EVALUATION OF NCHRP REPORT 472 PROVISIONS FOR SEISMIC DESIGN OF HIGHWAY BRIDGES IN NEW JERSEY

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Abstract

In current AASHTO LRFD Bridge Specifications (2004), the seismic design provisions are based on seismic design criteria and detailing provisions that are at least 10 to 20 years old. These provisions are mostly based on the Division I-A Seismic Design of the AASHTO Standard Specifications for Highway Bridges (1996) and the National Earthquake Hazards Reductions Program (NEHRP) (1997). TRB/NCHRP Project 12-49 "Comprehensive Specifications for the Seismic Design of Bridges" was initiated to address the inadequate performance of highway bridges in recent earthquakes and the deficiencies in the current seismic code, and NCHRP Report 472 was issued with the project completion. The proposed new seismic design specifications are to be considered by AASHTO as a replacement for its existing aged seismic design provisions. New Jersey saw some value in utilizing the proposed new specifications. Because of the several significant changes in the design criteria and approach provided in the new provisions, there are questions on how these new provisions will affect the design and performance of bridge in states nationwide as well as the retrofit of existing bridges. Hence, there was a need to evaluate the impact of the new seismic design for use in design and detailing of bridges in New Jersey. This paper provides an update on a project undertaken to do this evaluation using two 3-span bridge configurations.

Introduction

In 1998 AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) initiated a research project to develop a new set of seismic design provisions for highway bridges which was intended to be compatible with AASHTO LRFD (Load Resistance Factor Design) specifications. NCHRP Project 12-49 (FY'98) was conducted by a joint venture of Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER). NCHRP 12-49 was intended to reflect experience gained during recent damaging earthquake and results of research work conducted in the United States and elsewhere over the last decade, as well as to develop comprehensive specifications for seismic design of bridges considering all aspects of the design process including: (1) design philosophy and performance criteria, (2) seismic loads and site effects, (3) analysis and modeling, (4) design requirements, and (5) detailing. With the completion of NCHRP Project 12-49 "Comprehensive Specifications for the

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Seismic Design of Bridges", NCHRP Report 472, with the same name, was issued (NCHRP, 2002). Along with the report, a proposed new seismic design specification was issued. The purpose of the proposed new NCHRP 472 provisions is to provide seismic guidelines and procedures for bridge design that ensures safety of the public and to minimize structural and non-structural damage with considerations of structural performance, ground motion occurrence probability, cost-effectiveness and son on. Based on this report, MCEER/ATC (2004) recently published Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (Part I: Specifications; Part II: Commentary and Appendices). The proposed new seismic design specifications are to be considered and adopted by the American Association of State Highway and Transportation Officials (AASHTO) as a replacement for its existing aged seismic design provisions. While not adopted by AASHTO in the form presented, the proposed new specifications are available for consideration by jurisdictions to use them in their bridge designs. New Jersey saw some value in utilizing the new specifications. Because of the several significant changes in the design criteria and approach provided in the new provisions, there are questions on how these new provisions will affect the design and performance of bridge in states nationwide as well as the retrofit of existing bridges. There are also questions on the impact of new provisions on the design of abutments and retaining walls, an area beyond the scope of the original project. The purpose of this study is to evaluate the impact of new seismic design provisions proposed in NCHRP Report 472 on the seismic design and detailing of bridges and abutments in New Jersey. In recent years, several major bridges have collapsed and others sustained significant damage during an earthquake. Among the major changes in the proposed new provisions are the increased ground motion accelerations and soil amplification factors. These changes will have a major impact on seismic design of bridges in the Eastern United States. Several states including New Jersey are looking at the impact of these changes on their local, state, and federal bridges. The difference in ground motion acceleration between the expected earthquake (EE or EXP) and the maximum considered earthquakes (MCE) in the Northeast is significant compared to those in the Western United States. Designing all bridges for the MCE in Eastern US would be too conservative and need to be evaluated. The soil amplification factors F_a and F_v are higher than those in the existing provisions, especially for soft soils. These factors are not site-specific to the Northeast and were based on soils mostly in the Western US (Nikolaou, 2001). These factors can be different for different soils in different states. The evaluation of these site factors for New Jersey soils is needed.

The current guidelines for seismic design of bridges in New Jersey are based on the 1998 LRFD Specifications (with interims 1999-2003). These specifications require Safety Evaluation performance level as well as Functional Evaluation level for "critical" bridges. Local road bridges over major facilities, including State and Federal Highways, are designed based on the lesser of the two events. Single-span bridges that are less than 2300 m^2 (25,000 ft^2) in deck area are designed using the 1997 NEHRP response spectra while those exceeding 2300 m^2 require site-specific evaluation. Site-specific is also required for soil profile types E and F (as defined by NEHRP) or when a time-history response analysis to be performed as part of the design/retrofit. Some states design their bridges such that the substructures are capable of resisting all the seismic loads without any contribution from the abutments. Other states use the abutments as key components of the Earthquake Resisting System (ERS). Although the new NCHRP 472 provisions recognize that abutments can be an important part of the ERS and considerable attention should be given to their contribution to the global response of the bridge, more information and guidance is needed for their design and detailing. The proposed new NCHRP 472 provisions have made significant improvements over the current LRFD Specifications. It provides more detailed performance and hazard level criteria that would be very helpful to bridge owners and state officials. It also gives the designer more flexibility in the analysis and design procedures and it provides more updated seismic maps.

