr. – Design
Cynthia MacLeay	Earthquake Engr. – Prioritization
Gary Goff	Structures Local Assistance

2. Screening and Prioritization

Cynthia MacLeay spoke in detail about the screening and prioritization program developed by Caltrans. After the Loma Prieta earthquake, Caltrans was under tremendous pressure to evaluate and fix bridges that might pose a hazard in future earthquakes. Each bridge in the state was subjected to up to three screenings: (1) a general plan screening, (2) a detailed screening, and (3) a third screening if necessary.

General Plan Screening

The general plan screening was a very quick screening (a one page form) based on structural characteristics that determined whether the bridge would be included in the retrofit program. This was used to evaluate 24,000 bridges in the state, and took 30-40 people working over 6 months. The bridges that remained in the program after the general plan screening were grouped into project categories according to location. It proved valuable to retrofit all bridges in a given location together, and higher risk areas, such as LA County or SF County, were given attention first. Also, some bridges were placed in separate categories by type if it made sense to evaluate these bridges with different procedures. Examples are single-span bridges, all bridges along the aqueduct that runs through the center of the state, and Bay Area transit bridges (narrow bridges with high columns).

Detailed Screening

In their project categories, all bridges went through a second level of screening, which looked at structural details and abutment details according to bridge as-built records. At least two engineers evaluated each bridge independently; in some cases three or more evaluations were done to break ties. As the process was described, one gets the sense that it was a somewhat subjective process. If any doubt remained, the bridge was left in the program.

Third Screening

A third screening was started in 1994, which addressed remaining bridges that had not yet been dealt with. This was around the time of Northridge, which influenced the process. The screening had seven criteria, which included abutments, bents, and restrainers. The decision to retrofit these final bridges often came down to money; how much would it cost and how much money was available. Ultimately the bridges were ranked and every bridge above 0.1 rank was retrofitted. Bridges below 0.1 rank were given lower priority and Caltrans is still retrofitting those bridges.

Vulnerability and Risk Algorithms

Around the same time that the screenings were taking place, Caltrans engineers were developing prioritization/risk algorithms. Brian Maroney developed a risk algorithm in 1990 that was used to evaluate the relative risk of various bridges. In 1991-1992, this algorithm was fine-tuned and converted the risk to a rank. MacLeay stated that these risk algorithms were not valuable and ultimately were not used to select bridges for the retrofit program. They may have been used to make a case to the governor to get funding approved for the program, but were mostly academic.

Local Agency program: Design standards and retrofit program for local agencies was the same as for state bridges. The local agency retrofit program was fully funded by the state government, but many local agencies did not go through with the program due to lack of resources (staff) and lack of priority. This funding has now disappeared. The local programs website:

http://www.dot.ca.gov/hq/LocalPrograms has lots of information, including Program Guidelines (Chapter 6 of main Manual) and Procedures (Chapter 11 of Procedures Manual)

Several papers were provided that described the risk algorithms and/or screening:

(1) Development of Vulnerability Analysis Algorithm for Prioritization of Seismic Retrofit Projects – *internal document, not dated*

(2) A report to the governor on the Bridge Seismic Retrofit Program, *dated 1990*

(3) Report to the Seismic Design Advisory Board: Caltrans Screening Process for Seismic Retrofit, *by Cynthia MacLeay, Structures Notes, 1992*

(4) Seismic Risk Assessment and Prioritization of California Bridges, *by Maroney and Gates, Third NSF Workshop on Bridge Engineering, 1992.*

(5) Developments in Seismic Prioritization of Bridges in California, *by Ann Gilbert, 1994*

(6) Sensitivity Study of Bridge Seismic Risk Algorithms Used in the USA, *by Sundstrom and Maroney, not dated*

The last paper describes the results of a comparative study of screening procedures used by different DOTs across the country, and should prove valuable.

