COMPARATIVE DESIGN OF A BRIDGE IN NEW JERSEY USING THE NCHRP 472 COMPREHENSIVE SPECIFICATION FOR THE SEISMIC DESIGN OF BRIDGES AND THE 1998 LRFD

Harry A. Capers Jr., P.E.[1]

Abstract

With the completion of a National Cooperative Highway Research Program (NCHRP) project known as NCHRP Project 12-49 "Comprehensive Specifications for the Seismic Design of Bridges", NCHRP Report 472, with the same name, was recently issued. Along with the report, a proposed new seismic design specification was issued. The new seismic design specification was to be considered by the American Association of State Highway and Transportation Officials (AASHTO) as a replacement for its existing aged seismic design provisions. While not adopted in the form presented, the 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.

This paper discusses New Jersey's findings in performing two trial designs in support of the NCHRP project as well as current activities occurring in the State of New Jersey in tailoring the new specifications for use in its bridge designs.

[1] Harry A. Capers Jr., NJ State Bridge Engineer, New Jersey Department of Transportation

Background

In the fall of 1998, a project was initiated by NCHRP to develop new comprehensive seismic design specifications for highway bridges. The proposed specifications were intended to replace the existing specifications developed for the AASHTO Standard Specifications For Highway Bridges in 1983 and commonly referred to as Division 1-A of that publication. The new specifications were to be compatible with the new Load and Resistance Factor (LRFD) design philosophy of the latest design specification issued by AASHTO's Subcommittee on Bridges and Structures. It was to reflect the latest knowledge and design approaches and was to produce designs with a high level of seismic performance.

NCHRP Project 12-49, *Comprehensive Specification for the Seismic Design of Bridges* was began in August, 1998 and concluded in November, 2001 with the issuance of NCHRP Report 472, *Comprehensive Specifications for the Seismic Design of Bridges.* The research was performed by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering (ATC/MCEER).

To assist the Subcommittee on Bridges and Structures' evaluation of the specification, the ATC/MCEER team provided leadership with a group of states in performing trial designs of a sampling of bridges in various parts of the country. Trial designs were performed by the states of Alaska, New Jersey, Missouri, Washington, Arkansas, California, Oregon, South Carolina, Tennessee, Illinois, Nevada, Georgia, New York and the Federal Highway Administration's (FHWA) Central Federal Lands Highway Division. In all, 19 trial designs were performed. As can be seen from the list of States, the sample designs covered a variety of areas with differing seismicity. As such, an impact to a variety of bridge owners was demonstrated.

In support of this effort, New Jersey designers performed two trial designs. The bridges selected were bridges that had already been designed using the provisions of Division 1-A of the AASHTO Standard Specifications. This permitted a comparison between the two specifications. Consultants who had performed the original work for the on the selected bridges performed both trial designs. The following is a discussion of their findings:

The Doremus Avenue Bridge

The first structure for which a trial design was performed was the replacement of the Doremus Avenue Bridge over the Conrail Oak Island Railyard in the City of Newark, New Jersey. The existing Doremus Avenue Bridge was built in 1918 and was in poor condition with severe settlement throughout. The current bridge evaluation revealed that it should be classified as structurally deficient and functionally obsolete. Traffic estimates indicated that about 15,000 trucks per day or 1.4 million truck containers annually would cross the new bridge. This bridge is part of a very significant system of roadways in the Port Newark area. Doremus Avenue serves as a major truck route into and out of the port. This will help to very actual large vehicle configurations that are used in New Jersey. From this identification a permit vehicle configuration will be established for State.

