Implementation of Recommended LRFD Guidelines (NCHRP 12-49) for Seismic Design of Highway Bridges in New Jersey

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16. Abstract

The proposed new seismic code for highway bridges (NCHRP 12-49) is a major step forward reflecting the latest scientific developments and knowledge gained over the past two decades. It is a nationally applicable set of provisions where hazard is characterized uniformly for the entire nation by employing the latest maps developed by the US Geological Survey (USGS). The goal of this study was to evaluate its implications for the design of bridges in the State of New Jersey. The findings can be summarized as strong support for its implementation in its present form.

The results show that seismic design forces based on the proposed NCHRP 12-49 guidelines are not necessarily higher than existing code. Furthermore, the provisions provide simplified analysis and design procedure that can apply to a large number of bridges in the State of New Jersey. The cost implication of seismic design requirements for a wider region is expected to be minimal to none due to the addition of no analysis design concepts and simplified but more accurate procedures. It is recommended that NJDOT adopt NCHRP 12-49 in its present form and encourage AASHTO to do the same. Furthermore, NJDOT should oppose changes in hazard definition that results in a lower return period (e.g., 1,500 year or lower) since such a move can have serious adverse effect on seismic safety of bridges. Instead efforts to better characterize site specific ground motion and relaxing site-specific analysis limit(s) are recommended.

Future research and development efforts must instead concentrate on considerations to dual benefit multi-hazard problems such as response of highway bridges to vertical motion and blast load. Possible research topics for immediate considerations are effect of blast and vertical seismic loads on bridge superstructure, development of protective systems for both sub- and superstructure, and impacts of upward motions on pre-tensioned and post-tensioned bridges.

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EXECUTIVE SUMMARY

The proposed new seismic code for highway bridges (NCHRP 12-49) is a major step forward reflecting the latest scientific developments and knowledge gained over the past two decades. It is a nationally applicable set of provisions where hazard is characterized uniformly for the entire nation by employing the latest maps developed by the US Geological Survey (USGS). The goal of this study was to evaluate its implications for the design of bridges in the State of New Jersey. The findings can be summarized as strong support for its implementation in its present form.

The bottlenecks to adoption of NCHRP 12-49 by AASHTO reportedly are: perceived high increases in seismic forces, requirement of seismic design for a wider range of seismicity, complexity of the provisions, and control over the maps defining the seismic hazard. The latter is an issue that can easily be addressed. However, contrary to ongoing misperception and/or misunderstanding, it is the determination of this study that seismic forces are not necessarily higher when the proposed new provisions are employed. Indeed for the State of New Jersey for majority of designs (if not all) the new provisions can result in lower seismic forces. This is for several reasons more notably for the facts that the new provisions contains an incentive for performing more advanced analyses and that it has removed the conservatism associated with current AASHTO acceleration response spectrum for long period systems. Furthermore, although the new provisions are more involved and contain new concepts such as capacity spectrum design procedure, they are well documented and discounting the initial efforts of the learning curve, it is not expected that there will be significant change in the design effort compared to Division I-A. Indeed it is for availability of these new simplified design procedures, which will probably be applicable to a large number of bridges, that the impact of a wider region requiring seismic design is also minimal or none.

As a result of misunderstanding on the exact impact of NCHRP 12-49, an alternative version, which among its changes is a reduction in return period, is being explored. It is shown here that if the design spectrum is reduced by two-thirds (roughly corresponding to a 1,500 year return period) the seismic forces for relevant bridge periods will be minuscule leaving highway bridges vulnerable to even moderate events. The 1995 Kobe, Japan earthquake proved that rare earthquakes do indeed happen. Rather than reduction in return period, it is recommended that more emphasis be placed on site-specific analysis to further reduce uncertainties in defining the hazard and to allow taking steps to relax the limit on its applicability.

Furthermore, future research efforts and resources should be concentrated on areas of seismic design that can have multi-hazard benefits. Among such topics are explicit considerations to the effect of vertical motion and bridge superstructure vibration, which can also have significant implications on bridge safety vis-à-vis blast and reversed loadings.

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Thus, findings and recommendations can be highlighted as follow:

- Seismic design forces based on the proposed NCHRP 12-49 code are not necessarily higher than the existing code. Most designs (if not all) for the state of NJ can actually have lower seismic forces compared to the existing code.
- NCHRP provisions provide simplified analysis and design procedure that can apply to a large number of bridges in NJ.
- The cost implication of seismic design requirements for a wider region is minimal to none due to the addition of no analysis design concepts and simplified but more accurate procedures.
- Proposed provisions are nationally applicable and provide uniform hazard across the nation by using recent USGS maps.
- Adopt NCHRP 12-49 in its present form and encourage AASHTO to do the same.
- Consider making comparative designs based on NCHRP 12-49 and Division I-A a part of the bridge project scope in the state of NJ.
- Oppose changes in hazard definition that results in a lower return period (e.g., 1,500 year or lower) since such a move can have serious adverse effect on seismic safety of bridges.
- Encourage seismic provisions, such as NCHRP 12-49 that take advantage of advances in analytical procedures and rewards designs that are based on the state-of-the-knowledge.
- Collect dynamic soil properties as a part of future bridge construction. Gains in more efficient and better designs well justify minimal associated costs.
- Encourage relaxing site-specific analysis limit(s) to promote further collaboration among geology, seismology, geotechnical engineering, and structural engineering to better define seismic hazards and model soil-structure interaction.

Rather than revising NCHRP 12-49 by reducing return period and/or removing incentives for use of the state-of-the-art analysis procedures, the following topics are recommended for future research and development:

- Consideration to dual benefit multi-hazard problems such as response of highway bridges to vertical motion and blast load. Possible research topics for immediate considerations are effect of blast and vertical seismic loads on bridge superstructure, development of protective systems for both sub- and superstructure, and impacts of upward motions on pre-tensioned and post-tensioned bridges.
- Perform a sensitivity study on interdependency of bridge dynamic characteristics (period) and site conditions (hazard level) to more exactly quantify its impact on seismic demands for various bridges.
- Develop design tools to facilitate and streamline the application of new analysis and design concepts under NCHRP 12-49.

INTRODUCTION

The San Fernando Earthquake of February 9, 1971 was a turning point in the development and subsequent implementation of seismic design codes for highway bridges. Prior to 1971, seismic guidelines for highway bridges were very simple and without much consideration to bridge vibration and site conditions. Seismic design forces were constant regardless of system period and they were related to the bridge weight through a simple constant that ranged from 0.02 to 0.06. Following the San Fernando Earthquake many research and development projects were initiated. Among these was a major study sponsored by FHWA to develop seismic design guidelines for highway bridges, which was conducted by the Applied Technology Council (ATC) and published as ATC-06 [1]. In 1991, the American Association of Highway and Transportation Officials (AASHTO) adopted the guidelines as a standard specification (also known as Division I-A). During the 1990s changes (mostly terminology) were made to Division I-A specifications and they were adopted into AASHTO LRFD. Thus, existing seismic codes are mostly based on research conducted in the 1970s after the San Fernando Earthquake.

During the past two decades there have been several major earthquakes with significant impact on seismic performance of highway bridges. Among these are:
- October 17, 1989 Loma Prieta Earthquake,
- January 17, 1993 Northridge Earthquake,
- January 17, 1995 Kobe Earthquake,
- September 20, 1999 Taiwan Earthquake, and
- 1999 Earthquakes in Turkey.

In addition to reinforcing lessons learned from the San Fernando Earthquake, these earthquakes provided many other lessons to be considered in the seismic analysis and design of highway bridges. These earthquakes highlighted:
- Importance of site conditions,
- The need to consider near fault ground motion,
- Importance of connection details, and reliability of multi-column bent performance,
- The possibility of occurrence of rare events (an important lesson for low seismicity region such as NJ),
- The likelihood of premature failure of more recent bearings,
- The fling-step phenomenon (large unidirectional velocity pulses in the fault-normal direction), and fault rupture crossing the bridge site, and
- Development of advanced analytical tools.

Furthermore, these earthquakes provided the needed information on performance of some of the newer technologies developed after the San Fernando Earthquake of 1971.

To reflect the experience gained during these earthquakes, and the scientific knowledge created over the past two decades, in 1998 the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER) at SUNY Buffalo, initiated a joint venture to develop the next generation of seismic design specifications for highway bridges in the United States. The project was sponsored by AASHTO and conducted through the National Cooperative Highway Research Program (NCHRP), as NCHRP Project 12-49 [2]. It was entitled "Comprehensive Specifications for the Seismic Design of Bridges." The project continued the evolution of seismic codes for highway bridges from a single equation in the 60s to a performance-based specification. The work reflects a 5-year effort of a project team consisting of academicians, and practitioners from all parts of the nation, guided by equally distinguished project advisory committee and a NCHRP project panel. The final document addresses the state-of-the-art aspects of highway bridge seismic design, including the latest approaches for representing the seismic hazard, design and performance criteria, improved analysis methods, steel and concrete superstructure and substructure design and detailing, and foundation design. The proposed provisions are now being used in trial designs around the country, and are being considered (albeit probably in a modified form) for adoption by AASHTO as *"Recommended LRFD Guidelines for the Seismic Design of Highway Bridges."*

The objective of this study was to evaluate the NCHRP 12-49 guidelines and to assess its impact on the seismic design of highway bridges in the State of New Jersey.

ROCK DESIGN SPECTRA

A major advantage of NCHRP 12-49 is that unlike the current AASHTO code (Division I-A), which has three implied performance objectives; it is explicitly a performance-based approach. It provides more definitive performance objectives and damage states for two design earthquakes. Aside from the fact that performance based engineering is the evolutionary path for seismic design codes for both buildings and bridges, dual level approach to design earthquakes addresses an important lesson learned from the January 17, 1995 Kobe, Japan Earthquake. This earthquake proved that rare (or the maximum considered) events do indeed occur, as an earthquake of this magnitude was considered rare for that part of Japan.

Based on NCHRP 12-49, which employs USGS 1996 maps [3], the upper-level event termed the Maximum Considered Earthquake (MCE) corresponds to a 3% probability of exceedance (PE) in 75 years (approx 2500 years return period), while the lower-level design event, termed the expected earthquake, has ground motions corresponding to 50% PE in 75 year (or 108 years return period). The current provisions (Division I-A) employ the 1990 USGS maps, which provide contours of peak ground acceleration (PGA) for a PE of 10% in 50 years (about 500 years return period). Figure 1 shows response spectra for a rock site in Newark, NJ based on both

codes. This figure shows that for rock sites the response spectrum, even for the maximum considered earthquake (MCE) based on NCHRP 12-49 (2500 years return period), is lower than the 500-year event based on Division I-A (blue vs. red). Thus, even without consideration to other factors (such as higher R-Factor under NCHRP 12-49) on sites with competent soil the proposed new codes result in lower seismic forces for bridges in NJ.

Another important change is the general shape of the response spectra. The new spectral shape removes the conservatism associated with Division I-A acceleration response spectrum for long period systems. For long period systems the new response spectrum reduces as a function of the inverse of period (i.e., 1/T), while under the existing codes (Division I-A) it reduces more gradually with the inverse of period to the power of 2/3 ($1/T^{2/3}$). Information on design response spectrum construction based on NCHRP 12-49, using two-point method and online resources of USGS, is provided in Appendix I.

SOIL EFFECTS

More recent earthquakes, such as the 1989 Loma Prieta Earthquake, demonstrated the importance of site conditions and dynamic response of deep soft soil as a contributing factor to bridge collapse. Thus, another major advantage of the proposed new codes is that it provides new site classes and site factors. These are based on recent work adopted by CalTrans in 1999 and International Building Code (IBC) in 2000.

Site definitions are extended to five classes where specific information on site classification is provided. Shear wave velocity (V_s) is the fundamental property used in classifying both rock and soil sites. For example, sites with V_s greater than 5,000 ft/sec are classified as A, while at the other end of the spectrum is class E with V_s less than 600 ft/sec. Division I-A site classification includes four types (I through IV) and they are defined based on general descriptions that can become subjective. In addition to shear wave velocity, Standard Penetration Test (SPT) blow count (N-value) or undrained shear strength (S_u) can be used to classify soil profiles.

The new site coefficients are a significant change with regard to its impact on the level of seismic design forces. Site coefficients (or soil factors) are different for short period ranges and long period ranges, as shown in Tables 1 and 2. Important to states with low to moderate seismicity, such as the state of New Jersey, is also the fact that due to nonlinear response effects of soils, the site factors increase with a decrease in spectral accelerations. For example, the short period coefficient, F_a, for a mapped spectral acceleration equal to 0.25g ranges from 0.8 for class A to 2.5 for class E sites. For a mapped acceleration equal to 0.75g this coefficient ranges only from 0.8 to 1.2 for site classes A to E. The same pattern holds for 1-second period coefficient, F_v.

Figure 2 shows the response spectra for the range of site conditions based on NCHRP 12-49 (i.e., site classes A and E) and those for Division I-A (soil type I and IV) for Newark, NJ[1]. Note that the spectrum corresponding to NCHRP 12-49 corresponds to upper level event (MCE), which is 3% PE in 75 years or approximately 2,500 years return period. Division I-A has only one event with about 500 years return period. As discussed before (Figure 1), for competent soils the spectral accelerations are smaller based for the proposed new code (i.e., Class A/pink vs. Type I/red). For soft soil sites, the spectral accelerations based on NCHRP 12-49 (Class E/blue curve) are higher than those based on existing code (Type IV/black) only for low period (high frequency) systems. In this case, for systems with periods longer than 0.7 second the spectral accelerations are lower based on the proposed new code. This is an important point to be considered in the design process. Note that Division I-A spectrum is capped at a fixed value regardless of site condition. There is no scientific and/or empirical explanation on the derivation of this short-period cap, and site-specific results indicate that Division I-A can underestimate design spectral values by a factor of 1.5 or more [4].

Table 3 shows typical mapped short-period spectral values, S_s, for several counties in the state of New Jersey. For comparison purposes the peak ground accelerations based on Division I-A (and modified per NJ Bridge Design Manual) are also provided. It should be noted that NCHRP 12-49 gives spectral values for rock site while Division I-A gives peak ground accelerations. Therefore, appropriate factors should be used to compare the two codes and/or to determine spectral values for a different site condition. Division I-A spectral coefficient is capped at 2.5 regardless of soil type. Thus, based on Division I-A there is no difference between the short-period spectral values for Type I and Type IV soils (corresponding to Classes B and E in NCHRP 12-49). For example, in Figure 2 spectral values are equal for periods less than 0.3 seconds regardless of soil condition.

The highest increase in short-period spectral accelerations occurs in Salem County (zip code 08067). Inspection of contour maps of low period (0.2 seconds) spectral accelerations as shown in Figure 4 reveals the reason behind this. For the northern counties such as Bergen or Essex although S_s are high, so are the peak ground accelerations based on Division I-A (at 0.18g). Thus, in Bergen County the ratio of short-period spectral values for a rock site is almost unity (1.01) and it is around 1.83 for a soft soil site. However, for the Salem County the peak ground acceleration based on existing code is only 0.10g, while its northwest corner (e.g. zip code 08067) falls within a relatively high spectral acceleration contour based on the proposed new

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[1] Trends and patters discussed are general, and in the sake of brevity results of additional examples are not presented.

code. This causes an increase in spectral ratios to 1.28 and 2.92 for rock and soft soil conditions, respectively.

Of course the response spectrum by itself does not determine the level of seismic forces. Dynamic and nonlinear response characteristics of the bridge should also be considered. As it will be shown later when these factors are all considered seismic forces are mostly lower even for soft soil sites and low period systems when the proposed new provisions (NCHRP 12-4) are compared to Division I-A.

