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Seismic Design Considerations
Volume I: Technical Approaches and Results

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16. Abstract

NJDOT has adopted "AASHTO Guide Specifications for LRFD Seismic Bridge Design" approved by the Highway Subcommittee on Bridges and Structures in 2007. The main objective of research presented in this report has been to resolve following issues for an effective implementation of AASHTO Guide Specifications: (i) AASHTO Guide Specifications don't provide any specific guidelines for classification and performance requirements for critical bridges. This issue is resolved by proposing performance requirements and classification criteria for critical bridges in New Jersey. (ii) Guide Specifications present displacement based approach, which is significantly different than the force-based approach in previous versions of seismic guidelines. Nine examples of reinforce concrete and steel bridges of different characteristics (spans, skew, etc.) illustrating the use of newly adopted seismic guide specifications have been developed for training of engineers in New Jersey. (iii) NJDOT maintains an extensive electronic database of soil boring logs for the State of New Jersey. A zip-code based soil site map for New Jersey has been developed by analyzing soil boring data and other available New Jersey soil information. This map can be used for a rapid seismic hazard evaluation for the entire state or for a network of bridges in the state. (iv) AASHTO Guide Specifications introduce seismic design categories based on local seismicity and soil properties. Using the seismic soil map and zip code based seismic spectra provided in the AAHSTO Guide Specifications, seismic design category maps for critical and standard bridges in New Jersey have been developed. A detailed analysis has also been carried out to develop liquefaction potential maps for the state of New Jersey. These maps can be used to determine the need for a detailed liquefaction analysis for a particular bridge site. A detailed guideline on developing site-specific spectra has also been developed, since AASHTO Guide Specifications recommend site-specific spectra for critical bridges. (v) Existing bridges in New Jersey are required to be retrofitted on the basis of 2006 Edition of the "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges". Simplified guidelines for seismic retrofit of existing bridges, that are consistent with guidelines for the design of new bridges in AASHTO Guide Specifications, have been developed.

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CHAPTER 1: INTRODUCTION

Guidelines for the seismic design and retrofit of highway bridge structures in New Jersey are presented in Section 38 of New Jersey Department of Transportation Design Manual for Bridges and Structures, 5th Edition [NJDOT (2010)]. This manual recommends using "AASHTO Guide Specifications for LRFD Seismic Bridge Design" [AASHTO (2008)], (referred to as AASHTO-SGS) for the design of new bridges. FHWA publication titled "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges", dated January, 2006 [FHWA (2006)] has been adopted by the NJDOT for the seismic retrofit of existing bridges. The main objective of this project has been to resolve following issues for an effective implementation of these two guidelines adopted by NJDOT:

- AASHTO-SGS don't provide any specific guidelines for classification and design of critical bridges. A majority of bridges in New Jersey may be critical.
- AASHTO-SGS present displacement based approach, which is significantly different than the force-based approach used before the adoption of the AASHTO-SGS. There are very few examples illustrating the use of AASHTO-SGS.
- AASHTO-SGS propose different seismic design categories (SDC) based on zip code-based spectra and soil site classes. A seismic design category map can be developed if a zip-code based soil site class map can be developed. This map can be used for a preliminary seismic design, a rapid seismic hazard evaluation for the entire state or for a network of bridges in the state. A soil site class map can be developed using NJDOT electronic database of soil boring logs for different sites across the state.
- Liquefaction analysis is generally carried out during different NJDOT projects, although New Jersey is a region of low seismicity. AASHTO-SGS also recommend liquefaction analysis for Seismic Design Category B. Many of the critical bridges in New Jersey are likely to fall into this category. Currently, there is no liquefaction hazard map for the state of New Jersey to determine liquefaction potential at a particular bridge site during the preliminary design phase.
- AASHTO-SGS recommend site-specific spectra for critical bridges. NJDOT doesn't have an established procedure or tools to develop site-specific spectra. Since a majority of New Jersey bridges may be critical, development of site-specific procedure / tools will result in significant cost-savings.
- Existing bridges in New Jersey are retrofitted using the FHWA manual on "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges" [FHWA (2006)]. It has been observed that analysis requirements for seismic retrofit of existing bridges are significantly more complicated than those for new bridges.

Design Requirements for Critical Bridges

State of the Practice in Northeastern United States Region

Since New Jersey doesn't have historical earthquake ground motion data, review of the state of the practice in the Northeastern United States is the most relevant towards developing design requirements for critical bridges in New Jersey. We have investigated the relevance of the state of the practice in New Jersey by comparing different regions of the state on the basis of 1000 Yr (AASHTO-SGS) and 2500 Yr (USGS) return period spectra. These spectra have also been compared with those developed by NYCDOT (2008). Furthermore, comparison of these spectra with the AASHTO (2002) Division 1-A spectra has been done to establish a benchmark of the current practice.

In AASHTO Division 1-A guidelines, acceleration coefficient for horizontal force are prescribed on the county basis (i.e., each county is assigned a peak ground acceleration). If a bridge is located on the border between two counties with different acceleration coefficients, the larger value is used. Vertical component of acceleration is neglected. Figure 1.1 shows the map of New Jersey with regions of three different design peak ground accelerations highlighted in red, blue and green colors.

Following references have been critically examined and reviewed for this research on seismic design considerations for New Jersey:

- NYSDOT Seismic Hazard Practice [NYSDOT (2010)]
- NYCDOT Seismic Hazard Practice [NYCDOT (1998, 2008)]
- NCHRP 12-49 Seismic Hazard Practice [NCHRP (2001)]

Among the references listed above, NYCDOT and NYSDOT Seismic Hazard Practices may have the most significant relevance to the proposed research. Currently, NYSDOT has adopted AASHTO-SGS for the entire state, except for the New York City region. New York City Department of Transportation (NYCDOT) has been using modifications to the AASHTO (2002) Division 1-A based on findings of the "New York City Seismic Hazard and It's Engineering Application", prepared by the Weidlinger Associates in December 1998 [NYCDOT (1998)]. In 2008, Weidlinger Associates developed draft NYCDOT guideline based on the AASHTO-SGS. This document is currently under review by the NYSDOT for adoption.

NYCDOT bridges are classified as Critical, Essential and Other. Essential and other bridges are designed for seismic hazard of 1500 years return period for NEHRP soil classes A through E. A site specific analysis is required for soil class F, irrespective of the bridge importance category. Critical bridges are designed according to site-specific analysis using 500-year & 2500-year return period earthquakes.

Figure 1.2 shows Division 1-A (2002 AASHTO Standard Specifications for Highway Bridges) spectra for Northern New Jersey (A = 0.18g) for soil types III and IV. AASHTO-SGS provide zip code based spectra for 1000 Yr. return period earthquake in New Jersey. Figure 1.2 also shows the 1000 year return period spectra for the zip code in New Jersey that has the maximum spectral quantities and 1500 year return period NYCDOT spectra [NYCDOT (2008)] for soil classes D and E. It is noted that standard bridges (called as "Other Bridges" in NYCDOT guideline) are recommended to be designed by AASHTO for 1000 Yr return period earthquake, whereas these bridges are recommended to be designed for 1500 Yr return period earthquakes in the 2008 NYCDOT seismic guideline.

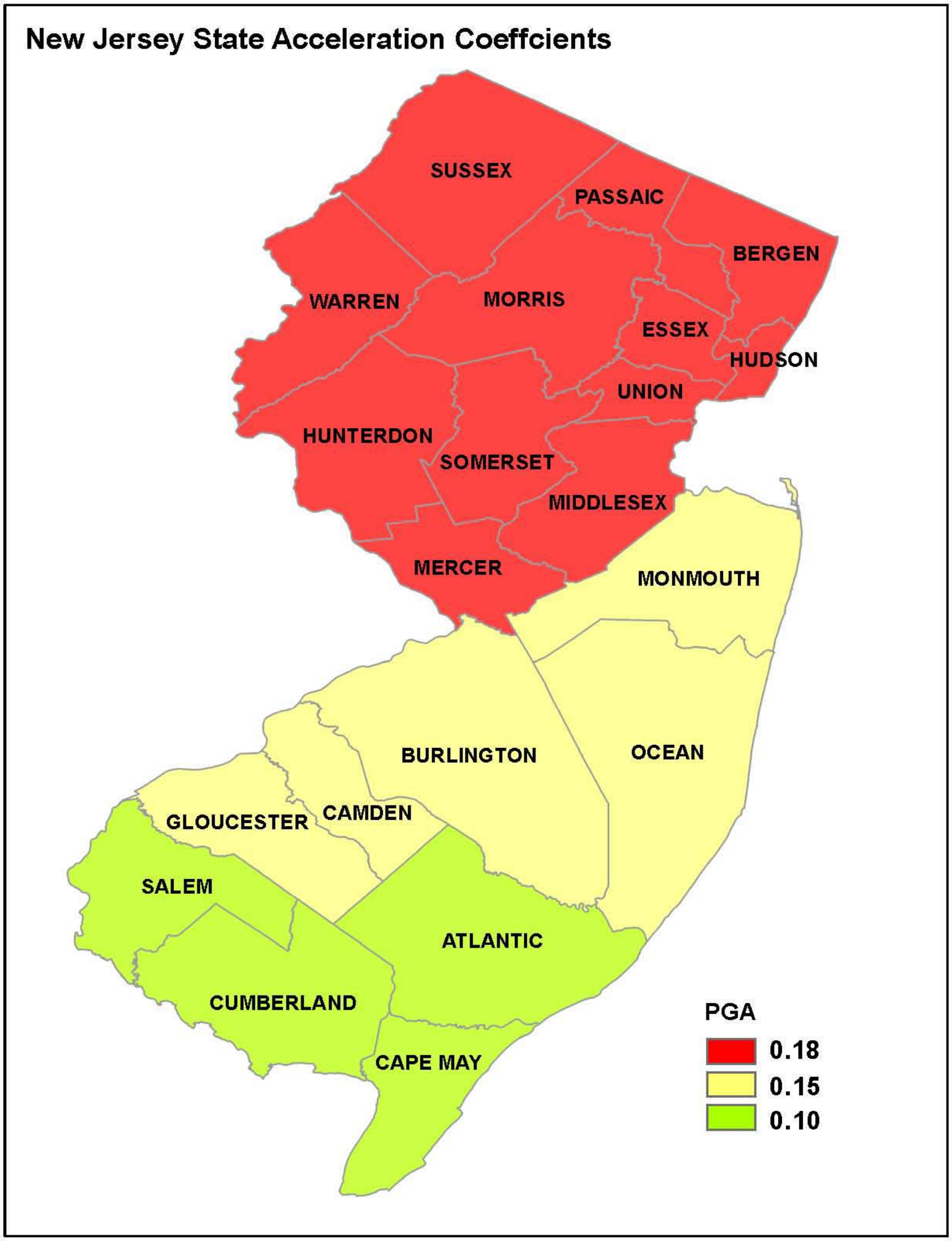


Figure 1.1 Seismic Map of New Jersey Based on 2002 AASHTO Standard Specifications, Division 1A.

It is observed from Figure 1.2 that the short period portion of the NYCDOT spectra is significantly higher than corresponding short period portions in Division 1-A [AASHTO (2002)] and AASHTO-SGS. Since the damage to bridges is associated with low frequency (high period) range, seismic design categories in AASHTO-SGS are based on spectral acceleration at 1-sec period. This acceleration at 1-sec period for 1500 Yr. NYCDOT spectra is smaller than that for the Division 1-A spectra, whereas it is significantly higher than that for 1000 Yr. spectra in AASHTO-SGS.

Figure 1.3 shows Division-1A spectra for Soil Types III and IV for Northern New Jersey along with 2500 yr spectra for New York City and New Jersey (USGS spectra). The 2500 yr USGS spectra for New Jersey is for a zip code for which spectral quantities have the maximum values among spectral quantities in the state. NYCDOT spectra for 2500 Yr. return period are applicable to “Critical” bridges. It is observed from Figure 1.3 that spectral accelerations at 1 second and higher periods are almost identical for soil class E for NYCDOT and New Jersey spectra. For soil class D, spectral accelerations at 1 sec or higher periods for New Jersey are significantly smaller than those for NYCDOT. Overall, all spectral values for 1 second and higher periods are smaller than those for Division 1-A spectra [AASHTO (2002)].

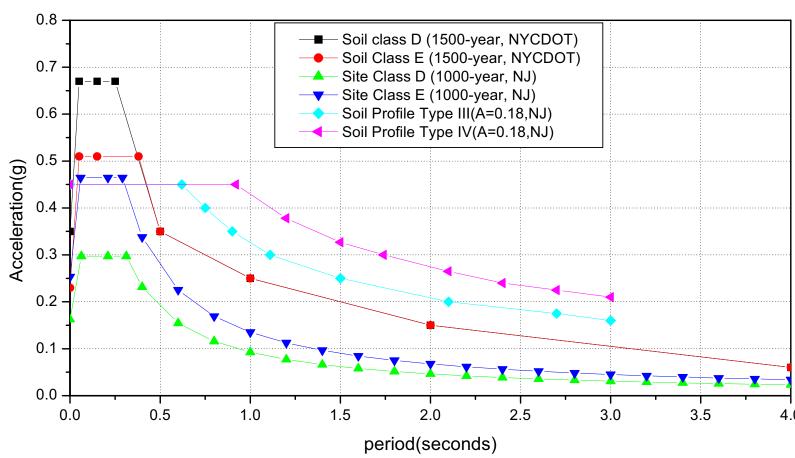


Figure 1.2 Div. 1-A, 1500 Yr. NYCDOT (2008) and 1000 Yr. AASHTO-SGS Spectra.

In order to investigate seismic intensity across the state, peak ground accelerations corresponding to 1000 Yr. return period for different zip codes have been plotted on a zip code map. Then, zip code areas have been combined to obtain approximate grouping of regions similar to regions of 0.18g, 0.15g and 0.10g in Figure 1.1. Figure 1.4 shows the NJDOT seismic map with three regions: Red region is similar to 0.18g region of Div-1A spectra in Figure 1.1, Yellow region is similar to 0.15g region of Div-1A in Figure 1.1 (although some counties from 0.10g regions are included in the Yellow region) and Green region is similar to 0.10g region in Div-1A spectra in Figure 1.1. For the three regions in Figure 1.4, single spectra (instead of zip code based spectra) corresponding to largest value of S_s in these regions is assigned for the entire region, as shown in a table in the lower right hand corner of the Figure 1.4. It is observed that the PGAs in Fig. 1.4 are significantly smaller than those for the Div-1A spectra in Fig. 1.1. Hence, AASHTO Guide-SGS are recommending significantly lower level of earthquake loading as compared to AASHTO (2002) Div-1A loading used in the past. This, in fact, has been achieved by improving capacities of bridge components through prescribed detailing (through different seismic design categories), as described later in this chapter.

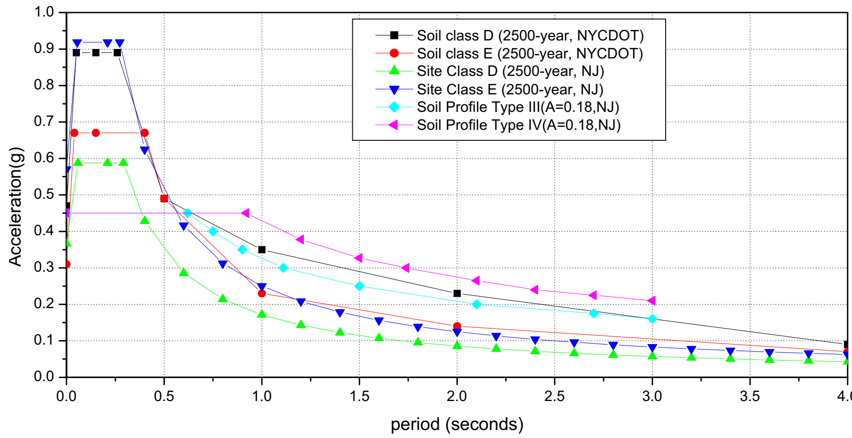
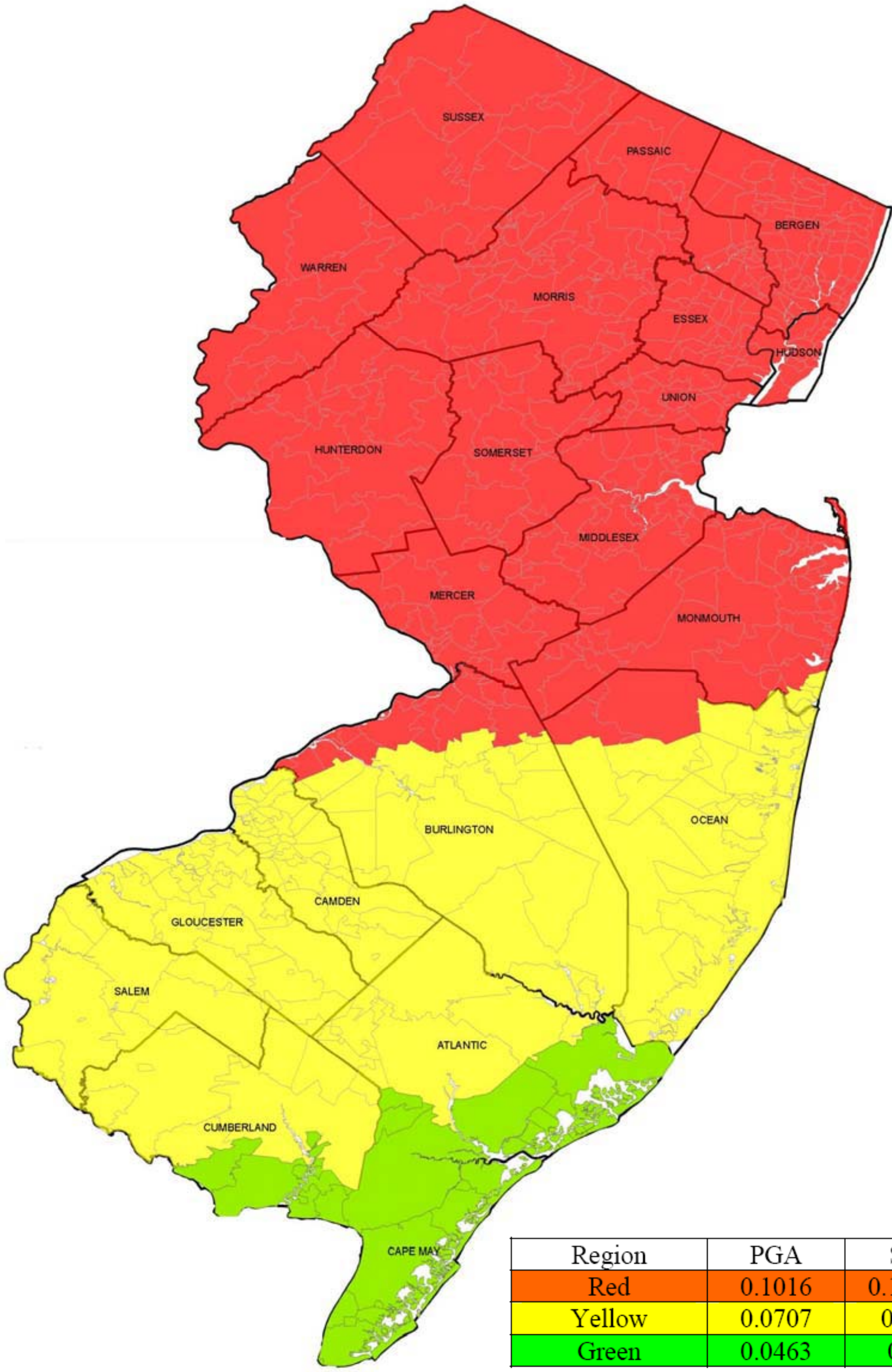


Figure 1.3 Comparison Between AASHTO Div.1-A , 2500 Yr. NYCDOT (2008) and 2500 Yr. USGS Spectra for New Jersey.

Seismic spectra for New Jersey, as recommended by the AASHTO-SGS, has also been investigated on the basis of seismic design categories. Tables 1.1 to 1.5 show seismic design categories when S_{D1} and S_{DS} are calculated from spectra corresponding to AASHTO Division 1A for New Jersey, NYCDOT (1998), NYCDOT (2008) and AASHTO-SGS. Spectra for 1000 yr return period in the AASHTO-SGS are for the zip code with highest values of spectral quantities among all zip codes in the state. Soil types for SDC's in Tables 1.1 to 1.5 have been assumed to be D and E. It is observed from Tables 1.1 to 1.5 that:

- Based on Division 1A , 2500 Yr NYCDOT (1998), or 2500 Yr NYCDOT (2008) spectra, bridges will be designed as per SDC B or C, depending on the bridge site zip code.
- Using 2008 NYCDOT spectra with 1500 Yr. return period earthquake will require the design of bridges by SDC B for Rock B and deep rock sites with the soil types D and E, and by the SDC A for the Rock A site.
- Using the 1000 Yr. return period spectra will result in the design of standard bridges in the entire state by SDC A.
- Using the USGS spectra with 2500 Yr. return period will require the design of some bridges in the Northern New Jersey by SDC B, while a majority of bridge will still be designed by SDC A.



Region	PGA	SS	SD1
Red	0.1016	0.1887	0.0386
Yellow	0.0707	0.14	0.0311
Green	0.0463	0.1	0.0288

Figure 1.4 Seismic Map of New Jersey Using 1000 Yr. Return Period AASHTO-SGS Spectra.

The map has been created by grouping counties in New Jersey to obtain a map similar to that shown in Fig. 1.1.

Table 1.1 SDC Classification Based on Division 1-A Spectra for Soil Classes D and E.

Soil / A(g)	Type D		Type E	
	S_{D1}*	SDC	S_{D1}	SDC
0.1	0.18	B	0.24	B
0.15	0.27	B	0.36	C
0.18	0.32	B-C	0.43	C

* Spectral value at 1 second.

Table 1.2 SDC Classification Based on NYCDOT (1998) Spectra for Soil Classes D and E.

Soil / NYCDOT	Type D		Type E	
	S_{D1}	SDC	S_{D1}	SDC
2500 Yr.	0.3	B-C	0.44	C
2/3rd of 2500 Yr.	0.2	B	0.3	B-C

Table 1.3 SDC Classification Based on 1500 Yr NYCDOT (2008) Spectra for Soil Classes D and E.

Soil	Type D		Type E	
	S_{D1}	SDC	S_{D1}	SDC
Bedrock A	0.13	A	0.13	A
Bedrock B	0.19	B	0.19	B
Deep bedrock	0.24	B	0.24	B

Table 1.4 SDC Classification Based on 2500 Yr NYCDOT (2008) Spectra for Soil Classes D and E.

Soil	Type D		Type E	
	S_{D1}	SDC	S_{D1}	SDC
Bedrock A	0.18	B	0.18	B
Bedrock B	0.27	B	0.27	B
Deep bedrock	0.34	C	0.34	C

Table 1.5 SDC Classification Based on 1000 Yr Spectra in ASSHTO-SGS and 2500 Yr USGS Spectra for Soil Classes D and E.

Soil / Hazard	Type D		Type E	
	S_{D1}	SDC	S_{D1}	SDC
1000 Yr	0.093	A	0.14	A
2500 Yr.	0.17	B	0.25	B

Following the completion of NCHRP 12-49 project, Mr. Harry Capers, the state bridge engineer of NJDOT during that time, led a comparative study of the provisions of 12-49 with those of Division 1-A spectra [AASHTO (2002)] to establish the applicability of NCHRP provisions to NJDOT practice [NJDOT (2005), Capers (2003)]. As a part of this study, he selected the "Doremus Avenue Bridge" and the "Scotch Road Bridge over Interstate 295" for the comparative study. NJDOT also sponsored a research project to investigate applicability of provisions of NCHRP 12-49 to NJ practice [NJDOT (2005), Capers (2003)]. These studies led to the following recommendations regarding the impact of NCHRP12-49 on the NJDOT practice:

(i) Even though the AASHTO Subcommittee on Bridges and Structures did not approve the adoption of the outcome of NCHRP 12-49 as the Seismic Guide Specifications , 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 AASHTO (2002) LRFD Specifications Division 1-A.

(ii) The 2500-year return period for the Most Credible Earthquake (MCE) is very conservative compared to other extreme events such as vessel impact and floods. A return period of 1500 years was being considered; however, USGS maps for 1500 years return period were not available. Hence, acceleration equal to 2/3 of that of the 2500-year event was recommended to be used.

(iii) The 1000-year event for which USGS seismic maps were available seemed to have lower accelerations than AASHTO LRFD specifications.

(iv) Number of bridges in New Jersey that can be classified as standard should be maximized for budgeting and economic reasons.

Important Observations for Critical Bridges New Jersey

From the review of past practice in New Jersey and current practice in the region surrounding New Jersey, following observations can be made:

(i) Multiplying 0.10 PGA in the Red region in Figure 1.4 by a factor of 1.8 will give 0.18g PGA, which is the same as 0.18g PGA used in Division 1-A spectra in Fig 1.1. This seems to imply that the AASHTO Seismic Guide Specifications is downgrading the seismic load by a significant factor. In realty, this downgrading in loading is compensated by increased capacity by a better detailing requirement for new bridges through prescribed SDCs [NCHRP (2006)].

(ii) Spectra with 1000 Yr. return period, as prescribed by the AASHTO Seismic Guide Specifications, is the minimum prescribed and is applicable to standard bridges for “collapse prevention” performance. For critical or more important bridges, 1000 Yr. spectra should be multiplied by a factor > 1 to ensure that critical bridges suffer minimal damage during an earthquake with 1000 Yr. return period or don’t collapse during stronger earthquakes (such as 2500 Yr earthquake).

(iii) Previous studies, including NJDOT (2005), Capers (2003) and NCHRP 12-49 [NCHRP(2001)] have established that a seismic design using the 2500 Yr. return period earthquake is too conservative for New Jersey.

(iv) Based on a similar rational, New York City Department of Transportation sponsored a study to revise seismic guidelines for New York City. This study, based on extensive study of rock motion and soil boring data, developed spectra for an earthquake with 1500 Yr. return period for standard bridges. For critical bridges, spectra for an earthquake with 2500 Yr. return period has been developed. The rational for New York City of using 2500 Yr. return period earthquake for critical bridges is because of high values of bridge inventories and their critical role in the global economy.

(v) Previous studies have also pointed out the appropriateness of using spectra for an earthquake with 1500 Yr. return period for New Jersey. Unfortunately, 1500 Yr. return period spectra aren’t available.

(vi) This deficiency can be resolved either by applying a factor to available zip code based spectra for 1000 Yr. earthquake or by developing 1500 Yr. spectra for different soil types (or zip codes) in New Jersey using Random Vibration Theory approach (RVT). The second option, although feasible, will require significantly large financial resources and may not result in substantial improvement in understanding of seismic risk in New Jersey because of lack of historical data. Hence, applying an appropriate factor to available zip code based spectra may be more appropriate.

(vii) The approach adopted in this project is to apply a factor > 1 to available zip code based spectra for 1000 Yr. earthquake for generating spectra for critical bridges. The selection of an appropriate factor is explained in the next section.

Design Spectra for Critical Bridges

Following the rejection of NCHRP 12-49 by the AASHTO because of extremely conservative design, Task 193 under NCHRP 20-07 was initiated to explore the development of acceptable seismic guideline. The final recommendations of this task formed the basis of the AASHTO Seismic Guide Specifications. As per NCHRP 20-07/Task 193 [NCHRP (2006)], “Selection of a lower return period for Design is made such that Collapse Prevention is not compromised when considering historical large earthquakes. This reduction can be achieved by taking advantage of sources of conservatism not explicitly taken into account in current design procedures. These sources of conservatism are becoming obvious based on recent findings from both observations of earthquake damage and experimental data." *Reduction here implies with respect to 2500 Yr return period used in NCHRP 12-49.*

Table 1.6 shows some of sources of conservatism that are not accounted for during the design and construction, but they contribute to increased resistance of bridge components during an earthquake. Considering this conservatism in the design and construction, seismic risk was decreased from 2500 Yr. return period earthquake to 1000 Yr. return period earthquake for collapse prevention performance. Overall, the AASHTO Seismic Guide Specifications contains a safety factor of 1.5 based on conservatism reported in Table 1.6 with the understanding that hinging mechanism will contribute to energy dissipation before collapse during earthquakes equal to or greater than 1000 Yr. return periods. For critical bridges where design requires "minimal damage" performance, this energy dissipation due to hinging mechanism isn't available since the expected behavior is essentially elastic. Hence, critical bridge components need to be designed by considering 1000 Yr. spectra multiplied by a factor of 1.5 to achieve "minimal damage" performance. Selection of 1000 Yr. return period earthquake in combination with different SDCs is assumed to ensure collapse prevention in case of 2500 Yr. earthquake. Critical bridges need to be designed for "repairable damage" performance during such earthquakes. Usage of 1000 Yr. return period earthquake spectra multiplied by a factor of 1.5 for "essentially elastic" performance will ensure reparable damage performance during a 2500 Yr. earthquake.

Table 1.6 Identified Sources of Conservatism in NCHRP 20-07/Task 193

Source of Conservation	**Safety Factor**
Computational vs. Experimental Displacement Capacity of Components	1.3
Effective Damping	1.2 to 1.5
Dynamic Effect (i.e., strain rate effect)	1.2
Pushover Techniques Governed by First Plastic Hinge to Reach Ultimate Capacity	1.2 to 1.5
Out of Phase Displacement at Hinge Seat	Addressed in Task 3

It should be noted that the 1000 Yr. spectra multiplied by a factor of 1.5 is not the same as 1500 Yr. return period spectra. Ideally, 1500 Yr. return-period spectra for different soil types in New Jersey should be developed by carrying out detailed modeling of rock motion in New Jersey and then using this rock motion in the random vibration theory (RVT) to develop ground motion spectra [Risk Engineering Inc. (2002)].