In this study, two bridge configurations, seat-type abutment bridge and integral abutment bridge, were chosen to evaluate the impact of new NCHRP 472 provisions on the seismic design and detailing of bridges in New Jersey. These two three-span bridges with each type of abutment were modeled, analyzed, and designed using proposed new and current provisions.

Proposed New Seismic Design Provisions (NCHRP Report 472)

The new seismic design provisions proposed in NCHRP Report 472 are substantially different and significantly more intricate than the existing provisions. It includes many new concepts and several major modifications compared to the existing AASHTO LRFD provisions (basically LFD). Among the new concepts and major changes in the proposed NCHRP 472 provisions are: (1) *adoption of the 1996 USGS Maps* as basis for the rock ground motion of seismic design. The parameters obtained from these maps include the peak ground acceleration (PGA), elastic response ground acceleration for 0.2 sec, 0.3 sec, and 1.0 sec periods of vibrations. These values are available in three different probabilities: 10 percent in 50 years (return period (R.P.) 500 years), 5 percent in 50 years (R.P. 1000 years), and 2 percent in 50 years (approximately 3 percent in 75 years; R.P. 2500 years); (2) *design earthquakes and performance criteria*: the proposed provisions provide two design earthquakes with definite performance objectives and design checks: an upper-level event termed the "rare" or Maximum Considered Earthquake (MCE) which has a 2% probability of exceedence (PE) in 50 years (3% in 75 years); and a lower-level event termed the "expected" earthquake (EE), which has a 50% probability of exceedence in 75 years (40% in 50 years or R.P. of ~100 years). The "rare" earthquake or MCE governs the limits of inelastic deformations in the substructure and the design displacements in the superstructure while the "expected" earthquake event essentially assures an elastic response of the structure with minimum or no damage; (3) *new soil site factors:* where soil sites are classified based on the average shear wave velocity, SPT (Standard Penetration Test) blow-count, or undrained shear strength in the upper 30 m (100 ft) of the site profile; (4) *new spectral shapes:* where the long-period portion of acceleration response spectrum is governed by a spectral shape that decays as 1/T rather than $1/T^{2/3}$; (5) *allowing ERS and Earthquake Resisting Elements* (*ERE)* that are not currently permitted in current AASHTO LRFD; (6*) four seismic hazard levels I, II, III, and IV* based on the design earthquake response spectral acceleration values $S_{DS}= F_vS_1$ and $S_{D1}=F_aS_s$; (7) *five defined SDAP (Seismic Design and Analysis Procedures) A, B, C, D, and E* that reflect the variation in seismic risk are based on seismic hazard level, performance objective, structural configuration, and type of ERS and ERE concepts; (8) *seismic detailing requirements (SDR);* and (9) *design incentives when performing "push over" analysis* in which higher values of the response modification factor, R are used to take ductility into account. The new provisions include pile cap lateral load capacity and displacement evaluation procedures. Liquefaction of soil is also addressed. Abutment design incorporated the research that has been done on bridge abutments over the past 10 years. The proposed LRFD limit state for seismic design for foundations is similar to the current LRFD limit state, which is Extreme Event I except that the γ_p factor for dead loads and earth pressures is taken as 1.0. The resistance factor Φ = 1.0. The load combination and load factors for this limit state is the following:

$$Ext\ Event\ I = 1.0(DC+DD+DW+EH+EL+EV+ES)+\gamma_{EQ}(LL+IM+CE+BR+PL+LS)+WA+FR+EQ$$

NJDOT Seismic Performance and Hazard Levels

The current guidelines for seismic design of bridges in New Jersey are based on the 1998 AASHTO LRFD Specifications (w/interims 1999-2003). The specs also require Safety Evaluation performance level as well as Functional Evaluation level for "critical" bridges including bridges carrying Turnpike traffic. Local road bridges over major facilities, including State and Federal Highways, are designed based on the lesser of the two events. As an alternative to the use of current AASHTO LRFD Specifications, NCHRP Report 472 may be used upon request to NJDOT Bridge Manager. The proposed NJDOT seismic performance and hazard levels based on the new NCHRP provisions are shown in **Table 1**.

Table 1 Proposed NJDOT Seismic Performance and Hazard Levels Based on New NCHRP 472 Provisions

Importance Category	Hazard Level	Seismic Event	Return Period	Performance Criteria (Service and Damage)		
					Operational	Life Safety
Critical Bridge (TPK & Special Bridges)	Upper Level (MCE)	2% PE in 50 years	2500 Years	Service - Damage-	Immediate Minimal	Significant Disrup Significant
	Lower Level (EXP)	10% PE in 50 years	500 Years	Service - Damage-	Immediate Minimal	Immediate Minimal
Essential Bridge	Single Level (2/3 of MCE)	2/3 (2% PE in 50 years	1500 Years	Service - Damage-	Immediate Minimal	Significant Disrup Significant
Other Bridges	Lower Level (EXP)	PE 10 % in 50 Years	500 Years	Service - Damage -	Immediate Minimal	Significant Disrup Significant

The proposed seismic performance criteria and hazard levels are consistent with NJDOT current philosophy in seismic design of bridges in New Jersey, and will be consistent with NYCDOT seismic design criteria (NYCDOT, 2000).