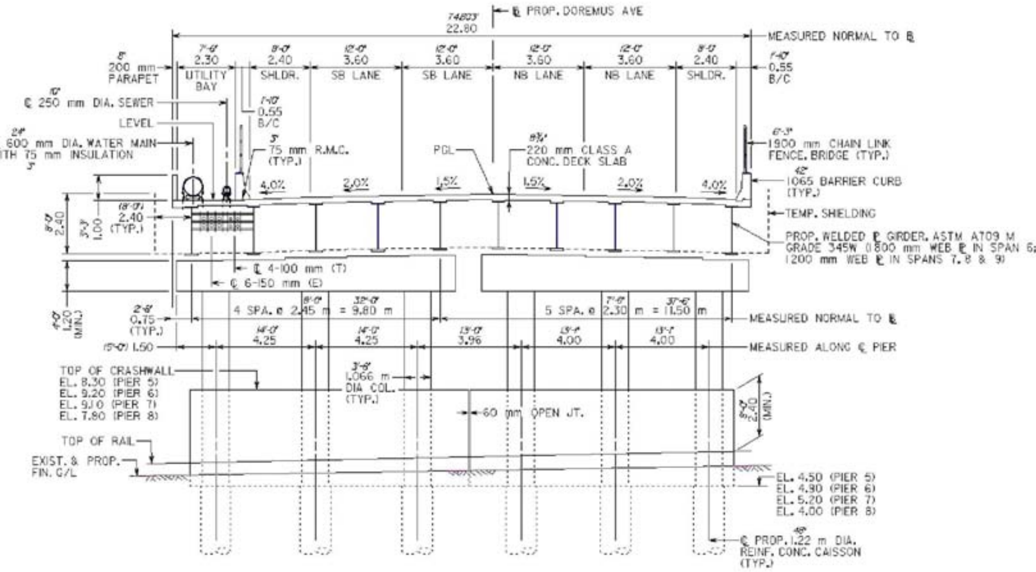

Figure 1 – Section of Doremus Avenue Bridge over the Conrail Oak Island Railyard

The new bridge is approximately 400 meters long and consists of three (3)- 3 span continuous units for a total of nine (9) spans. It will consist of variable depth steel plate girders (AASHTO M270M, Grade 345W) with a 220 millimeter (8.7 in) steel reinforced concrete deck. The foundations consist of drilled shafts embedded into bedrock. A cross section of the bridge is shown as Figure 1. Figure 2 provides an Arial view of the project site. The new bridge was designed by Parsons-Brinkerhoff-FG, Princeton, New Jersey.

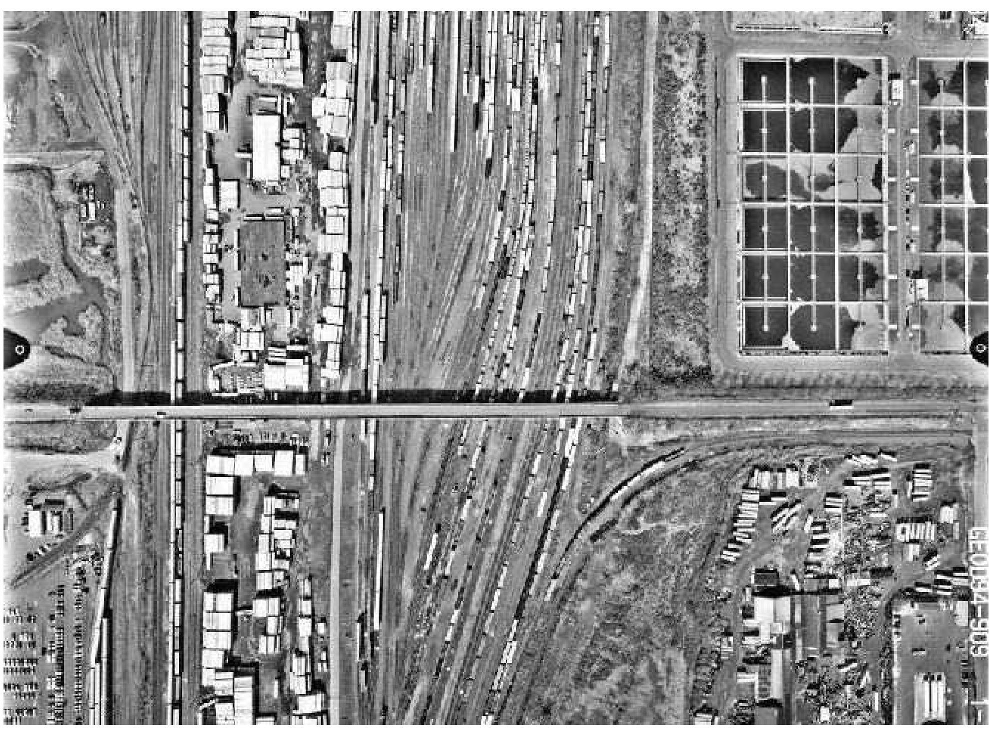

Figure 2 - Ariel view Doremus Avenue bridge over the Conrail Oak Island Railyard

The following is provided for those who are interested in the specific assumptions made by the designer in performing the comparative designs.