However, this will be a very complex and expensive understanding without any guarantee of better seismic performance, since no historical data on strong earthquakes in New Jersey exist.

The sufficiency and economical impact of the 1.5 factor can be understood by considering a comparative analysis. Among all zip codes in New Jersey, maximum value of 1-sec period spectral acceleration (S_1) for 1000 Yr. return period earthquake occurs in zip code 07003. For this zip-code:

S_1 = 0.0381 (for 1000 Yr. Return Period for bedrock)

For Soil Type D, S_{D1}= 0.0381*2.4*1.5 = 0.137 (SDC A)

For Soil Type E, S_{D1} = 0.0381*3.5*1.5 = 0.20 (SDC B)

Hence, only bridges on soil type E in Northern NJ are likely to be designed by SDC B. All other bridges are likely to be designed by SDC A. Values of S_{D1} for soil type E for earthquakes of different return periods are calculated as:

- 1000 Yr. Return Period (NJ): = 0.133
- 1.5 times 1000 Yr. Return Period (NJ): = 0.20
- 2500 Yr. Return Period (USGS Spectra for NJ) = 0.25
- 2/3rd of 2500 Yr. Return Period (NJ) = 0.17
- 1500 Yr. Return Period (NYC) = 0.13-0.24 (depending on Rock type in Table 1.3)

It is noted from above analysis that the spectral quantity S_{D1} for soil type E in New Jersey for “1.5 times 1000 Yr. Return Period” spectra is significantly below that for 2500 Yr. return period for New Jersey and is comparable (although slightly higher) to that for 2/3rd of 2500 Yr. return period (USGS) and 1500 Yr. return period (NYC). Figure 1.5 shows spectra for 1000 Yr (AASHTO-SGS), 2500 Yr (USGS Spectra for NJ) and 1000 Yr. (AASHTO-SGS) Spectra multiplied by a factor of 1.5 for soil sites C, D and E. It is observed that the spectra for “1000 Yr. multiplied by a factor of 1.5” lies almost in the middle of 1000 Yr. AASHTO-SGS and 2500 Yr. (USGS Spectra for NJ) spectra.

The application of 1.5 factor to the 1000 Yr. return period earthquake spectra recognizes the uncertainties in the hazard data . In addition to seismic loads, this factor will also improve the safety of bridge components during other hazards, e.g., blast, vehicular impact. Recent research has clearly shown that a better seismic capacity directly implies improved performance during other types of hazards, such as blast and vehicular impacts [Yi (2008), Agrawal et al. (2010)].

Based on the discussion above, following recommendation is proposed for the design of new bridges in New Jersey:

All critical bridges in New Jersey should be designed for minimal damage performance level for 1000 Yr AASHTO-SGS spectra multiplied by a factor of 1.5. In case a site specific analysis is required, rock spectra (spectra for Site B) for 1000 Yr AASHTO-SGS should be multiplied by a factor of 1.5 before carrying out the site specific analysis.

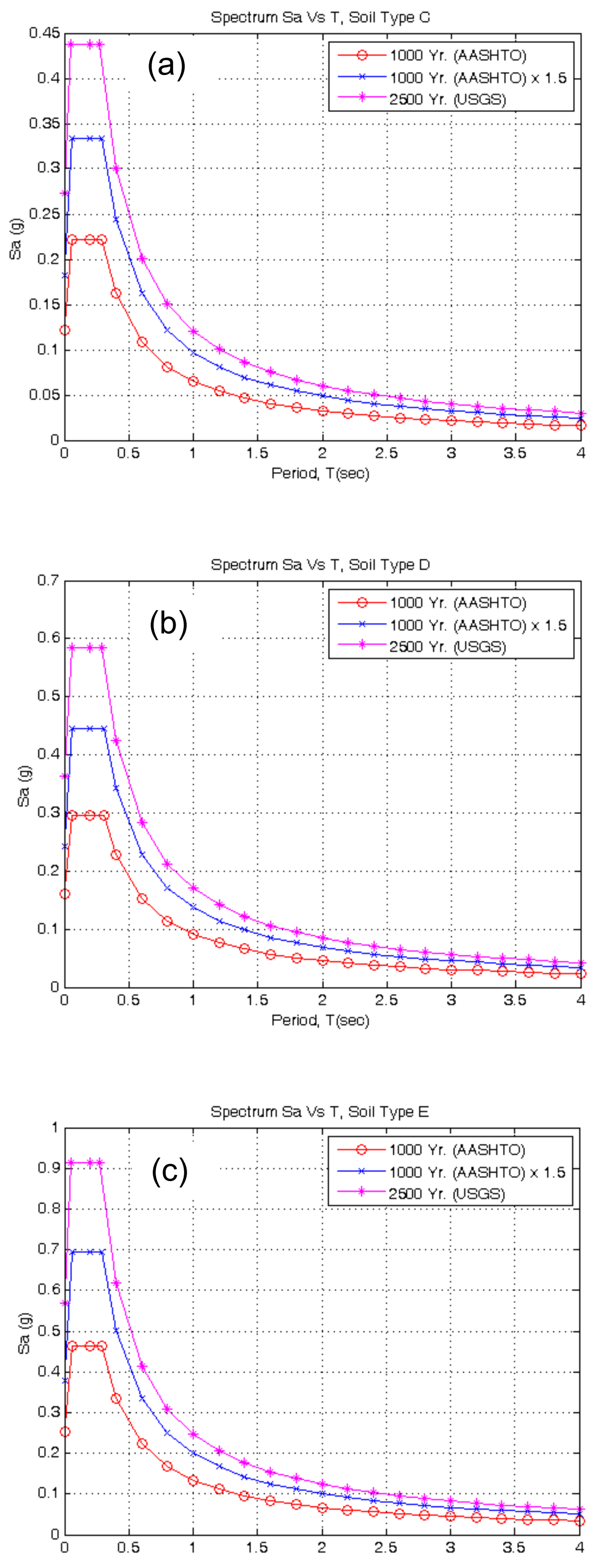


Figure 1.5 Comparison Between 1000 Yr (AASHTO-SGS), 1000 Yr. (AASHTO-SGS) ×1.5 and 2500 Yr. (USGS for NJ) Spectra for Soil Sites C, D and E.

Seismic Design Category Maps

New Jersey Department of Transportation maintains an extensive online database of soil boring data called “Geotechnical Database Management System (GDMS)”. These soil boring data, combined with other sources of information on soil types in New Jersey, can be used to develop soil site class map for New Jersey as per provisions of AASHTO-SGS. These seismic maps can be used to develop GIS based Seismic Design Category (SDC) maps for the State of New Jersey. Development of these maps has numerous advantages, e.g., zip code based preliminary seismic design of a planned new bridge, visual seismic risk assessment across the state, seismic risk assessment for a particular network of bridges, etc. A detailed description of procedures to develop soil site class map for the State of New Jersey and resulting maps are presented in Chapter 2 of this report. Seismic design category maps are presented in Chapter 4 of this report.

Liquefaction Analysis

AASHTO-SGS requires liquefaction analysis for SDC C and recommend it for SDC B. It has been observed that most of the critical bridges in New Jersey on soil classes D and E may be designed as per SDC B. However, not all soils classified as D or E may be liquefiable. Since a soil site class map has been developed based on available boring data (as described in Chapter 2), a liquefaction map can be developed by the first order liquefaction analysis. It has been observed from these maps that a major portion of New Jersey soils isn’t liquefiable. Hence, liquefaction maps can be used to avoid repeated liquefaction analysis during NJDOT projects. This itself may result in significant savings for NJDOT.

A conservative approach has been used in analyzing liquefaction potential, considering the uncertainty in the soil property regarding fine contents. Specifically, silts has been regarded as liquefiable soils, by treating them as sandy silt with a silt content of 35% according to AASHTO soil classification system. Procedure for the development of liquefaction maps for standard and critical bridges are presented in Chapter 4 of this report.

Site-Specific Analysis

AASHTO-SGS require site-specific spectra for the design of critical bridges. It is also required for Site Class F. Although AASHTO-SGS provide 1000 Yr. return period spectra on zip-code basis, they don’t reflect the effects of local soil conditions. In Chapter 4 of this report, a customized approach to develop site-specific spectra and ground motions is presented.

Examples on the use of AASHTO Seismic Guide Specifications

Since AASHTO-SGS, there are very few examples illustrating the use its provisions for the design of new Bridges. Development of examples is important for the training of engineers since AASHTO-SGS are based on displacement based approach, which is a significant departure from the traditional force based design of bridge components. Chapter 5 of this report presents nine examples of reinforced concrete and steel bridges (3 of each type) designed as per the provisions of AASHTO-SGS. These examples present step-by-step procedure to design bridges both in SDC A and B.

Seismic Design Issues for Existing Bridges in New Jersey

The FHWA retrofit manual prescribes two levels of earthquakes. A structure is expected to stay essentially elastic during the lower level earthquake. Collapse prevention is targeted during the upper level earthquake. Based on a preliminary review of spectral accelerations during lower level earthquakes, it is noted that lower level earthquakes are likely to have very little impact on bridges.

FHWA retrofit manual uses both S_{DS} ($S_{DS} = S_s \times F_a$) and S_{D1} in determining seismic retrofit categories. This is completely different from the AASHTO-SGS where only S_{D1} is used to determine seismic design category. Use of both S_{DS} and S_{D1} can place much higher requirement on retrofit of existing bridges compared to new bridges. The choice of high-frequency spectral indicator through the use of S_{DS} penalizes the Eastern USA (including NJ) for no credible justification, given that the damage to bridges is associated with low frequency range of interest. For example, for a Zip-Code 07022, Table 1.7 below shows comparisons of hazard levels for new and existing bridges using AASHTO-SGS and 2006 FHWA Seismic Retrofit Guidelines using 1000 Yrs. spectra.

Table 1.7 Comparison of Seismic Hazard Levels for New and Existing Bridges.

Soil Class	S_{DS}	S_{D1}	Hazard Level for Existing Bridges	SDC According to AASHTO-SGS
B	0.19	0.04	II	A
C	0.22	0.07	II	A
D	0.30	0.09	II	A
E	0.46	0.13	III	A

It is observed from Table 1.7 that existing bridges in soil type E may have to be retrofitted as per seismic retrofit categories based on desired level of performance, whereas new bridges will be designed as per SDC A (similar to hazard level I for existing bridges) for all soil types. For existing bridges, seismic retrofit category (SRC) A, B, C or D is assigned based on performance level requirements during a particular hazard. For Level III hazard at soil site E in Table 1.7, Seismic Retrofit Category (SRC) C will be required during the upper level earthquake which will require detailed capacity and demand analysis. This requirement is significantly higher than that for new bridges and should be resolved to minimize the use of resources on unnecessary retrofits.

Based on discussions above, it is clear that using S_{D1} only will place most of the bridges in Hazard level I in the FHWA Retrofit manual. Seismic design categories for new bridges and seismic retrofit categories for existing bridges may not correspond to identical levels of risks of damages. This may result in disproportionate level of risk management and more expensive retrofits for bridges than that may be needed. The guidelines for retrofit of existing bridges needs to be aligned with new bridges based on acceptable level of performance for all bridges in New Jersey. Chapter 6 of this report presents simplified guidelines for seismic retrofit of existing bridges. These guidelines meet or exceed the requirements of FHWA Seismic Retrofit manual currently being used by NJDOT and are consistent with AASHTO-SGS.

CHAPTER 2: DEVELOPMENT OF SOIL SITE CLASS MAP FOR NEW JERSEY

Introduction

According to AASHTO-SGS [AASHTO (2008)], soil sites for the purpose of seismic analysis and design can be classified into Site Classes A, B, C, D, E and F. Site Classes A and B are rock sites, Site Class C is very dense soil, Site Class D is dense soil, Site Class E is soft soil and Site Class F is special soil requiring site specific analysis. New Jersey Department of Transportation (NJDOT) has recently developed Geotechnical Database Management System (GDMS) which contains large number of soil boring data across New Jersey. These boring logs provide information on Standard Penetration Test (SPT) blow count and soil description. Although various methods can be used to carry out site classification, the method based on Standard Penetration Test (SPT) blow counts and soil description has been used to classify soil sites, considering the availability of soil boring data from GDMS.

The purpose of the site classification analysis is to generate a map of soil site class at a precision of zip code for the State of New Jersey. In another words, each zip code in New Jersey is assigned a site class based on its main soil condition. The following three sources of soil data have been used to generate the soil site classes:

- NJDOT soil borings database available at the following web link: (http://www.state.nj.us/transportation/refdata/geologic/),
- Surficial Geological Map

  (http://www.state.nj.us/dep/njgs/geodata/dgs07-2.htm) developed by New Jersey Geological Survey (NJGS)
- Soil site class Maps for nine counties in northern New Jersey (http://www.state.nj.us/dep/njgs/enviroed/hazus.htm) developed by New Jersey Geological Survey for the purpose of earthquake loss estimation with the support from FEMA, which are referred to as HAZUS soil maps hereafter.

General Procedure for Soil Site Classification

The approach to classify soil sites utilizes as much available information as possible while considering adequate conservativeness, given variability in soil profiles between different locations. The procedure is based on the site class definitions using average SPT blow counts and is shown in Appendix II. Some criteria for the site classification as per AASHTO-SGS were also conservatively adjusted based on the availability of data.

Due to the large amount of data available in the GDMS for the soil site classification (about 50,000 boreholes in the NJDOT soil boring data during the time of analysis in spring and summer of 2009), a system was established for data collection, grouping and analysis so that the relevant data could be analyzed according to their geological locations and conditions. Geographical Information System (GIS) was used for the selection and grouping of boreholes in a zip code such that boreholes were distributed across the zip code. For each zip code, a maximum of 30 boreholes were classified, resulting is the analysis of approximately 12,500 boreholes for the entire state. The maximum limit of 30 for each zip code was imposed based on considerations of both ground condition representation and the amount of effort involved. However, the number of data analyzed for many zip codes, e.g., zip codes in Hudson County, was significantly more than 30. The detailed procedure for site classification for each zip code is described in Appendix II.

Soil site class Site Class Maps for New Jersey

Using the procedure outlined in Appendix II, soil site class maps were generated for 21 counties of New Jersey. These maps have been generated using ArcGIS, and the digital maps are also provided for application purposes. In the digital map, the user will be able to identify the soil site class of each zip code in the state of New Jersey. Since not all zip codes have boring data, for zip codes with boring logs available in the GDMS, the user can also locate the borehole used to classify the site class of a specific zip code.

Soil site class maps for 21 counties of New Jersey are enclosed in Appendix II. Each of these maps shows the county name, zip code, soil site class and location of analyzed boring logs. Other information, such as municipality, can be overlapped on the maps using the digital file. The specific numbers of boreholes analyzed for a zip code are not shown on the map. This information can be retrieved from the digital file. Excel files containing information on all analyzed boreholes are provided on the CD enclosed with this report. Soil site class maps for 21 counties were combined together to yield a map for the whole state of New Jersey, as shown in Figure 2.1. This map doesn't show locations of analyzed boreholes so that zip code names are visible clearly.

Notes on the Use of Soil Site Class Maps

Although soil site class map has been developed based on all available soil information in New Jersey, following issues should be considered before using the map:

1. The map is for the purpose of preliminary seismic design and evaluation of bridges only. It cannot be used for foundation design and analysis.
2. The soil site class of a zip code is only a general representation of the soil condition and does not exclude the possibility of localized soil condition. Specifically, in some zip codes (such as zip code 087XX in Ocean county) where marsh deposits can be found, Class F sites could be found, which requires special attention and site specific analysis.
3. Considering the possibility of localized ground condition, it is recommended that geotechnical engineer(s) screen any site of interest to check if it belongs to Site Class F. If a borehole is found to be Site Class F, the bridge should be seismically designed according to site-specific procedure.
4. The soil site class map is based on the digital zip code map found on the website of the New Jersey Geological Survey. The zip code map of New Jersey used in the AASHTO Earthquake Ground Motion Parameter Software is slightly different from the zip code map used for soil site class map of New Jersey. The AASHTO software has 20 additional zip codes that occupy non-trivial areas. There are also

several PO zip codes. In order to ensure the applicability of the soil site class map, the locations of zip codes found in AASHTO Ground Motion Parameter Software have been mapped on the soil site class map. A user can conveniently locate the zip codes on this map to determine their soil site class. An electronic soil site class map containing the representation of zip codes in AASHTO software is also provided.

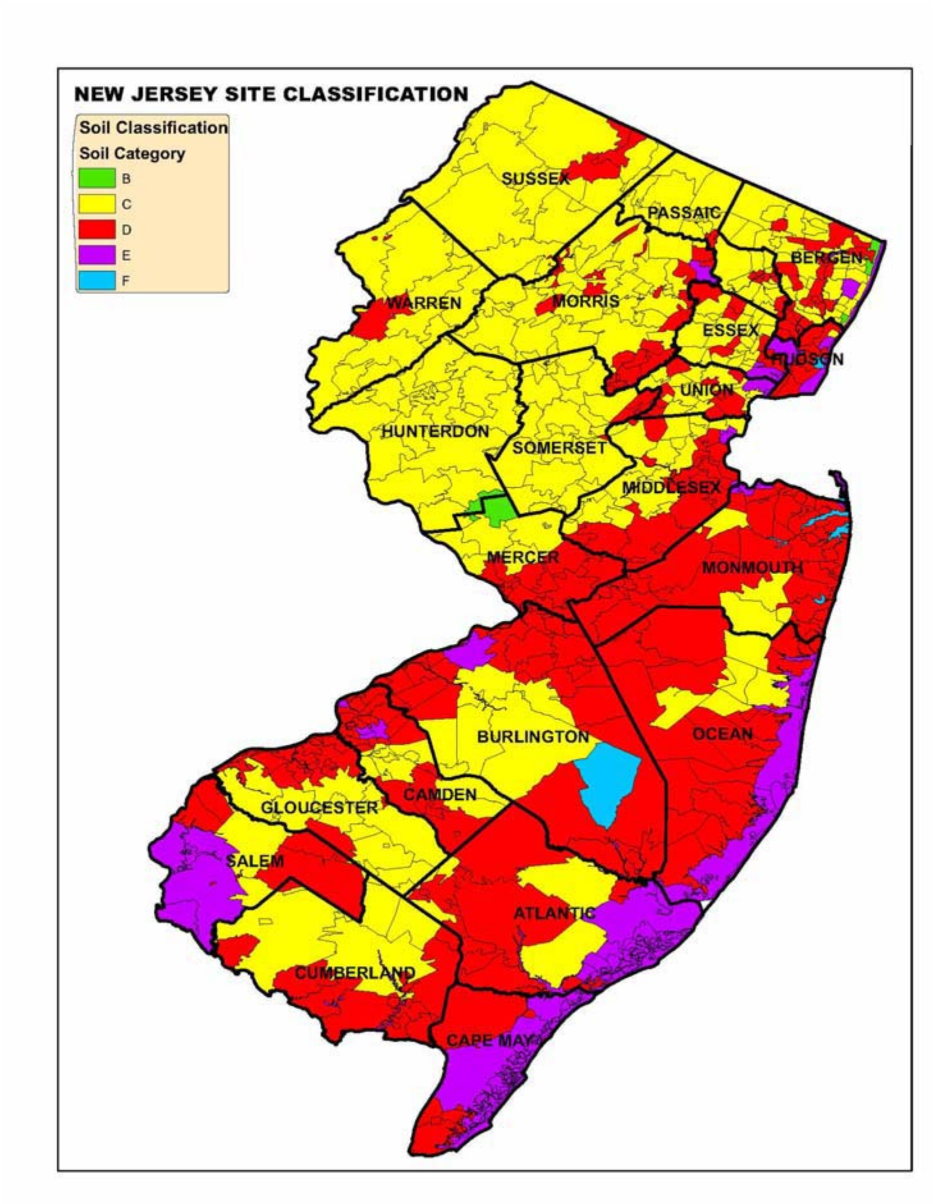


Figure 2.1 Zip Code Based Soil Site Class Map for Bridges in New Jersey.

CHAPTER 3: IMPORTANCE CLASSIFICATIONS OF BRIDGES IN NEW JERSEY

The provisions included in this document should be applied to the seismic design of normal Bridges. For purpose of these provisions, normal bridges are considered to be of conventional slab, beam, girder and box girder superstructure construction with spans not exceeding 500 ft (150 m). For complex bridge types (e.g., suspension bridges, cable-stayed bridges, truss bridges, arch type and movable bridges) and spans exceeding 500 feet, a site specific design specification, as directed by NJDOT, may be required.

Seismic effects for box culverts and buried structures need not be considered, except when they are subject to unstable ground conditions (e.g., liquefaction, landslides, and fault displacements) or large ground deformations (e.g., in very soft ground).

Bridges in New Jersey are recommended to be classified as critical and standard, depending on the importance assigned to the highway system carried on/under a bridge. It has been observed from 9 examples of bridge design in Chapter 5 that the behavior of bridges in New Jersey is likely to be essentially elastic (elastic or slightly plastic), even when 1000 Yr (AASHTO-SGS) spectra multiplied by a factor of 1.5 is used. Hence, based on feedback from NJDOT, “Essential” category isn’t considered for New Jersey bridges.

Criteria for classification for bridges in these two categories and performance requirements for bridges in critical category are contained in this Chapter. The provisions specified in the AASHTO Guide Specifications for LRFD Bridge Seismic Design (AASHTO-SGS) are for “Standard” bridges and should be taken as the minimum requirements. The provisions included in this document should supplement and/or supersede the AASHTO-SGS.

For design purposes, all bridges shall be classified as standard or critical based on the provisions of this document. However, New Jersey Department of Transportation has the discretion to classify a bridge either as critical or standard.

Seismic Ground Shaking Hazard

The seismic ground shaking hazard should be characterized using an acceleration response spectrum, which is determined in accordance with the general procedure Article 3.4.1 of the AASHTO-SGS or the site-specific procedure in Articles 3.4.3 of the AASHTO-SGS and modified by a factor of 1.5 applicable to critical bridges, as described later in this chapter.

Selection of Seismic Design Category (SDC)

Each bridge should be assigned to one of four Seismic Design Categories (SDC), A through D based on the one-second period design spectral acceleration for the design earthquake. A Seismic Design Category (SDC) based on design spectral acceleration (S_{D1}) corresponding to the 1.0 second period, T_1, is the minimum requirement which may be upgraded to a higher SDC based on the discretion of the bridge owner.

Seismic hazard level is defined as a function of the magnitude of the ground surface shaking as expressed by $S_{D1} = F_vS_1$ for standard bridges (non-critical, as defined later). For critical bridges, site specific analysis should be carried out after applying 1.5 factor to the input bedrock motion to determine the spectra and S_{D1}. A detailed rationale for using 1.5 magnification factor is presented in Chapter 1.

Bridges should be assigned Seismic Design Categories (SDC) A, B, C and D based on the values of S_{D1} as per Table 3.1. Each of the SDCs A to D should satisfy the requirements listed in Table 3.2. The partition of SDCs according to S_{D1} affects ground shaking hazards. Besides S_{D1}, other factors may affect the selection of SDC. For example, if the soil is liquefiable and lateral spreading or slope failure can occur, SDC D should be selected.

Table 3.1 Partitions for Seismic Design Categories A, B, C and D.

S_{D1}	SDC
$SD_1 < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

Table 3.2 Requirements for Different Seismic Design Categories.

Requirement	A	B	C	D
Identification of ERS	N/A	Recommended	Required	Required
Demand Analysis	N/A	Required	Required	Required
Implicit Capacity	N/A	Required	Required	Required
Push Over Capacity	N/A	N/A	N/A	May be Required
Support Width	Required	Required	Required	Required
Detailing – Ductility	N/A	SDC B	SDC C	SDC D
Capacity Protection	N/A	Recommended	Required	Required
Liquefaction	N/A	Recommended	Required	Required

The Seismic Design Category reflects the variation in seismic risk across the country and is used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures. If significant liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge may occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

Bridge Performance Criteria

Critical Bridges: A Critical Bridge must not collapse and provide immediate access (once inspected within a few hours) to function as a critical link to the lifeline network to serve the social/survival network, civil defense, police, fire department, and/or public health agencies to respond to a disaster situation after the event. The hazard level for the Critical Bridges is recommended to be 1000 year event (7% probability of being exceeded in 50 years) multiplied by a factor of 1.5.

A Critical Bridge should be designed to have only minimal damage. The bridge should essentially behave elastically during the design earthquake, although minor inelastic response could take place. Post earthquake damage should be limited to narrow flexural cracking in concrete and masonry elements. There should be no permanent deformations to structural members. Only minor damage or permanent deformations to non-structural members should take place.

Standard Bridges: Standard bridges will be classified as non-critical bridges and should be designed as per provisions of AASHTO-SGS for 1000 Yr. return period earthquake.

Criteria for the Classification of Critical Bridges

A bridge in New Jersey can be classified as on the basis of any of the three following criteria. New Jersey Department of Transportation (NJDOT) can select the criteria applicable to seismic risk management goals of the department. Selection of “Generic” and “Serviceability Based” criteria may provide maximum flexibility while managing the seismic risk effectively. As per AASHTO-SGS, a critical bridge is classified as

- Bridges that are required to be open to all traffic once inspected after the design earthquake and be usable by emergency vehicles and for security, defense, economical, or secondary life safety purposes immediately after the design earthquake.
- Bridges that should, as a minimum, be open to emergency vehicles and for security, defense, or economical purposes after the design earthquake and open to all traffic within days after that event.
- Bridges that are formally designated as critical for a defined local emergency plan.

These three criteria have been combined to propose the following generic criteria for the importance classification of bridges:

Generic Criteria: During the design phase of a bridge, bridge engineers and consultants can classify a bridge as critical if bridge satisfies functional requirements of the following criteria:

“A *Critical* Bridge must not collapse and it must provide immediate access after the design hazard level (1.5 times 1000 Years) event (i.e., operational performance) and continue to function as a part of the lifeline, social/survival network and serve as an important link for civil defense, police, fire department and/or public health agencies to respond to a disaster situation within 48 hours after the event, providing a continuous route. Any bridge that crosses a critical route should also be classified as critical if significant damage to such bridge may interfere with the critical route.”

Specific Criteria: A bridge can be classified as critical if it satisfies any of the following criteria.

- Bridges that are required to be open to all traffic once inspected after the design earthquake.
- Bridges that are on the Interstate Highway System.
- Bridges that provide access to the New Jersey Turnpike.
- Bridges on highways that lead up to major river crossings.
- Bridges on routes that don't have detour.
- Bridges that are required to be usable by emergency vehicles to provide secondary life safety to provide access to local emergency services such as hospitals immediately after a design level earthquake.
- Bridges that serve as a critical link in the security and/or defense roadway network. Now referred to as SHRAHNET, this defense highway network provides connecting routes to military installations, industries, and resources and is part of the National Highway System.
- Bridges that are formally designated as critical for a defined local emergency plan.
- Bridges that cross over critical routes (e.g., a bridge going over New Jersey Turnpike) providing secondary life safety or bridges crossing type of facilities as pertinent to defense, emergency, and economical considerations.
- Bridges that carry utilities and their relative importance on life safety (on the discretion of NJDOT).
- Bridges with foundation and site characterization that may require increased effort of post-earthquake investigation and response.

It should be noted that bridges crossing over critical routes (such as a bridge over New Jersey Turnpike) may be designed for lesser performance level of “acceptable damage”, depending on the functionality of the bridge.

Serviceability Based Criteria: Bridges can also be classified based on serviceability factors, such as average daily traffic, recovery time after an earthquake, detour length and time impact on emergency and defense vehicles. The classification based on these factors can be on the basis on bridge importance screening formula developed by Englot (2011).

The bridge importance screening formula (BISF) can be used to classify a bridge as critical based on Potential Delay of Transport Units (PDTU) calculated in the units of hours [Englot. (2011)]:

$$PDTU = [TVTU \times DD \times TDD] \tag{3.1}$$

Where TVTU = total volume of transport units (in Units/day), DD = Days of downtime when bridge or tunnel is not functional (in days), TDD = Time delay due to detour (in hours). In the calculation of TVTU, one automobile is considered one transport unit. One large truck is equivalent to two transport units. Hence, TVTU can be calculated as:

$$TVTU = (ADT - AADT) + 2AADT \tag{3.2}$$

where ADT and AADT are obtained from SA&I sheet. DD is equal to the maximum span length factor and is calculated on the basis of the following equation:

$$DD(inMonths) = 7.0 \times 10^{-06} MaxSpan^2 + 0.0168 MaxSpan \qquad (3.3)$$

where MaxSpan is the maximum span length of a bridge. The parameter TDD is calculated as

$$TDD = CountyMultiplier \times Detour\ \text{Length (SI \& A Sheet)} / Speed\ on\ Detour\ Route \qquad (3.4)$$

The speed on detour speed is assumed to be 25 miles/hour. County multiplier is based on the values provided in Table 3.3.

Table 3.3 County Multiplier for Detour Length in New Jersey.

County	County Multiplier	County	County Multiplier
Union	1.26	Passaic	1.28
Hudson	1.15	Camden	0.97
Bergen	1.33	Gloucester	0.83
Essex	1.25	Somerset	1.03
Mercer	0.97	Warren	0.63
Morris	1.15	Hunterdon	0.83
Cape May	0.72	Sussex	0.78
Monmouth	1.24	Middlesex	1.21
Ocean	1.25	Burlington	0.89
Atlantic	0.72	Cumberland	0.70
Salem	0.72		

A value of PDTU from Eq.(3.1) is indicative of the potential delay of transport units because of the loss of a particular bridge during the reconstruction period. A representation of PDTU in dollars can be obtained by multiplying PDTU by dollars/hour for delay of transportation units. For prioritization purposes, a value of $30/hour can be considered to calculate "Estimate Loss Because of Delay in Transport Units (ELBDT). This value of ELBDT can be used to designate a bridge as critical or standard. The value of ELBDT separating critical and standard bridges should be based on information provided by the NJDOT.