Acceleration Coefficients and Design Response Spectra

The acceleration coefficients for various return periods and various locations in New Jersey are summarized in **Table 2**. Short period acceleration S_s and long period acceleration S_1 are tabulated for the maximum earthquake (2% PE in 50 years) and for the expected earthquake (10% PE in 50 years). Also tabulated are acceleration coefficients for 2/3 of 2500 years event and the 1000 years event (5% PE in 50 years).

Table 2 Acceleration Coefficients in New Jersey for Various Earthquake Events

Location	Accel Coeff (%g)	10% PE in 50 years 500 Years Event (EXP)	2% PE in 50 years 2500 Years Event (MCE)	2/3 of 2% PE in 50 years ~ 1500 Years Event	5% PE in 50 years 1000 Years Event
New NCRP 12- 49 Provisions					
North NJ	S_s =	0.11 - 0.13	0.38 - 0.48	0.25 - 0.32	0.20 - 0.23
Central NJ	S_s =	0.09 - 0.11	0.32 - 0.38	0.21 - 0.25	0.15 - 0.20
South NJ	S_s =	0.06 - 0.09	0.18 - 0.32	0.12 - 0.21	0.10 - 0.15
North NJ	S_1 =	0.025 - 0.027	0.09 - 0.10	0.06 - 0.07	0.048 - 0.052
Central NJ	S_1 =	0.023 - 0.025	0.08 - 0.09	0.05 - 0.06	0.042 - 0.048
South NJ	S_1 =	0.02 - 0.023	0.06 - 0.08	0.04 - 0.05	0.03 - 0.042
Existing LRFD Specifications *					
North NJ	A =	0.18 - 0.20			
Central NJ	A =	0.16 -0.18			
South NJ	A =	0.08 - 0.16			
** PGA Maps*					

The design spectral accelerations for various return periods (2500 years, 2/3 of 2500 years (~1500 years), 1000 years) in North New Jersey in soil type E (soft type) is shown in **Figure 1**. Also shown are the spectral accelerations from the current LRFD specifications. The 2/3 of 2500 years return period accelerations are considered for the essential bridges. For local and ordinary bridges, designs will be based on the lower level (expected) earthquake.

The design response spectra for a bridge located in North New Jersey based on the proposed new provisions are shown in Figures 2 and 3 for different soil sites. **Figure 2** shows the spectra for the upper level earthquake (MCE) and **Figure 3** shows the spectra for lower level earthquake (EXP). A comparison of the response spectra from the new and existing provisions for a bridge located in New Brunswick, NJ in soil type E is shown in **Figure 4**. In **Figure 4** and for this particular location, the short period acceleration from the new provisions is approximately 60% higher than that of the existing provisions, while the long period acceleration is smaller beyond the 1 sec period. Although not shown herein, several plots were developed for various locations in New Jersey. For very stiff soil conditions, the existing provisions could result in higher short period accelerations compared to the new provisions. The soil type has a major effect on the design response spectra in the new provisions. This effect is more significant in the new provisions compared to the current provisions. Site-specific analysis is very important. The combination of higher return period and soft soil conditions will result in higher design accelerations in New Jersey and in the Northeast in general. The 2500 year return period could be applied to critical bridges, however, for essential and ordinary bridges in the state, lower return periods need to be considered.