A) The Doremus Avenue Bridge was originally analyzed for seismic loads according to AASHTO Division 1-A requirements using the following parameters:

Acceleration Coefficient, A=0.18
Seismic Performance category, SPC = B
Soil Profile Type III
Site Coefficient, S=1.5
Elastic Response Spectrum as per AASHTO
Damping Coefficient = 0.05
Response Modification Factor, R=3.0 per AAHSTO Table 3.10.7.1-1 for non-critical bridges.
The Extreme Event –1, LRFD loading Combination was used in combination with the Orthogonal Seismic Load Combination – 100/30 Rule.
Extreme Event 1 = 1.0 DL + γ_{eq}LL +1.0EQ
Where EQ can be obtained from LC1 = 100% Longitudinal + 30% Transverse
LC2 = 30% Longitudinal + 100% Transverse
$\gamma_{eq} = 0.5$

B) The trial design using the NCHRP Guidelines utilized the following key parameters:

Spectral acceleration parameters Ss of 0.421g and S1 0f 0.094g were used.
Seismic Design and Analysis Procedure (SDAP) D Elastic Response Spectrum Method was used with Seismic Detailing Requirements of 4
Site class of E was selected based on geotechnical conditions of the site.
Vertical Acceleration effects were ignored

An Elastic Response Spectrum Analysis using Multi-mode Dynamic Analysis was performed. The structure was modeled as a single line of frame elements using equivalent stiffness of the members. The multimode analysis used gross section properties of members and elements rather than cracked section properties. Cracked section properties usually result in smaller seismic forces and larger deformations.

The performance level selected for the bridge was life safety.
The use of Earthquake Resisting systems was permitted.
The seismic Hazard Level was IV.
R factor used was R=4.0
No Transverse displacements were assumed at abutments due to foundations being built on piles.

The Extreme Event –1, LRFD loading Combination was used in combination with the Orthogonal Seismic Load Combination – 100/40 Rule.

Extreme Event 1 = 1.0 DL + γ_{eq}LL +1.0EQ

Where EQ can be obtained from LC1 = 100% Longitudinal + 40% Transverse
LC2 = 40% Longitudinal + 100% Transverse

$\gamma_{eq} = 0.5$

Forces on the substructure obtained from performing the two designs are summarized in the following table:

Pier Number	AASHTO Division I-A			NCHRP Guidelines	
	F_x (kN)	F_y (kN)	Governing Load Comb.	F_x (kN)	F_y (kN)
Pier 1	110*	144	Strength I	110*	140
Pier 2	458	315	Extreme Event I	294	288
Pier 3	82.3*	628	Extreme Event I	82.3*	665
Pier 4	454	415	Extreme Event I	313	380
Pier 5	142*	278	Strength I	142*	255
Pier 6	376	487	Strength I	260	510
Pier 7	80*	226	Extreme Event I	80*	209
Pier 8	91*	87	Strength I	91*	83

Figure 3 Resulting Design Loads

In the above table, the listed forces are per girder, forces marked with * are forces from friction between the pier cap and the bearing. In general, NCHRP Guidelines resulted in smaller loads than the AASHTO Division I-A requirements except at Pier 6 in the y direction.. Pier 6 design was governed by Strength load combination originally, and it appears that the "square-root of the sum of the squares" combined seismic load would not govern the design if the NCHRP Guidelines were used.

Seismic loads for Pier 4 using the NCHRP Guidelines were tabulated and compared with the same loads obtained by using the AASHTO Division I-A guidelines. The load combination for the Extreme Event-1 was calculated using the proposed NCHRP 12-49 Guidelines Subsection 3.5 and Table 3.5-1, and modified with the Response Modification Reduction Factor of 4.0 as per Subsection 4.7. It should be noted that the largest moment for the extreme event load combination obtained using AASHTO Division I-A was 2673 kN-m, which governed the design of the columns. The largest moment for the extreme event load combination obtained following the NCHRP 12-49 code was 1212 kN-m, which is less than the moment obtained by the other strength limit states. This value would not govern the design.

For this particular structure, NCHRP Guidelines resulted in a more moderate design requirement when compared to the AASHTO Division I-A Seismic Design requirements. For the piers, where seismic loads governed the design using Division I-A, the design loads remained practically unchanged. Therefore, no significant change in substructure cost would result.