Application of the estimated loss because of delay of transport units may be illustrated by considering an example of a bridge in Hunterdon county with AADT = 180,000, AADTT = 10,000, Detour length = 5 miles and Max Span = 500 ft. Then,

TVTU = (180,000-10000) + 2×10,000 = 190,000

DD = 10.15 Months

TDD = 0.83×5/25 = 0.166

PDTU = 190,000*10.15*0.166 = 286,433 hours

Assuming $30 per hour as average cost for each PDTU hour,

ELBDT = 286,433 × $30 = $8.59 Million Dollars

In order to classify this bridge, impact of $8.59 M on local economy should be analyzed to classify the bridge as critical or standard.

The value of ELBDT for bridges owned by NJDOT has been calculated based on bridge inventory data of NJDOT. Figure 3.1 shows the plot of ELBDT for 100 NJDOT bridges. It is observed that ELBDT, expressed in million dollars, decreased from approximately $115 Million to less than $10 Million for the 10th bridge. This value further decreases to approximately $1 Million for the bridge with 100th highest value of ELBDT. An appropriate threshold for classifying critical and standard bridges based on this criteria can be identified by considering the fact that $1M of ELDBT represent a traffic delay of approximately 33,333 hours of delay to all traffic during the recovery period. This threshold should be determined by considering the impact of this delay on local economy and community.

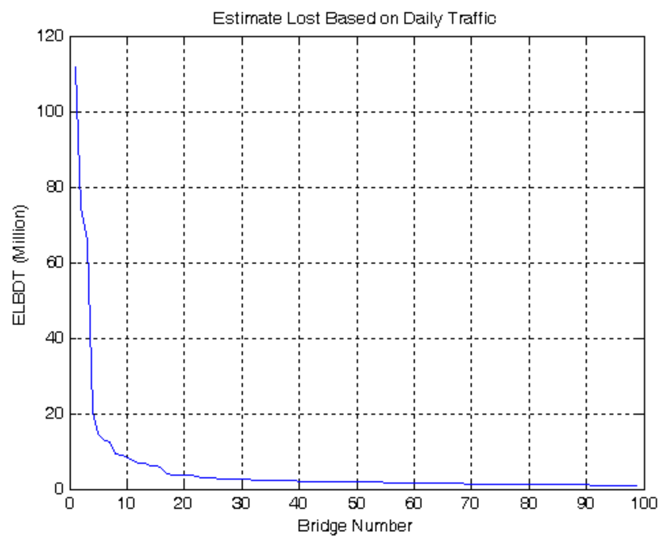


Figure 3.1 Plot of ELBDT for Bridges in New Jersey.

Recommended Performance Levels

The three performance service levels based on importance classification of a bridge are defined as:

Immediate: Full access to all traffic immediately following the earthquake. This service level is intended for Critical Bridges.

Maintained: Immediate access to emergency traffic. Short periods of closure to public with access typically restored within days of the earthquake. This service level is intended for critical bridges whose closure for a limited time will have acceptable level of impact on the local economy and traffic.

Impaired: Extended closure to public with access typically restored within months to a year after the earthquake. This service level is intended for standard bridges.

The three damage levels corresponding to the Immediate, Maintained and Impaired Performance Service Levels defined above are as follows:

Minimal: No risk of collapse. Essentially elastic performance of structure with no permanent deformation. May have limited plastic action (ductility demand up to 2).

Repairable: No risk of collapse. Concrete cracking, spalling of concrete cover, and minor yielding of reinforcement steel will occur. The extent of damage is expected to be sufficiently limited so that the structure can be essentially restored to its pre-earthquake condition without replacement of reinforcement or replacement of structural members. Damage can be repaired with a minimum risk of losing functionality. May have moderate plastic action (ductility demand up to 4).

Significant: Minimum risk of collapse. Permanent offsets may occur in elements other than foundations. Damage consisting of concrete cracking, reinforcement yielding, major spalling of concrete, and deformations in minor bridge components may require closure to repair. Partial or complete demolition and replacement may be required in some cases. May have significant plastic action (ductility demand higher than 4).

Normal Bridges defined as Critical Bridges shall be designed such that they suffer minimal damage level under the design ground motion. Table 3.4 shows recommended damage levels for components of critical bridges.

Table 3.4 Bridge Component Seismic Damage Limits.

Component	Damage to components of a Critical Bridge
Ductile Column	Minimal
Spread Footing	Minimal*
Pile Cap	Minimal*
Piles	Minimal*
Bent Cap	Minimal
Pad Key	Minimal
Diaphragm Cap	Minimal
Seat Abutments	Minimal
Stub Abutments	Minimal
Wingwall	Minimal
Piles At Abutment	Minimal*
Shear Keys At Abutment	Minimal
Stem Wall	Minimal
Ductile Steel Diaphragm	Minimal
Girder Connection to Concrete	Minimal

* These components should be designed for elastic behavior.

All standard bridges in New Jersey should be designed as per provisions of AASHTO Seismic Guide Specifications to achieve “Impaired Performance level” (significant damage) defined above. However, underground components for standard bridges should be designed to have elastic behavior.

CHAPTER 4: DEVELOPMENT OF SEISMIC DESIGN CATEGORY MAPS, LIQUEFACTION ANALYSIS MAPS AND SITE-SPECIFIC ANALYSIS PROCEDURE FOR NEW BRIDGES IN NEW JERSEY

As described in Chapter 2, the research team carried out an extensive analysis of boring data in New Jersey to develop the soil site class map for New Jersey. This map can be used in combination with zip-code based seismic spectra and spectral quantities (e.g., S_1, S_s, etc.) to develop hands on tools that can be used effectively to manage seismic risk to all bridges in New Jersey in a unified manner. In particular, the soil site class map has been used to develop the following GIS based maps:

- Seismic Design Category (SDC) Map for Standard Bridges
- Seismic Design Category Map for Critical Bridges
- Liquefaction Hazard Map for Standard Bridges
- Liquefaction Hazard Map for Critical Bridges.

This chapter describes the development of these maps and associated seismic design recommendations for new bridges in New Jersey. The seismic design recommendations are based on the AASHTO-SGS, considering the seismic hazard and ground condition in New Jersey.

Development of Seismic Design Category (SDC) Maps

The SDC maps for two types of bridges (i.e., standard and critical bridges) were generated at a precision of zip code based on the Soil Site Class Map as discussed in Chapter 2. The maps are based on the digital zip code map downloaded from the website of New Jersey Geological Survey [NJGS (2007)].

The following procedure has been used to develop the SDC maps:

1) Representative latitude and longitude of each zip code is obtained from the AASHTO Ground Motion Parameters Program (AASHTO GM 2.1).
2) The response spectral acceleration S_1 at period T = 1.0 for Class B rock is obtained from the AASHTO Ground Motion Parameters Program (AASHTO GM 2.1), as illustrated in Figure 4.1.
3) The zip-code location (i.e., latitude and longitude information) is then mapped to the Soil Site Class Map to determine soil site class of the zip code.
4) If the soil site class of a zip code is F, it is shown as Site Specific in SDC maps, since Site Class F soil requires site specific analysis. The SDC of the zip code in this case is obtained by following the approach in section on “Site Specific Analysis” presented later in this chapter.
5) The response spectral acceleration S_{D1} at period T = 1.0 for a standard bridge is obtained using Eq. (4.1)

$$S_{D1} = F_v S_1 \quad (4.1)$$

Similarly, S_{D1} at period T = 1.0 for critical bridges is obtained from

$$S_{D1} = 1.5 F_v S_1 \quad (4.2)$$

Here, the factor F_v depends on the soil site class according to Table 4.1. In Eq.(4.2), factor 1.5 is applied for critical bridges, as described in Chapter 3. For New Jersey, S_1 is smaller than 0.1. Hence only the values of F_v in the first column of Table 4.1 are relevant.

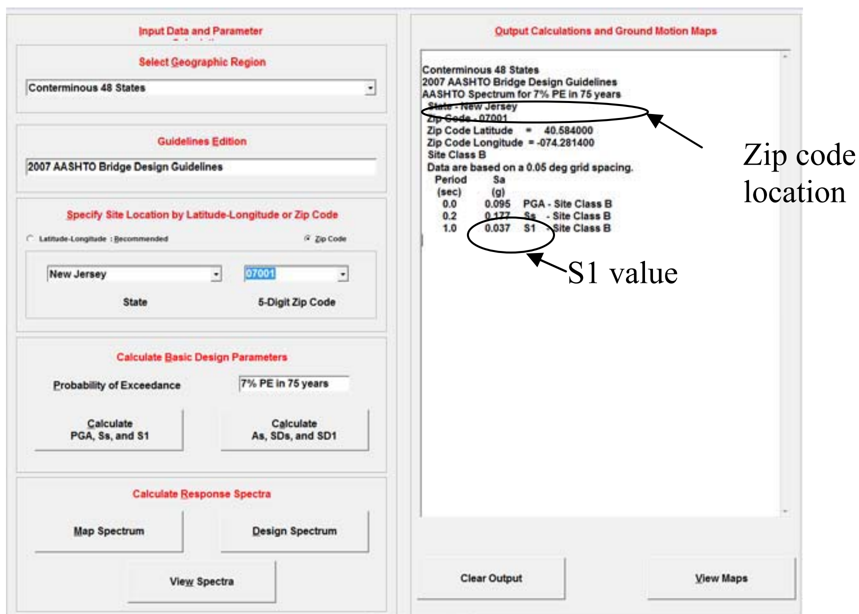


Figure 4.1 Zip Code Location and S_1 from AASHTO GM 2.1.

Table 4.1 Factor F_v in Equations (4.1) and (4.2)
(According to AASHTO-SGS)

Site Class	Mapped Spectral Response Acceleration Coefficient at 1 Second Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4

Note: Column 1 ($S_1 \leq 0.1$) is relevant to the seismic hazard in New Jersey.

6) Some locations in New Jersey don't have a zip code and S_1 value for Class B rock site for these zip codes cannot be obtained directly from AASHTO GM 2.1. In such cases, S_1 value for adjacent zip code has been used to obtain its S_{D1} according to Equation (4.1) or (4.2) for standard or critical bridges. For example, the area 070HH shown in Figure 4.2 shares the same S_1 value as zip code 07002. Hence, S_{D1} for 070HH has been obtained based on its site class and the S_1 value of zip code 07002 according to Equations (4.1) or (4.2) and Table 4.1.

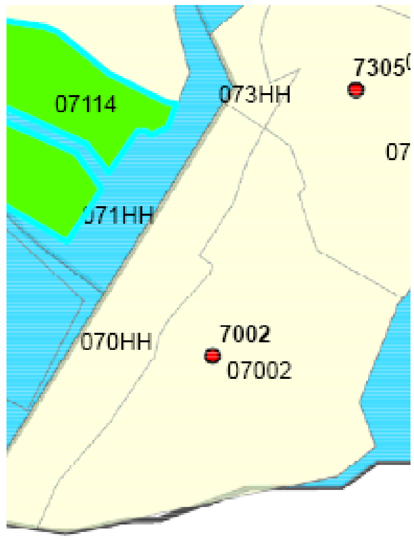


Figure 4.2 Classifying SDC for a Non-Zip-Code Region.

7) The SDC of a zip code was finally determined on the basis of S_{D1} calculated by Equations 4.1 or 4.2 following the criteria in Table 4.2. It has been observed that a majority of zip codes in New Jersey fall in SDC A with some locations falling in SDC B. Few zip codes are classified as site specific because of special soils. Such sites require site-specific analysis.

Seismic Design Category Map for Standard Bridges

The SDC map of the State of New Jersey for standard bridges is shown in Figure 4.3. For standard bridges, a majority of zip codes in New Jersey fall in SDC A category and few zip codes require site specific analysis.

Table 4.2 Criteria for Seismic Design Categories (SDC) as per AASHTO-SGS

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

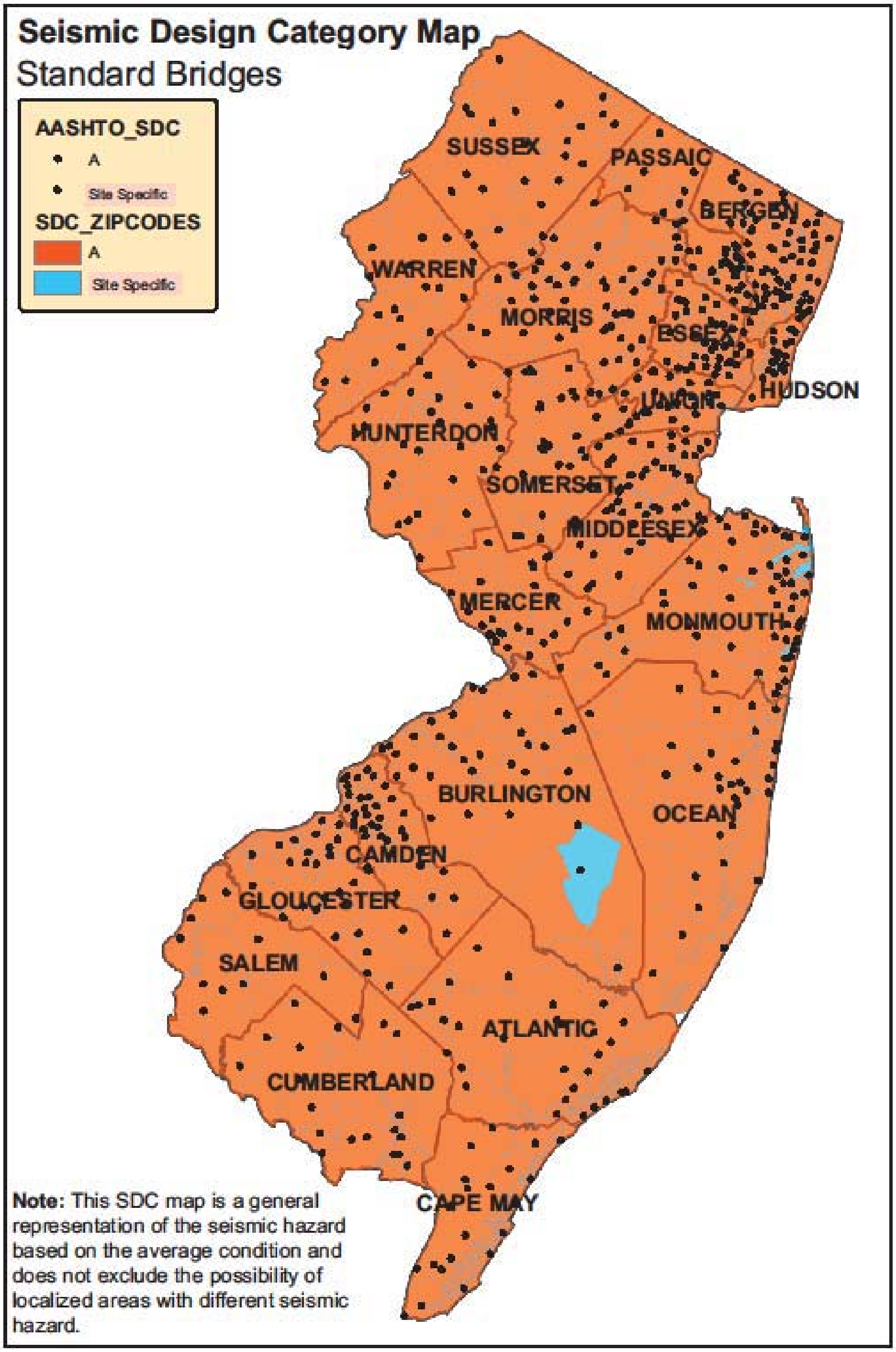


Figure 4.3 Seismic Design Category Map for Standard Bridges in New Jersey (The dots in the figure represent zip codes from AASHTO GM2.1)

Seismic Design Category Map for Critical Bridges

Figure 4.4 shows the SDC map for critical bridges in New Jersey. It is observed from Figure 4.4 that a majority of zip codes are in SDC A and few zip codes in Northeastern New Jersey in SDC B. The soil sites in these zip codes are site class E. SDC map for critical bridges in Figure 4.4 is based on the generic spectrum using Equation 4.2 and should be used as a reference since critical bridges require site-specific analysis to obtain the value of S_{D1}. The procedure for site specific analysis is discussed in a later section. For the convenience of users, digital SDC maps are also provided for application purposes.

Notes on the use of SDC maps

1) The seismic design category of a zip code in the map is only a general representation of the seismic hazard. It is suggested that geotechnical engineer screen the boring log of a specific site for Soil Site Class F. If such localized soil condition is encountered, site specific analysis procedure should be followed to obtain the response spectrum for seismic design of the bridge. Seismic design category is then determined as per the site specific response spectrum.
2) SDC maps in Figure 4.3 and 4.4 are based on the digital zip code map downloaded from the website of New Jersey Geological Survey. However, locations of some of the zip codes created during last few years cannot be indicated on SDC map in Figures 4.3 and 4.4. However, since the map covers entire state of New Jersey, SDC of such zip codes can be determined by plotting their geographical location (latitude and longitude) on the digital SDC map of the State of New Jersey.

Development of Liquefaction Hazard Maps for New Jersey

In conjunction with the seismic hazard analysis of New Jersey, liquefaction hazard analysis was conducted to assess the liquefaction potential of each zip code. The analysis utilized the Standard Penetration Test (SPT) blow counts of soil and followed the approach by Youd et al. (2001). The method is one of the approaches suggested by the AASHTO-SGS [AASHTO (2008)]. The liquefaction hazard analysis has been carried out to evaluate the liquefaction potential of New Jersey based on two types of earthquakes. The first type of earthquake with 1000-year return period for standard bridges is based on the peak ground acceleration (PGA) on Class B rock from AASHTO GM 2.1, as illustrated in Figure 4.5. The second type of earthquake applies a factor of 1.5 to the 1000 Yr earthquake, as recommended for critical bridges in New Jersey. Detailed procedure to analyze the liquefaction potential of a borehole is presented in Appendix III.

Definitions of Liquefaction Hazard Levels

According to Youd et al. (2001), a site is considered to liquefy if the factor of safety (FS) of any soil layer is smaller than 1.0. However, according to available studies (FHWA 2006), build-up of excess pore pressure could be considerable for FS between 1.0 ~ 1.5. Besides, considering limited sources of data and the inherent variability of soil

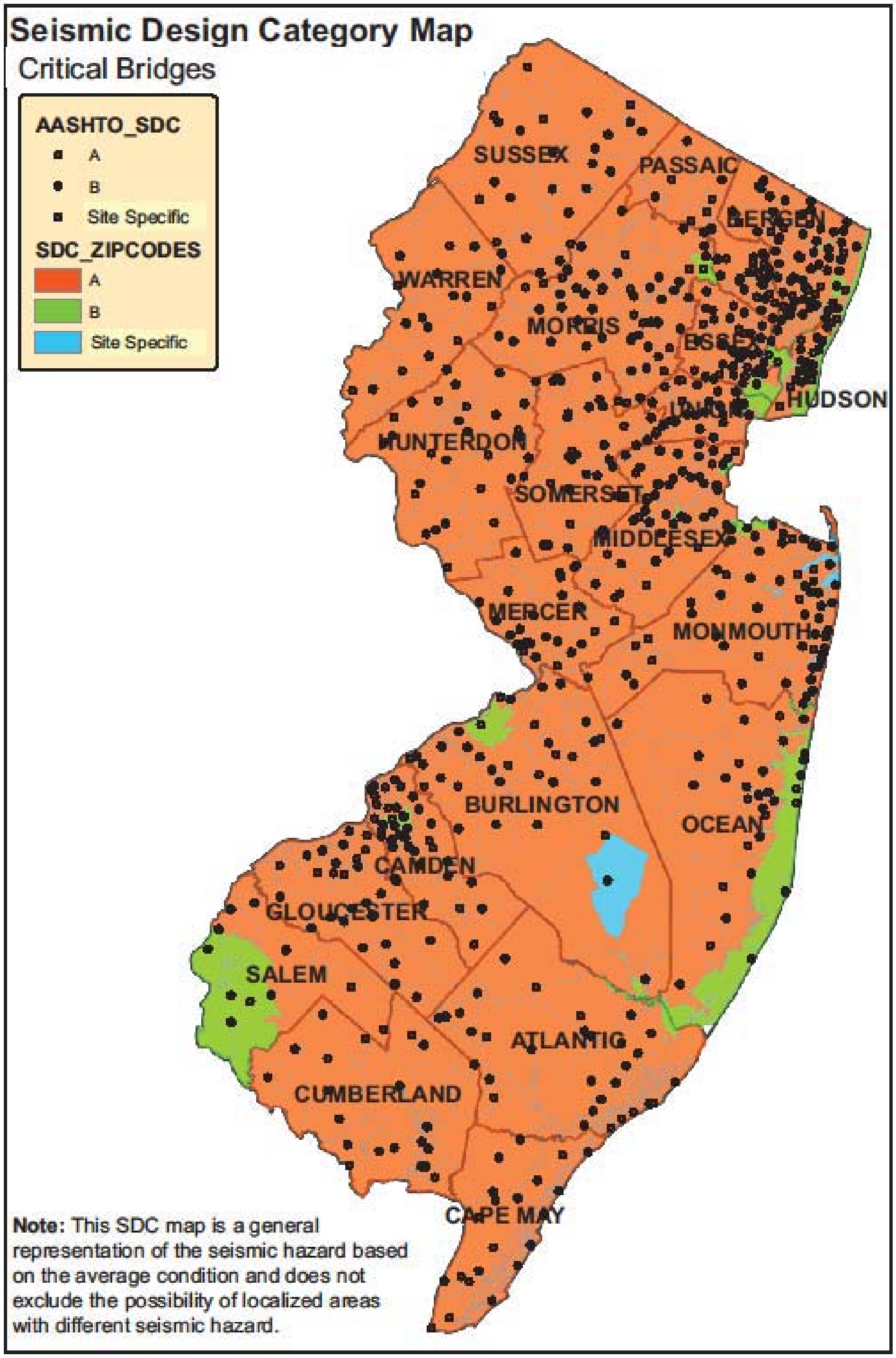


Figure 4.4 Seismic Design Category Map for Critical Bridges in New Jersey (The dots in the figure represent zip codes from AASHTO GM2.1).

conditions in a zip code, special attention should be paid to sites with FS slightly larger than 1.0. Hence, liquefaction hazard based on the FS of the site are assigned to a zip code based on the following criterion:

1) If more than 50% of boring logs of a zip code contain granular soil layers with FS smaller than 1.0, the zip code is assigned “High” liquefaction hazard level.
2) If more than 50% of boring logs of a zip code have FS in the range of 1.0 – 1.3, the zip code is assigned “Medium” liquefaction hazard level. However, a zip code with 30-50% liquefiable sites having FS < 1.0 is also assigned “Medium” hazard level.
3) Granular sites that are also site class D or E were “Low” liquefaction potential if the FS of boring logs are larger than 1.3.
4) All other sites were assumed to be non-liquefiable (a hazard level of “none”).

It should also be noted that a concept similar to above approach has also been used in FEMA’s seismic hazard analysis. [NJGS (1999-2009)]. In that analysis, liquefaction hazard levels are classified as “very high” to “none” based on the geological age of the soil deposit and underlined level of ground shaking.

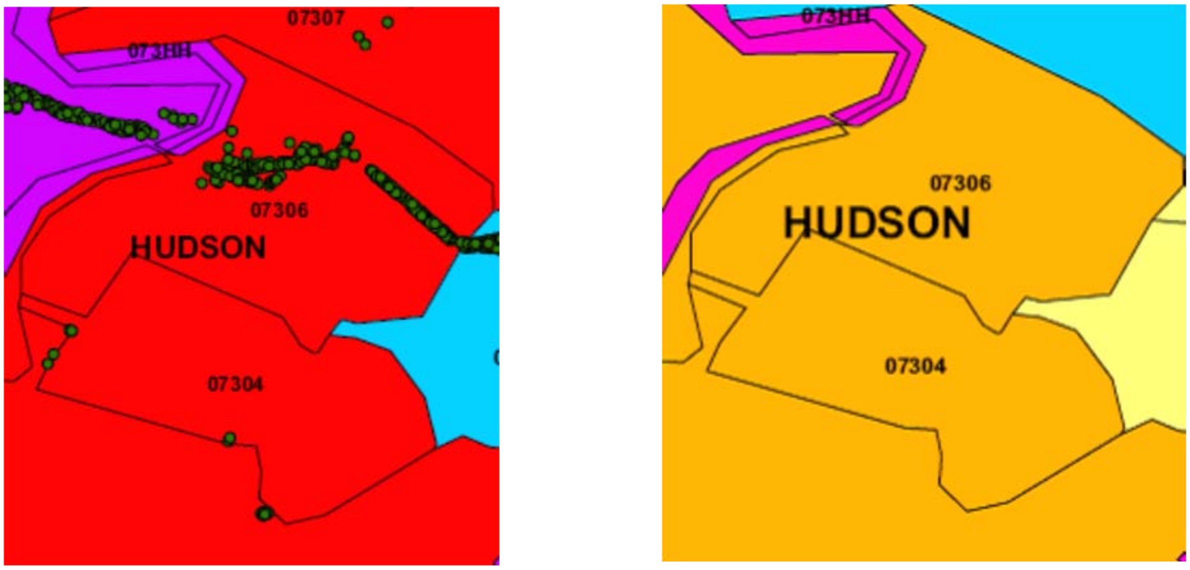
Liquefaction Hazard of a Zip Code with Sufficient Boring Logs

If a zip code has more than 30 boring logs and the site class of the zip code is D or E, all boring logs selected for site classification analysis were screened for granular soil layers. If a boring log contains granular soil layer, it was analyzed for factor of safety FS according to the procedure outlined in Appendix III.

The NJGS soil map was used to double-check the soil type in a zip code. If boring logs represent the soil type in the region, the liquefaction hazard of the zip code was then determined based on the criterion described in the previous section. If boring logs are too localized to represent the soil condition (this is not common for zip codes with 30 or more boring logs), the approach in the next section was adopted.

Liquefaction hazard of a Zip Code with Insufficient Boring Logs

If a zip code belonging to site class D or E has few or no boring logs, its liquefaction hazard was determined using an approach similar to that used for determining its site class. The NJGS soil map was used to determine if the ground in the region contains granular soil or not. If the soil in the region was mainly granular, liquefaction hazard of the zip codes in the vicinity that share the same type of soil and site class was assigned to the zip code of interest. For example, zip codes 07306 and 07304 in Hudson county are both Site Class D. Zip code 07306 has sufficient boring logs to determine its liquefaction hazard while zip code 07304 doesn’t have sufficient number of data. Since they share the same type of soil with significant granular content, both belong to site class D and are close to each other, they also have the same liquefaction hazard. Figure 4.5 shows the comparison between the two sites (i.e., zip codes 07306 and 07304).



(a) Soil class (red: Class D); (b) Liquefaction Hazard (dark yellow: Medium)

Figure 4.5 Example Illustrating Determination of Liquefaction Hazard of a Zip Code with Insufficient Data.

Liquefaction Hazard Maps for Standard Bridges

Using the 1000-year earthquake spectra in AASHTO-SGS, liquefaction hazard maps for 21 counties in New Jersey were generated, as shown in Figure III.2 to III.22 in Appendix III. The map for the whole state is shown in Figure 4.6. The electronic versions of these maps are also provided for application purposes. It can be seen from these maps that areas with higher liquefaction hazard are mainly in the northeast part of New Jersey.

Liquefaction Hazard Maps for Critical Bridges

Using a factor of 1.5 to the PGA of 1000-year earthquake, the liquefaction hazard maps for 21 counties in New Jersey were generated, as shown in Figure III.23 to Figure III.43 in Appendix III. The map for the whole state is shown in Figure 4.7. Compared to the hazard for 1000-year earthquake, the areas with "medium" liquefaction hazard are now classified as "high", and some areas with "low" hazard now have "medium" liquefaction hazard.

Similar to the SDC map, liquefaction hazard maps for critical bridges are for preliminary design and reference purposes only, since critical bridges require site specific analysis and the maximum acceleration a_{max} at ground surface that is needed for liquefaction potential analysis must be obtained using site-specific analysis. The procedure to determine a_{max} for critical bridges, as described in Appendix III is only approximate.