SENSITIVITY TO SITE CLASSIFICATION

As noted before, due to the nonlinear response of soil, and as shown in Tables 1 and 2 under the proposed new provisions, the site coefficients depend on the level of spectral values. Furthermore, the incremental difference between the site coefficients increases as the soil gets softer. For example, for mapped short-period spectral accelerations of 0.25g the site coefficient increases from 1 for Site Class B, to 1.2 for Site Class C (a 20% increase). However, it increases from 1.6 for Site Class D, to 2.5 for Site Class E (an increase of 56%). Thus, noting differences in the return period and the fact that design spectrum is only one parameter in the determination of seismic forces, for low period systems on soft soils the spectral accelerations are higher when the proposed new provisions are employed. In light of these facts, it is recommended that for major projects soil dynamic characteristics (such as shear wave velocity) are measured in order to more accurately classify a bridge site without much conservatism. This is more so when available data (such as SPT blow count) indicate the site is on the boundary between two soft soil classes (as illustrated in the following paragraph). Availability of advanced technologies makes such measurements very cost effective with the potential for significant cost savings in the final bridge design.

From a list of NJ bridges with more than 3-spans and built recently (since 1997), five bridges distributed throughout the state were selected to classify the sites using available data and determine the corresponding design response spectra using both the existing and proposed new codes. These bridges are:

- Union Avenue over Passaic River (Bergen County),
- NJ Rt. 15 over NYS Western Rail (Sussex County),
- NJ Rt. 35 over Navesink River (Monmouth County),
- US Rt. 206 over Raritan River (Somerset County), and
- Ocean Drive over Middle Thorofare (Cape May County).

For all the bridges SPT blow counts at various locations along the span are available. Using a combination of proximity to the center of the bridge and lowest SPT blow count, one test was

selected for each bridge site to classify the soil. The test was used as directly measured in the field without any correction and the procedure outlined in the proposed new provisions was followed to determine average SPT blow count (N-value). Site classes A (hard rock) and B (rock) can be classified using only shear wave velocity and, therefore, not used here.

The bridge site in Somerset County is classified as Class C with an N-value[2] of 64 blows/ft. The corresponding response spectrum is compared to Division I-A spectrum for Type II soil in Figure 4a. With N-values of 22 and 49 blows/ft, respectively, Bergen and Cape May sites are both classified as Class D and their spectra are compared to Division I-A spectra at these counties for Type II soil (Figures 4b and 4c). Note that with only four soil types in Division I-A compared to six site classes under the proposed new provisions there are some overlap in site definitions. The Monmouth site with an N-value of 3 blows/ft is clearly a Class E soil and its spectrum is compared to Type IV soil under Division I-A in Figure 4d.

The average N-value for the bridge in Sussex County is 17 blows/ft, just above the 15 blows/ft to be classified as Class D. The response spectrum is compared to Division I-A Type II soil type in Figure 5a. However, if the site is classified as Class E the spectral accelerations will be much higher as shown in Figure 5b. The short-period accelerations go up from 0.53g to 0.76g. This is an example of sensitivity of design spectrum to soil classification, especially for low seismic regions. Determination of dynamic soil characteristics (e.g., for example shear wave velocity) may result in classification of the site as Class D with more confidence.

Based on Division I-A, due to the use of not clearly justified cap, the short-period peak is the same regardless of the soil classification.

For many bridge sites considered in NJ (including some of the above examples) the rock layer is often within the top 100-ft. If the SPT count for the remaining depth to 100-ft depth is assumed equal to the blow count for the last 5-ft measured, then the soil classification for the above examples, except for the Monmouth site, goes up one class (i.e., C rather than D or D rather than E). In light of the importance of soil classification on spectral values, especially for low seismic regions, it is well justified to define the soil class more accurately and aggressively. Therefore, it is recommended that consideration be given to collecting additional dynamic properties such as shear wave velocity during soil exploration of future bridge sites. Fundamental soil dynamic properties can be used to classify the site more accurately and to compare to classification using SPT blow count to increase confidence in the latter as an adequate soil parameter for site classification. With the collection of adequate numbers of such data, empirical relationships can

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[2] SPT indicates shallow rock depth at 10-ft, thus, the N-value is a conservative estimate.

be developed among various soil dynamic properties for future use. Availability of such relationships will have important design utility. Associated cost in collecting this data is expected to be insignificant compared to the overall bridge construction cost. There is also the potential for significant cost savings in the bridge itself as well as savings in the design of future bridges.

SEISMIC FORCES

As it was mentioned, simply comparing NCHRP 12-49 response spectrum for an MCE event to Division I-A response spectrum is incorrect and ignores several facts or scientific and empirical knowledge; namely:

- The return periods for the two spectra are not the same,
- More recent earthquakes have demonstrated the need for consideration to rare events, especially for regions of low seismicity such as NJ,
- Lack of scientific and/or empirical explanation for the short-period cap used in the Division I-A response spectrum,
- Importance of site conditions as demonstrated by several recent earthquakes,
- Considerations to characteristics of bridge nonlinear response and dynamic properties as important parameters in determining the level of seismic forces, and
- The interdependency of the bridge dynamic characteristics and the site conditions (boundary springs or soil-structure interaction).

The new spectral shape as discussed before removes the conservatism associated with Division I-A acceleration response spectrum for long period systems. For long period systems the new response spectrum reduces, as a function of the inverse of period (i.e., $1/T$), while under the existing codes (Division I-A) it reduces at a slower rate, which is the inverse of period to the power of 2/3 (i.e., $1/T^{2/3}$). Therefore, system period plays a more critical role in determining seismic forces and designers can reduce the seismic forces substantially by adjusting the design and elongating the system period. It is not within the scope of this study to quantify various procedures to achieve this objective since development of such procedures require a holistic approach considering other loads (gravity and environmental) as well as due consideration to specific serviceability requirements.

Seismic Design and Analysis Procedure (SDAP) and R-Factor

Seismic design and analysis procedure (SDAP) determines the level of analysis required, which in turn determines how accurately and aggressively the system response characteristic is represented. The proposed new seismic provisions provide for six SDAPs, namely: A1, A2, B, C, D, and E. Seismic hazard level (SHL) determines the SDAP.

SDAP A1 and A2 require no rigorous analysis and only minimum design forces at the connections in the horizontal direction are specified.

SDAP B is a no analysis design concept, which is a new addition in NCHRP 12-49 with beneficial implications for low seismic hazard areas such as the State of NJ.

SDAP C is a capacity spectrum design method that is another new addition to NCHRP 12-49 and combines demand and capacity analysis, including the effect of inelastic action. Thus, it can be utilized to explicitly see the trade-off between the seismic design forces and displacements. The bridge is designed for non-seismic loads and then the adequacy of resulting displacements is assessed. The approach is simple and does not require determination of the period of vibration. A large number of bridges in NJ can qualify for this category. Therefore, a simple example is provided in Appendix II to show the application of this procedure. It is recommended that design tools using this method be developed to further simplify its application. A bottleneck to adoption of NCHRP 12-49 is that it requires seismic analysis for a wider range of seismicity. However, SDAP C can be used even for seismic hazard level IV, and as shown in Appendix II the method is quite simple and does not require much effort. Thus, cost implications of a wider range of each state affected by seismic requirements under NCHRP 12-49 is probably minimal to none.

SDAP D is a one step design procedure that uses elastic analysis using cracked section properties. Similar to the existing provisions Uniform Load or Multimode method of analysis can be employed. Elastic forces from the analyses shall be modified using the R-Factors shown in Table 4 to determine seismic design forces. The new provisions contain an incentive for performing more sophisticated analysis and design as the response modification factor increases by as much as 50% when SDAP E is employed compared to SDAP D. This is important since it appreciates/rewards quality in both design and construction, something missing in the existing provisions. For comparison purposes, Division I-A response modification values are also provided in Table 5. For a single column bent under the proposed new code, considering life safety performance, the R-Factor is equal to 4 if SDAP D is used, and is 6 for SDAP E. Under Division I-A for bridges classified as "Other" (i.e. not critical or essential) the R-Factor is 3 regardless of the design and analysis effort used. Building upon lessons from the Northridge Earthquake there is no difference between single and multi column bents under the proposed new specifications. Under the existing code the R-Factor for multi-column bents is 5 for "Other" bridges.

Since the proposed new seismic provisions are performance based, the performance level determines the value of response modification factor. Under existing Division I-A provisions (Table 5) the importance category determines the R-Factor. This difference must be considered

when seismic forces are compared. Performance related classifications are more appropriate and easier to define and quantify. There cannot be a direct correlation between the two definitions since one is response/performance based while the other is simply a subjective non-engineering definition. However, it appears reasonable to say that critical bridges are those that must remain operational after the upper level earthquake. On the other hand, bridges classified as non-critical or non-essential (i.e., "Other" per Division I-A) are those that one would design for only life safety performance within a performance-based approach. Thus, when seismic forces under the two guidelines are compared proper care must be exercised to correctly correlate these classifications. Design examples that compare the operational level of NCHRP 12-49 to bridge class "Other" under Division I-A are not correct and misleading.

SDAP E combines SDAP D with more advanced analysis methods to take advantage of higher R-Factors. As shown in Table 4, the increase in R-Factor can be as much as 50% (e.g., 6 as oppose to 4 for single and multi column bents). Displacement capacity verification or pushover analysis is employed to verify displacement capacities. The pushover analysis will use nonlinear two-dimensional models. Today's engineers are well qualified to perform such analysis and the capability does exist within many commercial packages. A simple example is provided in Appendix III to demonstrate this method. Significant advancements were made in nonlinear time history analysis and in development of design tools after the Loma Prieta Earthquake of 1989. It is a logical and natural evolution of the seismic codes to take advantage of such advancements in order to make future designs better and more economical. The incentive in using advance analysis goes beyond the increase in R-Factor. It is stipulated that if sufficient displacement capacity exists the substructure design forces may be reduced by an additional 30%.

R-Factor Modifier and Strength Reduction Factor Φ

A new addition to NCHRP 12-49, which can increase the seismic forces in lower period systems, is the period modifier on the response modification factor as discussed in article 4.7 of the provisions [2]. Extensive research over the past decade on the relationship between ductility demand and R-Factor has demonstrated that this modifier must be a part of any new code. Thus, the basic response modification factor of Table 4 is modified using the following equation:

$$R = 1 + (R_B - 1) \, T / T^* \leq R_B$$

Where T is system period and $T^* = 1.25 \, T_S$, and T_S is the intersection point between the horizontal and ascending segments of the response spectrum (see Figure I.3 of Appendix I).

Indeed it is a shortcoming of the Division I-A provisions for not recognizing this important factor. As it will be shown later, the impact of this reduction in R-Factor on seismic forces,

however, is not significant since the fundamental period for most bridges is well outside the period range that is impacted by this modifier.

On the other hand, there is a major difference in the resistance factors under the two codes, which when considered, will result in lower seismic forces under the proposed new code. Based on the existing AASHTO Division I-A the design, strength is determined by applying a reduction factor (Φ factor) that ranges from 0.5 to 0.9, depending on the axial load and the amount of longitudinal reinforcing. Under the proposed NCHRP 12-49 this factor is unity for both spiral and tied columns. Typical Φ values under Division I-A designs is about 0.6 to 0.7. This is equivalent to a 40 to 70 percent increase in the seismic forces in order to correctly compare the two provisions.

Response Spectra / R: Rock and Soft Soil Sites

A better comparison of the two seismic specifications is made by comparing the seismic forces. This is represented by dividing the design response spectra by the appropriate response modification factor (considering low period modifier) and the Φ factor. Note that, as it was just mentioned increasing the AASHTO Division I-A design spectrum by dividing it by the Φ factor is equivalent to reducing the ultimate capacity by this factor to get the design strength as required by the provisions.

Figure 6 shows response spectra divided by the appropriate factors discussed. Consistent with prior discussions, these are for a bridge in Newark NJ. Here the site is rock and the bridge is supported either on multi-column bents or single column bents. Assumptions used in developing the Division I-A seismic force spectrum are: Type I soil, importance category is non-critical non-essential (i.e., "Other") corresponding to response modification factors of 5 (multi-column) and 3 (single column), and response reduction factor, Φ, of 0.7 is assumed. Assumptions used in obtaining the corresponding NCHRP 12-49 seismic force spectrum are: Class B soil, life safety performance, upper level event (MCE or 2500 years return period), base response modification factor of 6 (SDAP E), reduction of 30 percent assuming displacement capacity verified, and response reduction factor of unity (Φ = 1). Because of the low period modifier, under NCHRP 12-49 the actual response modification factor depends on the period and can be as low as unity for short-period systems, thus, the reason for the significant change in the shape of the seismic force spectrum for NCHRP 12-49 compared to spectral acceleration spectrum.

Figure 6a indicates that except for systems with periods less than 0.08 seconds, the seismic forces based on proposed NCHRP 12-49 is smaller than those based existing AASHTO code. For periods less than 0.08 the seismic forces based on NCHRP 12-49 are only marginally higher. The relevant period range for typical highway bridges (2 to 4 spans slab-on-girder) is between

0.1 to 2.0 seconds. Furthermore, the dynamic response of such systems is dominated by the fundamental mode with a period more towards the high end of the above range. Thus, as it will be shown even when a less aggressive assumption is made for design under NCHRP 12-49, the seismic forces will still be mostly lower than the Division I-A forces.

Figure 6b shows similar results assuming the bridge is supported on single column bent(s). Based on NCHRP 12-49 there is no difference in the R-factor for single and multi-column bents, and the seismic forces stay the same. However, Division I-A forces increase significantly since the response modification decreases to 3 (from 5). Consequently, the seismic forces are lower under the proposed new code for all periods, and they are significantly lower for high period bridges.

Figure 7 shows similar curves for a bridge on a soft soil site. Thus, all assumptions have stayed the same as those used in developing Figure 6 spectra except for soil conditions (under NCHRP 12-49 the soil class is E and it is Type IV for Division I-A). Similar to rock site case, for typical highway bridges, where the fundamental period is normally higher than 0.5 seconds the seismic forces would be generally lower under the proposed new seismic code. It should be noted that for the very same reason a trial design of the Doremus Avenue Bridge, which was conducted as a part of MCEER / AASHTO T-3 Trial Design Project for NJDOT, also found no increase or lower forces in the columns when NCHRP 12-49 is used, despite the fact that the bridge is located on a very soft soil. Considering Figure 7, again for single column bents, the difference in seismic forces in the relevant period range is even more significant. Thus, contrary to existing misperceptions and misunderstandings the proposed new LRFD guidelines do not necessarily result in higher seismic demands. Designers who use more advanced analysis methods can actually produce designs that are much more economical while using state-of-the-art knowledge.

Despite the above observations, in response to ongoing misunderstandings on the actual level of seismic forces, it has been suggested that a 1,500 year return period rather than a 2,500 year return period be employed as the upper level event. As it can be inferred from these curves, such a move can significantly increase the vulnerability of bridges in New Jersey since the seismic forces will be reduced to a minimum. It would be similar to the mid-60s move by CalTrans in changing the design spectrum that was subsequently corrected through lessons learned during the earthquakes of the 70s and 80s (Figure 8).

In comparing the spectra shown in Figures 6 and 7, a major difference is the intersection point of the NCHRP spectrum with Division I-A spectrum. For the soft soil site and multi-column bents the intersection period is increased from 0.08 to 0.36 seconds. For the single column bents case and a soft soil site there is now a short period region (T < 0.18 sec) where NCHRP forces are

higher. Indeed this is an advantage of NCHRP 12-49 that reflects recent findings on nonlinear site factors and low period modifiers. Therefore, should there be designs with very low fundamental periods the design guidelines must correctly quantify the correct relationship between linear and nonlinear bridge responses, and properly address the impact of soil characteristics on input motion. However, it is not expected that consideration to these facts will result in increases in seismic forces in too many cases because only for soft soils the difference in short-period spectral values is notable. However, as explained below, when due consideration is given to proper modeling of the soil-structure interaction, it is expected that bridges on soft soil typically have long periods that do not fall within this narrow range of high spectral values.