Advice for developing a screening program: Evaluate the infrastructure; identify up front unique structures that should be placed in special categories. Fine tune your hazard map before starting the screening program. (Caltrans was not able to do this in 1990 since they had little knowledge of hazards and faults.) Factor in lifeline routes from the very beginning, such as existence of subsidiary routes. (Caltrans also did not think about lifelines until later on.) A screening program should be done in-house. Even with Utah's relatively modest bridge inventory, it would tie up 6 or 7 engineers for some time. Figure out how much money Utah has to work with and where it could provide the most benefit, such as dealing with hinges. When you consider widening a bridge or increasing capacity, evaluate the seismic capacity also and consider a seismic upgrade. This is an indirect means of accomplishing these goals without having money specifically allocated for a seismic retrofit program.

3. General Questions

Question 1: Does MTD 20-4, written in 1995-1996, represent current practice?

Response: MTD 20-4 is currently being rewritten and there will be a lot of revisions.

Question 7: What design spectra do you use?
Response: Caltrans uses deterministic motion for MCE rather than probabilistic motion. This is based on the maximum magnitude earthquake that a fault can produce, and acceleration contours have been generated for all locations considering all adjacent faults. Magnitude and soil combine to give PGA. These are listed in SDC and reported in ATC-32 for the whole state. Sometimes we use probabilistic motion for a bridge with a limited remaining service life. Mark Yashinsky had a paper about this in the conference.

Question 8: What kind of site assessment is carried out?
Response: Site assessment is based on as-built borings. If necessary, a geologist will take additional borings and comment on the potential for liquefaction. The geologist provides the ARS curves.

Question 9: Is STRUDL the primary software used for analysis and why?
Response: Caltrans no longer uses STRUDL and now uses SAP2000 nearly exclusively. Steve Mitchell made the decision to switch to SAP once the GT STRUDL contract expired. One of the considerations is that SAP has very good support for Caltrans, such as people like Bob Morris who will look at the input file, run it, and assess the problem. Also, due to their good relationship, SAP has developed tools for Caltrans such as pushover tool, a live load tool, a bridge model development tool, etc. When using GT STRUDL, Caltrans had to design a front end bridge model generator. ADINA is used occasionally for specialized analyses, and sometimes DRAIN. Caltrans has no experience with LARSA, but it is viewed as a competitor to SAP.

Question 10: How are existing conditions (ie., deterioration, etc.) included in the analysis?
Response: Engineers must analyze the bridge for its existing conditions regardless of the plan. A big issue is to consider scour and estimate the condition of the bridge with long term degradation due to scour, especially short columns.

Question 11: Is Caltrans concerned about lateral spread and its affect on piles?

Response: This rarely comes up because liquefaction is infrequent and even less frequently is accompanied by lateral spread. Options are to strengthen the foundation to take the lateral spread, or use a soil improvement technique.

Question 12: Does Caltrans consider the effects of foundation rocking for bents on spread footings?
Response: Caltrans likes rocking as a way to reduce the costs of construction or retrofit and frequently allows rocking. Suggestions are to consider constraints and fairly evaluate whether the foundation can really rock, and to consider whether something will be built in the future to prevent rocking. A textbook on seismic retrofitting by Priestly has a simple way of dealing with rocking; the newer edition may have a more sophisticated procedure. Ultimately, the iterative procedure definitively evaluates whether the foundation will rock and the resultant forces in the structure. Caltrans uses software written by Steve McBride for rocking, which considers degradation of the soil due to bearing. Caltrans sometimes assumes poorly designed piles, such that the connections to the piles will break and the foundation can rock on the piles. However, a pile footing is much smaller and may not be able to rock as much.

Question 13: When can seismic isolation be used and when is it cost effective?
Response: Isolation and dampers are generally used for large, tall bridges, such as the major toll bridges. It is needed in these cases to avoid costly foundation retrofit and to limit additional work on the superstructure and the substructure. The major toll bridges are steel bridges, so the approach works well. Many of Caltrans typical bridges are monolithically constructed (i.e. superstructure tied directly to column), so isolation is not a good option. Since the typical UDOT bridges have bearings at every bent, isolation might be a better option for UDOT. However, isolation is seen as being more costly for a typical bridge.