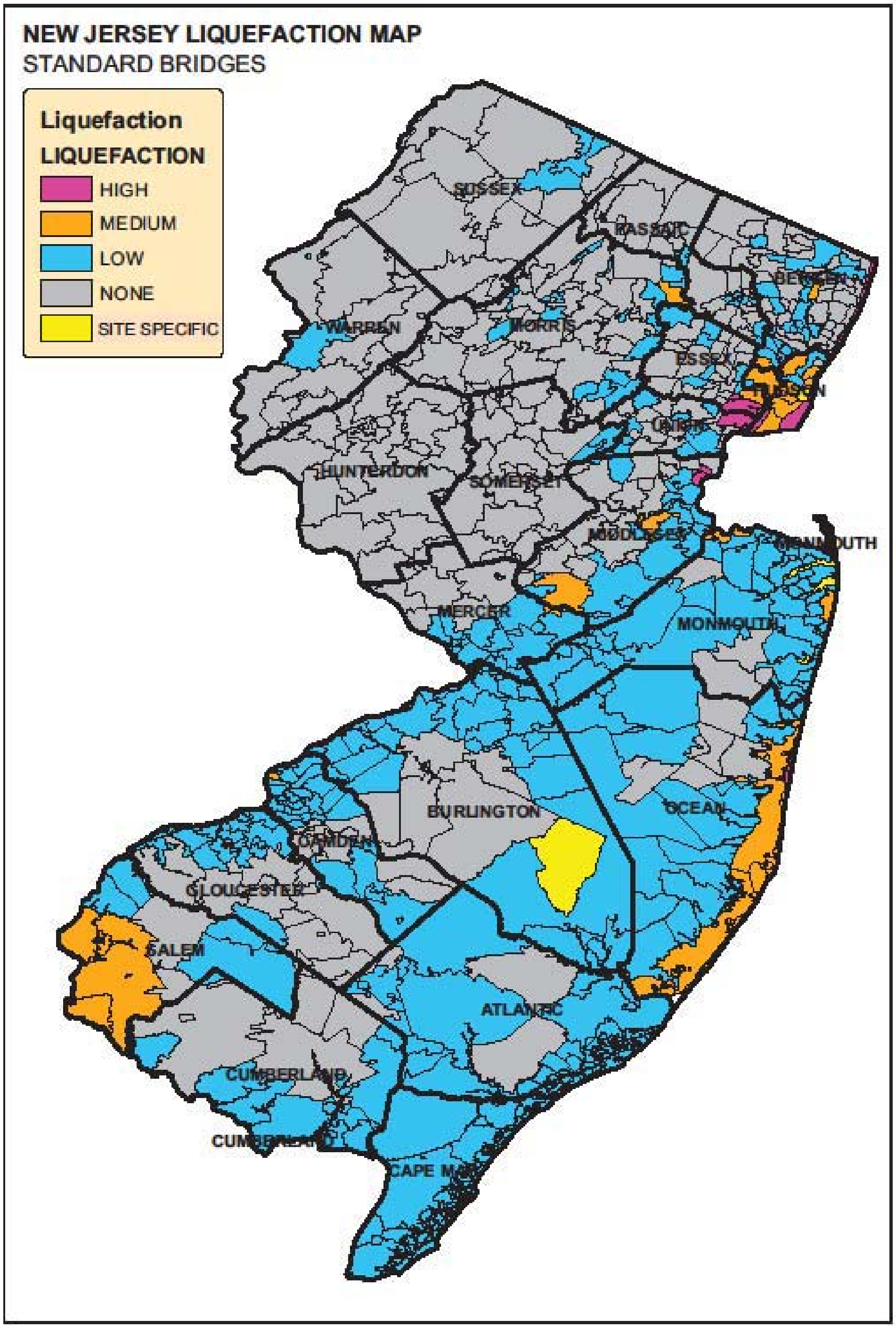


Figure 4.6 Liquefaction Hazard Map for Standard Bridges in New Jersey.

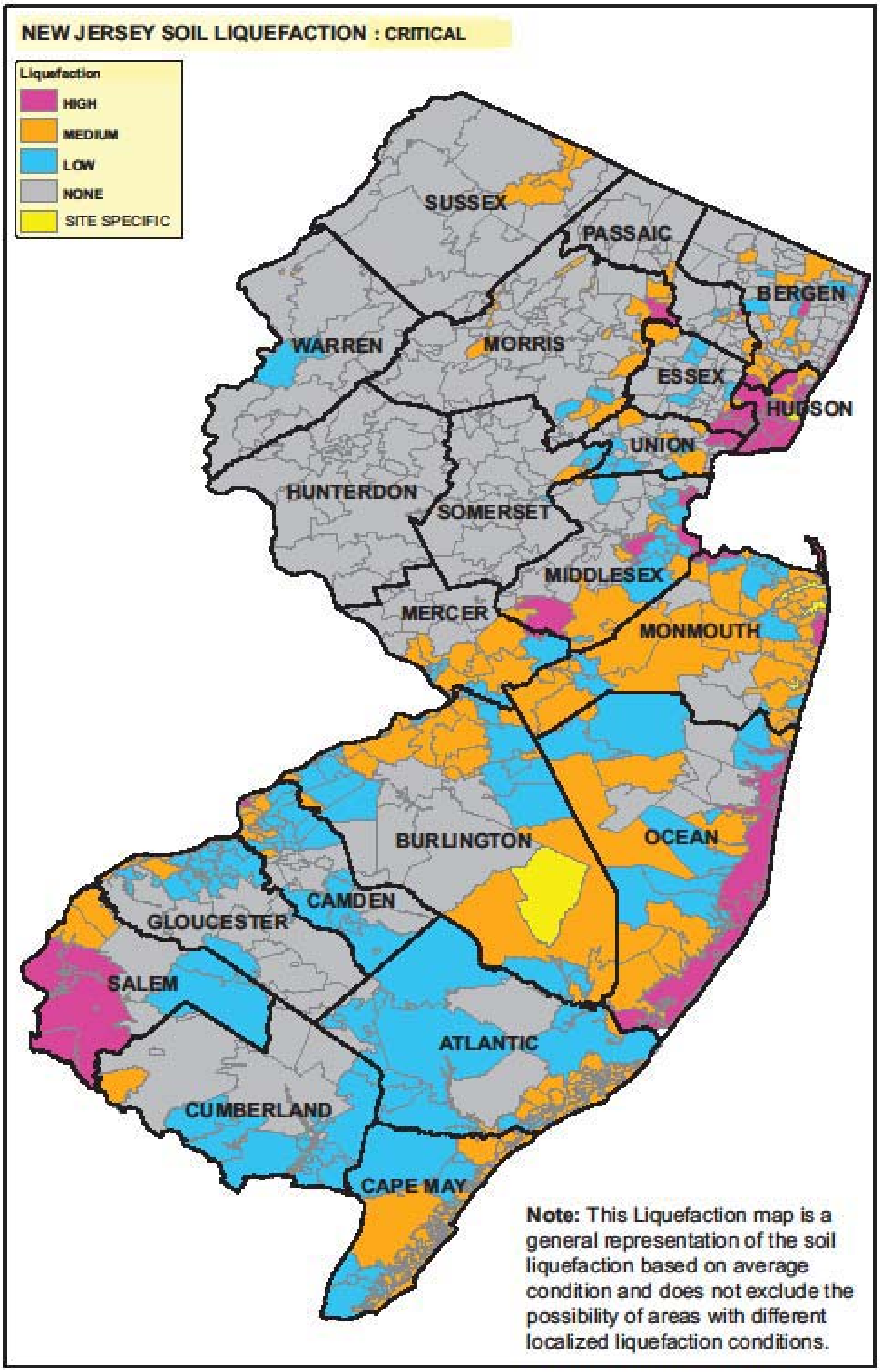


Figure 4.7 Liquefaction Hazard Map for Critical Bridges in New Jersey.

Notes on the use of Liquefaction Hazard Maps

1) For standard bridges, it is recommended that the geotechnical engineer screen the liquefaction potential of a site as per Youd et al. (2001), if a bridge is located in an area that is classified to have "high" or "medium" liquefaction hazard.
2) Similar to the seismic design category, the liquefaction hazard of a zip code is only a general representation. Very localized soil condition is possible in a zip code that is classified to have "None" or "Low" liquefaction hazard.
3) It is recommended that users of the maps identify the liquefaction hazard based on geographical location of the bridge (i.e., latitude and longitude), instead of the zip code. Digital maps are provided for this purpose.
4) The liquefaction hazard maps for critical bridges are for preliminary design or reference purposes only, since they are based on the generic response design spectrum. Detailed procedure to evaluate the liquefaction hazard of a critical bridge is discussed in site specific analysis section.

Recommendations on Seismic Design Based on SDC

Standard Bridges

For standard bridges, the SDC map in Figure 4.3 can be used to identify the seismic design category, unless screening of soil site condition indicates Soil Site F. In that case, site specific procedure should be used to determine the seismic design category. The following step can be followed in the seismic design.

1) If a site is found to be susceptible to liquefaction, but isn't susceptible to lateral spreading or lateral flow, the bridge can still be classified as SDC A and designed as per step (3) below. However, procedures to address liquefaction problem in the next section should also be followed and the change of foundation constraint should be considered in the seismic analysis.
2) If a site is found to be susceptible to lateral spreading or lateral flow due to soil liquefaction, the bridge must be designed according to SDC D.
3) If the zip code of a bridge falls in SDC A, the seismic design should follow section 4.6 of AASHTO-SGS [AASHTO (2009)].
4) If site specific analysis (for site class F) determines that a bridge must be classified as SDC B, the site-specific design spectrum must be generated based on the procedure described in a later section. Seismic design of the bridge should then follow the requirement of SDC B bridges in Chapter 4 of the AASHTO-SGS [AASHTO (2008)].

Critical Bridges

For critical bridges, site specific analysis is required to obtain S_{D1} spectral value. Seismic category for critical bridges is determined as per Table 4.2 using this value of S_{D1}. The SDC map in Figure 4.4 can only be used as reference during the preliminary design.

After seismic design category is determined, the following procedure must be followed in the seismic design:

1) If the site is found to be susceptible to liquefaction, but isn't susceptible to lateral spreading or lateral flow, the procedures to address liquefaction problem in a later section should be followed; the seismic design category of the bridge can still be obtained assuming that liquefaction does not occur but the change of foundation constraint should be considered in the seismic analysis.
2) If the site is found to be susceptible to lateral spreading or lateral flow due to soil liquefaction, the bridge must be designed according to SDC D.
3) If a bridge is classified as SDC A or B, performance criteria presented in Chapter 3 for critical bridges should be followed to design the bridge for "Minimal Damage" as per AASHTO-SGS.

It is not expected that any site in New Jersey will fall into SDC C or D unless there is susceptibility to lateral spreading or lateral flow.

Generation of response spectrum for standard bridges

The design spectrum should be generated for seismic analysis of standard bridges according to the following procedure:

1) PGA, S_s and S_1 of the site on Class B rock are obtained from AASHTO GM2 software according to the geographical location of the bridge.
2) The soil site class of the bridge can be obtained using the soil site class maps in Chapter 2.
3) The site factors F_{PGA}, F_a and F_v are obtained based on soil site class, PGA, S_s and S_1. These factors can be found in Chapter 3 of the AASHTO-SGS [AASHTO (2008)].
4) The design spectrum is then obtained according to Figure 4.8. A_s, S_{DS}, S_{D1} are obtained according to the following equations:

$$A_s = F_{PGA} PGA \tag{4.3}$$

$$S_{DS} = F_a S_s \tag{4.4}$$

$$S_{D1} = F_v S_s \tag{4.5}$$

Generation of generic response spectrum for critical bridges

For comparison purpose, generic response spectrum should be generated for critical bridges by multiplying Eqs. (4.3)-(4.5) for standard bridges by a factor of 1.5.

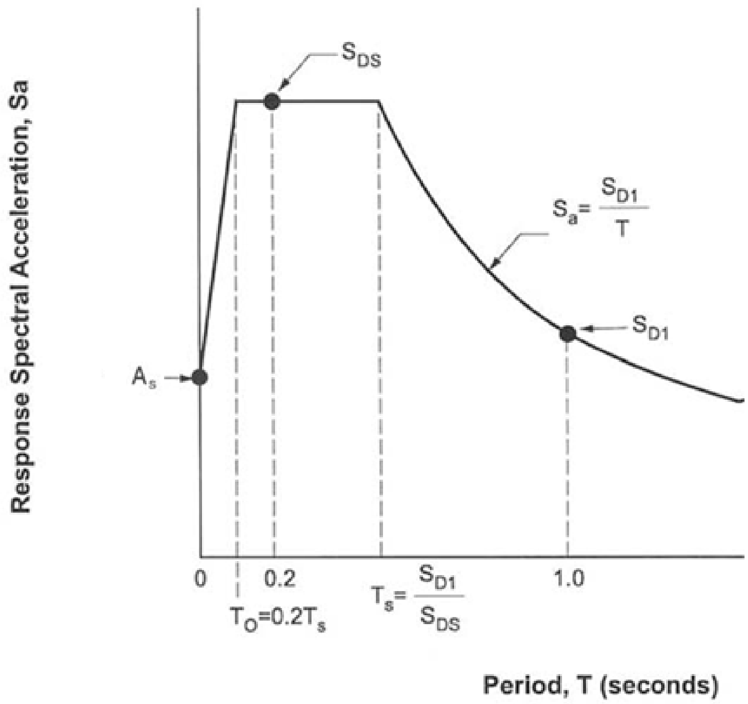


Figure 4.8 Construction of Design Spectrum Design spectrum for Standard and Critical Bridges.

Liquefaction Design Requirements

If a site is found to be susceptible to liquefaction, the foundation should be specifically designed to resist liquefaction damage or the ground should be improved so that liquefaction does not occur. Deep foundations must be used on these sites.

Lateral flow and lateral spreading

The geotechnical engineer should check if lateral flow or lateral spreading is possible at the site if it is determined that the site is susceptible to liquefaction. Possible procedures to evaluate lateral flow or lateral spreading are:

Lateral flow: To assess the potential for lateral flow, the static strength properties of the soil in a liquefied layer are replaced with the residual strength of liquefied soil. The residual strength of liquefied soil can be estimated using the curves reported in Seed and Harder (1990). A conventional slope stability check is then conducted without seismic force. If the resulting factor of safety is less than 1.0, lateral flow is probable.

Lateral spreading: To assess the potential for lateral spreading, the empirical method proposed by Youd et al. (2002) may be used.

Detailed design requirements and recommendations for lateral flow and lateral spreading have not yet reached a level of development suitable to be recommended in this document. The above procedures shall not be considered as design recommendations. Rather, they act as references to the geotechnical engineer and should be used by the geotechnical engineer appropriately to determine the evaluation procedure according to available knowledge in the field.

If a site is found to be susceptible to lateral flow or lateral spreading, then the bridge should be designed for SDC D and measures must be taken to resist associated damages. These measures include, but may not be limited to,

1) The engineer should consider the use of large diameter shafts;
2) A detailed evaluation of the effects of lateral flow on the foundation should be performed;
3) Detailed geotechnical analysis of the abutments may be required for single span bridges if lateral spreading of foundation soil is possible.
4) Box culverts and buried structures should also be properly designed to resist large ground deformation.

Other liquefaction design requirements

If appropriate measures have been taken to address associated damages because of lateral flow or lateral spreading, or if it is found that lateral flow or lateral spreading will not occur at the site, the following additional design requirements apply to the design of a bridge:

1) Bridges in liquefiable sites should be designed in the following two configurations:
    a) Non-Liquefied Configuration. The structure should be analyzed and designed by assuming that liquefaction doesn't occur using the ground response spectrum appropriate for site soil conditions.
    b) Liquefaction Configuration. The structure as designed in non-liquefied configuration above should be reanalyzed and redesigned, if necessary, assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance. The design spectra should be the same as that used in the non-liquefied configuration. All soil within and above the liquefiable zone should not be considered contributing to axial resistance. P-y curves for lateral pile response analyses consistent with liquefied soil conditions may need to be considered in this stage of analysis.
2) Foundation springs should be used to model pile or drilled shaft foundations while conducting seismic analysis, and they should reflect the change in support conditions due to soil liquefaction.
3) At a liquefiable site, deep foundations of standard and critical bridges are not permitted to form hinge below the ground line, considering the requirement on the performance of critical bridge as described in the Chapter 3.
4) If batter piles are used in a liquefiable site, consideration should also be given to the downdrag forces caused by dissipation of pore water pressures following liquefaction.

Site Specific Analysis

A site-specific procedure to develop design response spectra of earthquake ground motions should be performed if the bridge is critical, or the site belongs to Class F, and may be performed for any site. Depending on the bridge categories, the site specific design response spectra should be obtained according to:

1) Standard bridges: The site-specific probabilistic ground-motion analysis should be conducted in a manner to generate an acceleration response spectrum considering earthquake of 1000-year return period;

2) Critical bridges: The site-specific probabilistic ground-motion analysis should be conducted in a manner to generate an acceleration response spectrum considering earthquake of 1000-year return period multiplied by a factor of 1.5.

Principles and Assumptions

It is assumed that the seismic hazard at the location of interest is represented by the design response spectrum on the outcrop of the bedrock. Hence, in the site-specific ground response analysis, the input ground motion is generated from the bedrock design spectrum, and the motion propagates through the soil overlaying the bedrock to the bottom of footing. The motion at the bottom of footing is then used to generate the site-specific design spectrum at the site.

However, in order to take into account the uncertainties in ground motion and soil parameters, a series of analysis must be conducted, and the site-specific design spectrum should be taken as the envelope of motions obtained from these analysis.

Requirements on Subsurface Investigation

a) Shear wave velocity profile at the site should be obtained using appropriate measurement method before carrying out a site specific analysis. The soil at each layer needs to be classified by a geotechnical engineer to determine appropriate modulus reduction curve and damping curve for ground response analysis.

b) ASTM or AASHTO standardized methods for shear wave velocity measurements are recommended to be used. The measurement of shear wave velocity is required to reach full depth of the soil if the depth is smaller than 100 ft. The shear wave velocity of bedrock (top 20 ft) should also be measured. If the depth of soil is greater than 100 ft, it is strongly recommended that the full depth and the top 20 ft of rock be measured for accurate ground response analysis. However, the geotechnical engineer has the option to assume the depth of bedrock. In that case, at least three depths should be assumed in the ground response analysis, and rock class B should be assumed to exist below the soil deposit.

Generating Bedrock Design Response Spectrum

The class of bedrock is determined based on its shear wave velocity:

Class A: $V_s > 5000$ ft/s

Class B: $2500 < V_s \leq 5000$ ft/s

After the class of bedrock is determined, the corresponding design response spectrum should be generated according to the following criterion:

Standard bridges

1) The PGA, S_s and S_1 at the location on Site Class B is obtained from AASHTO GM 2.1;
2) The response spectral accelerations are obtained using Equations (4.3) - (4.5);
3) The design response spectrum is then generated according to Figure 4.8.

Critical Bridges

The design response spectrum for standard bridges is multiplied by a factor of 1.5 to obtain design response spectrum for critical briges.

Generating Ground Motion Time-Histories at the Bedrock

After the design response spectrum at the bedrock is obtained, response-spectrum compatible acceleration time-histories can be generated using appropriate program. The program SIMQKE that was developed by Gasparini and Vanmarcke (1976) is recommended in this report but the geotechnical engineers can also used other well-accepted programs. At least three time-histories must be generated for ground response analysis.

Ground Response Analysis

Ground response analysis should be conducted using appropriate program with the time-histories obtained in the previous section as input at the bedrock.

In order to take into account the uncertainty in measured shear wave velocity, it is recommended that three analyses be conducted for each input acceleration: (i) One using the measured shear wave velocities of soil and rock; (ii) one using 120% of the measured shear wave velocities of soil and rock; and (iii) one using 80% of measured shear wave velocities of soil and rock.

The geotechnical engineer is responsible for determining the modulus reduction curve and damping curve for each layer of soil according to soil classification and other available field data.

In this report, the computer program DEEPSOIL [Hashash et al. (2009), UIUC (2009)] developed by UIUC is recommended. The geotechnical engineer can also use other appropriate program, such as any of the SHAKE family programs.

Generating the Site-Specific Design Response Spectrum

Depending on the ground condition, at least 9 acceleration time histories at the bottom of footing should be obtained from site specific ground response analysis. If the depth of bedrock is assumed, then at least 27 time-histories should be obtained. The corresponding response spectrum (5% critical damping) of each acceleration time history should be obtained and the design response spectrum should be taken as the envelope of these spectra.

The owner can decide whether a peer review is necessary for the site-specific analysis. If peer review is not done, a two-third rule must be used in the final design spectrum:

the site-specific design response spectrum should at least be 2/3 of the generic design response spectrum in the region of 0.5TF to 2TF of the spectrum where TF is the bridge fundamental period. The generic response design spectrum for Site Class F should be obtained as per guidelines in subsections on “Generation of Response Spectrum for Standard Bridges” and “Generation of Response Spectrum for Critical Bridges”, assuming that the soil site is Site Class D. The generic response spectrum should be determined as per section “Recommendation on seismic design according to SDC”.

A detailed procedure to use SIMQKE and DEEPSOIL for site specific analysis is presented in Appendix III. However, it is the responsibility of the geotechnical engineer to prepare input to these programs and interpret results according to the principles and the procedure described in this section. In this recommended procedure, the duration of ground motion is assumed to be 20 seconds, which was obtained from the duration of ground motion on very hard rock (VHR) for New York City. The geotechnical engineers can use this duration, which is believe to be conservative for New Jersey, or can estimate it based on state-of-the-art in seismic hazard analysis [e.g. Kempton and Stewart (2006)].

Analysis of Liquefaction Potential

If saturated granular soil exists at the site, its liquefaction potential must be screened according to the procedure described in Appendix III. Alternatively, the analysis can also be conducted using procedures based on soil parameters other than standard penetration test (SPT) blow counts. In that case, the procedure described in Youd et al. (2001) must be followed.

While conducting the analysis of liquefaction potential for a site requiring site-specific analysis, the maximum ground surface acceleration a_{max} should be obtained from the ground response analysis assuming that liquefaction does not occur. It can be taken as the spectral value of the site specific response spectrum at period T = 0, as illustrated in Figure 4.9 below.

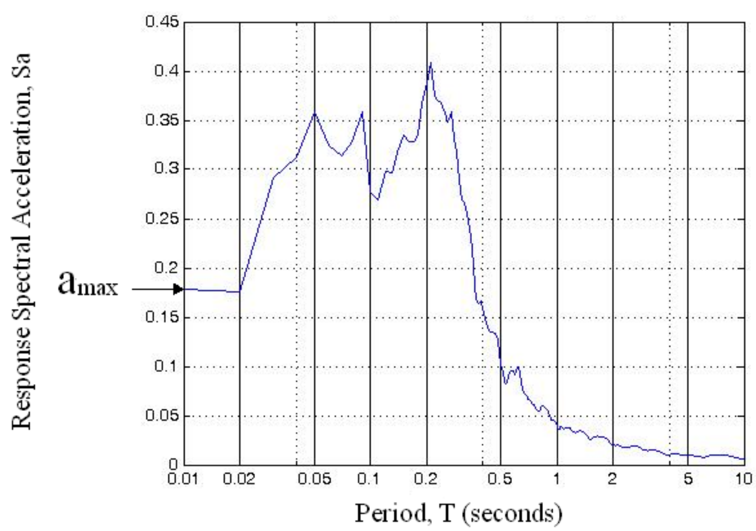


Figure 4.9 Maximum Ground Surface Acceleration a_{max} from Site-Specific Response Spectrum.

CHAPTER 5: EXAMPLES ON DESIGN OF NEW BRIDGES USING AASHTO SEISMIC GUIDE SPECIFICATIONS FOR LRFD BRIDGE SEISMIC DESIGN

Introduction

Based on discussions with New Jersey Department of Transportation, nine examples of bridges have been considered to illustrate step-by-step design of new bridges in New Jersey based on the 2008 AASHTO Seismic Guide Specifications. These examples and their seismic design categories are:

- Example 1: Design of a single span steel bridge in SDC A Category
- Example 2: Design of a single span steel bridge in SDC B Category.
- Example 3: Design of a Two-Span Steel Bridge in SDC B Category.
- Example 4: Design of a Three-Span Steel Bridge in SDC A Category.
- Example 5: Design of a Three-Span Steel Bridge in SDC B Category.
- Example 6: Design of a Single span Concrete bridge in SDC A Category.
- Example 7: Design of a Single span Concrete bridge in SDC B Category.
- Example 8: Design of Six-Span Concrete Bridge in SDC B Category.
- Example 9: Design of a nine-span Concrete Bridge in SDC B Category.

Seismic design has been illustrated by considering examples of existing bridges to eliminate the work related to sizing of bridge components for other loads, e.g., dead load, live load, etc. Supplementary information for these examples has been presented in different appendices in Vol. 2 of this report.

It should be noted that the examples of bridges are based on existing bridges in New Jersey. As built drawings of these bridges are based on on older versions of AASHTO code. Still, it has been observed that a majority of these bridges satisfy sesismic guidelines as per AASHTO-SGS [AASHTO (2008)].

Example 1: Design of a Single Span Steel Bridge in SDC A Category

Bridge Description

This example is based on single span steel bridge carrying Interstate Route 80 Westbound over Edwards Rd, Morris County, Structure Number 1415-151. The bridge is a single girder span supported by seat abutments. Figures 5.1 and 5.2 show the General Plan and Elevation of the bridge. Figures 5.3 and 5.4 show the superstructure Framing Plan and Part Section thru Deck. Figure 5.5 shows the bearing connection details reflecting current practice.