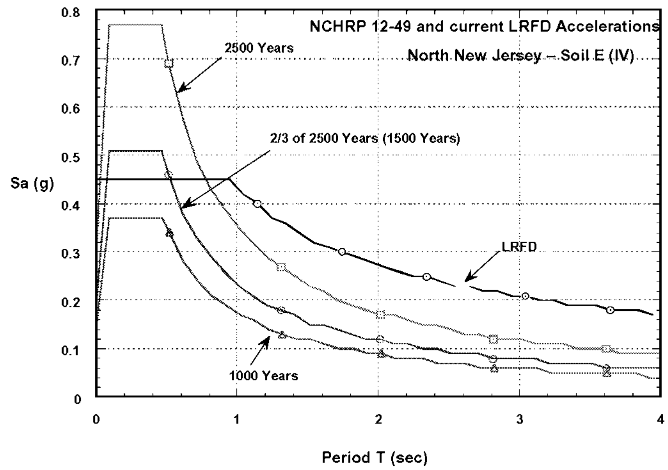

Figure 1 Design Response Spectra for Various Return Periods in North NJ

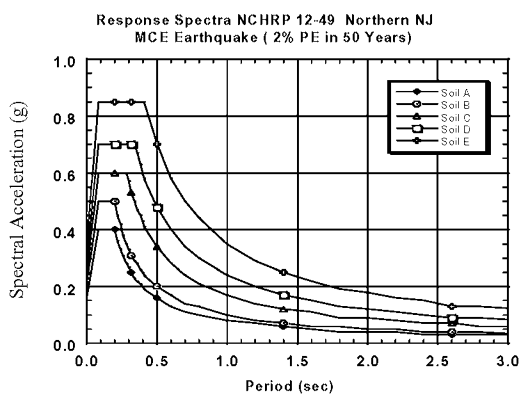

Fig. 2 MCE Response Spectra for Different Soils

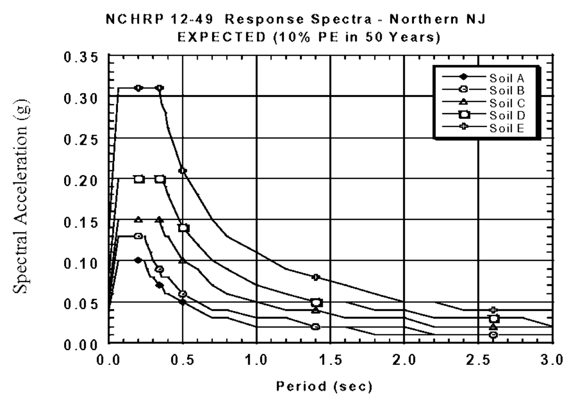

Fig. 3 EE Response Spectra for Different Soils

<u>Case Studies – Design Examples</u>

Bridge Description and Analysis Results

The structures considered in this study are: 1) a three-span seat-type abutment bridge with fixed bearings on two-column piers, and 2) a three-span integral abutment bridge with fixed bearings on two-column piers. The piers are supported on pile foundations. **Figures 5** and **6** show plans, elevations, and sections of the two structures. The span lengths are 24.5 m (80 ft), 30.5 m (100 ft), and 24.5 m (80 ft). The columns are 1.22 m (4 ft) in diameter. The footing dimensions are 9.1x4.6 mxm (30x15 ftxft). Integral abutment piles are HP 360x152 (HP14x73).

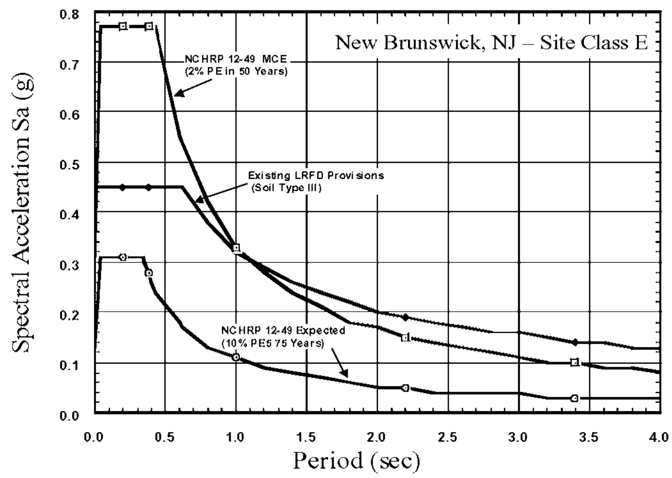

Figure 4 Design Response Spectra from Proposed New NCHRP and Existing LRFD Provisions

Two analytical models were created to model the two structures. These models are shown in **Figures 7** and **8**. Translation springs are used to model the pile foundations under the piers as well as the passive pressure behind the integral abutments. Springs were also used to model the transverse stiffness of the abutments. The height of abutment walls was taken as 3.05 m (10 ft) and the effective width of wing walls equal to 1.52 m (5 ft). The design response spectra from both provisions were input in the program. The soil chosen in this investigation was soft soil (V<180m/s). This soil is classified as Type E in the proposed new NCHRP provisions and Type IV in the existing LRFD provisions.

Modeling

The two bridges were modeled as shown in **Figures 7** and **8**. The pile foundations at the pier were replaced by two translation springs. In the transverse direction, the abutment restraints were two transverse springs that account for abutment piles and effective length of wing walls. The expansion bearings at the seat abutments were free to move with no soil resistance. In the integral abutment model, similar springs were used at the piers and in the transverse direction. In the longitudinal direction, the passive resistance of the soil was represented by a translation spring in the longitudinal direction. The spring stiffness was calculated using the guidelines in the new provisions. For the existing provisions, the spring constant was computed using passive resisting force of the soil and a certain prescribed soil movement.

Analysis Results

The two bridges were analyzed using both the new NCHRP provisions and the existing LRFD provisions, for three different geographic locations in the state of New Jersey – North, Central, and South. Results of the seismic analysis of the two models are shown in **Table 3**. **Table 3** shows base shears, bending moments, and displacements in the longitudinal and the transverse directions. Also tabulated are abutment forces and soil pressures.