For the original design of the Doremus Avenue Bridge, the seismic analysis was performed in accordance with Division I-A using Multimode Spectral Analysis Method - Procedure 3. SEISAB analysis software by Imbsen Software Systems was used. For this exercise, the One-Step Procedure suggested in the NCHRP Report was used. Except for a possible learning curve it was felt that there was no significant difference in the design effort required by either method.

The Scotch Road Bridge over Route Interstate 295

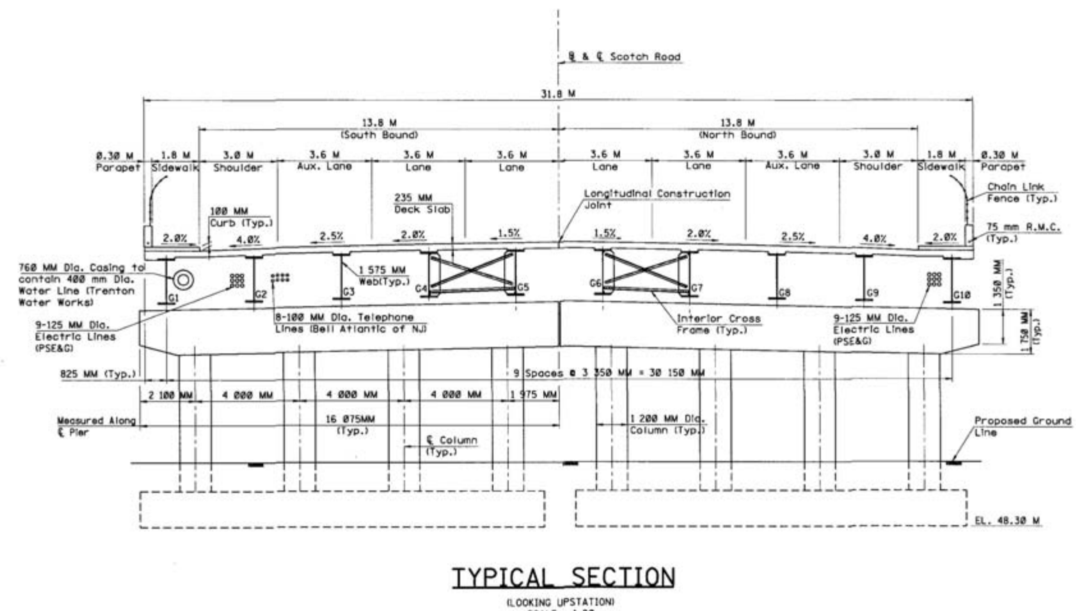

Figure 4 – Section of Scotch Road over Interstate Route 95

The second trial design was performed for the Scotch Road Bridge over Interstate Route – 95 in Ewing Township, New Jersey. The new bridge is a 90.9-meter long two span continuous structure. The span lengths are 45.45m with a web depth of 1.58m. The tangent alignment of the existing Scotch Road was maintained on the new bridge. The new bridge carries two 3.6m lanes, one 3.6 auxiliary lane, one 3.0m shoulder, and a 1.8m sidewalk in both the northbound and southbound directions. There is a 1.9m high, curved top chain link fence mounted on a 0.815m high parapet along each side of the bridge. The superstructure out-to-out dimension will be 31.8 m. Arora and Associates, P.C of Lawrenceville, New Jersey provided design services for this bridge. A cross section of the bridge is shown as Figure 3 and Figure 4 provides an aerial view of the project site.

Figure 5 – Ariel view of Scotch Road over Interstate Route 95

Specific notes on the sample design experience by the consultant were as follows:

The dynamic shear modulus was not measured at this site. The soil is comprised mainly of decomposed fractured shale. The shear wave velocity has been estimated to be between 760 meters per second and 1500 meters per second. Therefore, a classification of Site Class B was utilized. The ground surface shaking parameter was taken as
FaSs = 0.36. According to Table 3.10.3-1, the Seismic Hazard Level is equal to III. The structure meets the criteria for analysis under the SDAP D: Elastic Response Spectrum Method. Since this structure was evaluated with an Operational Performance Level, according to Table 3.10.3-2, the Seismic Detailing Requirement (SDR) is 5.

The seismic analysis of the structure was performed using SEISAB analysis software by Imbsen Software Systems. The model loading used in the analysis was the Arbitrary Curve Data. The response spectrum curve generated for this analysis is based upon the local ground motion values and Figure 3.10.2.1-3: Design Response Spectrum, Construction Using the Two-Point Method. A three second period was used for this analysis.