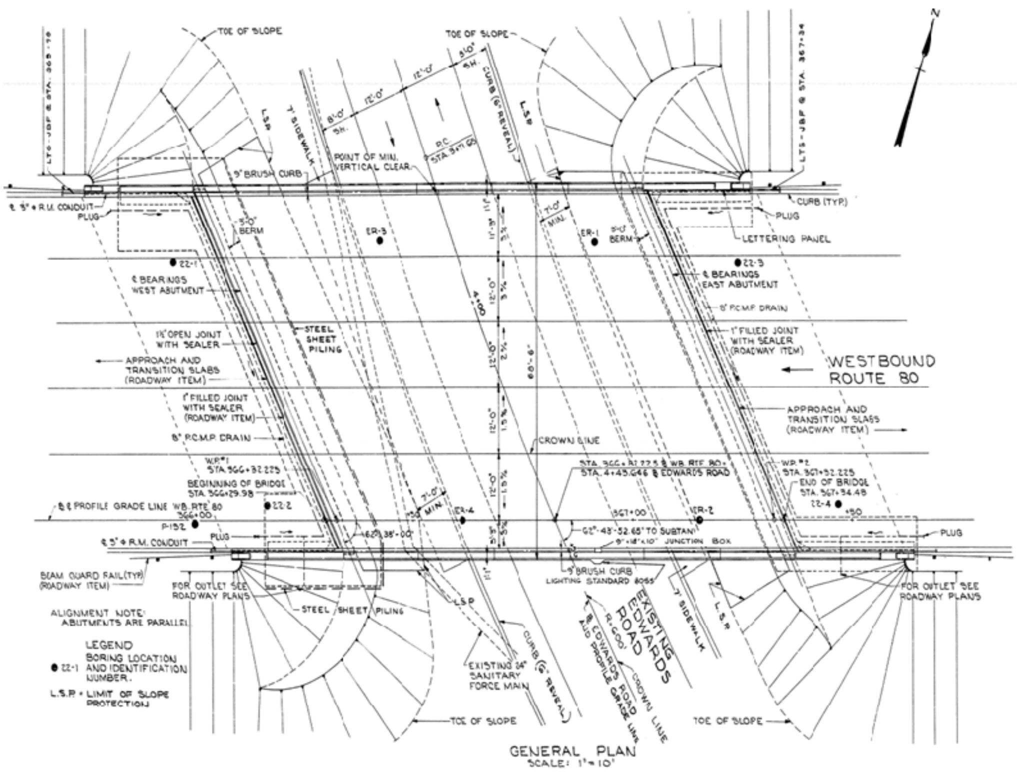


Figure 5.1 General Plan

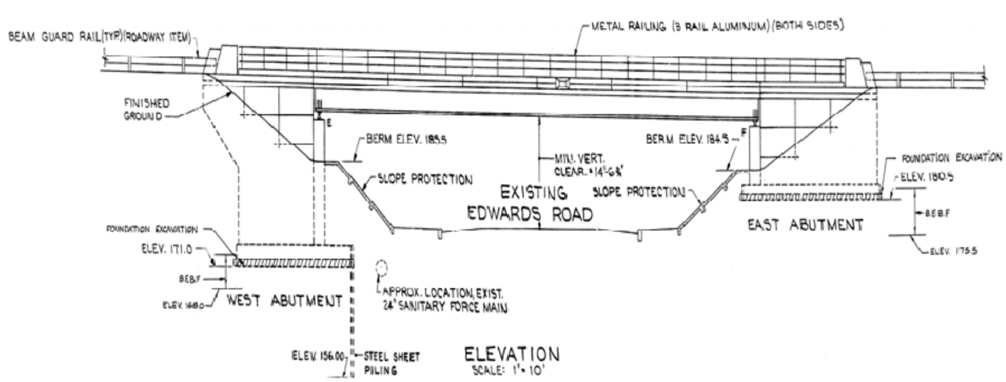


Figure 5.2 Elevation

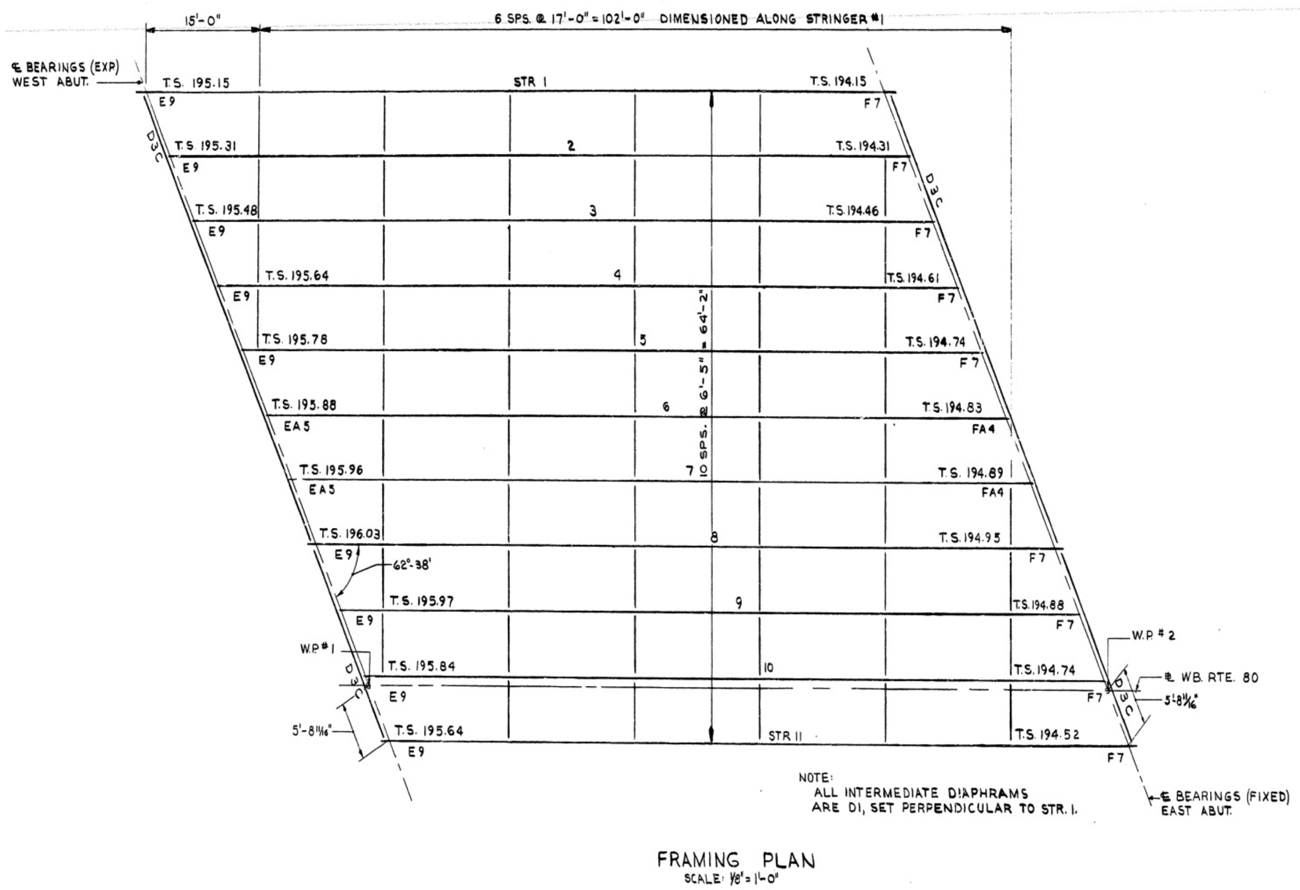


Figure 5.3 Framing Plan

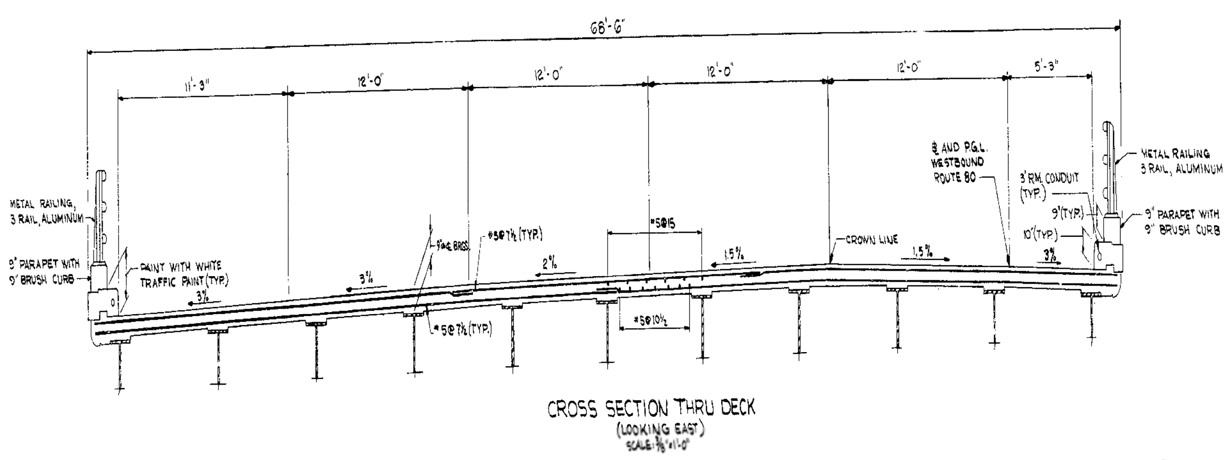


Figure 5.4 Part Section thru Deck

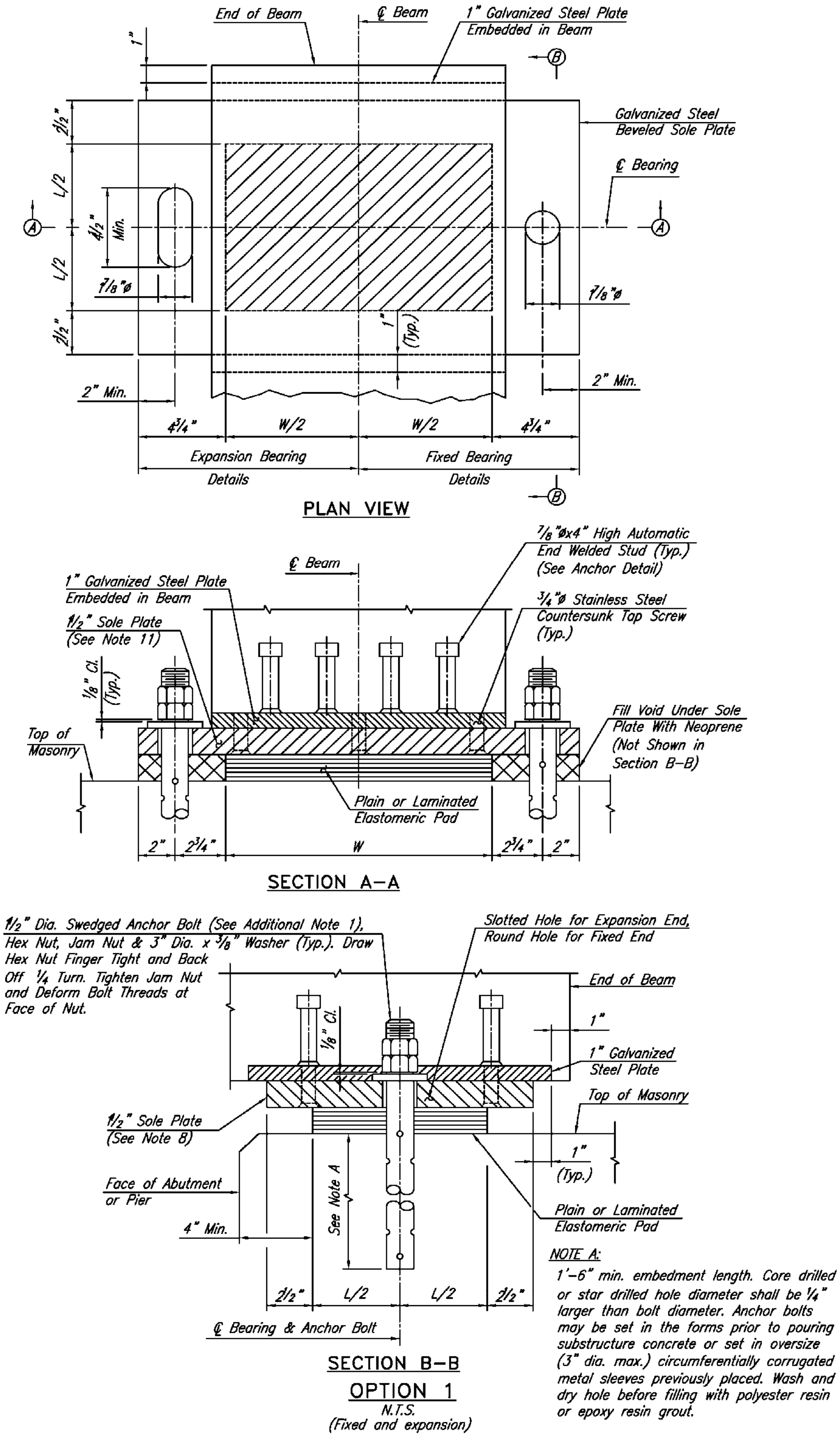
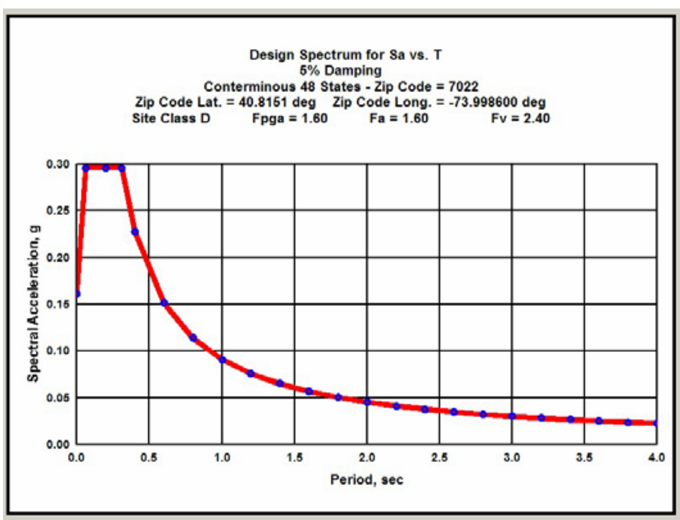


Figure 5.5 Bearing Connection Details

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.6. A site class D is considered for this example bridge. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.6 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D

PGA = 1.5 × 0.16 = 0.24
S_{DS} = 1.5 × 0.3 = 0.45
S_{D1} = 1.5 × 0.09 = 0.14

Flow Charts

The Guide Specifications were developed to allow three Global Seismic Design. Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 1 reflects a Type 3 bridge system with the bearing connections considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.7 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a single span bridge versus a multi-span bridge.

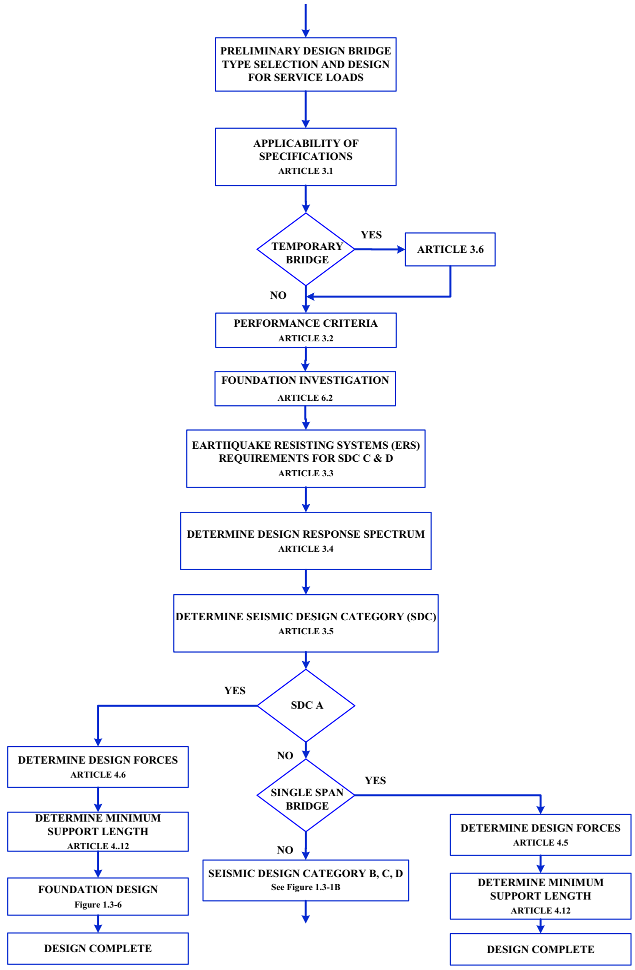


Figure 5.7 Seismic Design Procedure Flow Chart

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.1.

If liquefaction-induced lateral spreading or slope failure impacting the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

Table 5.1 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in the flowchart in Figure 5.8 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} =0.14, the example bridge is treated in SDC A with the following basic requirements:

- No Identification of ERS according to Article 3.3
- No Demand Analysis
- No Implicit Capacity Check Needed
- No Capacity Design Required
- Minimum detailing requirements for support length, superstructure/substructure connection design force, and column transverse steel
- No Liquefaction Evaluation Required

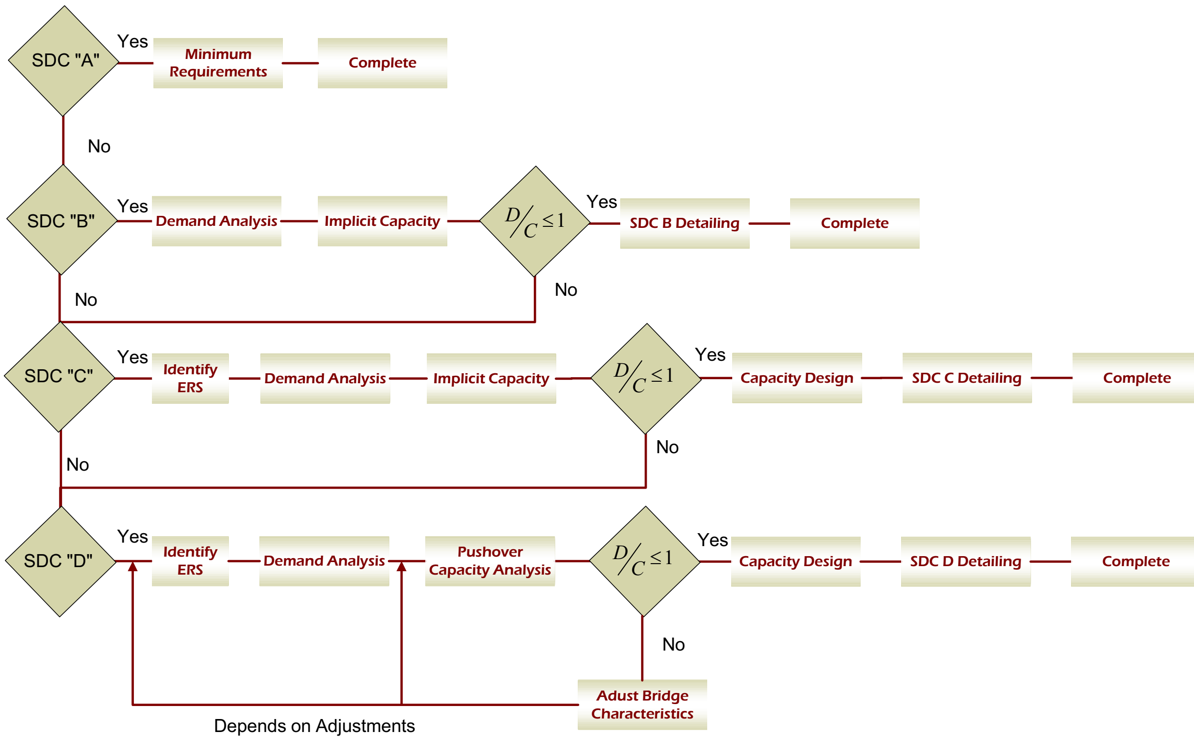


Figure 5.8 Seismic Design Category (SDC) Core Flowchart.

Seismic Analysis

Dead Load Calculation

Stringer Weight:

STRINGER SCHEDULE

STR NO.	MEMBER	LENGTH ℄ BRG. TO ℄ BRG.	COVER PLATE	MAX. SHEAR CONNECTOR SPACING 2 STUDS PER ROW (7/8"Φ)					CAMBER			
									DEAD LOAD DEFLECTION		VERTICAL CURVE @ 1/2 PT	TOTAL @ 1/2 PT
				PANEL I	PANEL II	PANEL III	PANEL IV	PANEL V	1/4 PT	1/2 PT		
1 & 11	36 WF 245	100'-0"	14×1 1/8×98'-8"	9"	10"	12"	14"	17"	2 3/4"	4"	9/16"	4 5/16"
2 THRU 10	36 WF 245	100'-0"	14×1 1/8×71'-0"	8"	9"	11 1/2"	13"	16"	2 3/4"	4"	9/16"	4 5/16"

Stingers 1 & 11: 0.245 × 102 = 25 Kips (Stringer Wt= unit weight×length)

Cover Plate: $\left(\dfrac{14 \times 1\frac{1}{8}}{144} \times 98 \right) \times 0.49 = 5.3\,\text{Kips}$ (Cover Plate Wt=volume×volumetric wt)

Subtotal 25.0 + 5.3 = 30 Kips

Stingers 2 through 10: 0.245×102=25 Kips (Stringer Wt=unit weight×length)

Cover Plate: $\left(\dfrac{14\times 1\frac{1}{8}}{144}\times 71\right)\times 0.49 = 3.8\,\text{Kip}$ (Cover Plate Wt=volume×volumetric Wt)

Subtotal: 25.0 + 3.8 = 29 Kip

Steel Superstructure Weight:

2×30+9×29=321 Kips

Slab Weight:

$$\left(\frac{8}{12}\times 68.5\times 102\right)\times 0.15 = 700\,\text{Kips}$$ (Slab Wt=volume×volumetric weight)

Overlay Weight: (overlay height $1\frac{1}{2}''$, Calculate as 2")

$$\left(\frac{2}{12}\times 68.5\times 102\right)\times 0.12 = 140\,\text{Kips}$$ Overlay Wt=volume×volumetric weight)

Superstructure Quantities: (As-built)

SUPERSTRUCTURE QUANTITIES			
ITEM	UNIT	QUANITY	As Built
CLASS B CONCRETE IN STRUCTURES, SUPERSTRUCTURE	C.Y.	208	213
REINFORCEMENT STEEL IN STRUCTURES	LBS.	45,101	44,471
PREFORMED ELASTIC JOINT SEALER-(4¾" DEEP)	L.F.	80	76
STRUCTURAL STEEL	LBS.	353,904	355,682
SHEAR CONNECTORS	UNITS	2,452	2,498
METAL RAILING (3-RAIL, ALUMINUM)	L.F.	272	273
3" RIGID METALLIC CONDUIT	L.F.	200	See Const. Book #52

Concrete (213 Cubic Yard):

(213×27)×0.15=863 Kips

(Concrete Wt=volume×volumetric weight)

Structural Steel: 356 Kips
Connectors: 3 Kips
Railing: 80 Kips
Subtotal: 356 + 3 + 80 = 440 Kips

Hence, total weight of superstructure is calculated as:

Concrete: 863 Kips
Structural Steel: 440 Kips
Overlay: 140 Kips

Total: 863 + 440 + 140 = 1450 Kips

Design Requirements for Single Span Bridges, SDC A

According to section 4.5 of AASHTO-SGS

- A detailed seismic analysis shall not be deemed to be required for single span bridges regardless of SDC as specified in Article 4.1.
- The connections between the bridge span and the abutments shall be designed in both longitudinal and transverse directions to resist a horizontal seismic force not less than the effective peak ground acceleration coefficient, As, as specified in Article 3.4, times the tributary permanent load except as modified for SDC A in Article 4.6.
- The minimum support lengths shall be as specified in Article 4.12.

Bridge Bearing Connections

According to Section 4.6 of the AASHTO-SGS, for bridges in SDC A, where the acceleration coefficient, A_s, as specified in Article 3.4., is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, assumed to exist during an earthquake.

The NJ PGA calculated in the Site Seismicity Section is shown equal to 0.24g. Therefore, the horizontal design connection force is considered at the minimum of 0.25g mentioned above.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Considering simply supported 11 stingers, the tributary permanent load per connection is calculated as:

$$\left(\frac{1450}{2}\right)\Big/11 = 66 \text{ Kips}$$

According to AASHTO-SGS Section 8.13.3, the principal tensile stress specified as $0.11\sqrt{f_c'}$ is used, where f_c' is the nominal concrete compressive strength (ksi).

The principal tensile stress of $0.11\sqrt{f_c'}$ corresponds to minimal concrete cracking and no yielding of reinforcement associated with the crack opening of concrete in the anchorage connection of the bearing.

Connection Lateral Load Demand (As described above according to AASHTO-SGS Sections 4.5 and 4.6) = 66×0.25 = 17 Kips.

Tensile stress in concrete (Corresponding to minimal damage of the bearing connection) = $0.11\sqrt{4}$ = 0.22 Ksi

Shear failure plane area for Seat Pull-out (as shown in Figure 5.9) = $\left[(5.25\times 2)\times\sqrt{2}\right]\times 18''$ = 267 in^2.

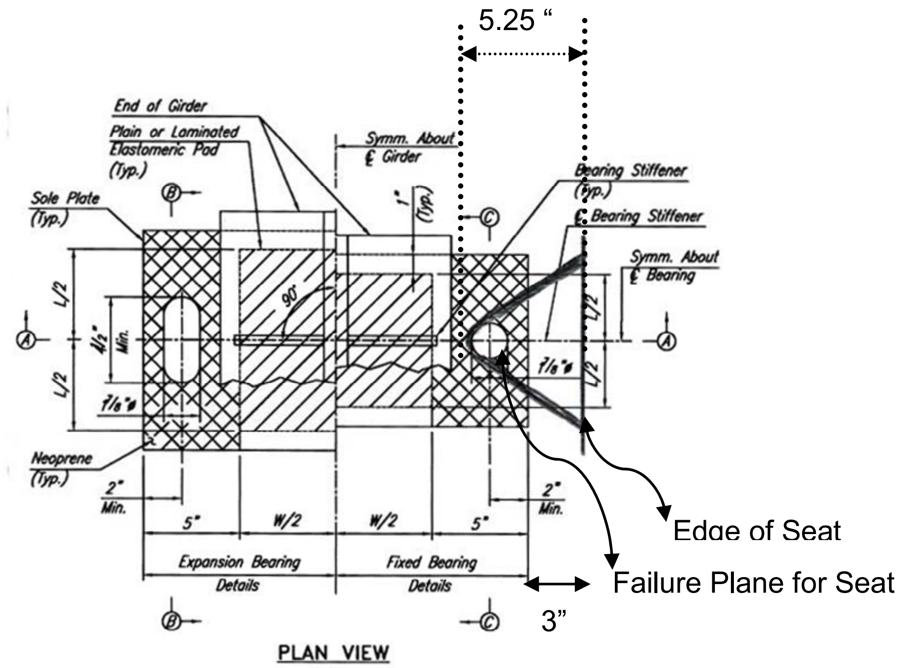


Figure 5.9 Anchor Bolt Shear Failure Plane (Connection Details Not Applicable See Figure 5.5)

In calculating the seat pull out area, 18” is the embedment length of the bolt. This calculation is performed to show that concrete pull out doesn’t govern. It is just a check to confirm that the bolt capacity is the focus in determining the strength of the connection.

Pull-out Capacity per Bolt: = Shear failure plane area × tensile stress in concrete = 267 × 0.22 = 59 Kips

Consider 1” ϕ bolt:

According to AASHTO-SGS section 6.13: $R_n = 0.48A_bF_{ub}N_s$

R_n = 0.48×0.785×60 = 22.6 Kips

$\phi_s R_n$ = 0.65×22.6 = 14.7 Kips (A307 bolts in shear ϕ_s = 0.65)

For 1" ϕ bolt (See Experimental Testing of Anchor Bolts in Appendix IV.A)
P_{crack} = 13.7 Kips @ Δ_{crack} = 0.96"

Consider Capacity @ 13.7 Kips based on Testing, considering Minimal Damage Requirement.

Connection Capacity Considering 2 bolts = 2×13.7 Kips = 27.4 Kips > 17 Kips, where 17 kips is the connection lateral load demand. (O.K.)

Consider $\frac{3}{4}^{"}\phi$ bolt:

R_n = 0.48×0.44×60 = 12.7 Kips

$\phi_s R_n$ = 0.65×12.7 = 8.2 Kips

Connection Capacity = 2×8.2 Kips = 16.4 Kips < 17 Kips (Marginally O.K.)

Hence, use minimum $\frac{3}{4}^{"}\phi$ bolts at the bearing connection.

Check minimum support length

Figures 5.10 and 5.11 show a typical abutment section and the corresponding seating detail.

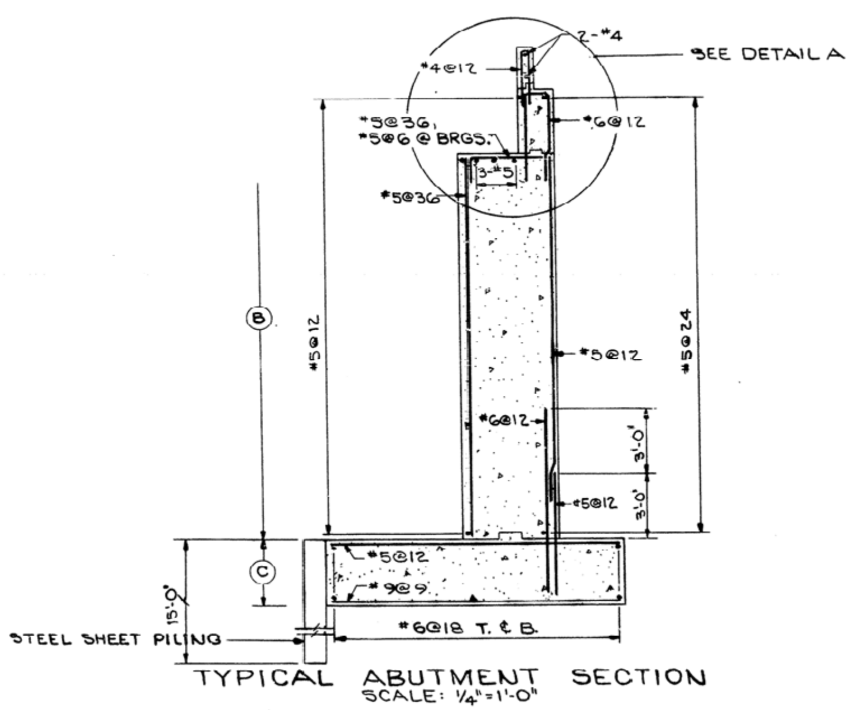


Figure 5.10 Typical Abutment Section

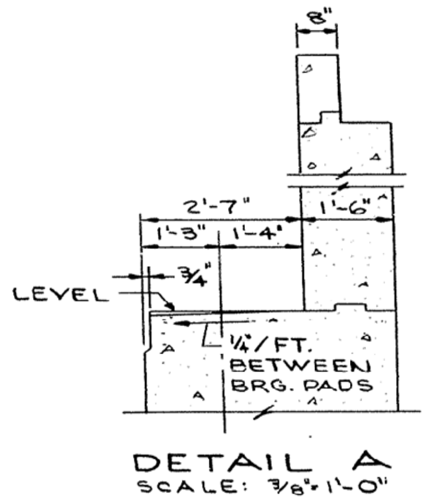


Figure 5.11 Details A of Typical Abutment Section

According to AASHTO-SGS Section 4.12.2, support lengths at expansion bearings without STU's or dampers for Seismic Design Categories A, B, and C shall be designed to accommodate the greater of (i) the maximum calculated displacement, except for bridges in SDC A, (ii) a percentage of the empirical support length, N, given by

$$N = (8+0.02L+0.08H)(1+0.000125S^2)$$

Where,

N = Minimum support length measured normal to the centerline of bearing (in.)

L = Length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (ft.)

H = For abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.) for columns and/or piers, column, or pier height (ft.); for hinges within a span, average height of the adjacent two columns or piers (ft.) 0.0 for single-span bridges (ft.)

S = Angle of skew of support measured from a line normal to span (°)

The percentage of N, applicable to each SDC, shall be calculated as per Table 5.2 below. (For example, for SDC A with As < 0.05, support length shall be calculated to be the greater of (i) the maximum calculated displacement, and (ii) 0.75N).

Table 5.2 Percentage N by SDC and effective peak ground acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percentage of N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC A:
$N = 1.0\ (8+0.02L+0.08H)(1+0.000125S^2)$

Roadway Elevation @ West Abutment (See Figure 5.12): 196.6'

Superstructure Depth (Stringer Depth + Deck Depth, See Dead Load Calculation):
= 36"+8"=3.6'

Bottom of Girder Elevation: 196.6'-3.6'=193'

Bottom of West Abutment Foundation (see Figure 5.2): 171'

Height of West Abutment: H=193'-171'=22'

For Single Span Bridges, H = 0.

Length of Bridge Deck (See Fig. 5.15): L=102'

Angle of Skew of Support (see Fig. 5.3): S=27.3°

$N=1.0(8+0.02\times102'+0.08\times0')(1+0.000125\times27.3^2) = 11"$

Available Seat Width: 2'-7" or 31" (See Figure 5.11 Detail A)

Available Seat Length: 31"-1" joint = 30". Available Seat greater than required support length N (O.K.).