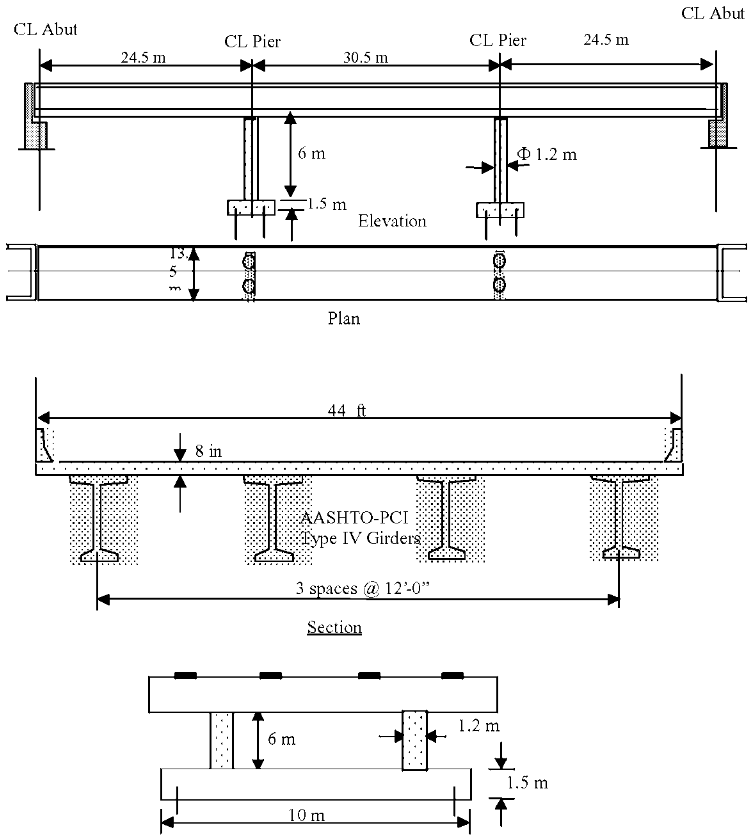

Figure 5 Plan, Elevation and Sections of Seat-type Abutment Example

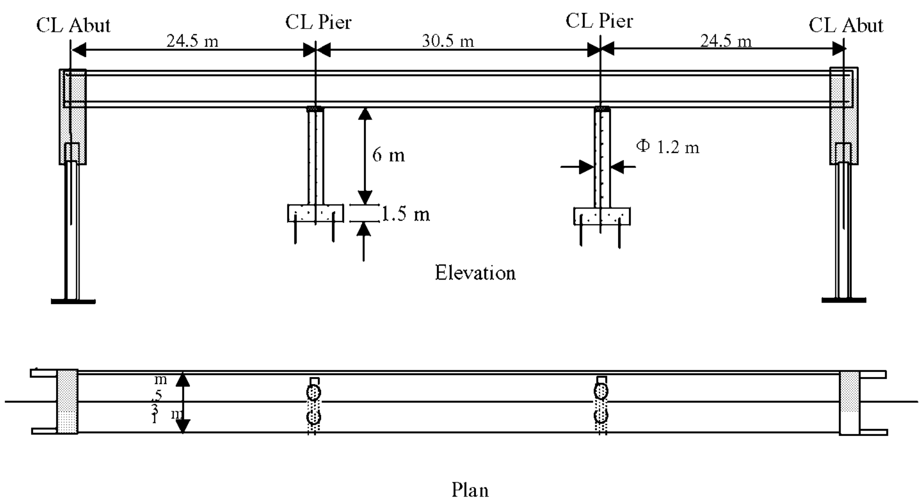

Figure 6 Plan and Elevation of Integral Abutment Example

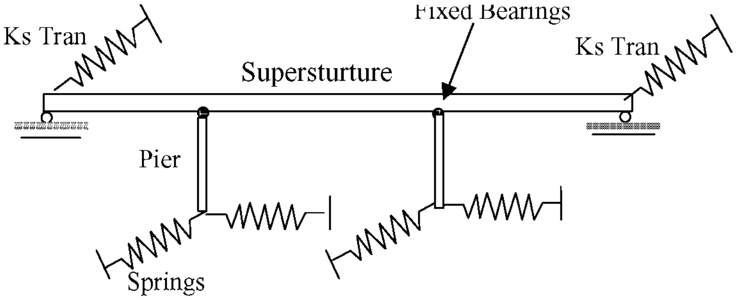

Figure 7 Model of Seat-type Abutment Example

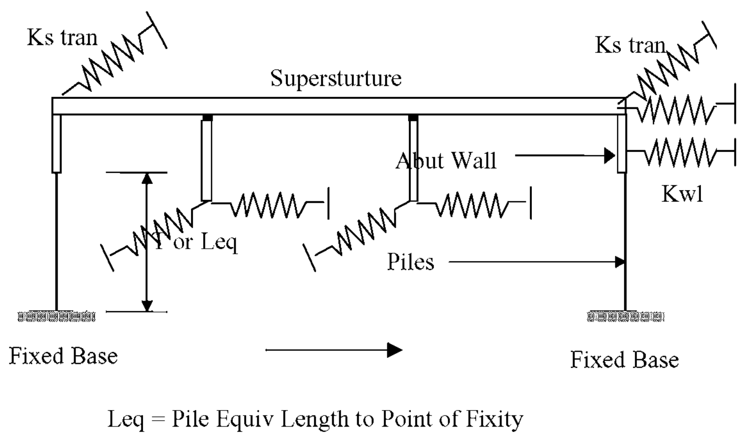

Figure 8 Model for Integral Abutment Example

Table 4 shows the response modification factors (R-factor) from both provisions used to modify the earthquake loads obtained from the analysis. For the bridge configuration chosen in this study, the maximum displacement of seat-type abutment under the maximum earthquake (2500 years event) for the structure located in North New Jersey was about 90 mm (3.54 in) using the new NCHRP provisions and about 105 mm (4.13 in) using the current LRFD specifications. For the integral abutment bridge, these displacements were 15 mm (0.6 in) and 35 mm (1.7 in) respectively. The passive soil pressure behind the abutment wall increased the stiffness of the bridge and reduced its period of vibration ($T_1 = 0.485$ sec) compared to the seat-type abutment bridge ($T_1 = 0.967$ seconds). The maximum soil pressure behind the integral abutment wall was 1.45 KN/m^2 (3.5 ksf) for the 2500 years event. This value was less than the maximum value specified by NCHRP 472 provisions, however, it was about three times the pressure resulting from temperature variations.

Design Procedures in NCRP 472 Provisions

The six SDAP (Seismic Design and Analysis Procedure) given in the proposed new provisions are: A1, A2, B, C, D, and E depending on the seismic hazard and performance levels. In these two examples, the seismic hazard levels were III for Central and South New Jersey and IV for North New Jersey for the safety level performance of the MCE earthquake (2% PE in 50 years). Based on Table 3.7-2 in the proposed Specifications in NCHRP 472, SDAP B, C, D, and E can be used for level III and SDAP C, D, and E can be used for level IV. The SDR (Seismic Detailing Requirements) was 3 for Central and South New Jersey and 4 for North New Jersey. SDAP C is a Capacity Design Spectrum Method (CDSM) in which demand and capacity analyses are combined. This SDAP can only be used in bridges that satisfy the requirements of Section 4.4.2 of the new provisions. The examples analyzed here satisfy these limitations and hence this SDAP can be used in both directions. Capacity Design Spectrum Method is a