Discussion ensues related to confusion over the premise that some of the bridges are partially isolated; ie. bearings are not located at every support. An example is the west span of the Bay Bridge, which only has bearings at certain locations. This is feasible because the Bay Bridge is a suspension bridge, and presumably has a more flexible superstructure. Isolators were used at some locations to: (a) reduce the force transferred to the superstructure at these locations, (b) accommodate large relative displacements when the substructure elements were overly flexible and tried to pull the superstructure away, or (c) to adjust the bridge modes such that the towers would move in-phase rather than out-of-phase (several retrofits like this such as Richmond Bridge). Also, the Bay Bridge was only isolated longitudinally and was completely tied in the transverse direction.

Some comments were made about the lack of faith in isolators, but it was clarified that the bridge components are always designed considering the effect of the isolators, and there is a program in place to periodically remove, test, and replace isolators. Thus, isolation tends to be more costly due to maintenance issues. Caltrans feels less confident about viscous dampers.

4. MTD 20-4 Questions

Question 14: Demand/capacity analysis procedures…
Response: Generally, linear analysis is used to determine demands while pushover analysis is used to determine capacity.
DEMAND: A nonlinear model and history analysis may be needed for more complex structures such as toll bridges. The main procedure referenced in MTD 20-4 is a ductility demand procedure. In this case, they were allowing a moment demand (determined from linear elastic analysis) to moment capacity ratio of 1.5 for rectangular columns and 2 for round columns. This was suspected to be overly conservative, and Nigel Priestly and others started looking at a displacement approach. Caltrans now uses the displacement ductility procedure almost exclusively. To use displacement ductility, the relation between force reduction factor Z and displacement ductility μ must be determined. Mark Mahan states that for a single span structure that behaves like a SDOF system, the relation between Z and μ is a factor of 1.1 to 1.3 (presumably $\mu = 1.3Z$). This factor is for material modeling and so on. Also noteworthy is that the displacement ductility demand procedure is based on cracked section properties rather than gross section properties, which is conservative for a displacement-based procedure but not for a force-based procedure. The equal displacement rule is discussed (displacement demand in a yielding bridge will be the same as if it had remained elastic), which applies only to longer period bridges. Mahan states that Caltrans now recommends to its designers not to design bridges with periods < 0.7 seconds (preferably 1 to 1.5 seconds), so that the equal displacement rule can be applied. For instance, the use of pier walls is strongly discouraged. This led to the

question of whether, for retrofit analysis, a displacement based procedure is appropriate for shorter period structures. Yes, it is appropriate for all periods.

CAPACITY: Capacity analysis involves a section analysis of the columns to establish the moment-curvature relationship, identify plastic hinges and determine a plastic hinge length; then developed a lumped plasticity model that converts the curvature to rotation, and finally do a pushover analysis of the structure to determine the ultimate displacement. The allowable curvature or rotation, which ultimately determines the displacement capacity, is a function of confinement. Section analysis can be done with a program like XTRACT. Caltrans has some older internal programs for section analysis or pushover analysis, like Xsection and Wframe, which are DOS-based, have not been updated for a while, and they do not recommend that we use.

Questions 18, 23, 24: Use of Class P versus Class F retrofit? How to obtain adequate shear capacity, and how to differentiate between small and large (collapse causing) displacements?

Discussion: One apparent difference between Utah and California bridges is that Utah bridge superstructures are supported on bearings while California superstructure to substructure connections are monolithic. Thus, the Class P retrofit for Caltrans bridges involves a pin only at the base while for Utah bridges such an assumption would lead to a pin-pin connection. There was discussion as to how to do this for UDOT bridges, since a pin-pin column will be unstable even for ordinary loads. To clarify, the majority of UDOT bridges are short, up to 3-span structures with a total length of not more than 140 ft, have brittle columns with lap splices from the footing into the column, and these are the bridges that we suspect are deficient.