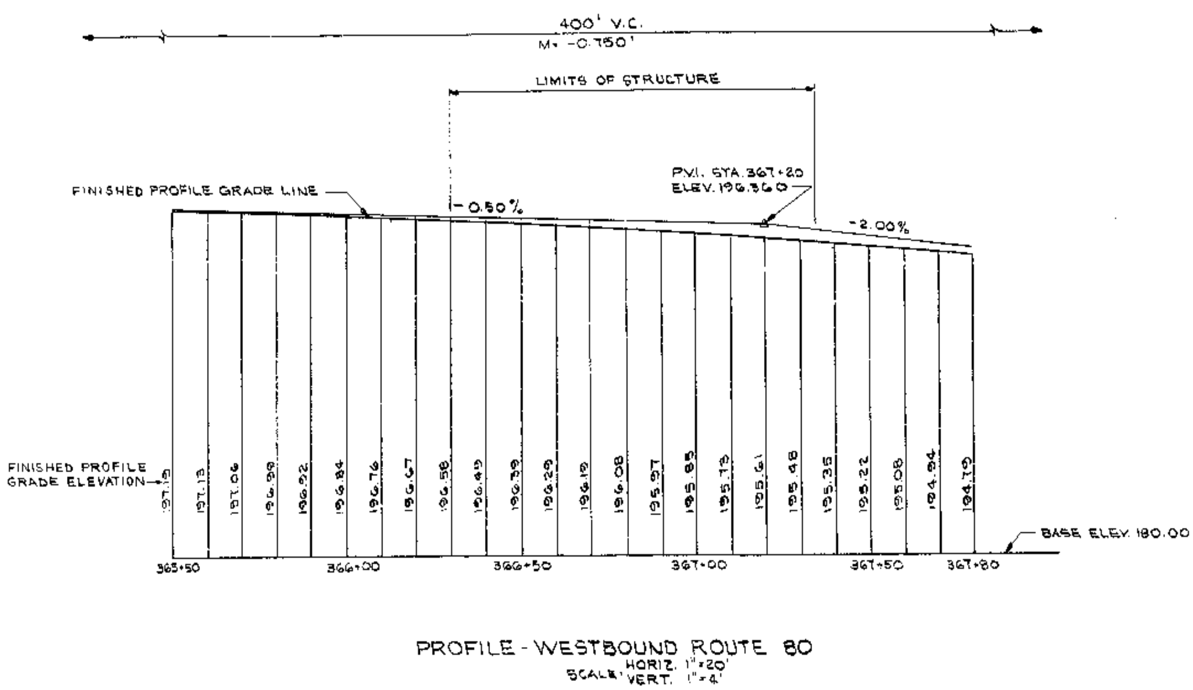


Figure 5.12 Roadway Profile

Example 2: Design of a Single Span Steel Bridge in SDC B Category

Bridge Description

This example is based on single span steel bridge carrying Interstate Route 80 Westbound over Edwards Rd, Morris County, Structure Number 1415-151. The bridge is a single girder span supported by seat abutments. Figures 5.13 and 5.14 show the General Plan and Elevation of the bridge. Figures 5.15 and 5.16 show the superstructure Framing Plan and Part Section thru Deck. Figure 5.17 shows the bearing connection details reflecting current practice.

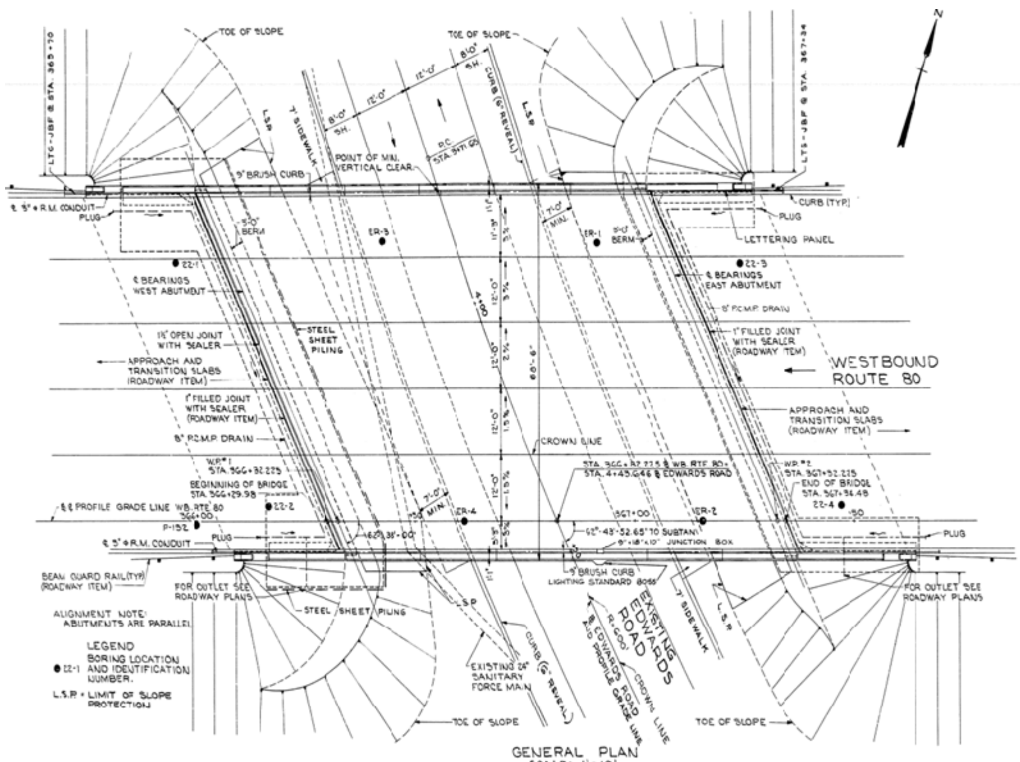


Figure 5.13 General Plan

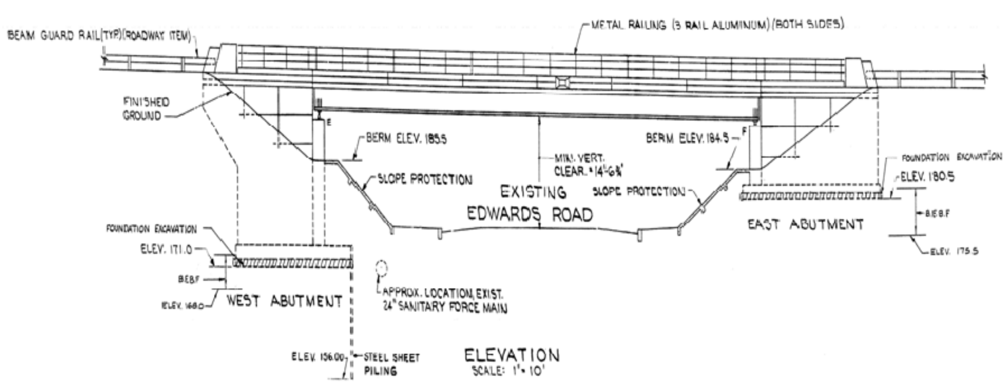


Figure 5.14 Elevation

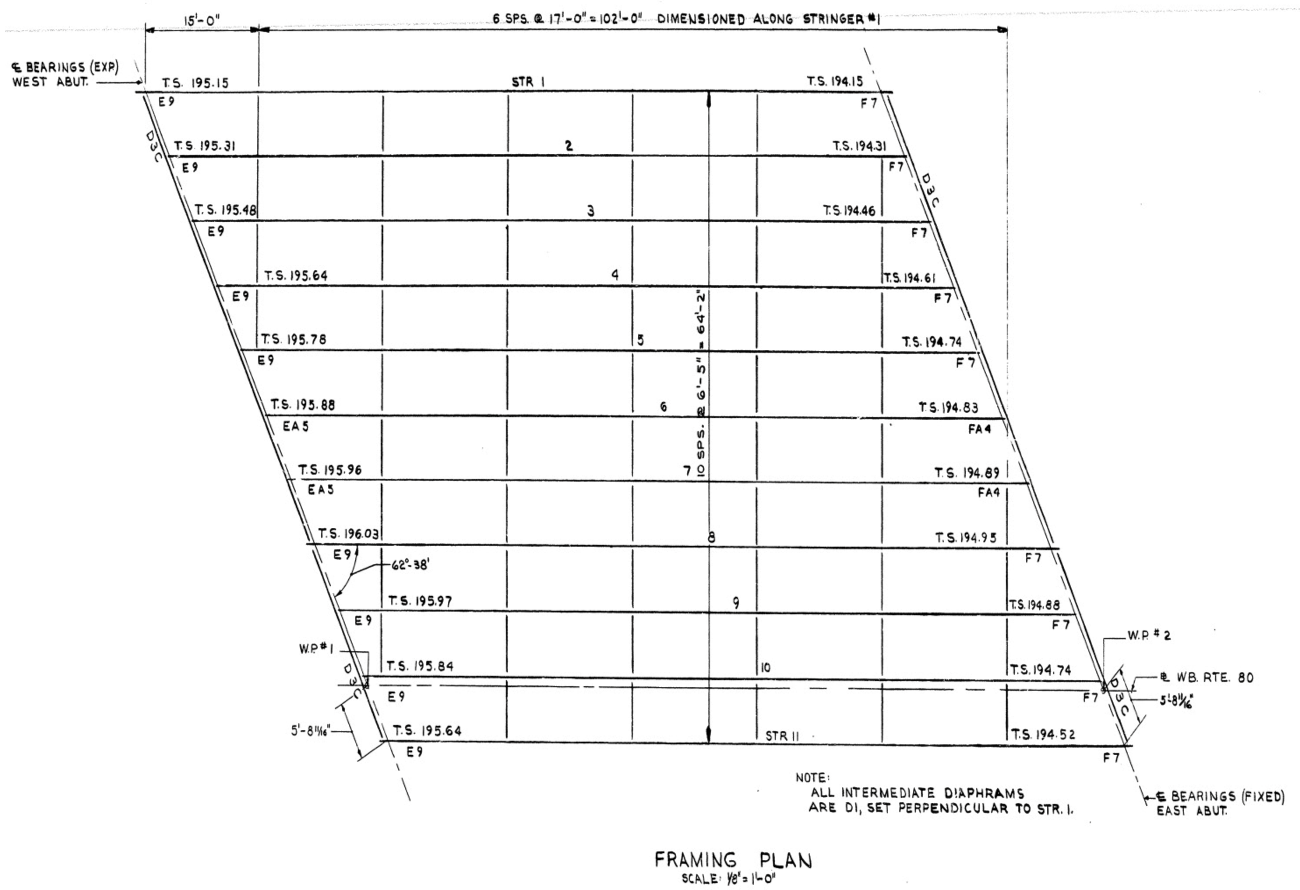


Figure 5.15 Framing Plan

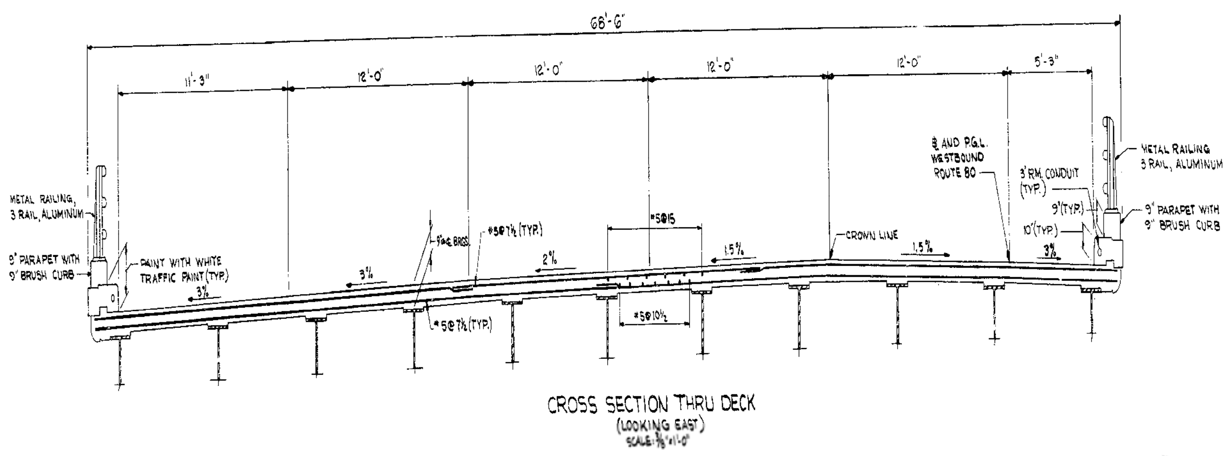


Figure 5.16 Part Section thru Deck

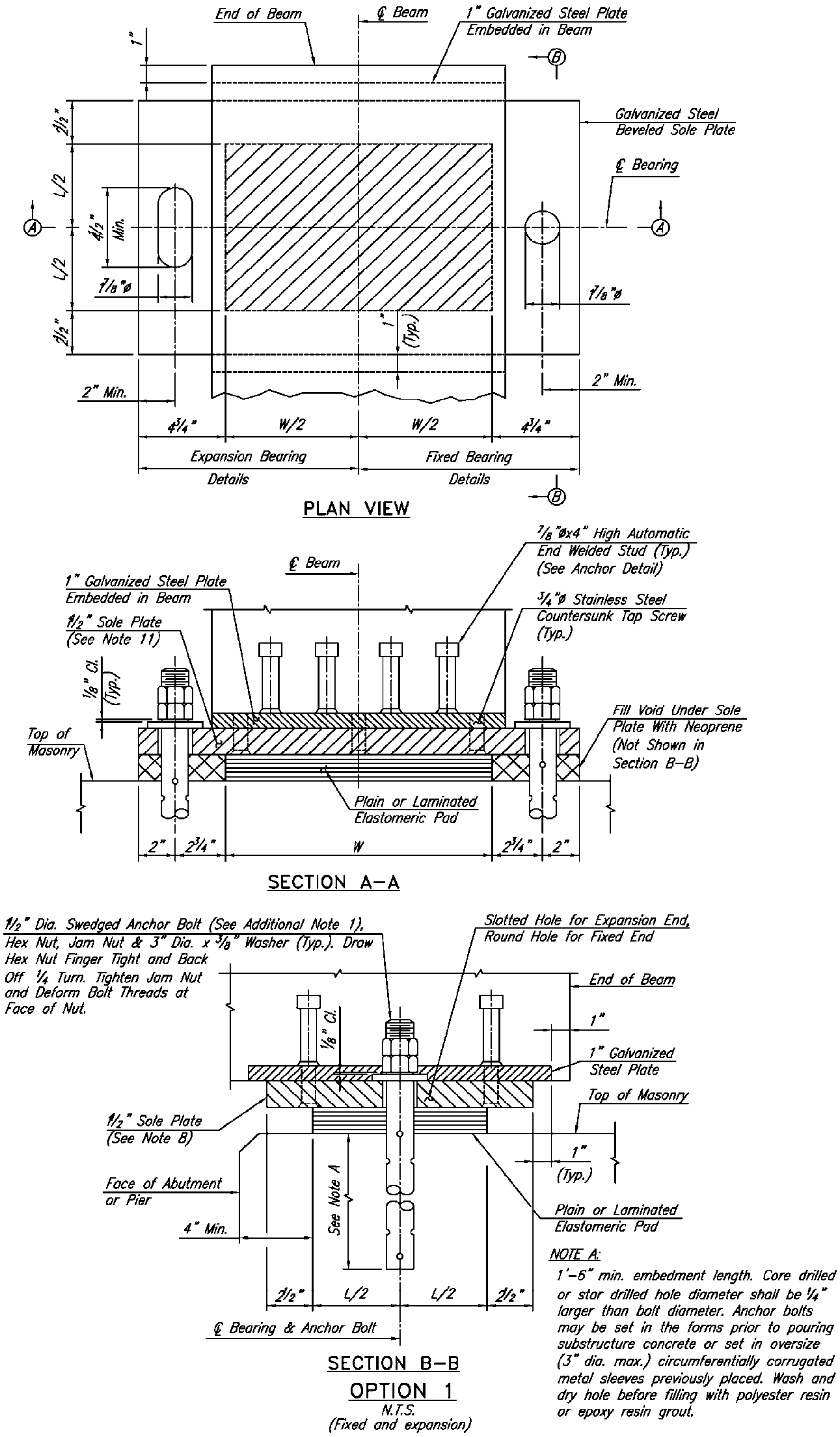
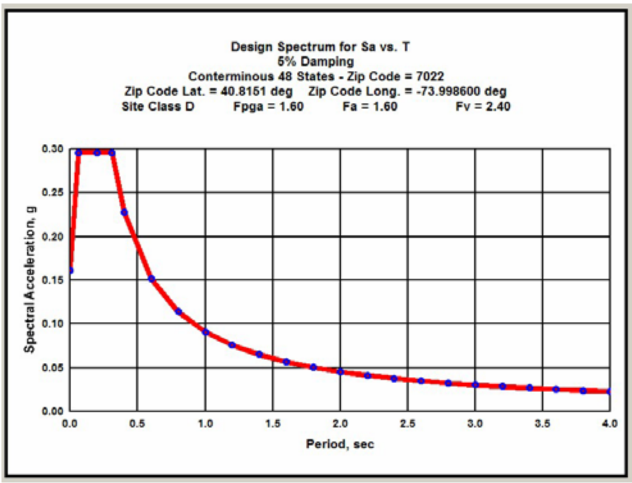


Figure 5.17 Bearing Connection Details

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.18. A site class D is considered for this example bridge. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.18 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D

PGA = 1.5 × 0.16 = 0.24
S_{DS} = 1.5 × 0.3 = 0.45
S_{D1} = 1.5 × 0.09 = 0.14

Flow Charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 1 reflects a Type 3 bridge system with the bearing connections considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.19 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a single span bridge versus a multi-span bridge.

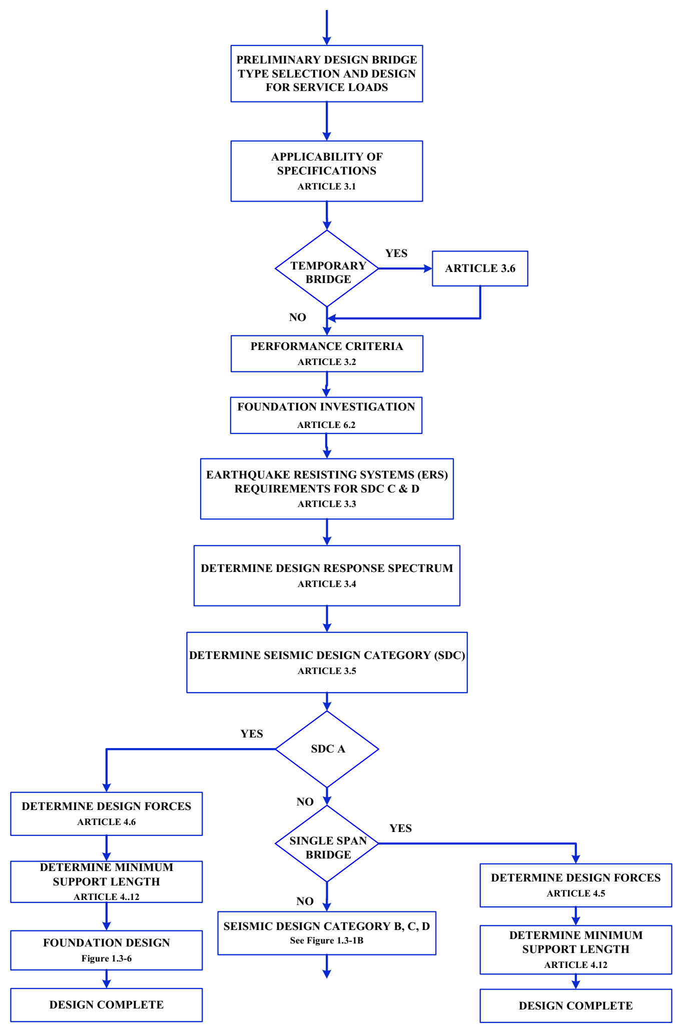


Figure 5.19 Seismic Design Procedure Flow Chart

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.3.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}

Table 5.3 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in the flowchart Figure 5.20 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} =0.14, the example bridge is treated in SDC B with the following basic requirements:

- Identification of ERS according to Article 3.3 should be considered
- Demand Analysis
- Implicit Capacity Check Required (displacement, P-Δ, support length)
- Capacity Design should be considered for column shear; capacity checks should be considered to avoid weak links in the ERS
- SDC B Level of Detailing
- Liquefaction check should be considered for certain conditions

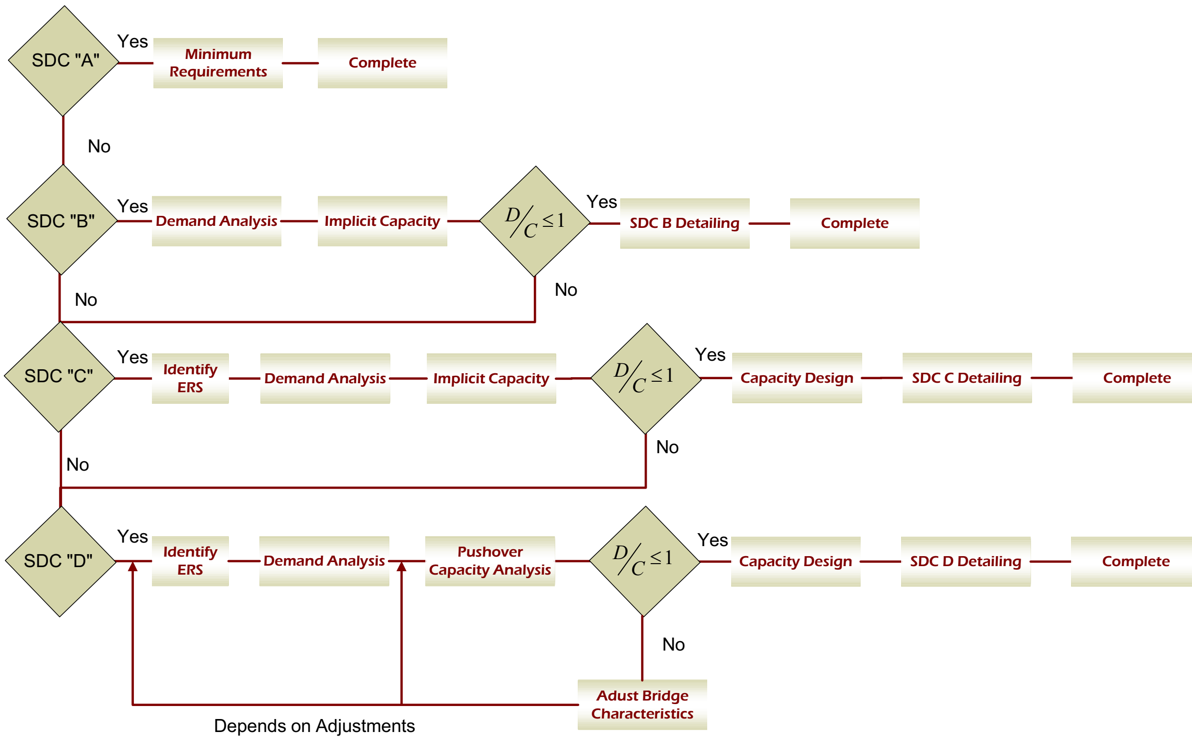


Figure 5.20 Seismic Design Category (SDC) Core Flowchart.

Seismic Analysis

Dead Load Calculation

Stringer Weight:

STRINGER SCHEDULE

STR NO	MEMBER	LENGTH ℄ BRG. TO ℄ BRG	COVER PLATE	MAX. SHEAR CONNECTOR SPACING 2 STUDS PER ROW (7/8"Φ)					CAMBER			
									DEAD LOAD DEFLECTION		VERTICAL CURVE @ 1/2 PT	TOTAL @ 1/2 PT
				PANEL I	PANEL II	PANEL III	PANEL IV	PANEL V	1/4 PT	1/2 PT		
1 & 11	36 WF 245	100'-0"	14 × 1 1/8 × 98'-8"	9"	10"	12"	14"	17"	2 3/4"	4"	9/16"	4 9/16"
2 THRU 10	36 WF 245	100'-0"	14 × 1 1/8 × 71'-0"	8"	9"	11 1/2"	13"	16"	2 3/4"	4"	9/16"	4 9/16"

Stingers 1 & 11: 0.245 × 102 = 25 Kips (Stringer Wt= unit weight×length)

Cover Plate: $\left(\dfrac{14\times 1\frac{1}{8}}{144}\times 98\right)\times 0.49 = 5.3\text{ Kips}$ (Cover Plate Wt=volume×volumetric wt)

Subtotal 25.0 + 5.3 = 30 Kips

Stingers 2 through 10: 0.245×102=25 Kips (Stringer Wt=unit weight×length)

Cover Plate: $\left(\dfrac{14\times 1\frac{1}{8}}{144}\times 71\right)\times 0.49 = 3.8\,\text{Kip}$ (Cover Plate Wt=volume×volumetric Wt)

Subtotal: 25.0 + 3.8 = 29 Kip

Steel Superstructure Weight:

2×30+9×29=321 Kips

Slab Weight:

$$\left(\frac{8}{12}\times 68.5\times 102\right)\times 0.15 = 700\,\text{Kips}$$ (Slab Wt=volume×volumetric weight)

Overlay Weight: (overlay height $1\frac{1}{2}''$, Calculate as 2")

$$\left(\frac{2}{12}\times 68.5\times 102\right)\times 0.12 = 140\,\text{Kips}$$ Overlay Wt=volume×volumetric weight)

Superstructure Quantities: (As-built)

SUPERSTRUCTURE QUANTITIES			
ITEM	UNIT	QUANITY	As Built
CLASS B CONCRETE IN STRUCTURES, SUPERSTRUCTURE	C.Y.	208	213
REINFORCEMENT STEEL IN STRUCTURES	LBS.	45,101	44,971
PREFORMED ELASTIC JOINT SEALER-(4¾" DEEP)	L.F.	80	76
STRUCTURAL STEEL	LBS.	353,904	355,682
SHEAR CONNECTORS	UNITS	2,452	2,498
METAL RAILING (3-RAIL, ALUMINUM)	L.F.	272	273
3" RIGID METALLIC CONDUIT	L.F.	200	See Const. Book #52

Concrete (213 Cubic Yard):

(213×27)×0.15=863 Kips

(Concrete Wt=volume×volumetric weight)

Structural Steel:	356 Kips
Connectors:	3 Kips
Railing:	80 Kips
Subtotal: 356 + 3 + 80 =	440 Kips

Hence, total weight of superstructure is calculated as:

Concrete:	863 Kips
Structural Steel:	440 Kips
Overlay:	140 Kips

Total: 863 + 440 + 140 = 1450 Kips

Design Requirements for Single Span Bridges According to SDC B

According to section 4.5 of AASHTO-SGS

- A detailed seismic analysis shall not be deemed to be required for single span bridges regardless of SDC as specified in Article 4.1.
- The connections between the bridge span and the abutments shall be designed both longitudinally and transversely to resist a horizontal seismic force not less than the effective peak ground acceleration coefficient, A_s, as specified in Article 3.4, times the tributary permanent load except as modified for SDC A in Article 4.6.
- The lateral force shall be carried into the foundation in accordance with Articles 5.2 and 6.7.
- The minimum support lengths shall be as specified in Article 4.12.

Bridge Bearing Connections

According to Section 4.6 of the AASHTO-SGS, for bridges in SDC A, where the acceleration coefficient, A_s, as specified in Article 3.4., is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, assumed to exist during an earthquake.

The NJ PGA calculated in the Site Seismicity Section is shown equal to 0.24g. Therefore, the horizontal design connection force is considered at the minimum of 0.25g mentioned above.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Considering simply supported 11 stingers, the tributary permanent load per connection is calculated as:

$$\left(\frac{1450}{2}\right)\Big/11 = 66 \text{ Kips}$$

According to AASHTO-SGS Section 8.13.3, the principal tensile stress specified as $0.11\sqrt{f_c'}$ is used, where f_c' is the nominal concrete compressive strength (ksi).

The principal tensile stress of $0.11\sqrt{f_c^{'}}$ corresponds to minimal concrete cracking and no yielding of reinforcement associated with the crack opening of concrete in the anchorage connection of the bearing.

Connection Lateral Load Demand (As described above according to AASHTO-SGS Sections 4.5 and 4.6) = 66×0.25 = 17 Kips.

Tensile stress in concrete (Corresponding to minimal damage of the bearing connection) = $0.11\sqrt{4}$ = 0.22 Ksi

Shear failure plane area for Seat Pull-out (as shown in Figure 5.21) =
$\left[(5.25\times 2)\times\sqrt{2}\right]\times 18^{"}$ = 267 in^2

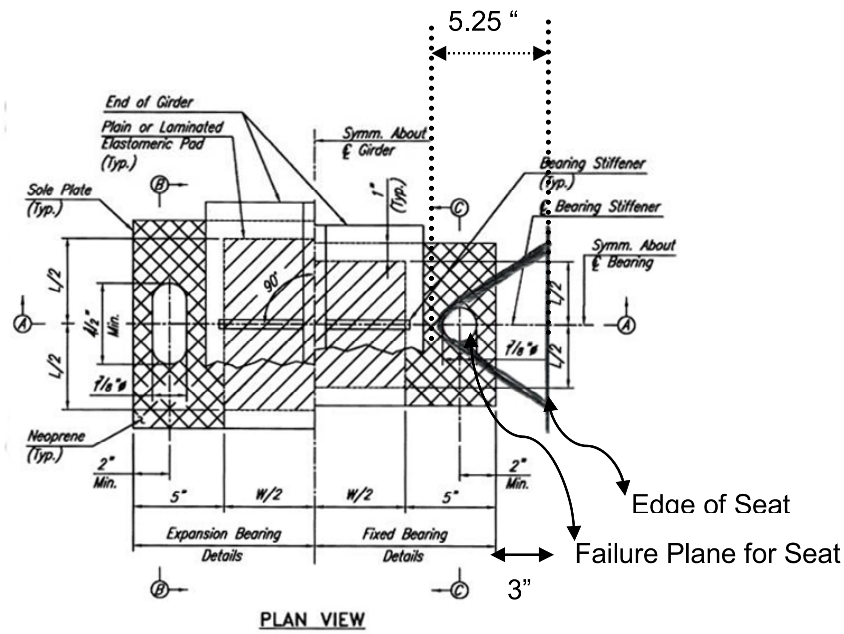


Figure 5.21 Anchor Bolt Shear Failure Plane (Connection Details Not Applicable, See Figure 5.17)

In calculating the seat pull out area, 18" is the embedment length of the bolt. This calculation is performed to show that concrete pull out doesn't govern. It is just a check to confirm that the bolt capacity is the focus in determining the strength of the connection.