relatively simple procedure and should be used whenever applicable. SDAP D is an Elastic Response Spectrum Method (ERSM). This procedure is a one step procedure using elastic (cracked section properties) analysis. Either the Uniform Load or Multimode method of analysis may be used. These two examples were analyzed and designed using SDAP D. Results of the analysis are shown in **Table 3**. R-factors and the design summary of columns and piles are shown in **Tables 4** and **5** respectively. SDAP E is an Elastic Response Spectrum Method with displacement capacity verification (pushover analysis). This SDAP is similar to SDAP D except that the response modification factors R are increased. This SDAP was not used in these two design examples.

Table 3 Summary of Forces, Moments and Deflections from Elastic Spectrum Analysis

Seat -Type Abutment SDAP D - Elastic Response Spectrum Ananlysis I eff = 0.4 Ig

T 1 = 0.967 sec T 2 = 0.485 sec

Provisions	Location	Column shears and moments				Piles shears and moments				Displacements				Abutment forces		Long Soil Pressure
		V L	V T	M T	ML	V L	V T	M T	ML	Abut x	Abut y	Pile x	Pile y	Px	Py	
Exisiting	South NJ	1370	351	8264	2143	1392	378	10480	2705	58.7	14.2	9.7	2.8	-	1010	0
LRFD 1998	Central NJ	2055	525	12547	3214	2091	569	15727	4057	86.6	22.4	14.5	4.2	-	1517	0
	North NJ	2469	632	15051	3855	2509	685	18866	4597	105.7	27.2	17.3	5.1	-	1824	0
New	South NJ	1632	818	9956	4909	374	890	40913	6296	69.1	34.8	11.7	6.6	-	2335	0
NCHRP	Central NJ	1859	916	11327	5597	427	996	46535	7060	78.5	37.3	13.5	7.6	-	2624	0
12-49	North NJ	2046	1023	12463	6253	470	1116	51196	7888	90.2	41.9	14.7	8.4	-	2927	0

Integral Abutment SDAP D - Elastic Response Spectrum Ananlysis I eff = 0.4 Ig

T 1 = 0.485 sec T 2 = 0.358 sec

Provisions	Location	Column shears and moments				Piles shears and moments				Displacements				Abutment forces		Long Soil Pressure
		V L	V T	M T	ML	V L	V T	M T	ML	Abut x	Abut y	Pile x	Pile y	Px	Py	
Exisiting	South NJ	182	351	1121	2156	227	383	1474	2720	8.4	14.5	1.8	2.8	2397	1010	0
LRFD 1998	Central NJ	276	529	1695	3234	338	574	2227	4082	12.4	21.6	2.8	4.3	3590	1512	0.669
	North NJ	334	636	2027	3878	409	689	2673	4898	15.0	25.9	3.3	5.3	4315	1819	0.805
New	South NJ	600	818	3648	4990	374	890	15172	6296	25.9	34.0	5.3	6.6	5800	2335	1.078
NCHRP	Central NJ	716	916	4459	5597	427	996	18544	7061	31.8	37.8	6.6	7.6	7086	2624	1.317
12-49	North NJ	810	1027	4795	6253	470	1116	20470	7888	35.1	44.2	7.4	8.4	7828	2927	1.454