A Note on Soil-Structure Interaction

An advantage of NCHRP 12-49 is that it provides explicit information on foundation modeling known as FMM (Foundation Modeling Method). Guidelines, including tables and charts, are provided on how to model boundary springs representing various foundation types. Table 6 shows typical equations used in determining boundary springs representing a spread footing. As it can be seen the spring constants are directly related to the shear modulus, G, which in turn is related to the shear wave velocity through the well-known wave equation (power of two relationships). As discussed, shear wave velocity, V_s, is the fundamental soil property used to define soil class (e.g., $V_s > 5,000$ ft/sec is Class A and $V_s < 600$ ft/sec is Class E). Thus, the spring stiffnesses for a bridge on a Class A (rock) site can be <u>two orders of magnitude higher</u> than those for a bridge on a Class E (soft soil) site. That is, for rock (stiff soil) sites the stiffnesses of the boundary springs are higher and so will be the bridge fundamental frequency. However, for rock (stiff soil) sites the seismic forces based on NCHRP 12-49 <u>is about the same or even lower</u> than Division I-A design spectrum at all frequencies (Figure 6). This takes place despite the fact that response spectrum for the 2500-year return period (NCHRP 12-49) is compared to the response spectrum for the 500-year return period (Division I-A).

On the other hand, for bridges on soft soil sites the stiffnesses of the springs modeling the foundation are significantly smaller, thus, elongating the bridge's fundamental period. Therefore, the system period will fall into the velocity (long-period) region of the response spectrum where the spectral accelerations <u>based on NCHRP 12-49 are less than those based on Division I-A</u> spectrum even for soft soil condition (Figure 7). Referring to Figure 7, seismic forces based on NCHRP 12-49 go below that based on Division I-A for periods longer than 0.36 second. Typical viaduct bridges on soft soil are expected to have their fundamental period towards the high-end of relevant period range, around 1.5 to 2 seconds. A parameter study to more accurately quantify this interdependency is timely and can shed more light on the exact impact of the proposed new provisions on the level of seismic forces.

Response Spectra / R: Medium Soil Site

The above discussions dealt with extreme soil types, namely rock and soft soil. Figure 9 shows response spectra for a medium soil type (Class C or Type II) in Newark, NJ. Furthermore, seismic force spectra under the proposed NCHRP 12-49 for two additional situations, where the design method does not use advance analysis or the design is less aggressive are also plotted. These are: i) a case where under SADP E no additional 30 percent reduction is made and, ii) a case where similar to current approach modal analysis (SDAP D) is assumed to be used (i.e., no pushover analysis performed). Under SDAP D the response modification factor is 4 compared to 6 for SDAP E. These NCHRP 12-49 spectra, which are the same for single and multi-column bents are compared to Division I-A spectra for single and multi-column bents.

As it can be seen from comparison of these curves, even when the less advance analysis method is employed the NCHRP 12-49 seismic forces are much smaller than Division I-A for the relevant period range (shaded area in Figure 9). In the higher end of this region (e.g., T = 1 second) the seismic forces using SDAP D is almost half of Division I-A for multi-column bents and about one-third for single column bents. Seismic forces based on proposed NCHRP are even smaller when a more advance analysis approach is employed. For example, for a system with 1-second fundamental period the seismic forces for SDAP E (with 30 percent reduction) are one-sixth and one-fourth of Division I-A forces for single and multi-column bents, respectively.

It must be emphasized that the observations discussed in this report are not unique to this site and applies to the entire state of NJ. Indeed the trend is the same for all over US. To this end, it is expected that for eastern and central US states seismic forces based on NCHRP 12-49 are in general comparable and often lower than those based on Division I-A. Therefore, as it was stated before and shown in the next section any move to lower the seismic hazard by reducing the return period may have a serious implication with regard to future events. The Kobe earthquake demonstrated that rare events do indeed happen. Future seismic design provisions must take lessons learned during past two decades into account, as the NCHRP 12-49 has done.

IMPACT OF LOWER RETURN PERIODS

As demonstrated qualitatively within the previous sections of this report, it is not expected that the proposed new provisions will increase the level of seismic forces significantly. Indeed, it is more likely that for most bridges the seismic forces will decrease. Parameter studies performed as a part of the NCHRP 12-49 project also has shown that [2] "the net effect on the cost of a column and spread footing is on the average 2 percent less than the current Division I-A provisions for multi-column bents and 16 percent less than Division I-A provisions for single column bents." However, there still persists the misunderstanding that the proposed NCHRP 12-49 provisions increases the seismic demand significantly. This is a major contributor to the lack

of a decision to adopt these state-of-the-art provisions. There are two reasons for this misunderstanding. One is due to the fact that conclusions are made by merely comparing the design spectra. As discussed, the response spectrum is only one parameter in determining the seismic forces. Response and dynamic characteristics of the system must be considered in order to determine the exact level of seismic forces. When these factors are considered the proposed new provisions do not necessarily result in higher seismic forces and indeed in most cases the seismic forces will be lower.

Another reason behind the perception that NCHRP 12-49 results in higher demands than Division I-A is that the comparisons made are often not necessarily correct. For example, in a major study performed for the Illinois Department of Transportation (IDOT) design of two bridges based on both provisions are compared [6]. Bridge sites based on Division I-A are characterized as having soil Type I and III. However, based on NCHRP 12-49 both sites are characterized as Class D. This is clearly a major inconsistency and incorrect. Both soil Type I and III under Division I-A cannot be equivalent to Class D under NCHRP 12-49. Furthermore, both bridges are classified under Division I-A as non-critical and non-essential (i.e., "Other") and an R-Factor of five (R = 5) is used to determine seismic forces. However, under NCHRP 12-49 the performance level "Operational" with an R-Factor of 1 and 0.9 is employed to determine the seismic forces, and to compare the designs. Again, this is a gross and fundamental mistake whereby designs that allow for nonlinear actions are compared to ones that are expected to remain fully elastic. Therefore, it is not surprising that the final designs are significantly different.

Nevertheless, the misunderstanding with regard to higher demands under NCHRP 12-49 persists. Consequently, an approach where the hazard is based on a lower return period (either 1,500 or even 1,000 years as oppose to 2,500 year in NCHRP 12-49) is being pursued as a possible revision. For example, it is suggested that the design response spectrum equal to two-thirds (2/3) of the response spectrum for 2 percent in 50-Yr event be employed. Figure 10 attempts to quantify the impact of such a proposal on seismic forces. Figure 10 is similar to Figure 9, except that the spectral values of NCHRP 12-49 are reduced by a factor of two-thirds. Here also a Class C or Type II soil is assumed. Similar to Figure 9 various analysis approaches are assessed.

Figure 10 indicates that when a lower return period is used, regardless of the level of sophistication employed in the procedure, and for all system periods, the seismic forces are lower compared to Division I-A for single column bents. This is also true for multi-column bents when SDAP E (with or without additional 30% reduction) is used. More importantly for bridges with a period around one-second, which is a more typical value for a fundamental system period, the seismic forces are dangerously small when compared to the existing provisions. For example, at

one-second period the seismic forces are only 2.6 percent of the system weight when SDAP D is used. They reduce to 1.7 percent when SDAP E is used and are merely 1.2 percent when an additional 30 percent reduction is taken assuming displacement capacity is adequate, which very likely is the case. These numbers are clearly too minuscule and a bridge designed to carry gravity and traffic loads will easily satisfy any seismic requirements associated to these levels of forces. <u>This will leave the highway system seriously vulnerable to occurrence of even moderate earthquakes.</u> It is, therefore, strongly recommended that a hazard as defined under the existing version of NCHRP 12-49 be retained.

SITE SPECIFIC ANALYSIS

Attempts were made to develop site-specific response spectra for several sites in NJ in order to assess the adequacy of those developed based on USGS maps. However, the results were not conclusive due to the lack of a reliable empirical relationship among various soil data (e.g., STP blow count, shear wave velocity, damping, etc.). It was observed that spectral values are much more sensitive to dynamic soil properties at the site than rock motion time histories. It is recommended that consideration be given to the collection of additional soil data (such as dynamic properties) either through a comprehensive research project or as a part of future bridge constructions. Using such data, a guide for estimating the dynamic properties of NJ soil for site specific analysis can be developed, and can be used as a resource document. A similar effort for South Carolina soils has been successfully completed [7]. Combined with available resources from USGS (such as interactive deaggregation) practitioners and researchers would have the needed tools to perform more reliable site-specific analysis.

A point to be considered to facilitate adoption of the NCHRP 12-49 is to relax the limit on site-specific spectra. Currently, it is specified that the response spectra determined from a site-specific study shall not be lower than two-thirds of the response spectra determined using the general procedure. As more knowledge is being created in this area and newer and "less wrong" models are being developed and considering the desires in part of some states to take risks at the expense of savings of public funds, revisiting this limit may indeed be a good option. A byproduct of site-specific analysis as a more common practice would be better, and more collaboration between the structural and geotechnical engineers. Combined with the availability of more detailed information on site conditions, this can have significant cost savings and safety implications. Soil structure interaction is an important area in the field of earthquake engineering and seismic design of structural systems that can benefit from additional collaborations and advancements.

ADDITIONAL NOTES ON NCHRP 12-49

The proposed new seismic code for highway bridges (NCHRP 12-49) contains many other additions and modifications, which were not the subject of this evaluation. Among the more important additions and changes are:

- Earthquake Resisting Systems and Elements (ERS and ERE) – A new concept to encourage implementation of some seismic resisting systems and elements not allowed under existing AASHTO provisions. Furthermore, classification of ERS and ERE into three categories including one that requires owner's approval will trigger due consideration to seismic objectives and performance.

- Abutments – Incorporating recent research results the provisions recognizes that the abutment can be an important part of the ERS. This can have significant beneficial effect for bridges in the state of New Jersey. In a study sponsored by NJDOT to assess seismic performance of exiting bridges the beneficial impact of abutments in resisting the seismic forces is demonstrated [8, 9].

- Liquefaction – An important change in the new provisions where recent research results are employed.

- Steel and Concrete Design Requirements – While there are no major additions to concrete provisions the seismic requirements for steel bridges are new.

- Bearing Design Requirements – Three design alternatives are included to address the performance issues associated with some bearing types. These include a design concept that permits the girders to slide on a flat surface if the bearings fail. Again, this is a very important addition that can reduce seismic risk while providing cost savings. Saadeghvaziri, et al. [8, 10] considered this mode of response in assessing the response of exiting bridges and it was determined that such a behavior, if stable, can have significant cost saving implications for the State of New Jersey vis-à-vis seismic retrofit (or lack of need for retrofit) of bridges.

CONCLUSIONS AND FUTURE RESEARCH NEEDS

This study was an attempt to qualitatively assess the impacts of the proposed NCHRP 12-49 provisions on the seismic design of bridges in the state of New Jersey. The findings can be summarized as strong support for implementation of the proposed new provisions in its present form. Besides the fact that the new provisions are based on state-of-the-art knowledge and recent scientific developments, it is determined that contrary to some misperceptions and misunderstandings the proposed new LRFD guidelines do not necessarily result in higher seismic demands. Indeed, for the State of New Jersey, for the majority of designs (if not all) the new provisions can result in lower seismic forces. This is for several reasons, more notably for the facts that the new provisions contains an incentive for performing state-of-the-art analyses, and

that it has removed the conservatism associated with current AASHTO acceleration response spectrum for long period systems.

Currently the NJDOT Design Manual contains an article that allows the use of the NCHRP 12-49 provisions as an alternative [11]. Results of this study indicate that it is well justified to take this one step further by making comparative designs based on both codes a part of project scope. In addition to the potential for significant cost savings availability of such information can shed more light on advantages and shortcomings of the proposed new provisions.

Another objective of this study was to develop resources to conduct training workshops on the application of the new guidelines since it is a more advanced and complicated document than existing Division I-A provisions. A seminar entitled *"Seismic Performance and Design of Highway Bridges: Past, Present, and Future (and Implications for NJ),"* was conducted on May 17, 2004 at the NJDOT headquarters. Over eighty consultants and staff engineers attended this seminar. NJIT will continue to work with the NJDOT to plan additional workshops and courses on the more specific aspects of designs using these provisions and recent developments. However, such efforts should consider the fact that T-3 Technical Committee for Seismic Design of AASHTO Subcommittee on Bridges and Structures is considering the possibility of major revisions to the provisions before its adoption. An important revision considered is to reduce the seismic hazard (by lowering the return period). Such a move can have significant adverse effect on seismic vulnerability of bridges in the state and should be avoided. Perhaps a more appropriate direction to take is to provide the resources to create the knowledge base to gain more confidence in site-specific analysis, thereby, relaxing the limit on the results of such analysis.

A major shortcoming of both the existing and proposed new seismic provisions for highway bridges is the lack of explicit consideration to the effect of the vertical component of earthquake ground motion on the seismic response of bridges. Thrust of seismic considerations and guidelines for highway bridges so far has been mostly substructure (more specifically columns) and horizontal motions [12]. Consideration to vertical motion can have great dual benefits vis-à-vis blast loads and bridge security, which can also cause reversed loading. Furthermore, consideration to deck detailing and response become equally important under extreme event loads, if not more important due to larger areas and shape of the decks vis-à-vis compressive pressure blast waves. Thus, additional research work on this subject should be seriously considered since it will greatly enhance bridge security while decreasing seismic vulnerability. To this end, possible research topics for immediate consideration are the effects of blast and vertical seismic loads on pre-tensioned and post-tensioned bridges, and simulations on shape/design optimization to lessen the adverse effects of compressive stress waves and upward

motion. Prestressing profiles are gravity based and inadequate. Simple consideration to detailing under reversed loads can have significant safety and seismic implications. Analytical simulations can also be used to develop simple design tools. In addition to flexural strength, shear capacity and development of reliable load path under reversed loading are important issues that require a comprehensive experimental and analytical study. For example, the author is of the opinion that superstructure shear under reversed loading was indeed the cause of a bridge collapse during the Northridge earthquake [13]. Use of CFRP for strengthening and work on recovery measures (such as development of deployable/movable bridges) are further topics, related to existing bridges, which should be explored. A pilot study sponsored by NJDOT has demonstrated the effectiveness of CFRP composites to enhance serviceability performance of continuity connections [14] while increasing structural efficiency.

Another area that can benefit from additional research is soil-structure interaction. A major change under NCHRP 12-49 as a result of recent developments is new soil factors, which can have significant impact on the level of seismic design forces. However, although spectral values are high for short-period systems on soft soil, systems on soft soil normally have long periods when soil-structure interaction is modeled using boundary springs. In general, and regardless of soil class, the long-period spectral values are much smaller under the proposed new provisions. A parameter study to more accurately quantify this interdependency is timely and can shed more lights on the exact impact of the proposed new provisions on the level of seismic forces.