Pull-out Capacity per Bolt: = shear failure plane area × tensile stress in concrete =

= 267 × 0.22 = 59 Kips

Consider 1" ϕ bolt:

According to AASHTO-SGS section 6.13: $R_n = 0.48A_bF_{ub}N_s$

R_n = 0.48×0.785×60 = 22.6 Kips

ϕ_sR_n = 0.65×22.6 = 14.7 Kips (A307 bolts in shear ϕ_s = 0.65)

For 1" ϕ bolt (See Experimental Testing of Anchor Bolts in Appendix IV.A)
P_{crack} = 13.7 Kips @ Δ_{crack} = 0.96"

Consider Capacity @ 13.7 Kips based on Testing, considering Minimal Damage Requirement.

Connection Capacity Considering 2 bolts = 2×13.7 Kips = 27.4 Kips > 17 Kips, where 17 kips is the connection lateral load demand. (O.K.)

Consider $\frac{3}{4}^{"}\phi$ bolt:

R_n = 0.48×0.44×60 = 12.7 Kips

$\phi_s R_n$ = 0.65×12.7 = 8.2 Kips

Connection Capacity = 2×8.2 Kips = 16.4 Kips < 17 Kips (Marginally O.K.)

Hence, use minimum $\frac{3}{4}^{"}\phi$ bolts at the bearing connection.

Abutment Lateral Load Path into the Foundation

According to AASHTO-SGS Sections 5.2 and 6.7, abutments in SDC B are expected to resist earthquake loads with minimal damage. For seat-type abutments, minimal abutment movement could be expected under dynamic passive pressure conditions. Testing at UCLA Report 2007/02 summarized in Appendix IV.B show that friction contribution is sufficient for satisfying SDC B requirement for lateral load path into the abutment foundation.

Check Minimum Support Length

Figures 5.22 and 5.23 show a typical abutment section and the corresponding seating details.

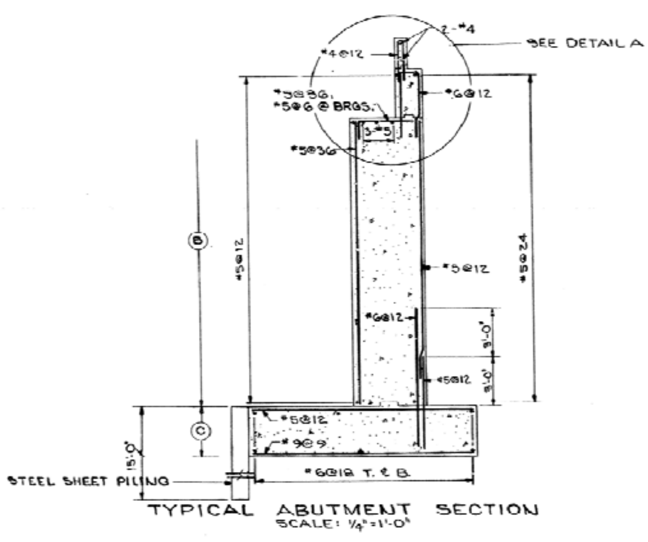


Figure 5.22 Typical Abutment Section

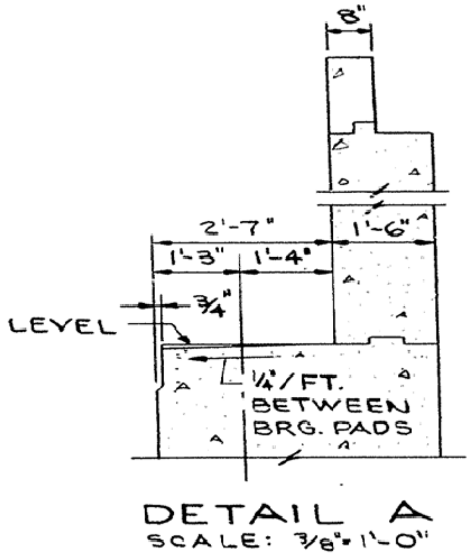


Figure 5.23 Detail A of Typical Abutment Section

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below. The percentage of N, applicable to each SDC, shall be as specified in Table 5.4 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

Where,

N = Minimum support length measured normal to the centerline of bearing (in.)

L = Length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (ft.)

H = For abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.) for columns and/or piers, column, or pier height (ft.); for hinges within a span, average height of the adjacent two columns or piers (ft.) 0.0 for single-span bridges (ft.)

S = Angle of skew of support measured from a line normal to span (°)

Table 5.4 Percentage N by SDC and effective peak ground acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percentage of N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC B:

$N = 1.5\ (8+0.02L+0.08H)(1+0.000125S^2)$

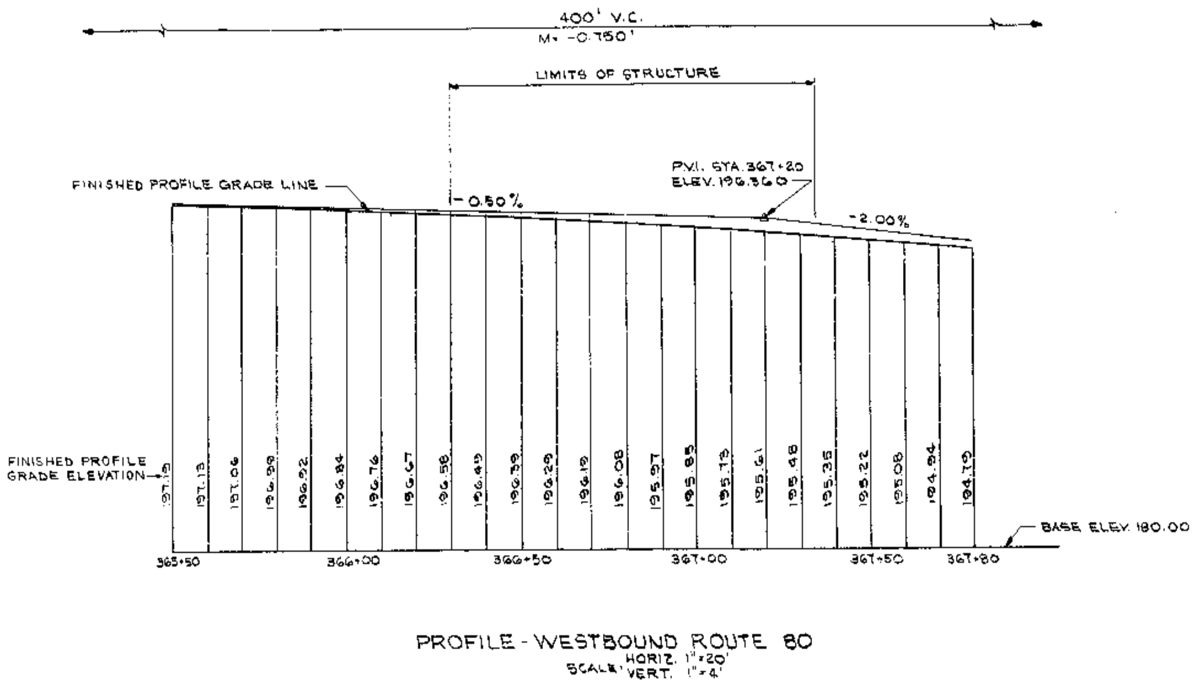


Figure 5.24 Roadway Profile

Roadway Elevation @ West Abutment (See Figure 5.24): 196.6'

Superstructure Depth (Stringer Depth + Deck Depth, See Dead Load Calculation):
= 36"+8"=3.6'

Bottom of Girder Elevation: 196.6'-3.6'=193'

Bottom of West Abutment Foundation (See Figure 5.2): 171'

Height of West Abutment: H=193-171=22'

For Single Span Bridges, H = 0.

Length of Bridge Deck (See Fig. 5.15): L=102'

Angle of Skew of Support (See Fig. 5.3): S=27.3°

$N=1.5(8+0.02\times102'+0.08\times0')(1+0.000125\times27.3^2) = 16.4"$

Available Seat Width: 2'7" or 31" (See Figure 5.23, Detail A)

Available Seat Length: 31"-1" joint =30". Hence, available seat is greater than the required support length N (OK).

Example 3: Design of a Two Span Steel Bridge in SDC B Category

Bridge Description

This example is based on a two-span steel bridge carrying Scotch Road over I-95, Structure No. 1120-153. The bridge is a two span continuous superstructure supported by monolithic abutments. Figures 5.25 and 5.26 show the General Plan and Elevation of the bridge, respectively. Figure 5.27 shows a typical selection at the bent location. Figures 5.28 and 5.29 show the superstructure Framing Plan and a typical girder elevation.

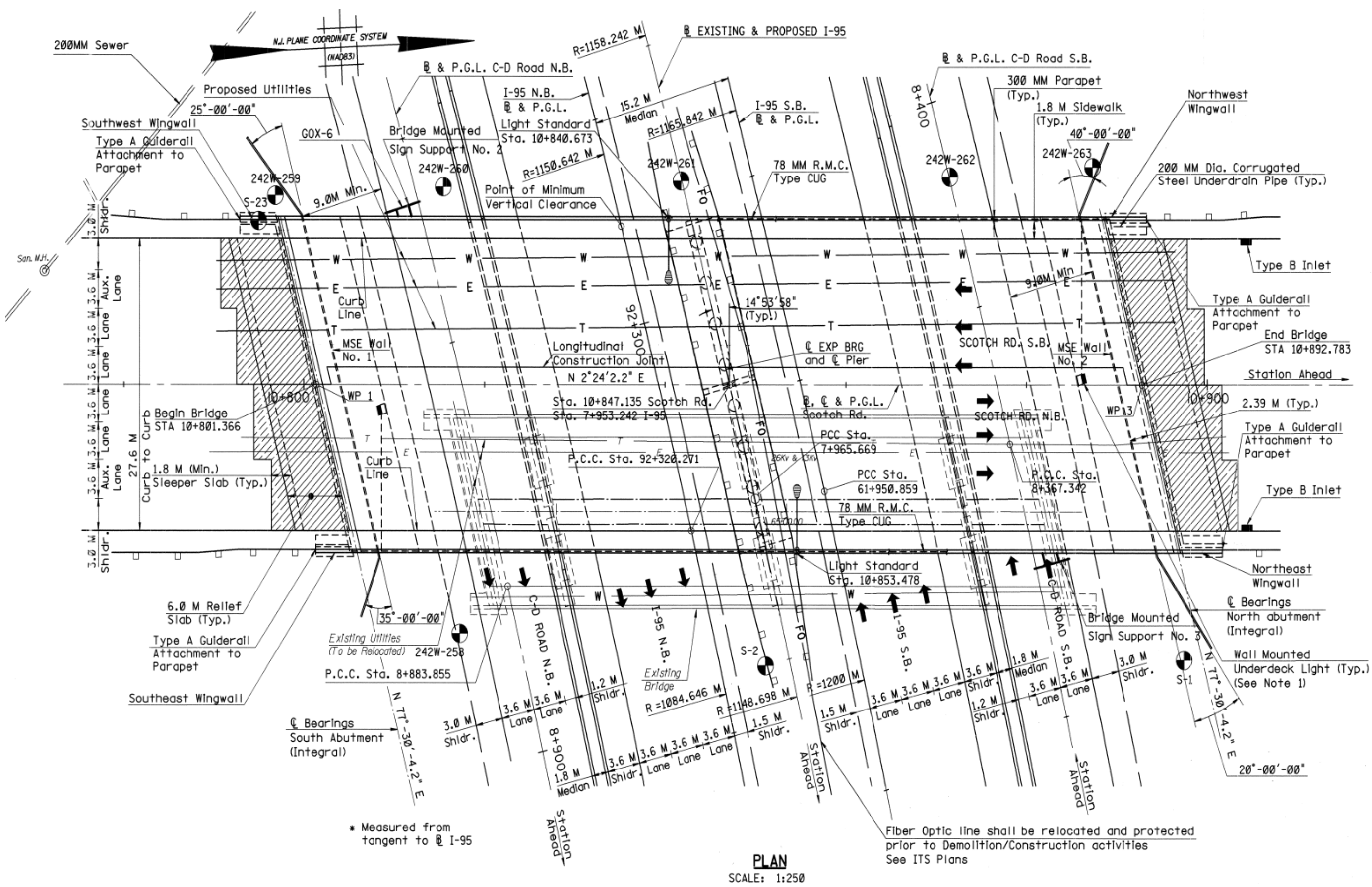


Figure 5.25 General Plan

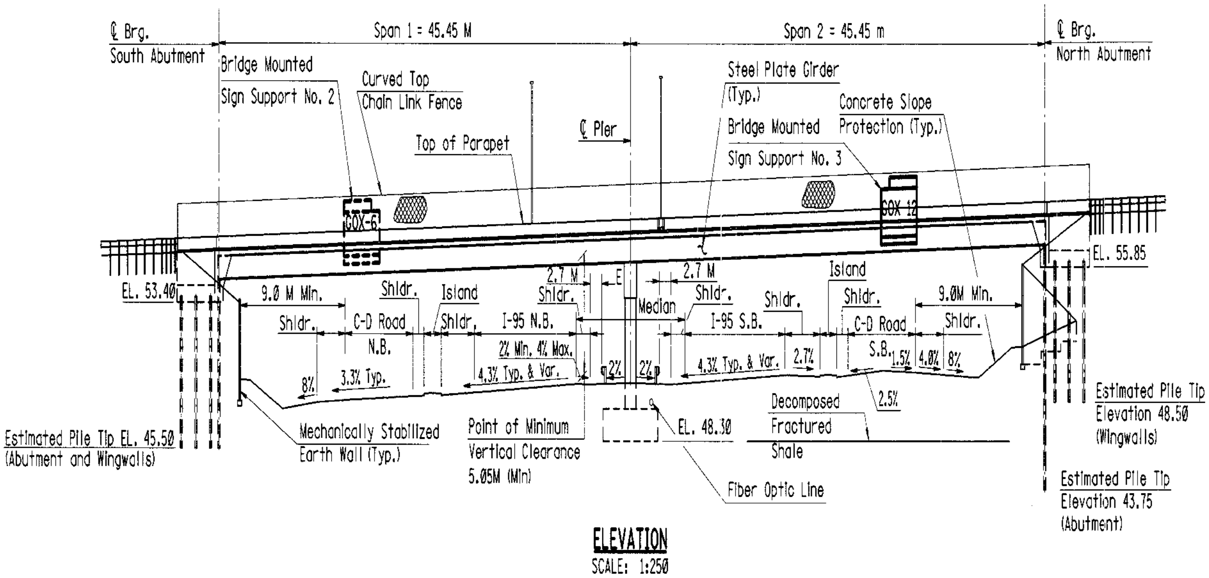


Figure 5.26 Elevation

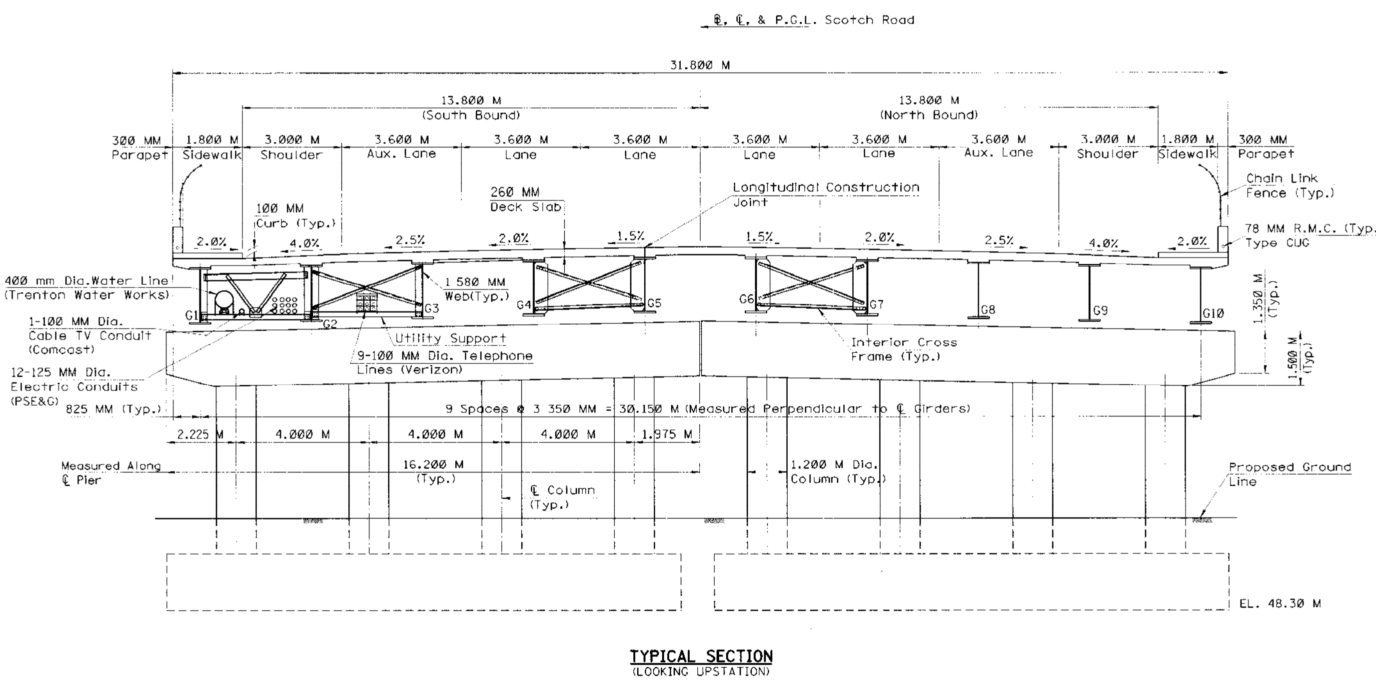


Figure 5.27 Typical Selection

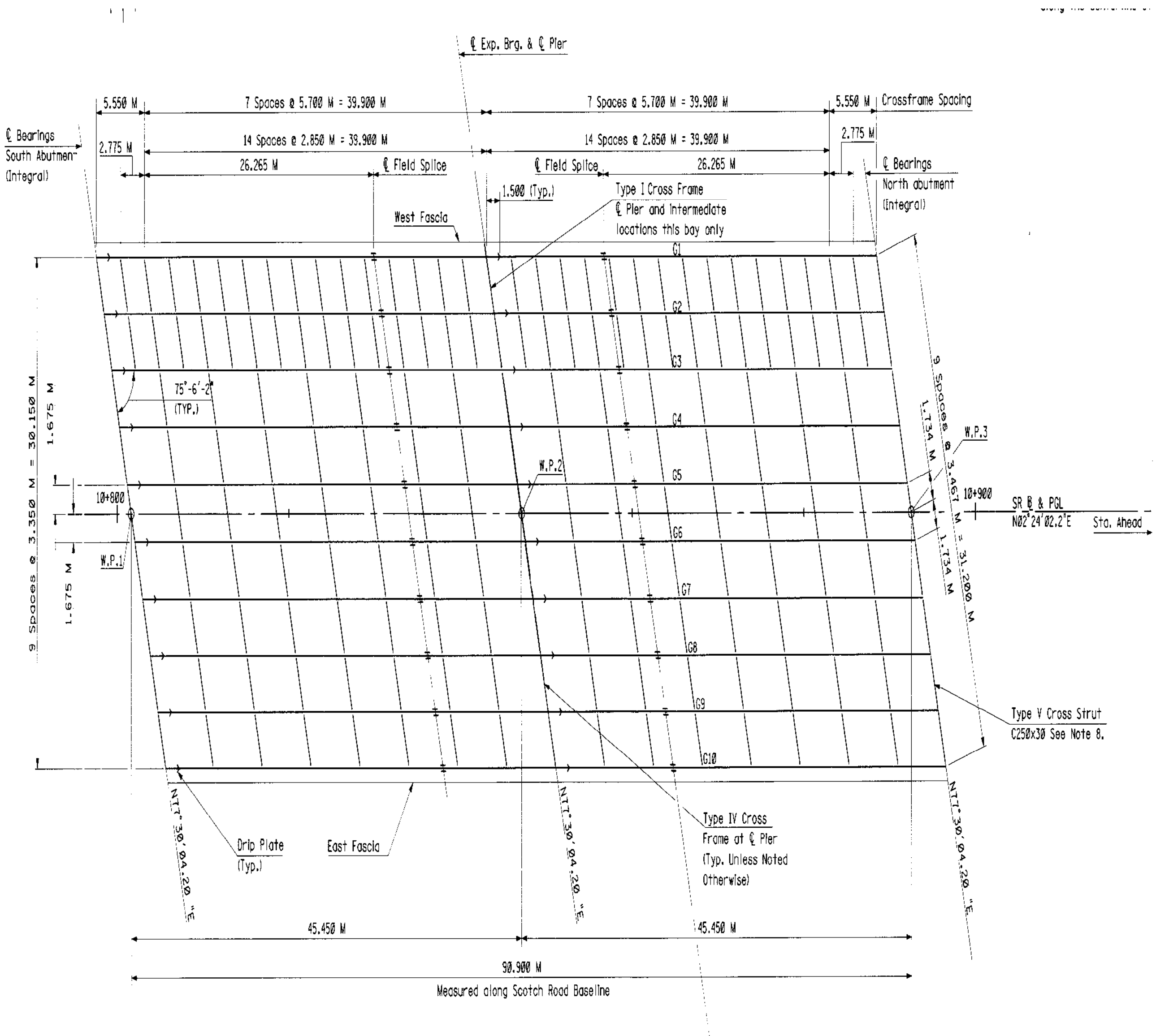


Figure 5.28 Superstructure Framing Plan

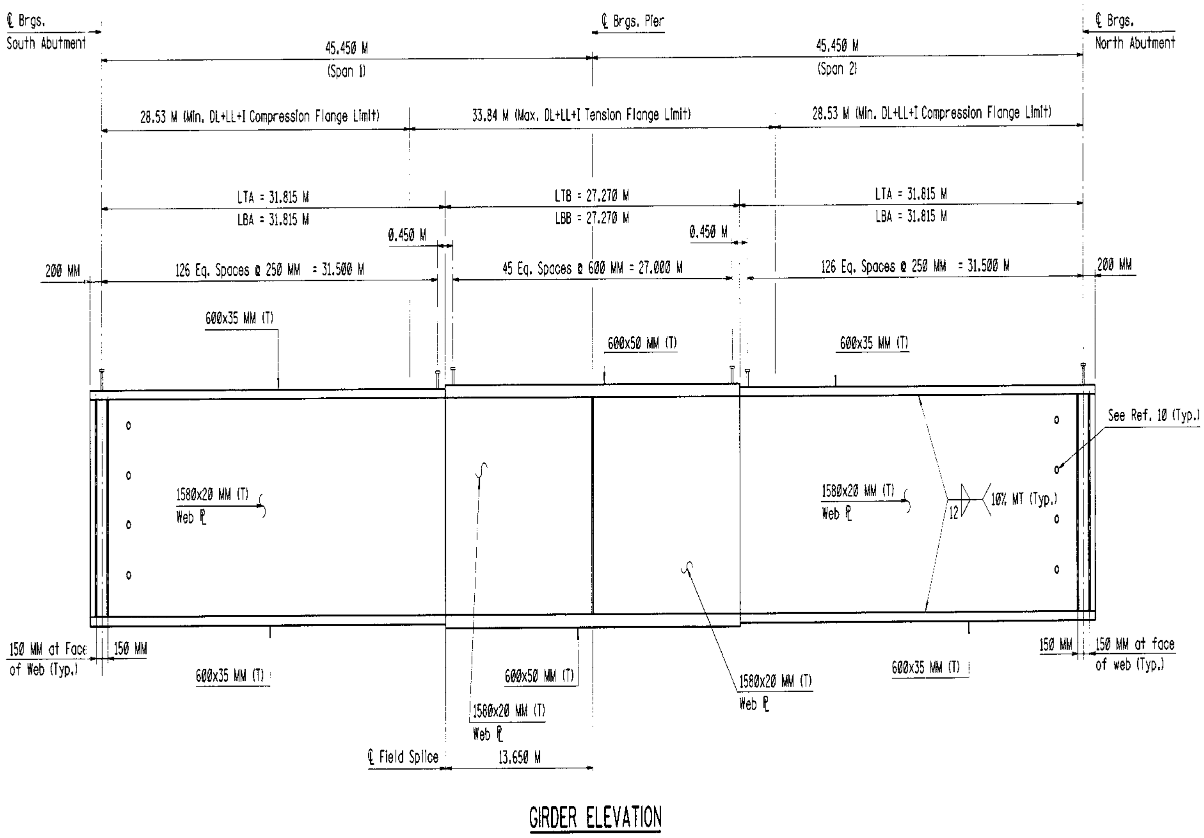
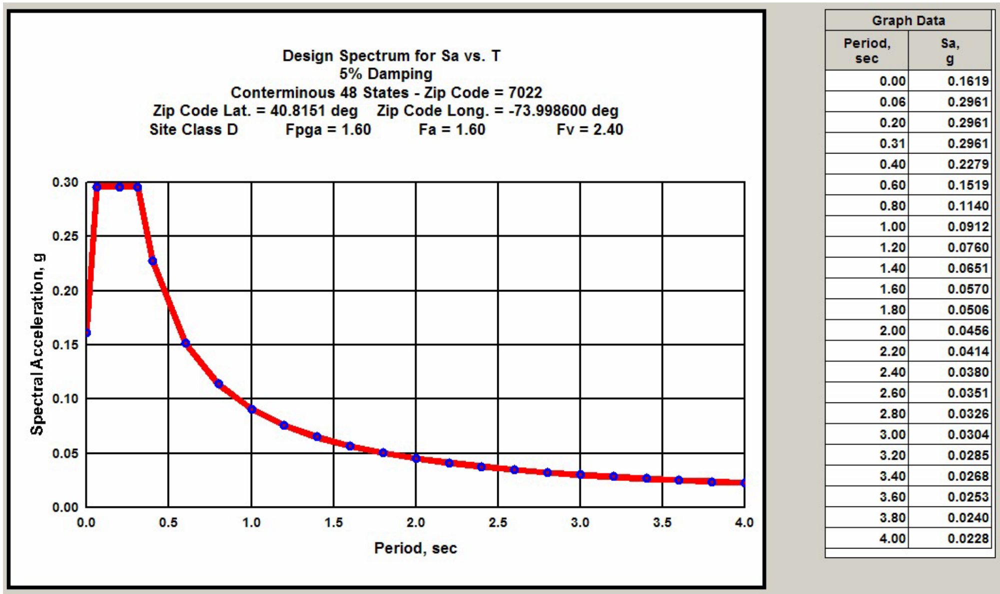


Figure 5.29 Girder Elevation

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum Shown in Figure 5.30. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.



Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.30 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D

PGA = 1.5 × 0.16 = 0.24
S_{DS} = 1.5 × 0.3 = 0.45
S_{D1} = 1.5 × 0.09 = 0.14

Flow charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 2 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path.

Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.31 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a multi-span bridge. Figure 5.32 shows the core flow chart of procedures outlined for bridges in SDC B, C, and D. Figure 5.33 outlines the demand analysis. Figure 5.34 directs the designer to determine displacement capacity. Figure 5.35 shows the modeling procedure. Figure 5.36 shows the foundation and abutment design applicable mainly for SDC C and D.