Units: Shears in KN, Moments in KN-m, Displacemnts in mm, Soil Pressure in KN/m^2

Table 4 Comparisons of R-Factors for Both Provisions

Location	Substructure Element	Existing LRFD			New NCHRP 12-49 SDAP D			
		Critical	Essen	Others	T	T_s	R_B	R
Seat Type Abutment								
South NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.967	0.443	4.0	**4.0**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.967	0.443	2.0	**2.0**
Central NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.967	0.409	4.0	**4.0**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.967	0.409	2.0	**2.0**
North NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.967	0.412	4.0	**4.0**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.967	0.412	2.0	**2.0**
Integral Abutments								
South NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.485	0.443	4.0	**3.6**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.485	0.443	2.0	**1.9**
Central NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.485	0.409	4.0	**3.8**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.485	0.409	2.0	**1.9**
North NJ	Multipl Column bent	**1.5**	**3.5**	**5.0**	0.485	0.412	4.0	**3.8**
	Vertical Piles	**1.0**	**1.75**	**2.5**	0.485	0.412	2.0	**1.9**

Table 5 Column and Pile Design – SDAP D

Provisions	Location	Column Design				Piles	
		ρ_{long}	Pu	Mu	Φ Mn	V_{EQ}/R	N_{piles}
Seat Type Abutment							
Exisiting LRFD	South NJ	1%	2313	2712	2712	721	12
	Central NJ	1.5%	2313	3390	3390	1083	24
	North NJ	2%	2313	4068	4068	1299	28
New NCHRP 12-49	South NJ	1.2%	2,313	2,780	2,983	943	20
	Central NJ	1.5%	2,313	3,159	3,390	1072	24
	North NJ	2%	2,313	3,485	4,068	1183	26
Integral Abutment							
Exisiting LRFD	South NJ	1%	2268	651	2712	254	6
	Central NJ	1%	2268	1037	2712	379	8
	North NJ	1%	2268	1252	2712	689	16
New NCHRP 12-49	South NJ	1%	2268	1718	2712	588	14
	Central NJ	1%	2268	1882	2712	684	16
	North NJ	1%	2268	2075	2712	761	18

Units: Shears in KN, Moments in KN-m

V_{EQ}/R = design earthquake load on piles

Transverse Column Reinforcement in Plastic Hinge Zones

The requirements for column design in the new provisions are more complicated than those in the current LRFD provisions. Column design based on the new NCHRP provisions showed that the design was controlled by the requirements to restrain longitudinal bars in plastic hinge zones. In the current LRFD provisions, column transverse reinforcement was also controlled by confinement in the plastic hinge zones. **Table 6** shows comparison of transverse reinforcement requirements for the new provisions and the existing provisions. In the new provisions, these requirements are usually governed by the need to restrain longitudinal reinforcement in plastic hinge zones. The required transverse reinforcement is dependent on the longitudinal steel ratio, diameter of the column, and strength of steel but independent of the strength of concrete. In the current LRFD provisions, the transverse reinforcement in plastic hinge zones is independent of the longitudinal steel ratio and column diameter. The table shows that for 0.91 m (3 ft) and 1.22 m (4 ft) diameter columns with 1% steel ratio, the existing provisions require about 10% - 20% more confining reinforcement than the new provisions. For 0.91 m (3 ft) and 1.22 m (4 ft) diameter columns with 2% steel or more and for larger column sizes, the new provisions require more confining reinforcement than the existing provisions. A 50% to 100% increase in transverse reinforcement could be expected for 1.52 m (5 ft) and 1.83 m (6 ft) diameter column with 2% steel when using the new NCHRP provisions.

Table 6 Comparison of Transverse Reinforcement for New and Existing Provisions

Column	Long steel	NCHRP Project 12-49[1]			Existing LRFD[2]		
Diameter	ρ_t	ρ_s	spiral size	spacing mm	ρ_s	spiral size	spacing mm
0.9 m	1%	0.0058	# 13	98	0.008	# 13	71
(3' - 0")	# 25 bars	0.0058	# 16	152	0.008	# 16	109
	(# 8 bars)	0.0058	# 19	216	0.008	# 19	155
	2%	0.0102	# 13	55	0.008	# 13	71
	# 29 bars	0.0102	# 16	85	0.008	# 16	109
	(# 9 bars)	0.0102	# 19	121	0.008	# 19	155
1.2 m	1%	0.0068	# 13	62	0.008	# 13	53
(4' - 0")	# 29 bars	0.0068	# 16	96	0.008	# 16	82
	(# 9 bars)	0.0068	# 19	136	0.008	# 19	116
	2%	0.0121	# 13	35	0.008	# 13	53
	# 32 bars	0.0121	# 16	54	0.008	# 16	82
	(# 10 bars)	0.0121	# 19	77	0.008	# 19	116
1.52 m	1%	0.0085	# 16	62	0.008	# 16	66
(5' - 0")	# 29 bars	0.0085	# 19	87	0.008	# 19	93
	(# 9 bars)	0.0085	# 22	119	0.008	# 22	127
	2%	0.0136	# 19	55	0.008	# 19	93
	# 36 bars	0.0136	# 22	75	0.008	# 22	127
	(# 11 bars)	0.0136	# 25	98	0.008	# 25	167
1.83 m	1%	0.0091	# 16	48	0.008	# 16	55
(6' - 0")	# 32 bars	0.0091	# 19	68	0.008	# 19	78
	(# 10 bars)	0.0091	# 22	93	0.008	# 22	106
	2%	0.0163	# 22	52	0.008	# 22	106
	# 36 bars	0.0163	# 25	68	0.008	# 25	139
	(# 11 bars)	0.0163	# 29	86	0.008	# 29	176