The original analysis was based upon the 1998 AASHTO LRFD Bridge Design Specifications 2nd Edition with current interims and as modified by the NJDOT LRFD Bridges and Structures Design Manual – 4th Draft Edition. Subsection 3.10.3 of the NJDOT LRFD Bridges and Structures Design Manual – 4th Draft Edition modifies the AASHTO LRFD Specifications by directing the design engineer to assume the bridge to have an Importance Category of "critical". Therefore, according to AASHTO, the response modification factors used were equal to 1.5 (the foundation R- Factor is 1.0). The bridge foundation was originally designed to be elastic under the design seismic loading.

No mechanisms developed in the pier by using the NCHRP provisions. This is because the effective seismic loading using the NCHRP provisions reduced the effective seismic loading on the pier by approximately 30 to 40 percent. In addition, the R-Factors used in the NCHRP provisions are also 1.5. No mechanisms developed in either the cap, columns or footing. The abutments are integral for this structure. Therefore, no mechanisms formed in the abutment.

Earthquake Resisting Systems (ERS) used at the pier include guided sliding bearings. A portion of the transverse seismic loadings are resisted by the bearings and transferred to the supporting pier. The longitudinal seismic loads are dissipated into the soil through the use of integral abutments.

The footing sizes are 15.500m x 6.000m footings. They support 4 - 1.200m diameter columns at each bent. There are two adjacent bents for this structure. The columns support a 1.400m wide x 1.500m deep cap beam. The abutments are integral with the superstructure. The majority of the seismic loading at the abutment is transferred to the soil. The transverse seismic loading at the abutment is resisted in part by the soil and also by the integral abutment pile system.

The structure was designed elastically. Since an Importance Category classification was designated for this structure, and also since the seismic forces were appreciably lower than the current AASHTO provisions, the pier components, as designed, will behave elastically.

The overall structure construction costs, using the NCHRP provisions, would be lower than the AASHTO LRFD provisions. The column capacity is significantly increased when the NCHRP provisions are checked against the AASHTO LRFD specifications. Either less reinforcement can be utilized or perhaps a smaller column diameter could be used. The footing size could also be reduced. However, the cost savings are not very significant. The seismic eccentricity requirements for spread footings did not significantly reduce the footing dimensions at this location.

The overall design effort for this example was not as significant as it would have been had a reduced Importance Category been designated. The higher performance level classification at this location made an elastic design the appropriate design method for this bridge. Design time using an elastic approach was less than what would have been required had a capacity approach been used. Reduced performance levels would make the capacity design method the appropriate design approach to use; thereby increasing the design time. Seismic detailing requirements would also increase. The designer felt that the provisions were clearly written and the learning curve was not significant.

Recommendations for revisions to the code:

The Designer of the Doremus Avenue Bridge made the following recommendations regarding the new code that was developed under NCHRP Project 12-49:

1. The NCHRP Guidelines require the analysis to be done using the cracked section. Cracked section properties in some textbooks are given as 50% of the gross section properties; however, it could also be assumed to be 70% or any other percentage. Clarification should be given whether this number should it be left to the designer or should it be the owner's decision.
2. The R factor depends on the T (bridge period). It is unclear how this is determined.
3. Guidance on developing site-specific data is needed.
4. Clarification is needed on decision making as to whether a bridge should be judged to have a critical or an essential Importance Category classification.
5. The NCHRP SDAP D, Elastic Response Spectrum Analysis Method has been used for this bridge structure. Regularity requirements, given in Subsection 5.4.2 for the uniform load method covers bridges up to 6 spans. The subject bridge has 9 spans that are composed of three 3-span continuous units. In addition, the South Abutment and Piers 1 through 4 have no skew while Piers 5 through 8 and the North Abutment have a skew of 15 degrees. The applicability of the multi-mode dynamic analysis method to cases similar to this should be addressed in future revisions.

The Designer for the Scotch Road Bridge suggested potential areas of refinement as follows:

1. The bridge performance level needs to be defined more clearly. Significant design parameters are highly dependant on this determination. This is typically done on a state-by-state basis.

2. It should be noted that the AASHTO LRFD designation of a "critical" structure is very similar to the NCHRP's "operational importance" SDAP D designation. The R-Factor values are similar to each other. The LRFD foundation R-Factor is equal to R/2 which cannot be less than 1.0. The NCHRP report says that "If the elastic foundation forces are less than the forces resulting from column hinging, they may be used for the foundation design. The foundation shall be designed using an R-Factor of 1.0". Therefore, effective foundation R-Factors can vary depending on the column hinging forces (NCHRP 15.8).