REFERENCES

1. Applied Technology Council (1981), "Seismic Design Guidelines for Highway Bridges," ATC-6, pp. 210.
2. MCEER, (2001), "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, NCHRP Project 12-49," MCEER, Buffalo, NY.
3. Frankel, et al., (1997), "USGS national seismic hazard maps: uniform hazard spectra, de-aggregation, and uncertainty." Proc. of FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities, *NCEER Technical Report 97-0010*.
4. Takasumi, D.L., et al., "Design Seismic Ground Motions for a Short-Period Bridge," Proc. of 4th National Seismic Conference on Bridges and Highways, Memphis, TN, Feb. 9-11, 2004.
5. AASHTO, "AASHTO LRFD Bridge Design Specifications," 2nd Edition, Washington, D.C., 1998 (2000 Interim).
6. Panahshahi, et al., "A Comparison Study of Seismic Design of Illinois Highway Bridges Using Proposed NCHRP and AASHTO Standard Specifications," Proceedings, 4th National Seismic Conference on Bridges and Highways, Memphis, TN, Feb. 9-11, 2004.
7. Andrus, et al., "Guide for Estimating the Dynamic Properties of South Carolina Soils for Ground Response Analysis," Final Report, Dept. of CEE, Clemson Univ., Clemson, 2003.
8. Saadeghvaziri, M. Ala, and A. Yazdani-Motlagh, "Nonlinear Response of MSSS Bridges under Earthquake Ground Motions," Final Report, FHWA/NJ-99-026-7270, Oct., 1999.
9. Saadeghvaziri, M. Ala, A. Yazdani-Motlagh, and S. Rashidi, "Effects of Soil-Structure Interaction on Longitudinal Seismic Response of MSSS Bridges," Soil Dynamics and Earthquake Engineering, Vol. 20, No. 1-4, pp. 231-242, December, 2000.
10. Saadeghvaziri, M. Ala, A. Yazdani-Motlagh, and S. Rashidi, "Seismic Response and Evaluation of MSSS Bridges in New Jersey," 2001 Structures Congress, ASCE, Washington, DC, May, 2001.
11. NJDOT, "Bridges and Structures Design Manual," Fourth Edition, Trenton, NJ, 2002.
12. Saadeghvaziri, M. Ala, and D. A. Foutch, "Inelastic Response of R/C Bridges under Horizontal and Vertical Earthquake Motions," SRS 540, University of Illinois @ Urbana-Champaign, Dept. of Civil Engineering, Urbana, IL, June, 1988, 270-Pages.
13. Saadeghvaziri, M. Ala "A Case Study from the Northridge Earthquake," Paper No. 1890, Eleventh World Conference on Earthquake Engineering, Elsevier Science Ltd., June, 1996.
14. Saadeghvaziri, M. Ala, L. Yin, and W. R. Spillers, "New Continuity Connection Using Fiber Reinforced Composites," Proc. of the 4th International Conf. on Advanced Composite Materials in Bridges and Structures, Calgary, Alberta, July 20-23, 2004.
15. "Competing Against Time," Report to Governor Deukmejin from The Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake, May, 1990.
16. Mast, R. M., Marsh, L., Spry, C., Johnson, S., Griebenow, R., Guarre, J. and Wilson, W. (1996). "Seismic Design of Bridges Design Example Nos. 1-7," Publication Nos. FHWA-SA-97-006 to 012, Federal Highway Administration, Washington, D.C.

Table 1. Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration [2]

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25$ g	$S_s = 0.50$ g	$S_s = 0.75$ g	$S_s = 1.00$ g	$S_s \geq 1.25$ g
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	a	a	a	a	a

NOTE: Use straight line interpolation for intermediate values of S_s, where S_a is the spectral acceleration at 0.2 seconds obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

Table 2. Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration [2]

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
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	a	a	a	a	a

NOTE: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.

a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3). For the purpose of defining Seismic Hazard Levels in Article 3.7 Type E values may be used for Type F soils.

Table 3 Low Period Spectral Accelerations and Corresponding Ratios for Class B and E

County	NCHRP 12-49		Division I-A		Ratios: NCHRP / I-A	
	S_s	F_a: Class E	A	Low Period Cap	Class B	Class E
Bergen	0.425	1.94	0.18	2.5	1.01	1.83
Monmouth	0.381	2.08	0.15	2.5	0.94	2.11
Cape May	0.183	2.50	0.10	2.5	0.73	1.83
Sussex	0.350	2.18	0.18	2.5	0.78	1.69
Somerset	0.390	2.05	0.18	2.5	0.86	1.78
Salem	0.322	2.27	0.10	2.5	1.28	2.92
Burlington	0.320	2.25	0.15	2.5	0.87	1.96
Atlantic	0.250	2.50	0.10	2.5	1.0	2.50
Camden	0.327	2.25	0.15	2.5	0.87	1.96
Cumberland	0.269	2.44	0.10	2.5	1.07	2.62

NOTE: Class B ratios are determined by dividing S_s by 2.5A (Division I-A has 2.5 cap regardless of soil type). Class E ratios are equal to $F_a S_s$ / 2.5A. Highest ratio is for Salem County (zip code 08067). It should be emphasized that spectral values by themselves do not determine seismic forces.

Table 4. Base Response Modification Factors, R_B, for Substructures – NCHRP 12-49 [2]

Substructure Element	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall Piers – larger dimension	2	3	1	1.5
Columns – Single and Multiple	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – above ground	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – 2 diameters below ground level-No owners approval required.	1	1.5	1	1
Pile Bents and Drilled Shafts – Vertical Piles – in ground - Owners approval required.	N/A	2.5	N/A	1.5
Pile Bents with Batter Piles	N/A	2	N/A	1.5
Seismically Isolated Structures	1.5	1.5	1	1.5
Steel Braced Frame – Ductile Components	3	4.5	1	1.5
Steel Braced frame – Nominally Ductile Components	1.5	2	1	1
All Elements for expected Earthquake	1.3	1.3	0.9	0.9

Table 5. Response Modification Factors for Substructures – Division I-A [5]

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers - larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents • Vertical piles only • With batter piles	1.5 1.5	2.0 1.5	3.0 2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents • Vertical pile only • With batter piles	1.5 1.5	3.5 2.0	5.0 3.0
Multiple column bents	1.5	3.5	5.0

Table 6. Surface Stiffnesses for Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space

Stiffness Parameter	Rigid Plate Stiffness at Surface, K_i'
Vertical Translation, K_z'	$\dfrac{GL}{1-v}\left[0.73+1.54\left(\dfrac{B}{L}\right)^{0.75}\right]$
Horizontal Translation, K_y' (toward long side)	$\dfrac{GL}{2-v}\left[2+2.5\left(\dfrac{B}{L}\right)^{0.85}\right]$
Horizontal Translation, K_x' (toward short side)	$\dfrac{GL}{2-v}\left[2+2.5\left(\dfrac{B}{L}\right)^{0.85}\right]-\dfrac{GL}{0.75-v}\left[0.1\left(1-\dfrac{B}{L}\right)\right]$
Rotation, $K_{\theta x}'$ (about x axis)	$\dfrac{G}{1-v}\,I_x^{0.75}\,\left(\dfrac{L}{B}\right)^{0.25}\left(2.4+0.5\dfrac{B}{L}\right)$
Rotation, $K_{\theta y}'$ (about y axis)	$\dfrac{G}{1-v}\,I_y^{0.75}\,\left[3\left(\dfrac{L}{B}\right)^{0.15}\right]$

1. See Figure 8.4.2.1-1** for definitions of terms
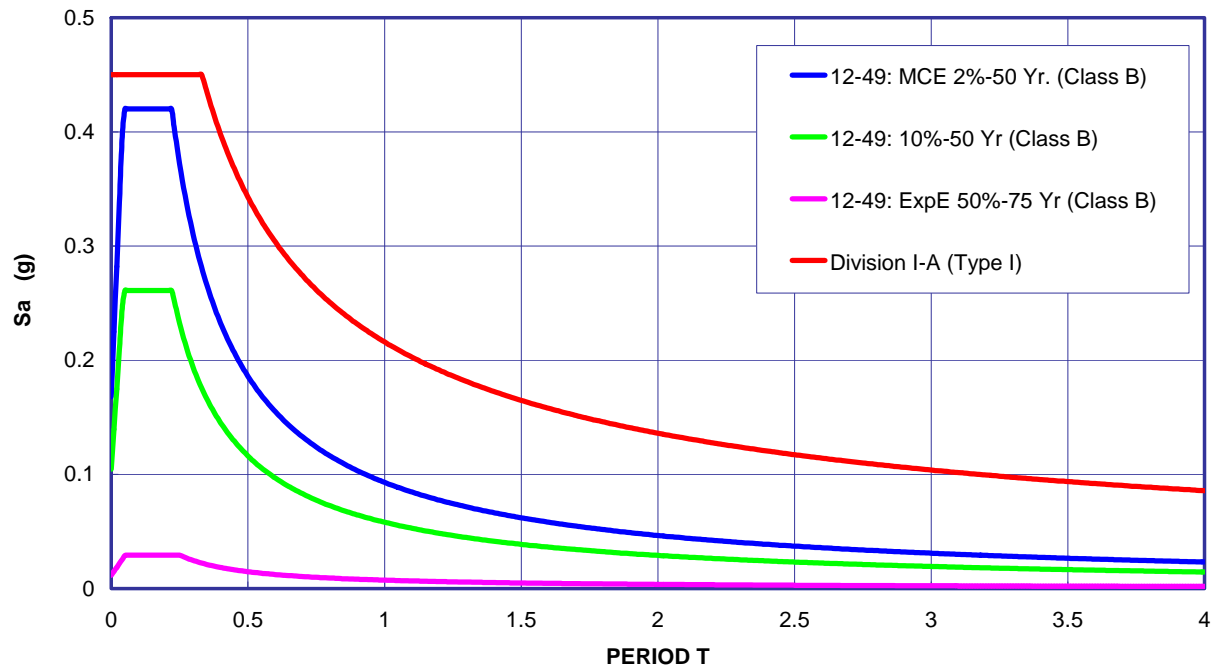
Figure 1. Rock Design Spectra based on Existing and Proposed New Seismic Codes (Newark, NJ)
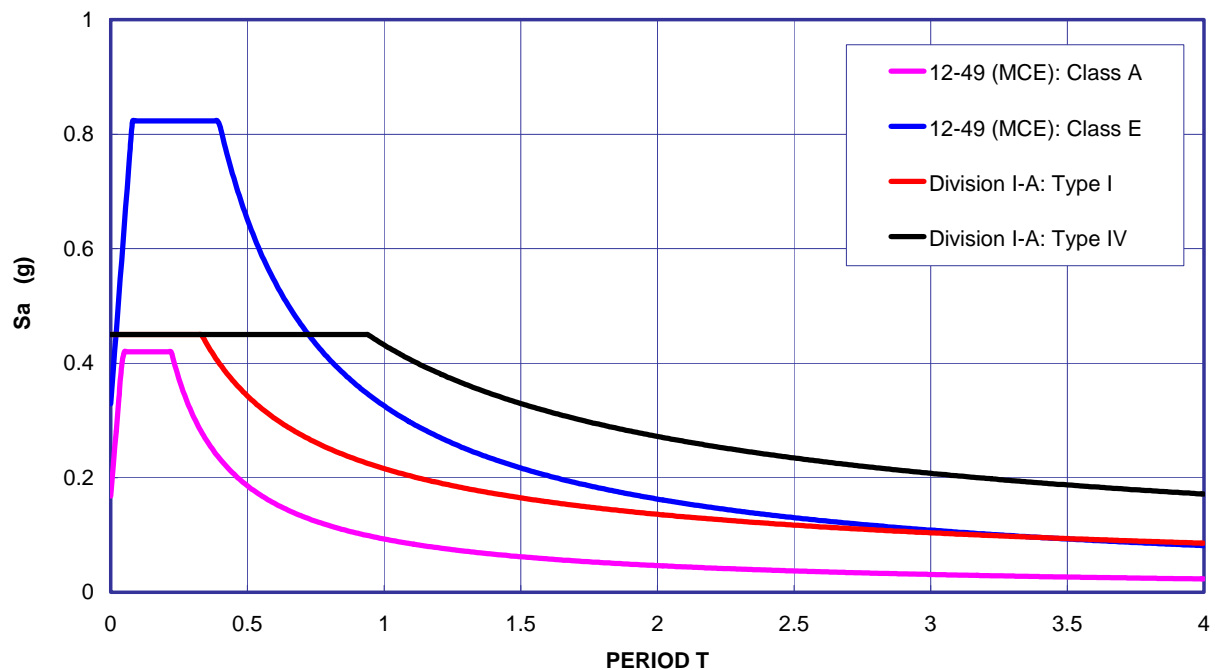
Figure 2. Design Spectra for the Range of Site Condition (Newark, NJ)
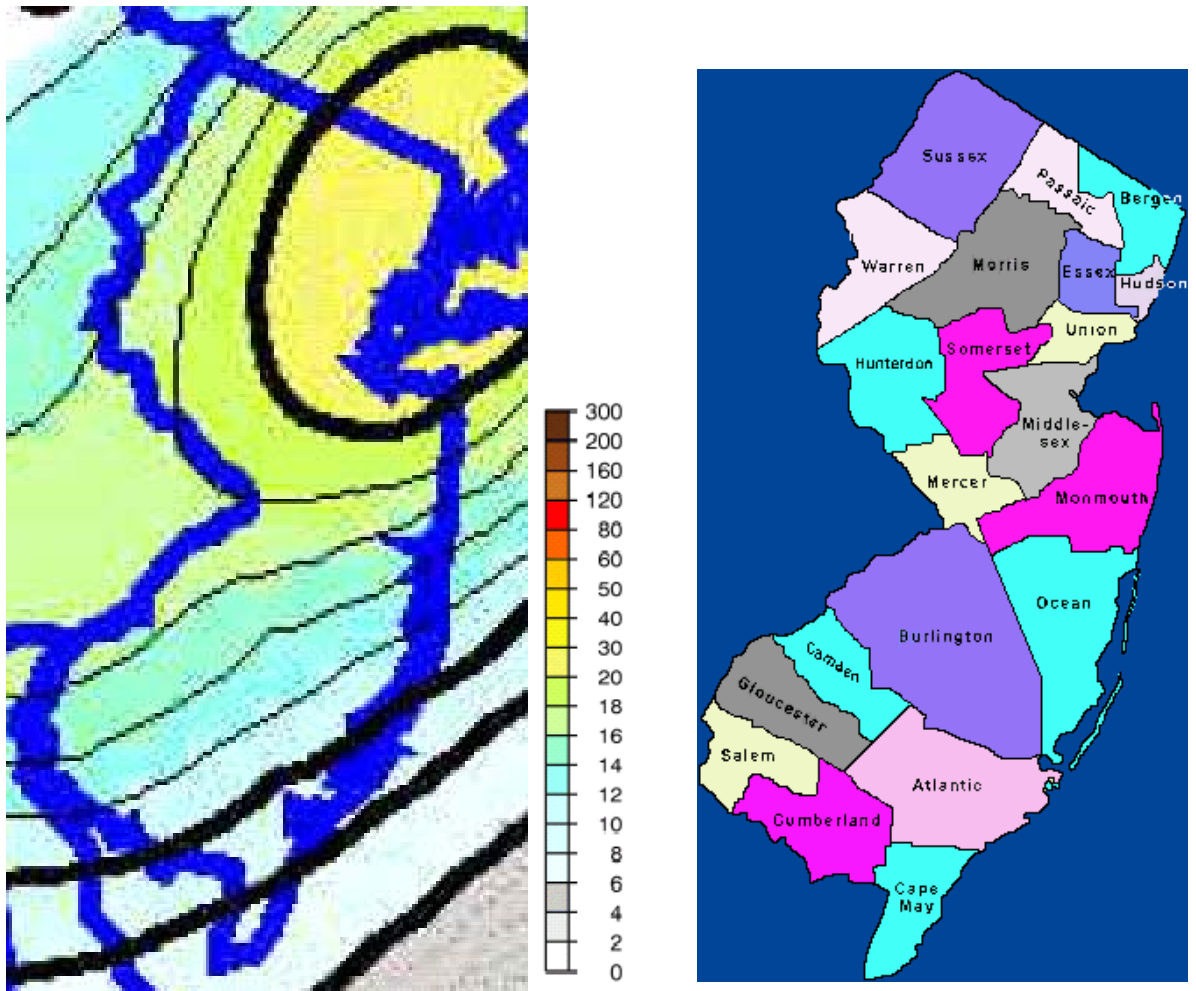
Figure 3. Contour Map of Short-Period Spectral Acceleration for 2% PE in 50 Yr and Counties in the State of NJ.
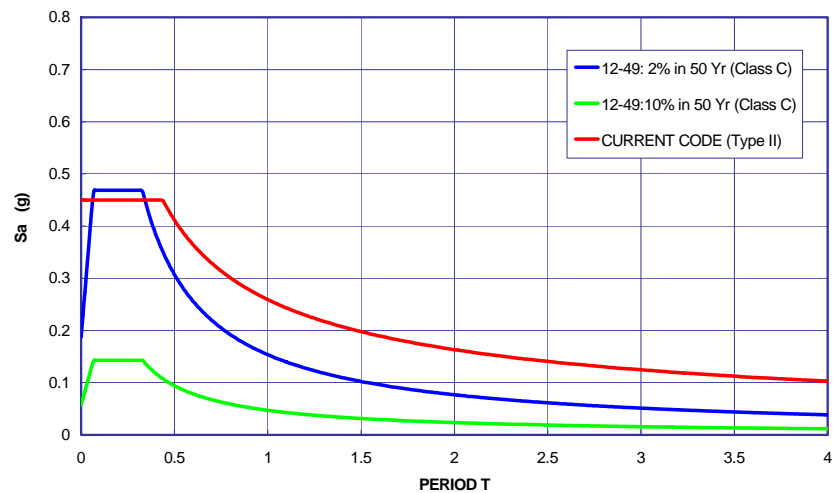
a) Somerset County: US Rt. 206 over Raritan River
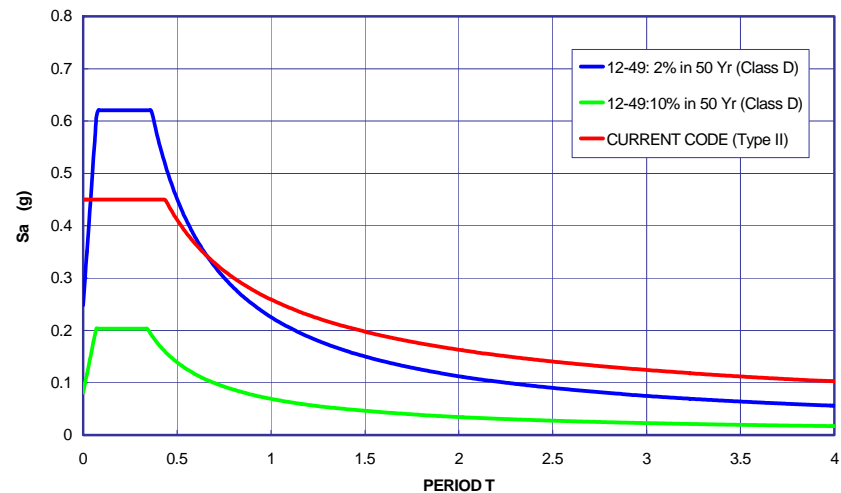
b) Bergen County: Union Avenue over Passaic River
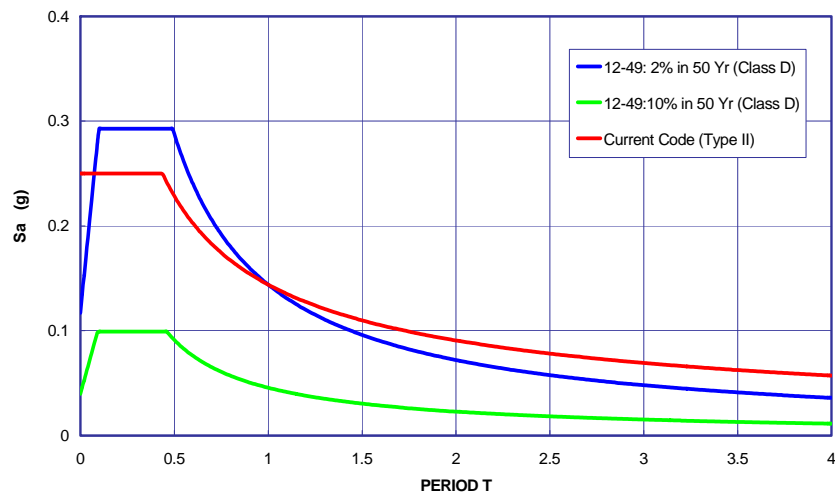
c) Cape May County: Ocean Drive over Middle Thorofare
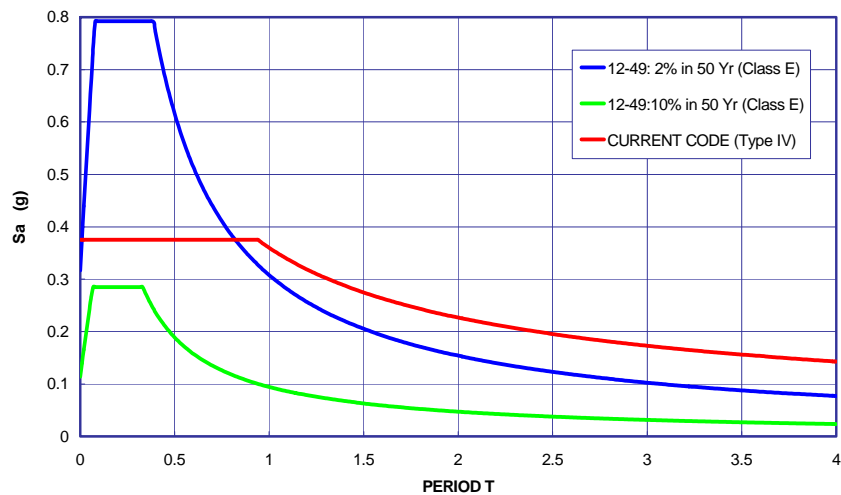
d) Monmouth County: NJ Rt. 35 over Navesink River