Question: Class P column retrofit must be designed for adequate shear, so how is the shear demand obtained if it is a pin?

Response: (1) Abutments are taking most/all of the load, so model abutments as springs. (2) Before the connection degrades into a pin at the bottom, it will transfer full shear. If the peak of the moment-curvature diagram is reached, prior to degradation, then the column transfers full shear. However, this is not desirable because the column is designed for full shear but the design does not take advantage of the plastic hinge. (3) Class P retrofit is characterized by a polystyrene shell over the length of the column (partial height column shells are no longer advised). The column shell prevents total collapse of the column, ensuring that it will be able to carry axial load after an earthquake, and should be able to provide adequate shear capacity. The polystyrene wrap allows concrete to expand and lap splices to slide/pin, so that moment is not transferred into the footing and the footing does not require retrofit. A multi-column bent can use a class F retrofit, since moment is not transferred down to the footing anyway.

Question: How do I differentiate between small displacements and displacements that can cause collapse?

Response: (1) Don't allow displacement demand to exceed capacity; use engineering judgment. (2) Caltrans has lots of pin-pin columns. This works if the superstructure is tied to the abutments such that it limits displacement. Retrofit measures for abutments include a dead weight behind the abutment and tied to the superstructure to restrain movement, waffle approach slabs, or 5 foot diameter piles holding the abutment together. (3) The displacement should stay within the middle third of the column; i.e., the displacement demand should be less than 1/3 the column width or diameter. This applies regardless of column height, although height enters indirectly because it increases displacement demand and likelihood of failing these criteria.

Question 19: What is the rationale behind the tension/compression model and is there research to determine its conservatism?

Response: This is really needed for a curved bridge. The tension model represents when the direction of the earthquake acts to open things up, and truss elements are used in the model. The compression model represents when the earthquake acts to close up hinges, and this is modeled by just locking up the hinges. This is a bounding response analysis that considers the worst case if different modes are activated, which otherwise could only be determined with a history analysis. We generally just use the upper bound response provided by the tension or compression model, and don't worry that it will be overconservative. It does not make much difference in cost.

Question 20: Does Caltrans allow pile yielding on new designs?

Response: Yes, Caltrans allows pile yielding in soft and liquefiable soils. The metric is to calculate the ductility demand on the piles. Ductility demand in the range of 1.5 to 2.5 is allowed, which is fairly conservative. In columns, 1.5 translates to barely cracking and 2.5 translates to barely spawling. Abutment piles are treated differently, because if they fail it is not catastrophic, so this type of pile yielding is not really addressed. In soft or liquefiable soils (when piles are designed to yield), better designed piles are used. These are called PISF (pipe with another pile inside).

Question 21: Caltrans allows an abutment displacement limit of 0.2 feet. Is this current practice and what is the justification?

Response: This may be a rule of thumb that the designers use, but practically speaking there is no limit on abutment movement because Caltrans builds very large seats on new bridges. To clarify, this refers to the retaining wall being sheared off. In other words the superstructure plunges into the backwall of the abutment, and the backwall breaks and pushes the soil behind it. In this case, large movements of up to 18 inches are possible. The backwall is designed for back soil pressure and requires reinforcing steel. To analyze, use an assumed abutment stiffness of 20 k/in, and push until the soil reaches its ultimate capacity, 7.7 ksf times the abutment area. When this state is reached, we say the soil is yielded, and no specific displacement is calculated. There is a section in SDC that discusses modeling the abutment soil spring. Caltrans is also doing research to verify the 20 k/in assumed abutment stiffness and this was reported at the conference.

Question: UDOT's bridges are integral with the piles, thus the abutment can move 0.2 feet and mobilize passive pressure, but this yields the top of the piles. The pile would yield before breaking the backwall.