Further clarification of Subsection 15.8 and how it may impact Subsection 3.10.3.7 would be useful.

3. The transverse reinforcement for longitudinal reinforcement in the columns required prestressing steel. This was due to the stringent design requirements. It was assumed that this should only be used for structures with an a critical Importance Category classification and that typical bridges would not require this measure.

4. Although the pier in this example did not include any pile foundations, NJDOT modifies the AASHTO LRFD Specifications and directs the engineer to design deep foundations using the 17th Edition of the AASHTO Standard Specifications. Therefore, unless the NCHRP guidelines address NJDOT's concerns regarding deep foundation designs, deep foundations will require a similar modification when using the NCHRP provisions.

5. The combination of seismic loads using the AASHTO LRFD Specifications is a (100% - 30%) combination. The NCHRP guidelines increase the seismic force effects to a (100% - 40%) combination. Whereas, a simple hand calculation can convert the LRFD seismic loads to the NCHRP load combination, seismic analysis software companies should gear their software to accept both the traditional method or the NCHRP method.

6. The major editorial suggested was to include a summary table of the major similarities between the two codes. This table can be shown in the commentary. For example, the load factor for DL changed from γp (1.25) to 1.0, or the combination of seismic force effects changed from (100% - 30%) to (100% - 40%)...

New Jersey Department of Transportation's Reaction.

Even though the AASHTO Subcommittee on Bridges and Structures did not approve the adoption of the NCHRP recommended specification as a Guide Specification in the form presented for consideration, based on New Jersey's experience in these two trial designs the Department directed that, NCHRP Report 472, "Comprehensive Specification for the Seismic Design of Bridges" may be used as an alternative to the use of the AASHTO LRFD Specifications Division 1-A. Designers may submit a request to the Manager, Bureau of Structural Engineering for the use of the NCHRP Report guidelines. A comparison of the effects on the design of a project between the two documents should be made to validate the request.

In conjunction with the issuance of this guidance NJDOT undertook a research project with Rutgers University's School of Civil and Environmental Engineering. Their initial findings include the following:

1. NCHRP 12-49 provisions do not necessarily result in higher seismic loads compared to existing AASHTO LRFD specifications. For example, longer span bridges on stiff soils will have lower seismic forces using the NCHRP 12-49 provisions compared to the AASHTO LRFD specifications.

2. The 2500-year return period for the MCE (rare) earthquake is very conservative compared to other extreme events such as vessel impact and floods. A return period of 1500 years is being considered; however, no USGS maps are available. An acceleration equal to 2/3 of that of the 2500-year event could similarly be used with the IBC provisions.

3. Also being considered is the 1000-year event for which USGS seismic maps are available; however, this event seems to have lower accelerations than current AASHTO LRFD specifications.

4. Soil amplification factors Fa and Fv are high for medium to soft soils and have a major impact on the design response spectra and the selection of the seismic hazard level. For locations with soft soils and high accelerations, site specific response spectra needs to be considered to lower the seismic demand. (See Figure 6)