Figure 4. Design Spectra based on Existing and Proposed New Seismic Codes

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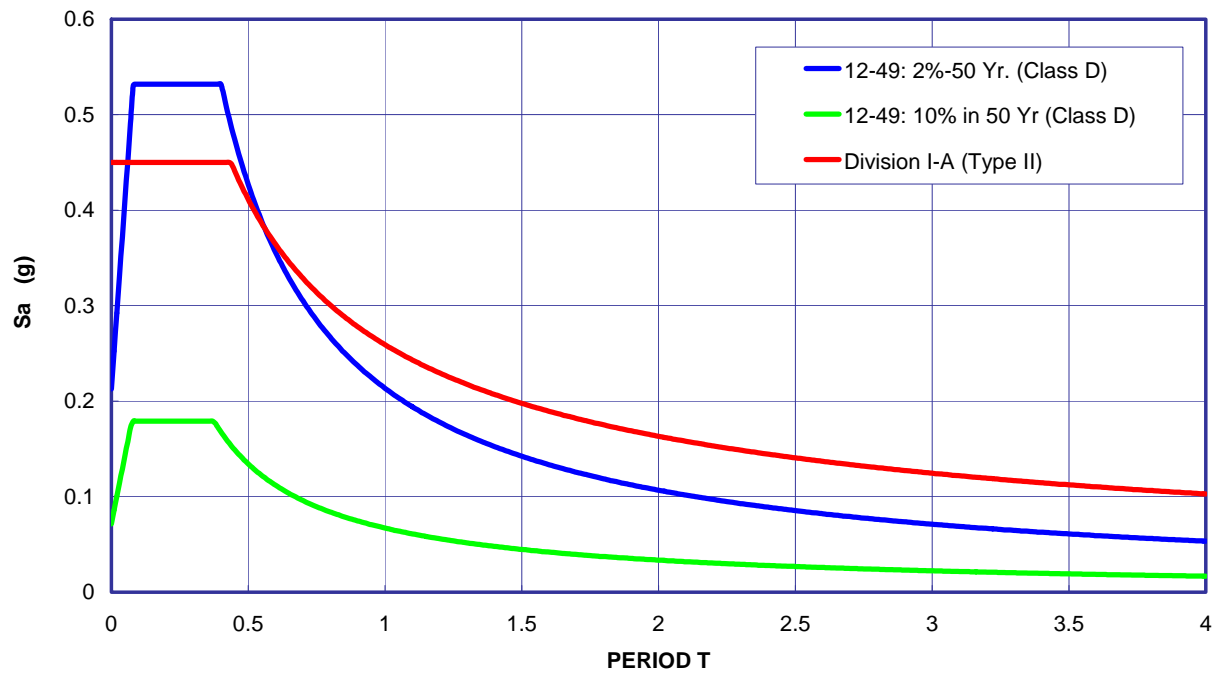
a) Sussex County: Class D and Type II
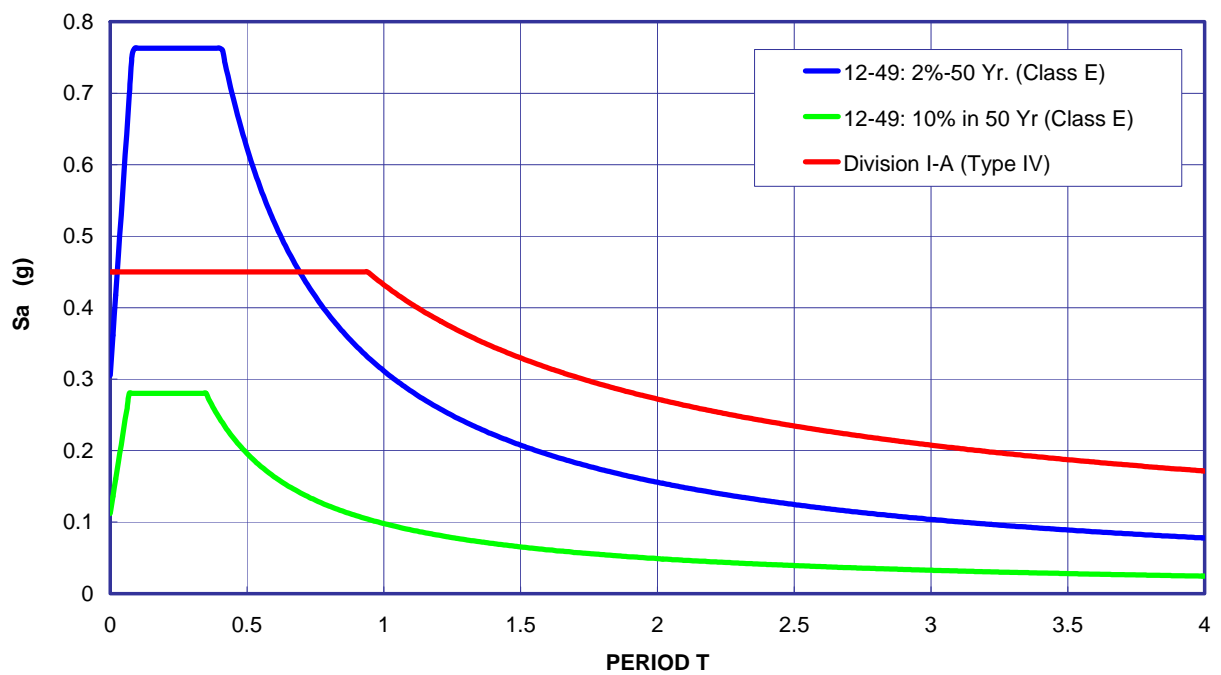
b) Sussex County: Class E and Type IV

Figure 5. Design Spectra based on Existing and Proposed New Seismic Codes: Sussex County
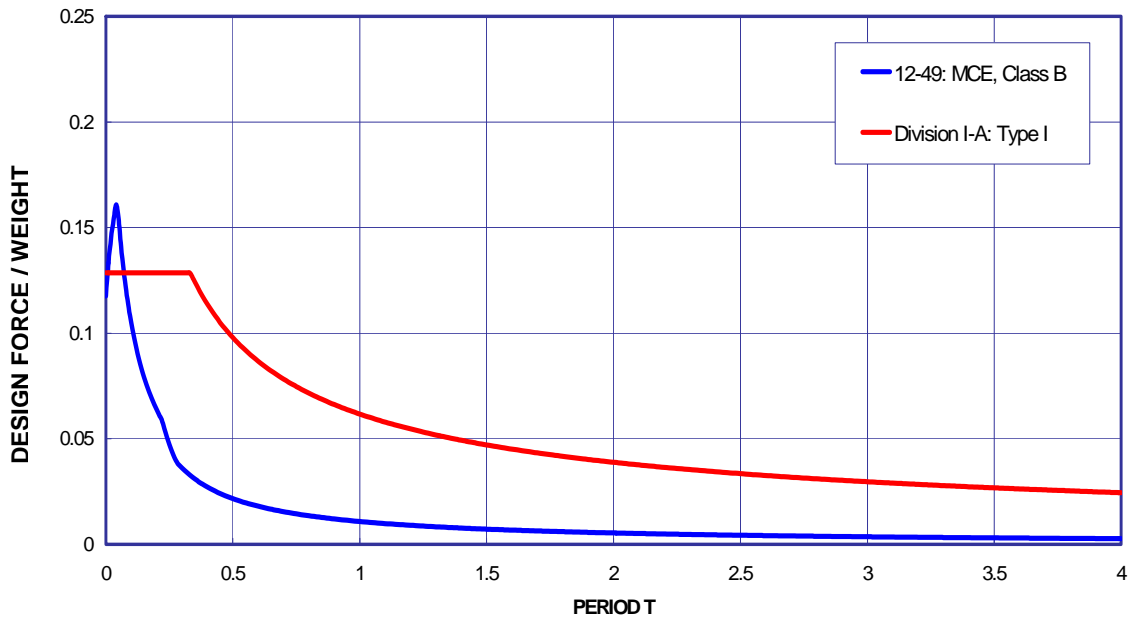
a) Multi-column bent
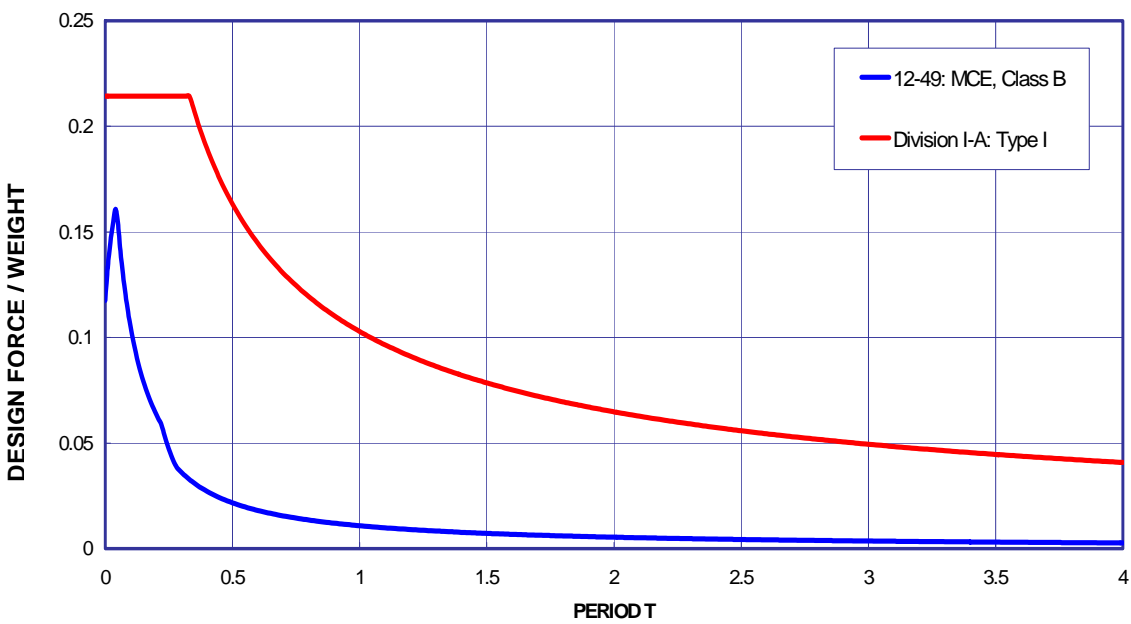
b) Single column bent