Response: Yielding the piles may cause failure, but not a catastrophic failure. Even if the foundation gets completely destroyed, this is acceptable as long as the bridge does not fail or collapse, because Caltrans approach is to do the minimum possible to prevent collapse. Preventing pile yielding is a very expensive retrofit. As for yielding the piles before breaking the backwall, it may not be avoidable. Cutting down on the amount of rebar in the backwall would create service load problems.

Related discussion: With regard to *transverse* shear, we ask our engineers to calculate the pile capacity, go down to 75% of that, and design the shear keys for values lower than that. Our shear key design is changing; see SDC. In the old days we provided some rebar for the shear key and some other temperature bar passing through. This generates capacities up to 3 times the calculated values. People did not realize they needed to capacity protect the abutments. Under that condition we lowered our lateral capacity to 0.3 g – really 0.9 g. We did some testing at UCSD using new details; we have a new way of estimating and it is much closer to the actual capacity.

Question 22: Is the 40 k/pile EQ resistance independent of pile type? If pile demands exceed the pile capacity, would you include soil resistance beneath the footing?

Response: 40 kip/pile is a number used in the old days, and cannot be supported now. We used to say that if the soil was good, 40 kip/pile was probably a good number. If the piles are good, we could use a number greater than 40 kip. Mark Yashinsky studied the lateral capacity of piles and has a publication on this. 40 kip/pile is not feasible on all piles, but is an average. All the testing was on L piles. UCLA is now doing some testing on 2 foot H piles, which was extended above the ground. We would have to convert to see what it would do below ground.
Question: How is lateral capacity of piles determined?
Response: There are varying degrees of sophistication, such as moment-curvature analysis. If prestress is included, there is specialized software. For our research on L piles, we had a geotech engineer provide good soil properties, one for clay and one for sand. We analyzed the L piles, and came up with values from 12 kips to 50 or 60 kips. This was limited to yield. The abutments are different, because for abutments it is okay to use standard piles or cheaper piles; this study was on standard piles. Now we are trying to compare to shear key piles. There is a report that answers this exact question. We could provide it to you but it is an in-house study so please do not distribute.
Question: What is the maximum length to allow integral abutment?
Response: It is based on thermal, but UDOT allows 350 feet. You could put an expansion joint in the middle, but this might require extra maintenance.

5. Other Discussion

Pipe seat extenders: Caltrans uses a new pipe seat extender, applicable to box girders. The extender spans through the joint and prevents the span from falling if the seat width is exceeded. The pipe seat extender can hold the weight of the span. Restrainers require extremely large forces to prevent unseating, which have other negative impacts. Restrainers are still used, but pipe seat extenders are relied on more and more. It seems that UDOT will not be able to use the pipe seat extender, because it is specially detailed for box girders. Whitten recommends that we first examine bridges for adequate seat width for girders. This is a common problem and an inexpensive fit.

Instability in steel girder bridges: For steel girder bridges with girder depth exceeding 4 ft, inadequate cross-bracing between the girders is a common problem that causes instability in an earthquake; the girders will want to rack over.

Welded hoops: Caltrans uses a standard set of details for new bridges, regardless of location and seismicity. For a little added cost, much better performance can be achieved. This means across the board quality detailing. An example is ultimate splices for hoops. Welded hoops are sampled and tested at every job. Caltrans has gone away from spirals because they have not found a successful ultimate splice for spiral hoops. For smaller piles, we do *extensing* spirals, because it is not feasible to do smaller diameter hoops due to bending and welding and sampling, etc. MTD 20-9 states what diameter hoops are acceptable and what diameter spirals are acceptable.

Epoxy-coated bars: Caltrans uses epoxy-coating only in salt environments and in mountains where the roads are commonly salted. In inland bridges, epoxy coating is not even used in the decks. Caltrans has faith in epoxy-coating for preventing corrosion. There are two kinds of epoxy, and Caltrans uses the newer. In the San Mateo Bridge westbound widening project, epoxy was used everywhere, even for hoops. Epoxy-use is dictated by policy and is not up to designers.

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