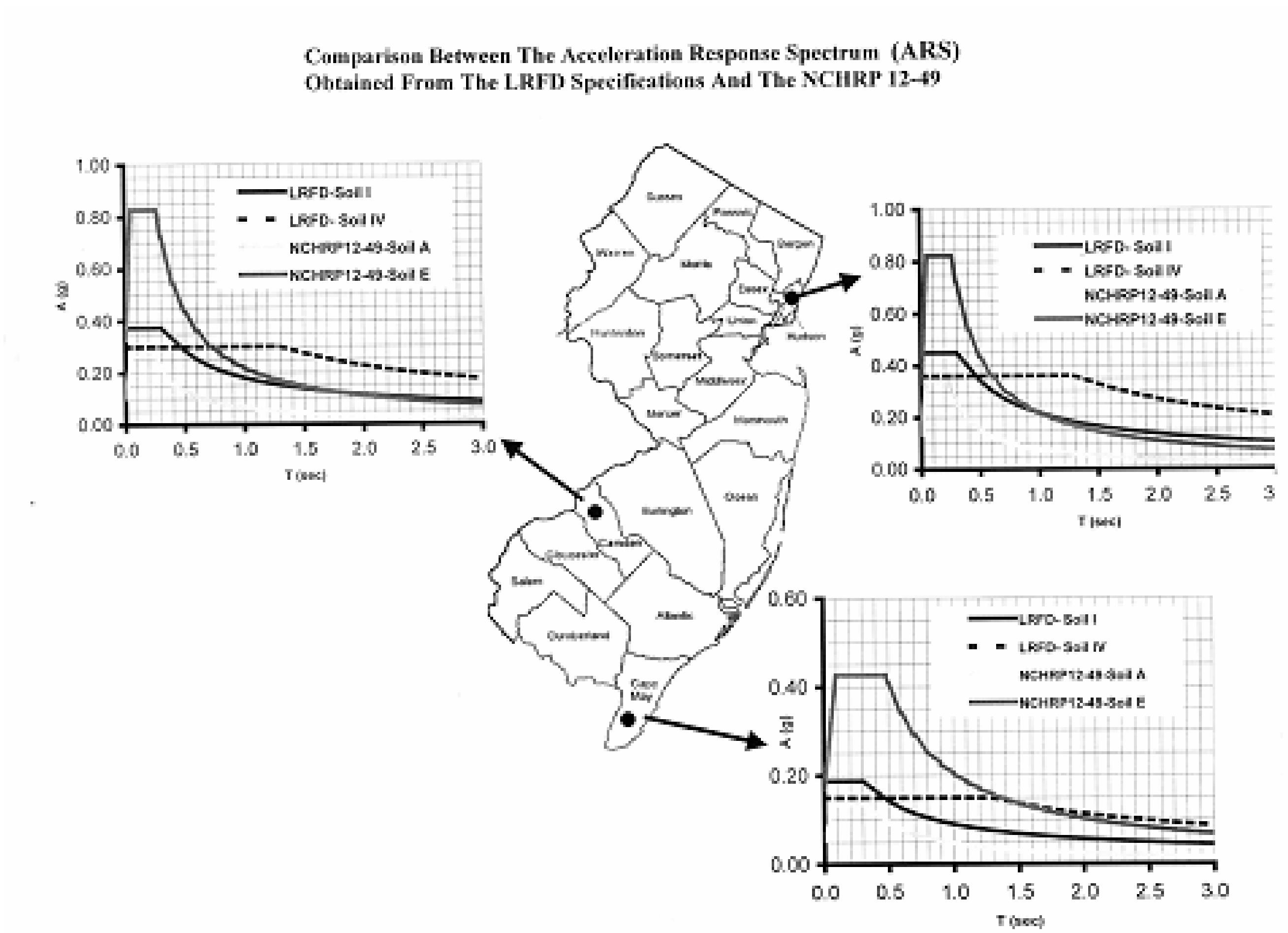

Figure 6

5. The new specification provides more information on the analysis of integral and seat abutments and on foundation stiffness.

6. The new specification provides more flexibility in the analysis and design procedures to be used. For example SDAP C, D, and E could be used in most cases for bridges in New Jersey.

7. Reduction factors of elastic seismic loads R are tabulated in more details for various cases. Pushover analysis is rewarded by allowing larger R-values when this analysis is used.

The researchers at Rutgers have suggested the following preliminary recommendations for consideration by the New Jersey Department of Transportation:

1. Extreme EQ event (MCE) based on 2/3 of the 2500 years spectral acceleration seems more realistic and will not be overly conservative compared to other extreme events in the AASHTO LRFD Specifications. For secondary bridges, a lesser return period may be used.

2. Maximize the number of bridges in New Jersey that can be classified as ordinary and for which detailed seismic analysis and design is not needed. This will depend on soil factors, return period and importance of bridge.

3. The NCHRP provisions include detailed procedures for analysis and design. Most of these will be followed and will be designated as design criteria. Provide specific guidelines and analysis models for the seismic design of abutments.

4. Provide detailed examples for the analysis and design of integral and seat abutments consistent with the NCHRP 12-49 provisions

5. Provide state-of-the-art information of the seismic analysis and design of retaining walls and buried structures

To date several requests have been made to use the NCHRP guidelines and several more projects are being examined relative to their potential use. While, mainly for the reasons outlined above, no designs have been produced using the new specifications, the outlook appears promising that we will be able to give more specific guidance in the near future that will address many of the barriers to our full deployment of this state of the art specification.

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