Figure 6. Seismic Force Spectra: Rock site, Newark, NJ
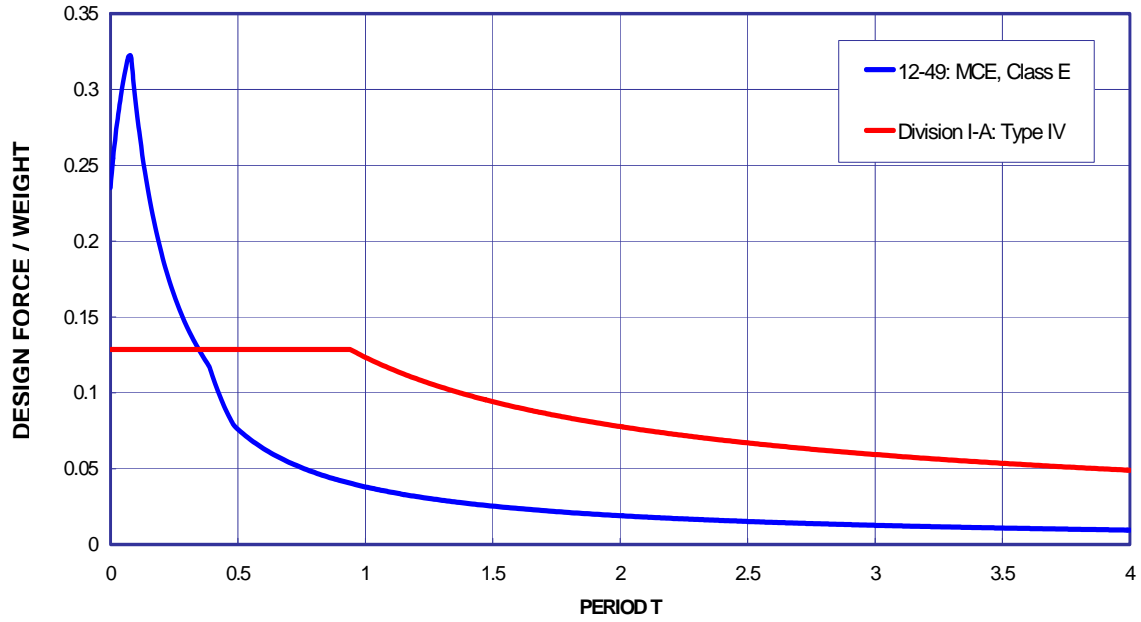
a) Multi-column bent
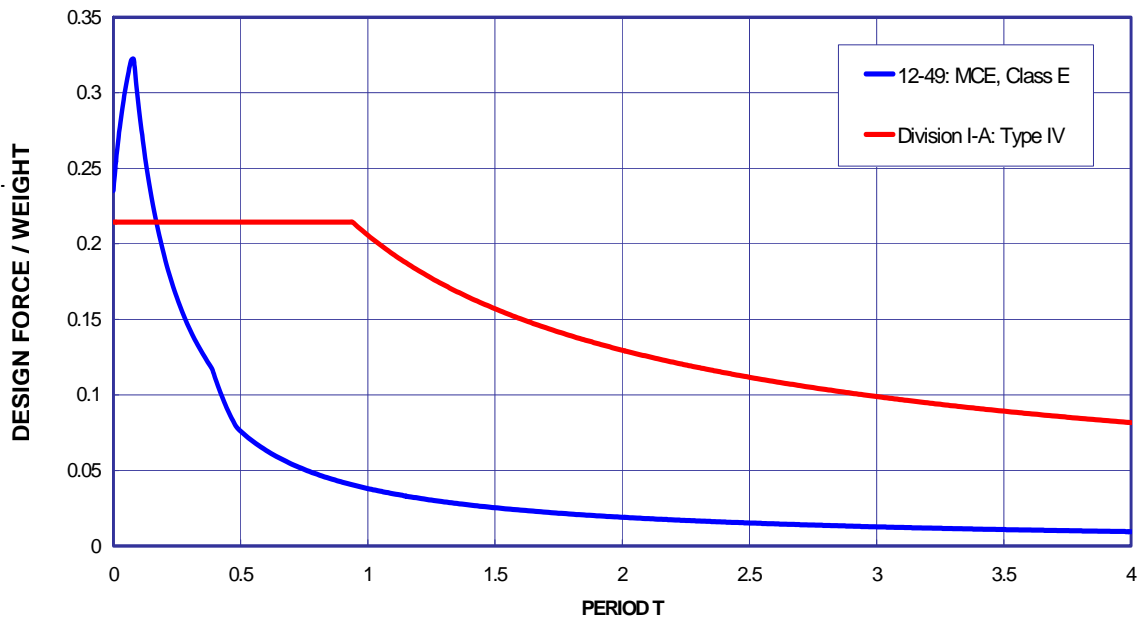
b) Single column bent

Figure 7. Seismic Force Spectra: Soft soil site, Newark, NJ

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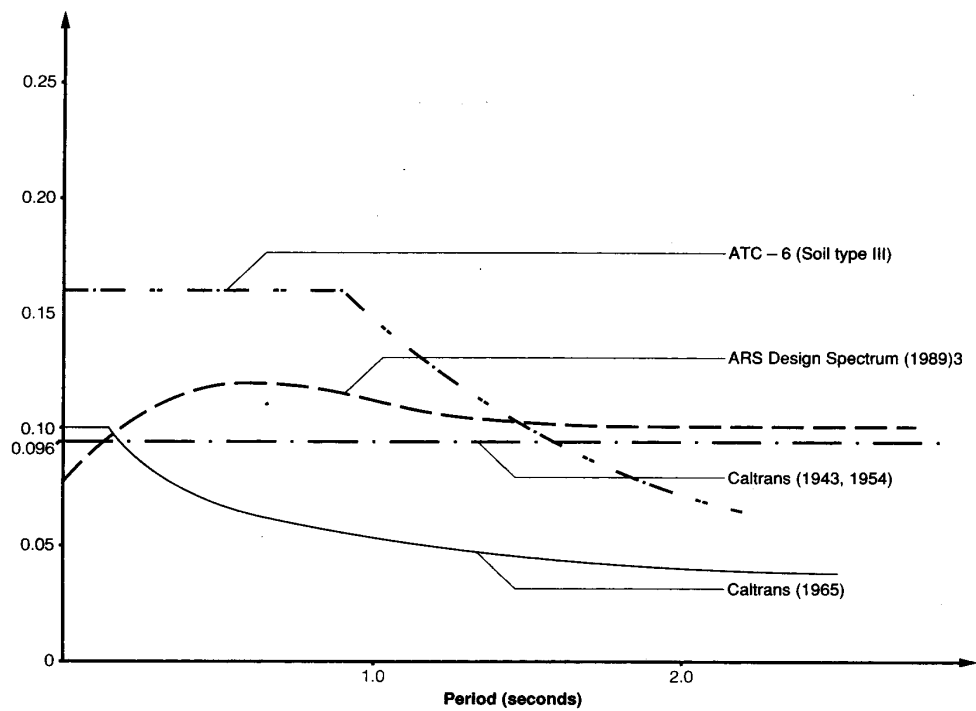
Figure 8. CalTrans strength design spectra (43, 54, 65, 89) and ATC strength design spectrum (81) for multi column bents on soft soil sites [15]
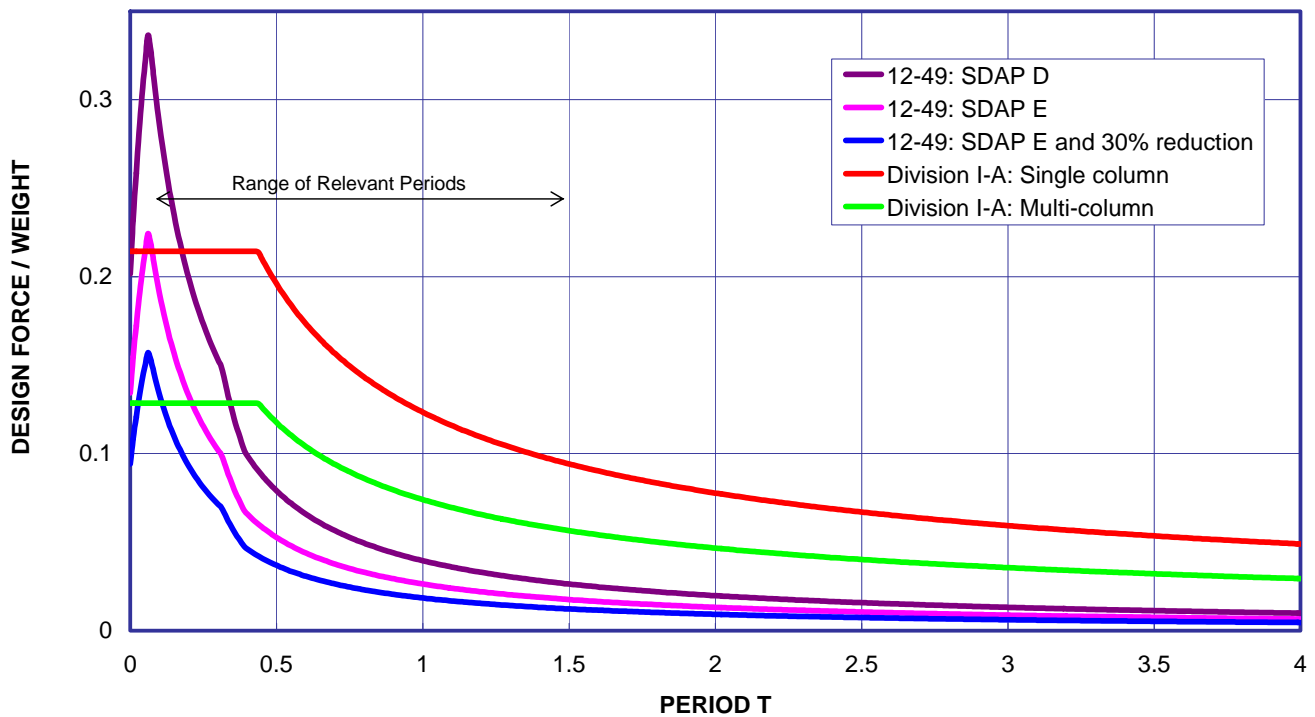
Figure 9. Seismic force spectra: Medium soil (Class C/Type II), Newark, NJ
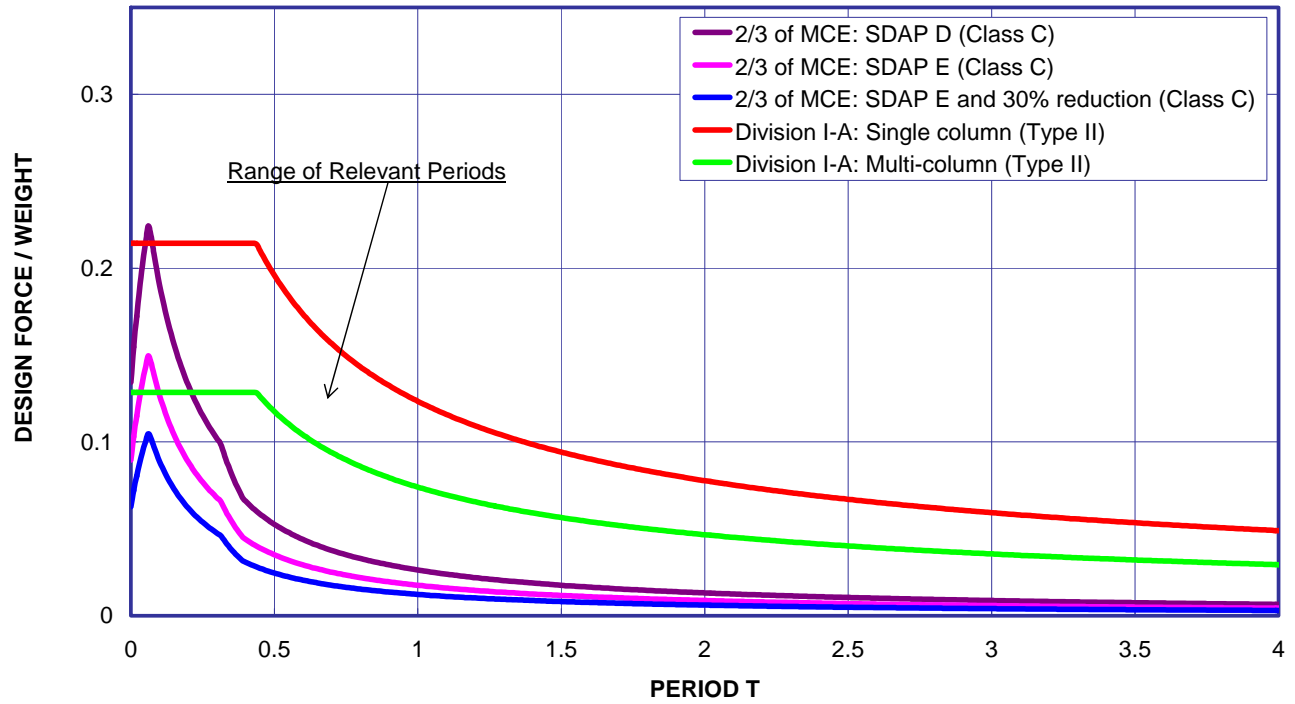
Figure 10. Seismic force spectra: Lower return period vs. Division I-A

In this appendix it is shown how USGS online resources and/or CD-ROM can be used to construct NCHRP 12-49 design response spectrum for a site in Newark NJ (Zip code: 07102).

Step 1: Select "Hazard by Zip Code" from the site **http://eqhazmaps.usgs.gov/** and enter 07102

Figure I.1 Snap shots of USGS online site.

Step 2: Record short-period and long-period spectral values for various return periods, as shown. This is for soil class B (rock).
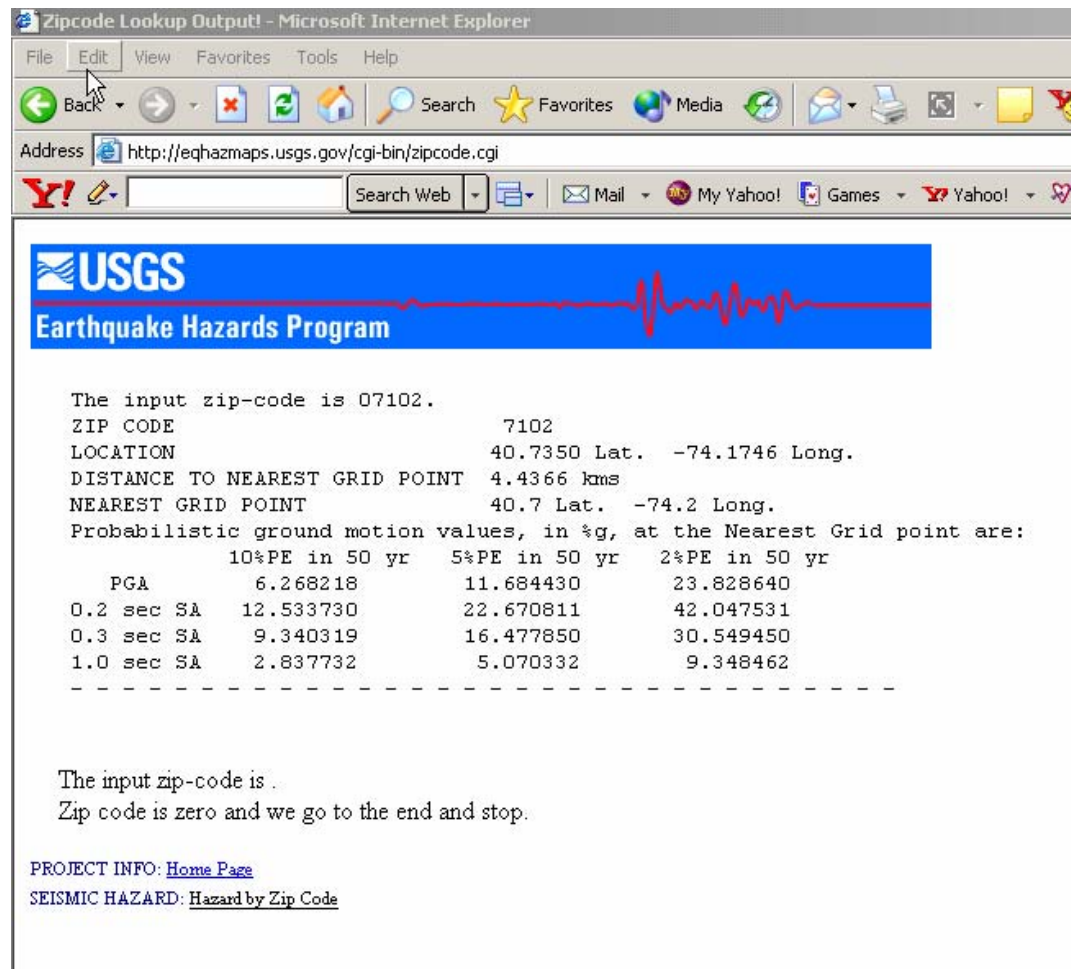
Figure I.2 PGAs and spectral values for rock site and different return periods.