ρ_s = ratio of transverse reinforcement $\quad$ ρ_t = ratio of longitudinal reinforcement

1 $\rho_s = 0.016*(D/s)*(s/d_b)*\rho_t\ (f_y/f_{yh})$ $\quad$ 2 $\rho_s = 0.12*(f'_c/f_y)$

$f'_c = 27.5$ MPa, $f_{yh} = 413$ MPa

Discussion of Results

Seat-Type Abutments

For the bridge configuration considered and for this abutment type, the maximum longitudinal seismic displacement for soil class E in North New Jersey was approximately

105 mm (4.13 in) without abutment resistance (K = 0). For a typical expansion gap, the soil behind the abutment wall is not likely to be mobilized and the passive pressure resistance will not be significant. In Central and South New Jersey, the maximum longitudinal seismic displacements for soil class E were 58 mm (2.3 in) and 86 mm (3.4 in) respectively. Hence no passive soil pressure behind abutment and the only resistance to lateral loads will be provided by the piers. For this bridge, both existing LRFD and new NCHRP provisions gave comparable displacements for North and Central NJ, however, for South New Jersey, the new NCHRP 472 gave higher forces and displacements.

Integral Abutments

For the bridge configuration considered and for this abutment type, the soil is engaged in resisting the seismic loads and, hence, longitudinal as well as transverse springs are used. The displacements and forces from new NCHRP 472 provisions were higher than those of existing LRFD provisions. This can be attributed to higher accelerations and soil amplification factors. The higher displacements from NCHRP will result in higher forces on the embankment behind the diaphragm. In both provisions, the soil seismic loads did not exceed the 2.75 KN/m^2 (6.67 ksf) ultimate passive pressure specified in the new NCHRP 472 provisions for this abutment height. Because of the larger displacements from new NCHRP 472 provisions, more soil movement behind abutment wall is expected. The longitudinal forces in the piers in the integral abutment were less than those of a seat-type abutment because of soil participation in resisting seismic loads. Engaging the soil behind the abutment will make the structure stiffer because of the added soil stiffness. This increased stiffness reduces the period of bridge and may result in higher seismic loads depending on the soil class and earthquake intensity.

Conclusions

1. The proposed new NCHRP 472 provisions do not necessarily result in higher seismic loads compared to existing LRFD provisions. For example, relatively long span bridges on stiff soils can have lower seismic forces using NCHRP 472 provisions compared to the existing LRFD provisions.
2. The 2500 year return period for the Maximum earthquake is very conservative compared to other extreme events such as vessel impact and floods. While this maximum earthquake can exceed the expected earthquakes by 50% in the Western United States, it could exceed the expected earthquake by 4 times in the Eastern United States. A lower return period would be more reasonable in the Eastern US. Accelerations for 2/3 return period 2500 year event would be reasonable for certain bridges, and are being considered. However, no USGS maps are available.
3. Soil amplification factors F_a and F_v are high for medium to soft soils and they have a major impact on the design response spectra and the selection of the seismic hazard level. Also they have an impact on the R-factors. The soil factors were based on soils from a specific region in the US (mostly the Western US). These factors need to be evaluated for states in the Eastern United States.

4. The new NCHRP 472 provisions provide information on the analysis of integral and seat abutments and on foundation stiffness. It also gives more options in analysis and design procedures. For example, SDAP C, D, and E could be used in most cases. Reduction factors of elastic seismic loads (R) are tabulated in more details for various cases. Allowing larger R-values when SDAP E is used rewards pushover analysis.
5. Because a large number of bridges can qualify under the category where the Capacity Design Spectrum Analysis (SDAP C) can be used, this procedure should be used because it is relatively simple and does not require complicated analyses. Design aids for this procedure can also be prepared to further simplify the procedure.
6. Transverse column reinforcement in plastic hinge zones are significantly affected by the bar size of longitudinal steel. This reinforcement is independent of the longitudinal steel in the existing provisions. For 0.91 m (3 ft) and 1.22 m (4 ft) diameter columns with 1% longitudinal reinforcement, the existing specifications require 10% to 20% more transverse reinforcement. For larger size columns with 2% steel, the new NCHRP provisions require approximately 50% to 100% more transverse reinforcement

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