MCE (2%-50 Yr): S_s = 0.42g and S_1 = 0.093

10%-50 Yr: S_s = 0.125 and S_1 = 0.028

ExpE (50%-75 Yr) using interpolation: S_s = 0.0292 and S_1 = 0.0078

Note PGAs and compare to 0.18g for Essex county based on I-A.

Step 3: Determine site coefficients using Tables 1 and 2.

Class E: $F_a = [(0.5-0.42)/(0.5-0.25)](2.5-1.7) + 1.7 = 1.96$ for MCE event
  $F_a = 2.5$ for ExpE and 10% in 50-Yr
  $F_v = 3.5$ for all events

Class C: $F_a = 1.2$ for all events
  $F_v = 1.7$ for all events

Step 4: Determine spectral value for the soil class in question and construct the spectrum using two-point as shown.

For Class E: $S_{DS} = 1.96 * 0.42 = 0.82$ compare to
        Division I-A maximum of $C_S = 2.5 * 0.18 = 0.45g$

        $S_{D1} = 3.5 * 0.093 = 0.325g$ compare to
        Division I-A $C_{T=1} = 1.2 (0.18) 2 / (1)^{2/3} = 0.432g.$

Note: $0.4 * S_{DS} = 0.328g$; $T_s = 0.325 / 0.82 = 0.4$ sec; $T_0 = 0.2 * 0.4 = 0.08$ sec; and For $T > T_s$ $S_a = 0.325g / T$
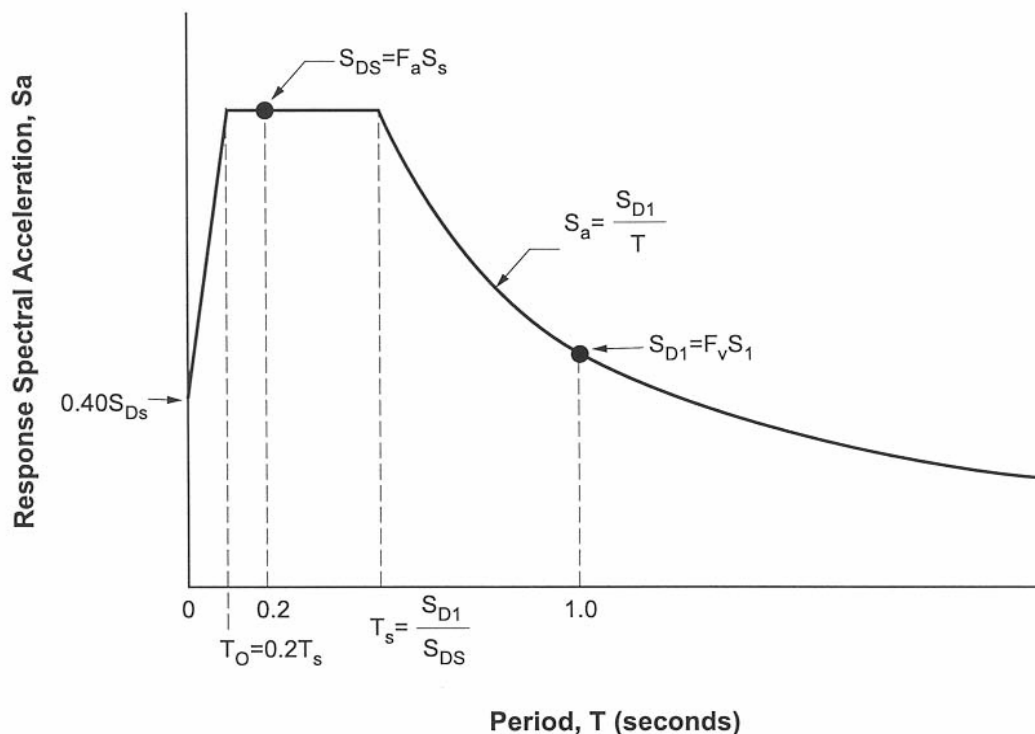
Figure I.3 Design response spectrum, construction using two-point method [1]

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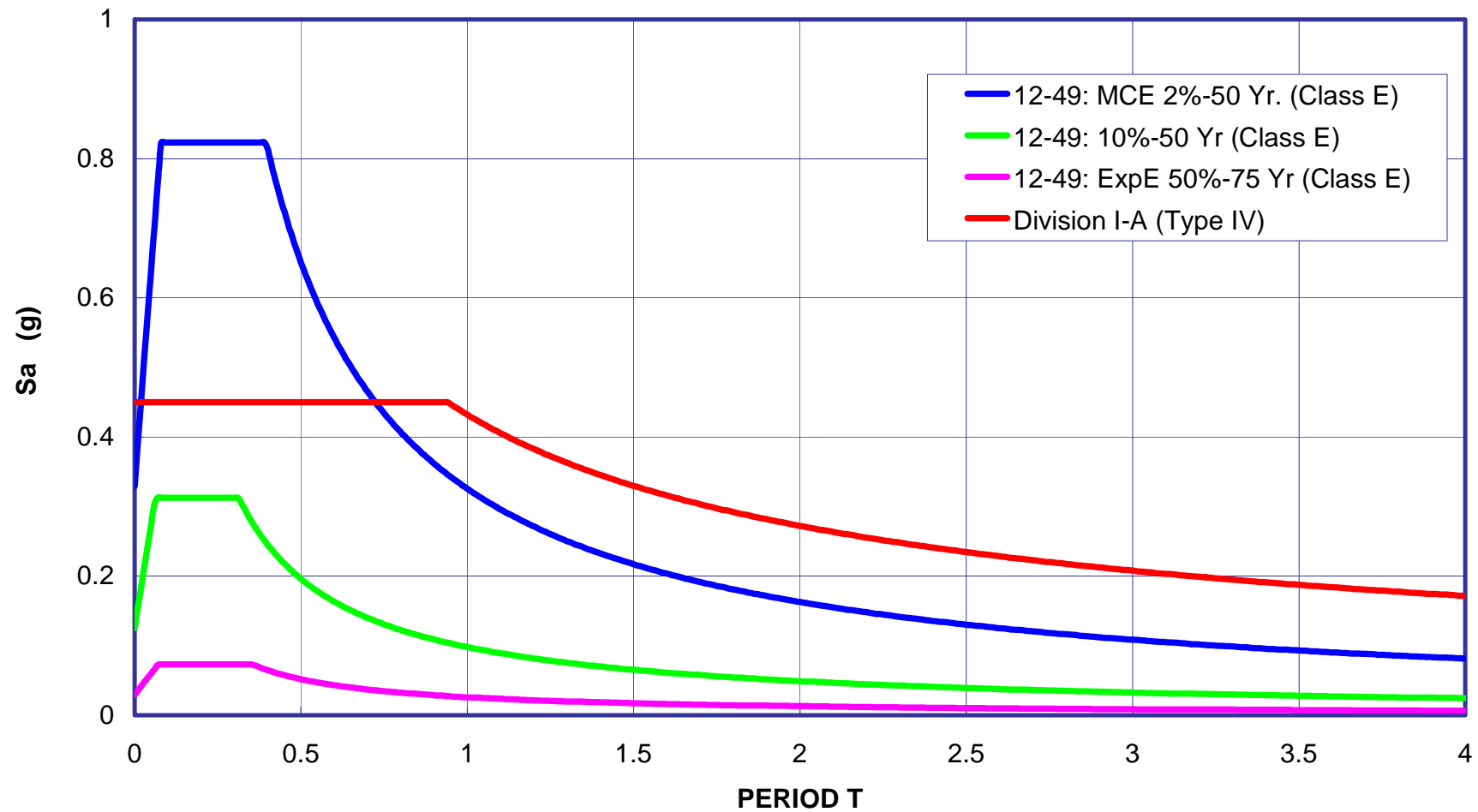
Figure I.4 Design response spectrum based on values discussed and compared to Division I-A

The following pages contain a simplified example demonstrating application of SDAP C. This example was presented during a seminar organized and conducted by the author at NJDOT headquarters in Trenton. About 80 engineers and consultants attended the seminar entitled: **Seismic Performance and Design of Highway Bridges: Past, Present, and Future (and Implications for NJ).**

The bridge used is that of FHWA Design Example 2 [16].

SDAP C Example: Plan & Elevation

Similar to FHWA design example 2[*].

3-Span continuous & skewed.

Steel girder.

Four column bent and wall type.

Seat abutment.

Footing foundation.

PLAN

ELEVATION
(LOOKING PARALLEL TO SKEW)
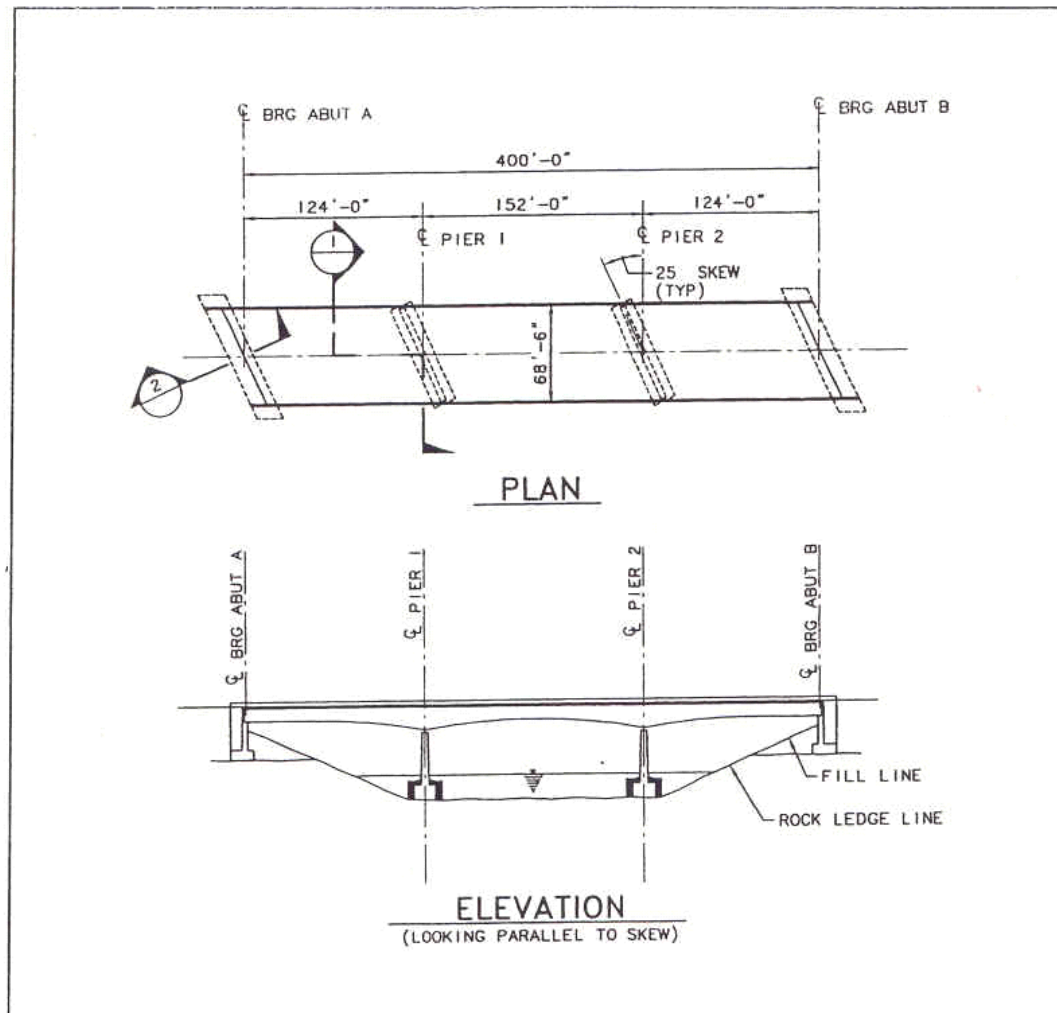
* Mast, et al., Seismic Design of Bridges Design Example No. 2, Publication No. FHWA-SA-97-007

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SDAP C Example: Pier Elevation
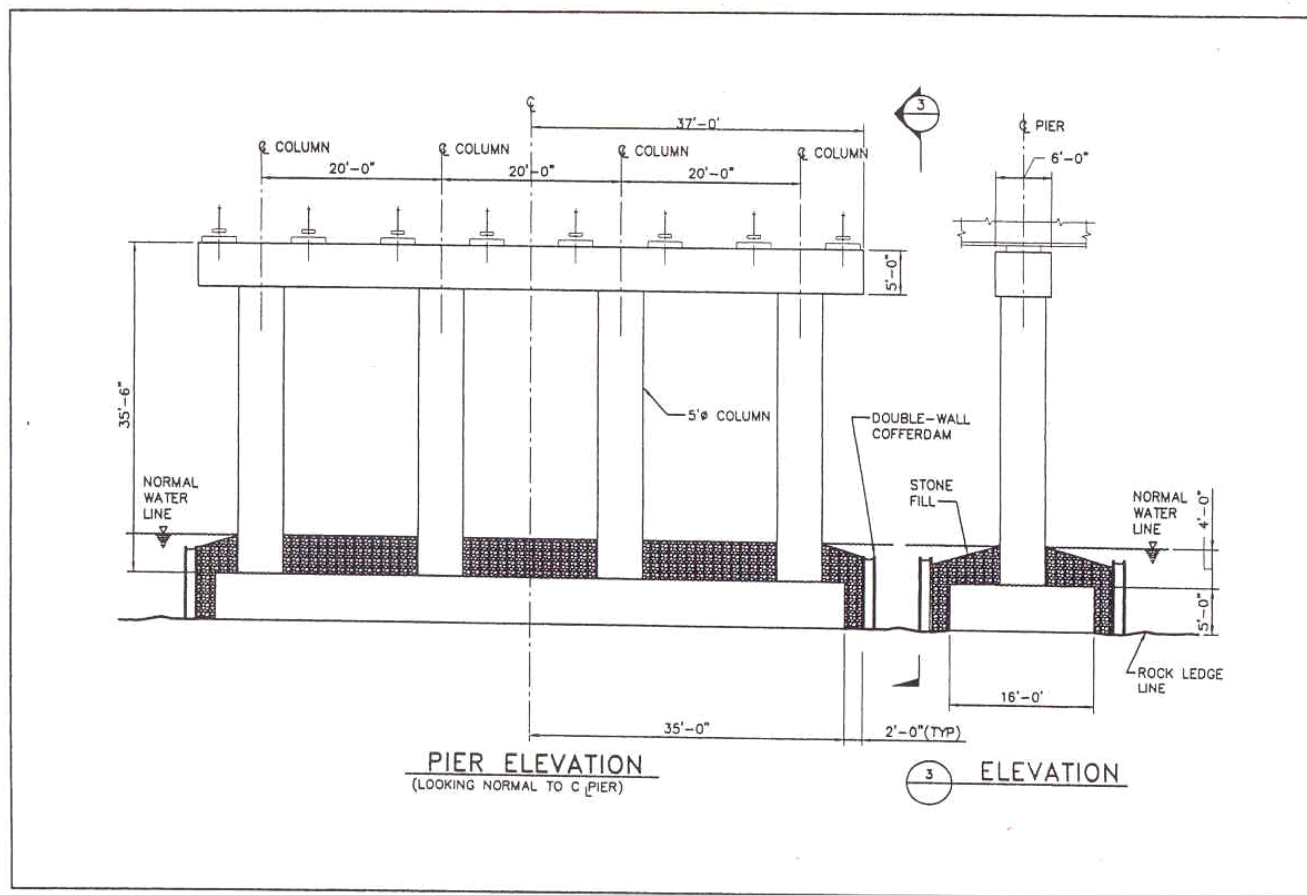
SDAP C: Regularity Requirements

- **Bridge has 3 span, less than the max of 6 and equal to minimum of 3.**

- **Sliding bearing at abutments do not resist much seismic forces in either directions.**

- **Maximum span of 152' < 200' maximum.**

- **Span length ration 1.22 < 1.5 maximum**

- **Skew angle 25 < 30 the maximum and piers are parallel.**

SDAP C: Regularity Requirements

- Bridge is not horizontally curved.
- Bent stiffnesses are equal (ratio of 2 is max).
- Bent strengths are equal (ratio of 1.5 is max).
- There is no potential for liquefaction.
- $P < 0.2\, f'_c\, A_g$
- $\rho > 0.008$
- $D = 300$ mm (12 inches).

$W_{super} := 5540$ **kips** $P := 705$ **kips**   **Axial Load per Column**

$5 - \phi$ $\rho := 1$ **%**

INTERACTION DIAGRAM:

$M_n := 4500$ **ft-k**

$H := 36$ **ft Pier Height**

$E_c := 3650$ **ksi** $I_g := 30.8$ **ft⁴**

$I_{cr} := 0.5 \cdot I_g$ $I_{cr} = 15.4$ **ft⁴**

$\Delta := \dfrac{M_n \cdot H^2}{3\, E_c\, I_{cr}}$ $\Delta_y := 0.24$ **ft**   Based on: $\Delta = \dfrac{PL^3}{3\,EI}$

$1.3 \cdot \Delta_y = 0.312$ **ft**

LATERAL STRENGTH:

$$V := \frac{M_n}{H}$$ $V = 125$ **Kips per Column**

$$V_n := 8 \cdot V$$ $V_n = 1 \cdot 10^3$ **Kips , 2 Bent and 4 Columns per Bent**

$$C_c := \frac{V_n}{W_{super}}$$ $C_c = 0.181$

NEWARK (NJIT) SITE:

ExpE $S_1 := 0.0078$ $F_v := 1.7$ **For Class C**

$F_v := 3.5$ **For Class E**

USE CLASS E

$$C_c \cdot \left(1.3\ \Delta_y\right) = 0.056$$

$$\left(\frac{F_v \cdot S_1}{2\ \pi}\right)^2 = 1.888 \cdot 10^{-5}\ \ \text{ft} \qquad\qquad \textbf{So, Column OK for Frequent Earthquake}$$

MCE EARTHQUAKE:

$$\Delta_{max} := \frac{1}{0.18} \cdot \left(\frac{3.5 \cdot 0.093}{2\ \pi \cdot 1.6}\right)^2 \cdot g \qquad\qquad \Delta_{max} = 0.18\ \text{ft}\ \ \text{required}$$

DEFORMATION CAPACITY:

USE CONSERVATIVE DEFAULT VALUE FOR $\quad \theta_p := 0.035$

$$\Delta_{max} < \theta_p \cdot H \quad = 1.26\ \text{ft} \qquad \text{OK}$$

$$\Delta_{max} < 0.25 \cdot C_c \cdot H \ \ = 1.62\ \text{ft} \qquad \text{OK} \qquad\quad \textbf{P-}\Delta\textbf{, Required Only For MCE}$$

The following pages contain a simplified example demonstrating pushover analysis. This example was presented during a seminar organized and conducted by the author at NJDOT headquarters in Trenton. About 80 engineers and consultants attended the seminar entitled: **Seismic Performance and Design of Highway Bridges: Past, Present, and Future (and Implications for NJ).**

SDAP E Example: Plan and Elevation

5-Span continuous & straight.

CIP Concrete box girder.

Two columns integral bent.

CIP concrete piles with steel casing.
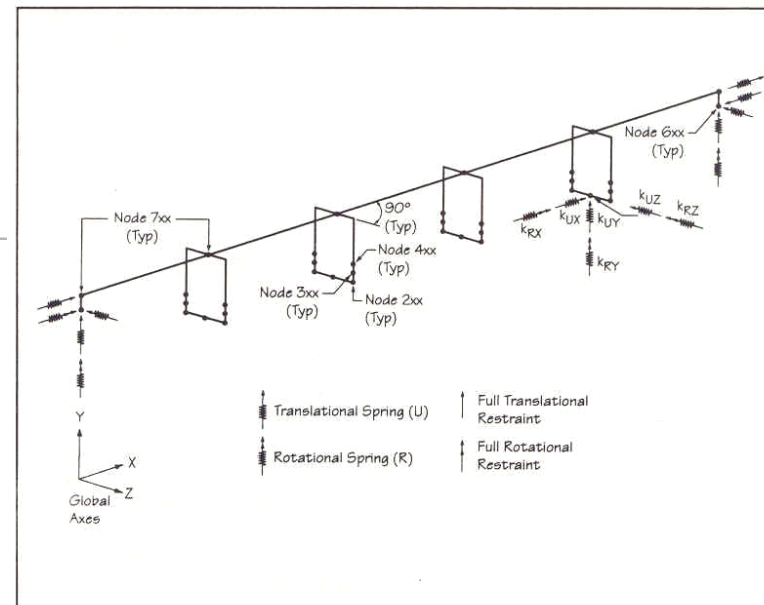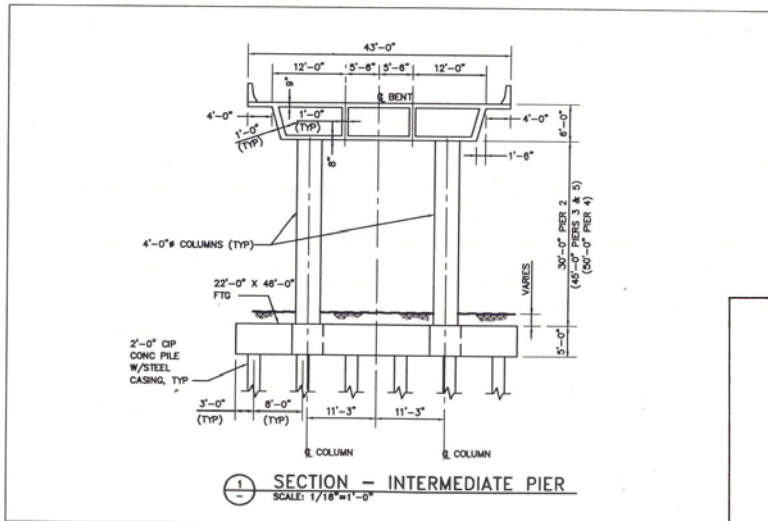
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SDAP E Example (Cont'd)

SDAP E Example (Cont'd)

Weight superstructure
+ Column weight
+ Others

$P_{DL} = 812$ kips

$4' - \Phi$; $\rho = 1.4 \%$

$M_n = 3250\ 'k$

Mechanism
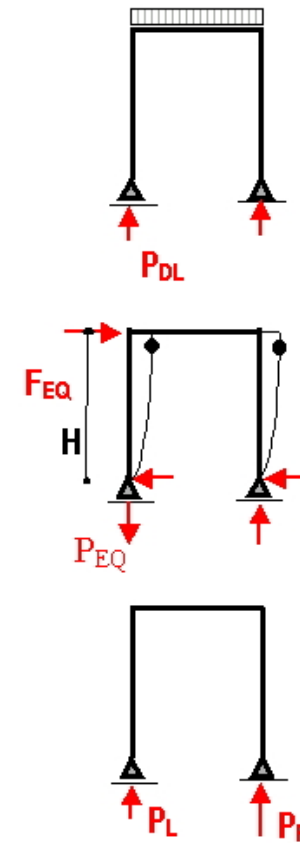Single curvature
Rigid Deck

$F_{EQ} / 2 * \text{Height} = M_n$

$H = 50' \Rightarrow F_{EQ} = 130$ Kips

$P_{EQ} = 130 * 50 / 23$
$\quad = 282$ Kips

Axial Load Due to EQ

$P_L = 812 - 282$
$\quad = 530$ Kips

$P_R = 812 + 282$
$\quad = 1,094$ Kips

Moment-Curvature Plot

SUBJECT: **SDAP E**

DATE: **May 2004**
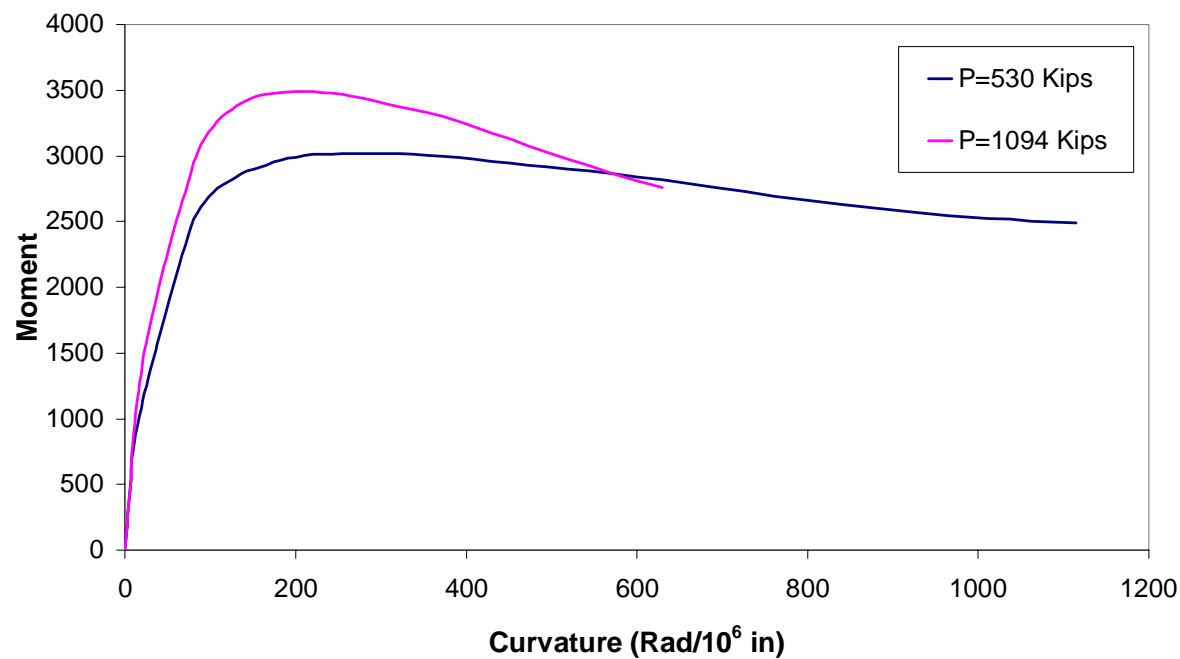
CALCULATED BY: **M. Ala Saadeghvaziri**

FOR: **Workshop**

RESPONSE 2000

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SDAP E Example (Cont'd)

SUBJECT: **SDAP E**   DATE: **May 2004**

CALCULATED BY: **M. Ala Saadeghvaziri**   FOR: **Workshop**

Nominal Moments

$M_L = 3015$ 'K $M_R = 3490$ 'K

$\Phi u = 1,115 * 10^{-6}$ rad / in $\Phi u = 630 * 10^{-6}$ rad / in

$\Phi y = 150 * 10^{-6}$ rad / in $\Phi y = 50 * 10^{-6}$ rad / in

$$(EI)avg. = 360 * 10^6 \quad K - in^2$$

Close to EI based on I_{cr}.

First Plastic Hinge

$F / 2 * H = M_L$
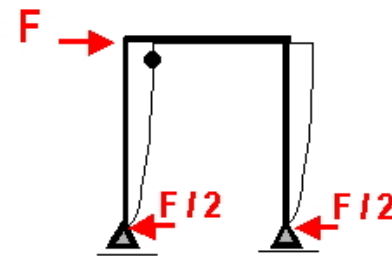
$F = 121$ Kips

Deflection use (EI)avg.

$\Delta_1 = PL^3 / 3\ EI = M_L\ H^2 / 3\ (EI)avg.$

$\Delta_1 = 12$ in.

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SDAP E Example (Cont'd)

SUBJECT: **SDAP E**   DATE: **May 2004**

CALCULATED BY: **M. Ala Saadeghvaziri**   FOR: **Workshop**

Additional Moment carried by Right Column	$\Delta M = M_R - M_L$

$\Delta F = \Delta M / H = (3{,}490 - 3{,}015) / 50 = 9.5$ Kips

$\Delta_2 = 9.5 * (50 * 12)^3 / 3 \, (EI)_R = 2$ in.

Final Force

$F = 130$ Kips.

12-49

Plastic Rotation Capacity

$\theta_p = 0.11 \, Lp / D' \, (N_f)^{-0.5}$ **OR (ATC-12):** $\theta_p = L_p \, (\Phi_u - \Phi_y)$

$N_f = 3.5 \, (T_n)^{-1/3}$

$2 \leq N_f \leq 10$

$Lp = 0.08 \, M / V + 4400 \, \varepsilon_y \, d_b$

$\quad = 0.08 \, H + 4400 * 0.002 \, d_b$

$\quad = 0.08 \, (50 * 12) + 8.8 * d_b$
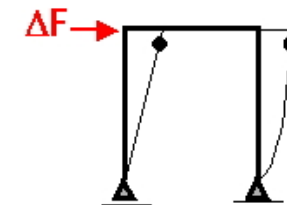
SDAP E Example (Cont'd)

SUBJECT: **SDAP E**   DATE: **May 2004**

CALCULATED BY: **M. Ala Saadeghvaziri**   FOR: **Workshop**

Cont'd   12 – 49

Thus, $\theta_p^L = 0.061$ rad. $\theta_p^R = 0.035$ rad.

When 2nd Plastic Hinge Forms:

$$\theta_L = \Delta_2 / 12 (50) = .0033 \text{ rad.}$$

$$\theta_L^{\text{Remaining}} = 0.0579 - 0.0033$$

$$= 0.054 \; > \; \theta_p^R$$

Continue until θ_p^R is reached.

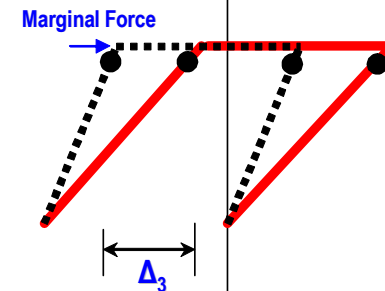$$\Delta_3 = (0.035) (50 * 12) = 21''$$

$$\Delta_{\text{capacity}} = \Sigma \Delta = 12 + 2 + 21 = 35'' \approx 3 \text{ ft}$$

Deflection is high because pinned have been assumed.

CHECK

$$\Delta_{\text{capacity}} > 1.5 \Delta$$

$$\Delta = R_d \Delta_{\text{elastic}}$$
From initial analysis using cracked properties.

Marginal Force

Δ_3

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Load – Deformation