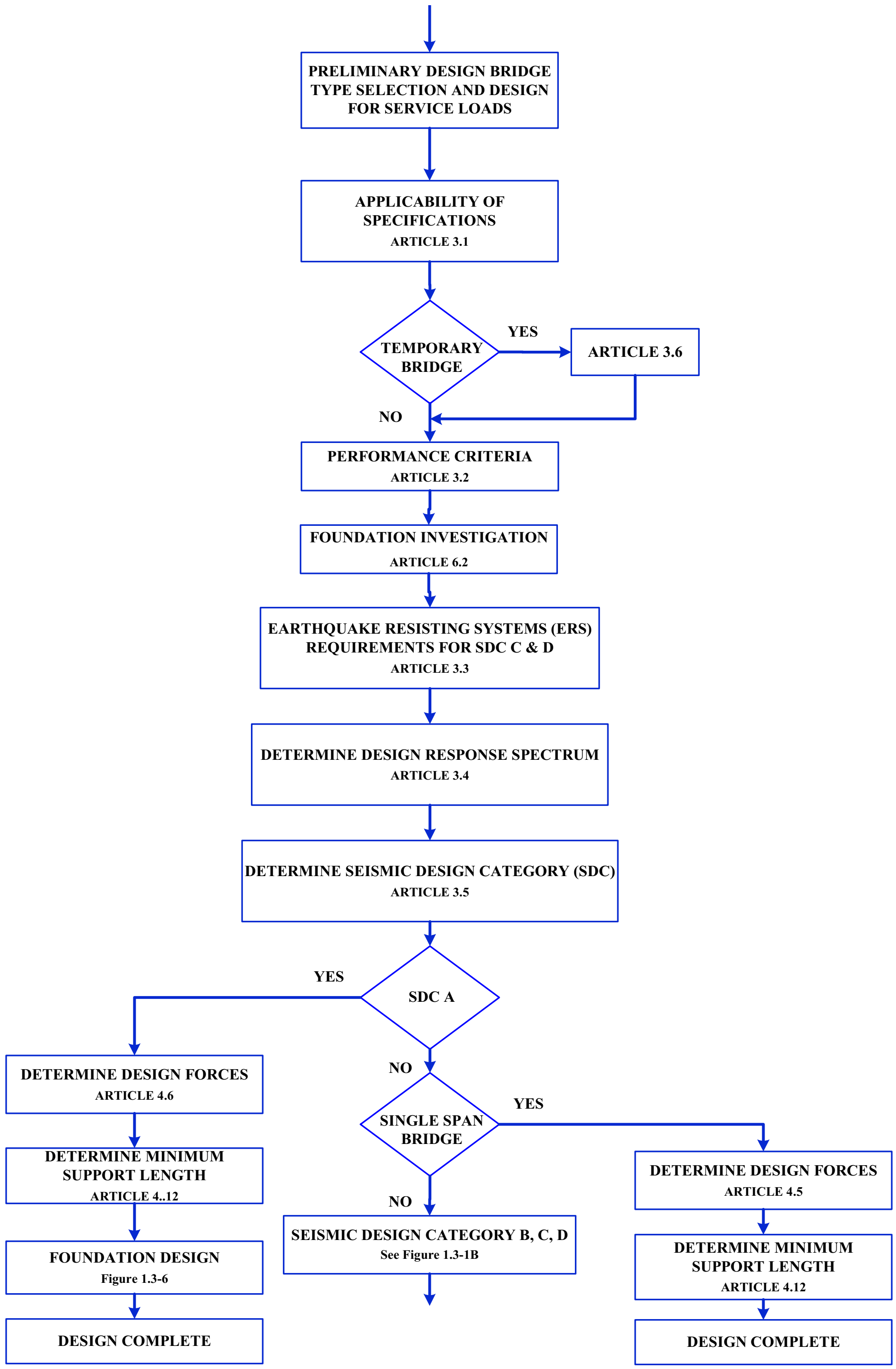


Figure 5.31 Seismic Design Procedure Flow Chart 1a

(Continued From Figure 1.3-1A)

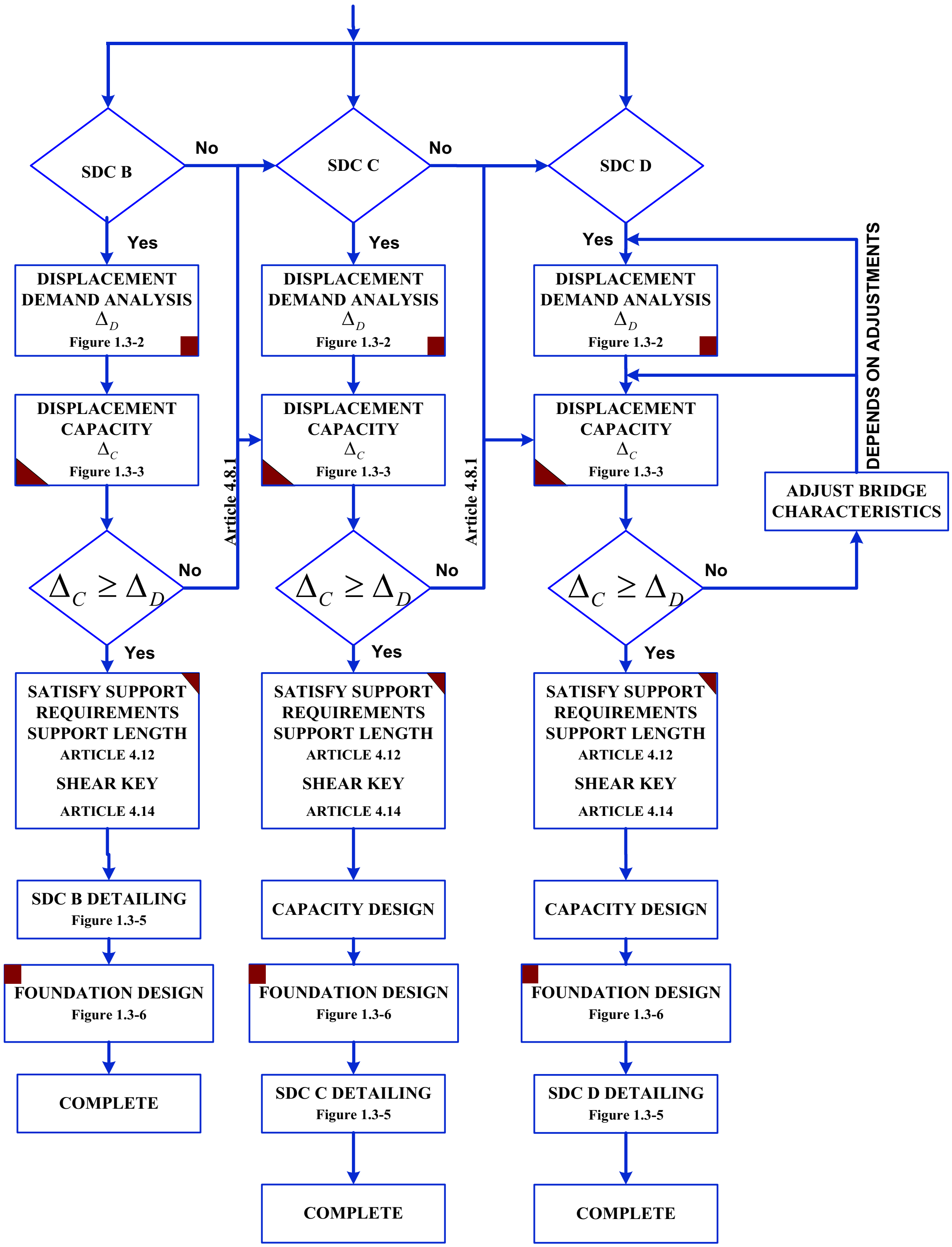


Figure 5.32 Seismic Design Procedure Flow Chart 1b

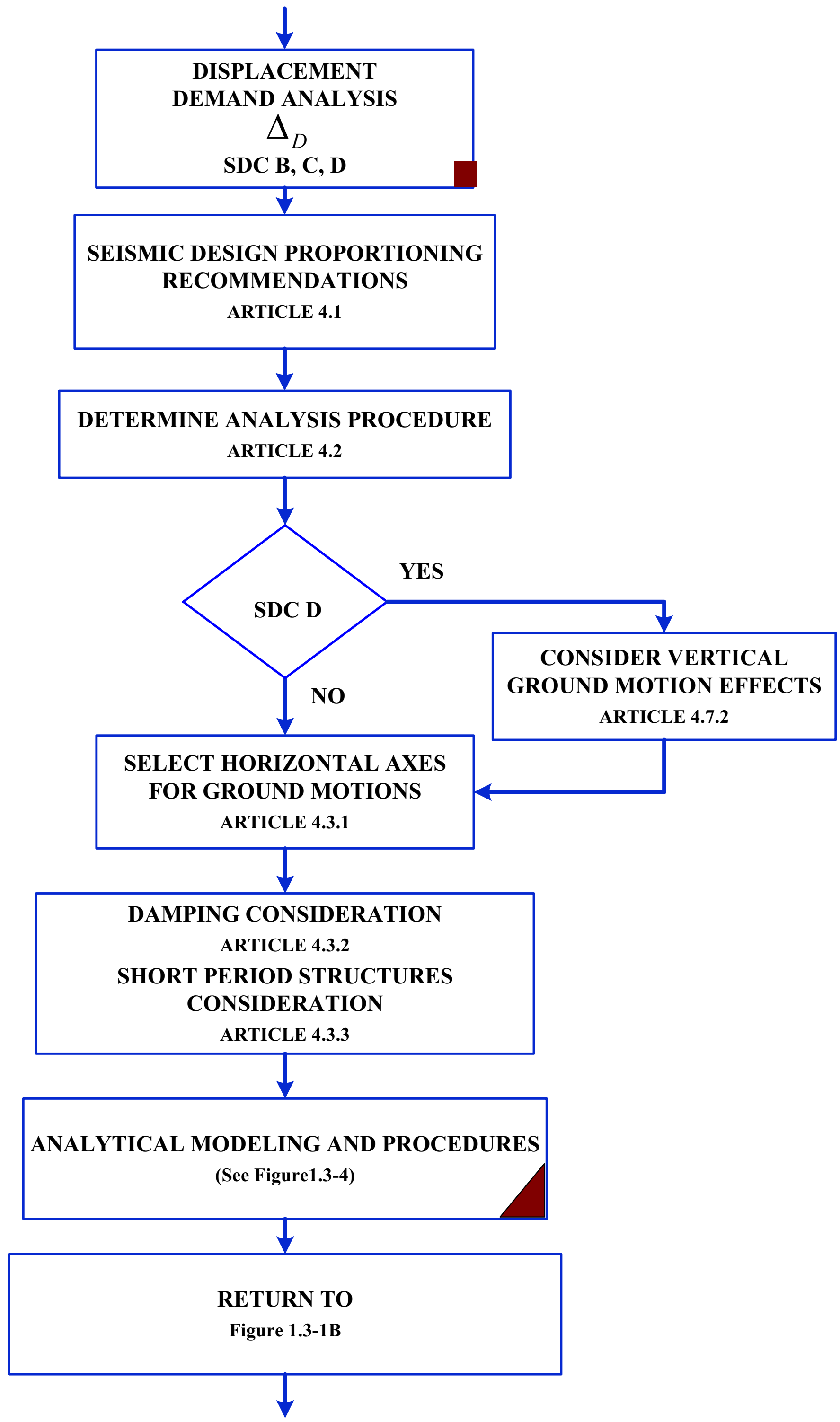


Figure 5.33 Demand Analysis Flow Chart 2

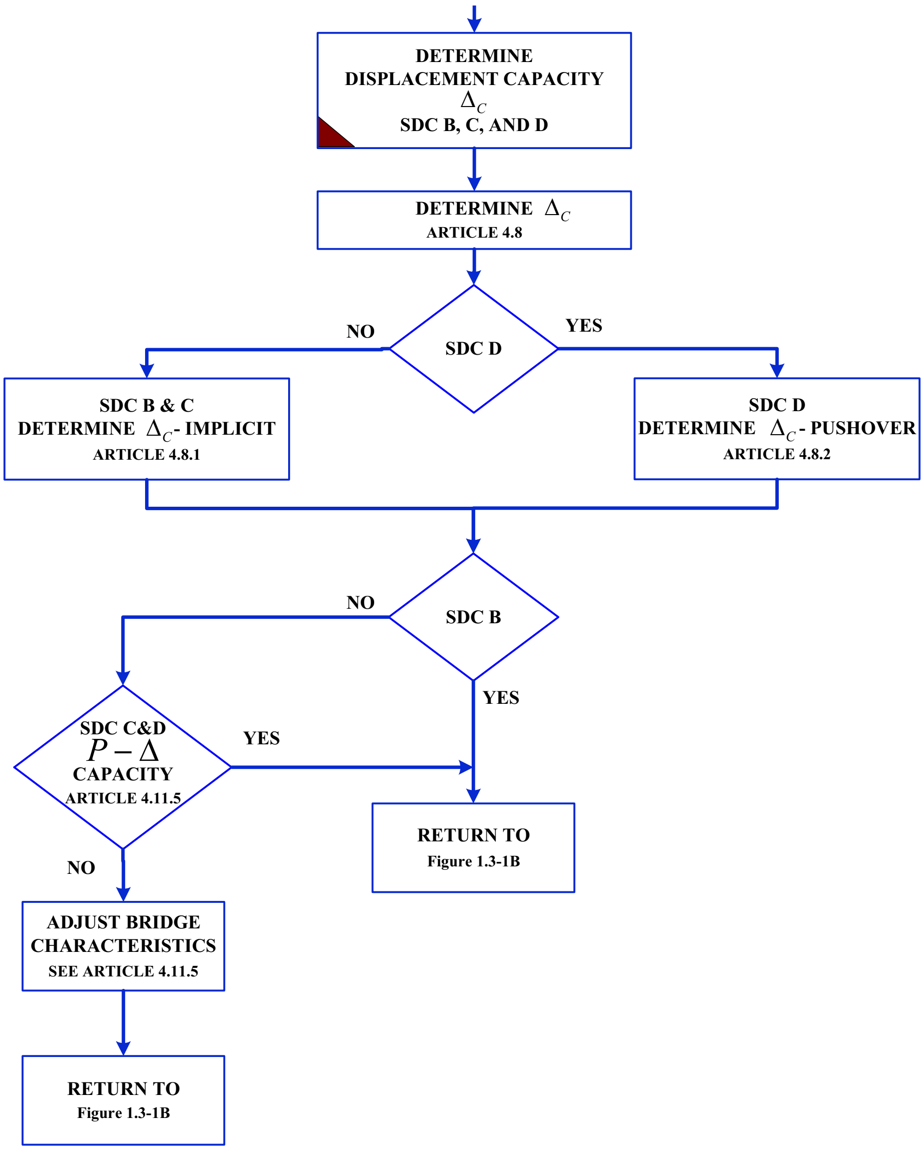


Figure 5.34 Displacement Capacity Flow Chart 3

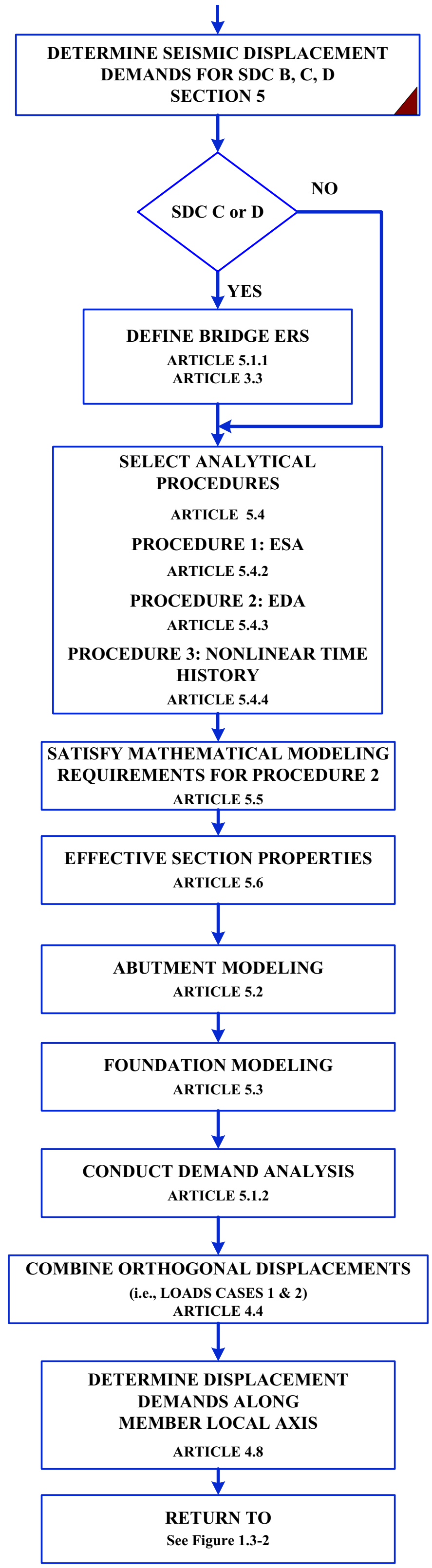


Figure 5.35 Modeling Procedure Flowchart 4

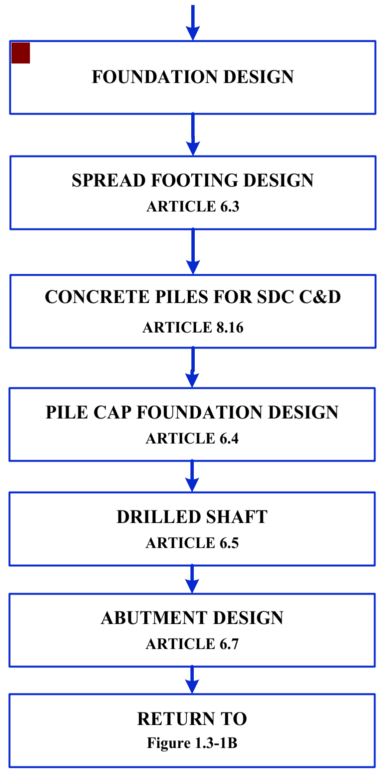


Figure 5.36 Foundation Design Flowchart 6

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.5.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}

Table 5.5 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.37 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Although S_{D1} is 0.14, the example bridge is designed by SDC B with the following basic requirements:

- Identification of ERS according to Article 3.3 should be considered
- Demand Analysis
- Implicit Capacity Check Required (displacement, P-Δ, support length)
- Capacity Design should be considered for column shear; capacity checks should be considered to avoid weak links in the ERS
- SDC B Level of Detailing
- Liquefaction check should be considered for certain conditions

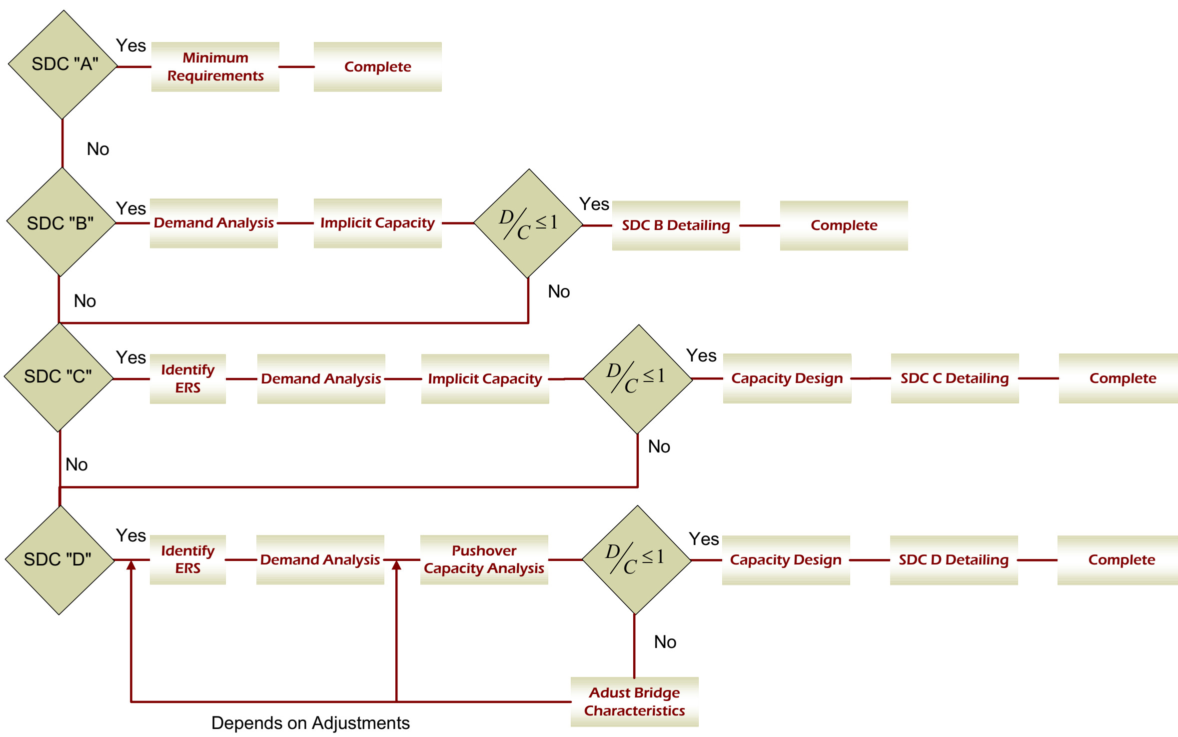


Figure 5.37 Seismic Design Category (SDC) Core Flowchart.

Considering a skew angle less than 20 degrees, the effect of skew is deemed negligible. Considering the continuity in the superstructure and the presence of integral abutments, a multi degree of freedom analysis is deemed not necessary to evaluate the displacement demand. The displacement demands are derived based on a combination of translational and rotational mode shapes as shown in the following analysis.

Seismic Analysis

Dead Load Calculation

Girder Weight:

South Abutment to Field Splice:

31.9m×600×35mm Top & Bottom Flange:

Volume: $2\times\dfrac{32\times10^3\times600\times35}{304.8^3}$

31.9m×1580×20mm Web Plate:

Volume: $\dfrac{32\times10^3\times1580\times20}{304.8^3}$

Subtotal Volume: 2×23.7+35.7=83.1 ft^3

Field Splice to Field Splice:

27.3m×600×50mm Top & Bottom Flange:

Volume: $2\times\dfrac{27.3\times10^3\times600\times50}{304.8^3}=2\times28.9\ ft^3$

27.3m×1580×20mm Web Plate:

Volume: $\dfrac{27.3\times10^3\times1580\times20}{304.8^3}=30.5\ ft^3$

Subtotal Volume: 2×28.9×30.5=88.3 ft^3

Total Weight: (83.1+88.3+83.1)×0.490 Kips/ft^3 = 125 Kips

Weights of 10 Girders for Superstructure = 125 ×10 = 1250 Kips

Deck Slab 260 mm, Width of deck 31m, length 91m

Deck Weight:

$$\frac{260\times31\times10^3\times91\times10^3}{304.8^3}\times0.15=3885\,\text{Kips}$$

Increase 10% for Fillets: 3885×1.1=4274 Kips

Concrete in Sidewalk 62 cm:

$$\frac{62}{0.3048^3}\times0.15=330\,\text{Kips}$$

Concrete in Parapet:

$$\frac{202 \times 10^3 \times 815 \times 300}{304.8^3} \times 0.15 = 262\,\text{Kips}$$

Concrete in Columns and Caps (109 C.M.): (See Figure 5.38 & 5.39 for Dimensions)

Column 1.2ϕ 4.55m Average height

Cap 1.4m×1.5m×16.2m

Pad Thick average 0.1 m

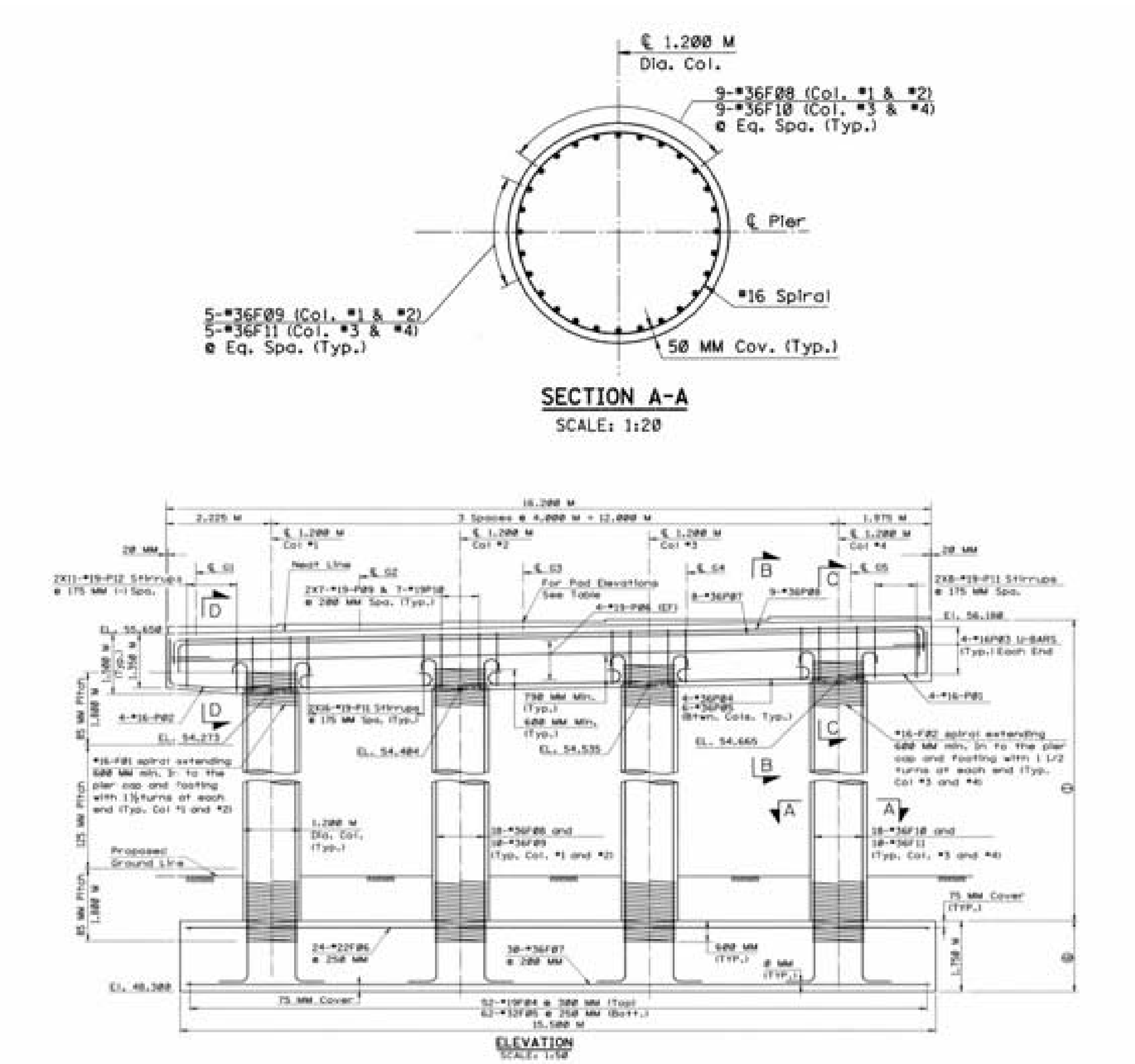


Figure 5.38 Bent Elevation

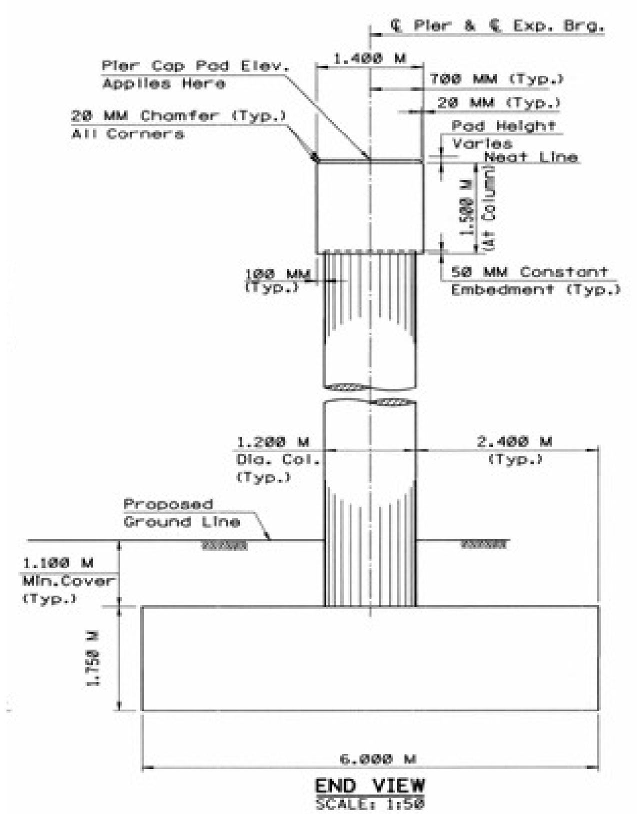


Figure 5.39 Bent End View

Abutment diaphragm: (See Figure 5.40 for Elevations)

El North Abutment 55.85 Bottom of diaphragm

El North Abutment 59.44 Max. Deck Elevation

North Diaphragm Height: 59.44-55.85 = 3.6 m

El South Abutment 53.4 Bottom of diaphragm

El South Abutment 57.1 Max. Deck Elevation

South Diaphragm Height: 57.11-53.4 = 3.7 m

Consider diaphragm dimension 12.1’×108’×3’

Weight of 2 Abutment diaphragms: $\left(12.1\times108\times3\times0.15\right)\times2=1176$ Kips

$\frac{1}{2}$ Columns Weight:

$$\frac{1}{2}\left(\frac{\pi 4^2}{4}\right)\times15\times8\times0.15=0.5\times12.56\times15\times8\times0.15=113\text{ Kips}$$

(Cap + Pad) Weight:

$$\left(4.6'\times5.25\times106\right)\times0.15=384\text{ Kips}$$

$2^{"}$ Overlay:

$$\frac{2}{12}\times 299\times 102\times 0.12 = 610\,\text{Kips}$$

Future Overlay:

$$\frac{31m\times 91m}{0.3048^2}\times 25psf = 760\,\text{Kips}$$

Summary:

Girders:	1250 Kips
Deck+10%:	4274 Kips
Sidewalk:	330 Kips
Parapet:	262 Kips
$\frac{1}{2}$ Columns:	113 Kips
Cap + Pad:	384 Kips
Abutment diaphragm:	1176 Kips
Overlay:	610 Kips
Total Not Including Future Overlay:	8399 Kips
Total Including Future Overlay:	8400+ 760= 9160 Kips

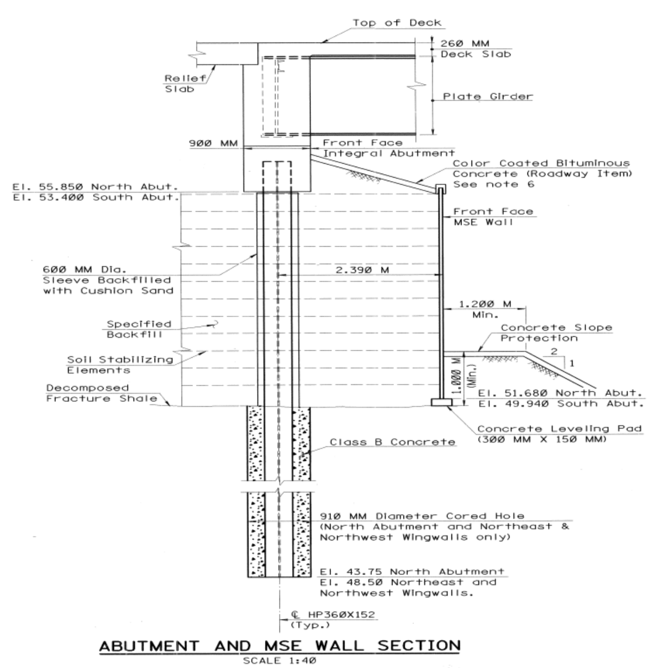


Figure 5.40 Abutment Section

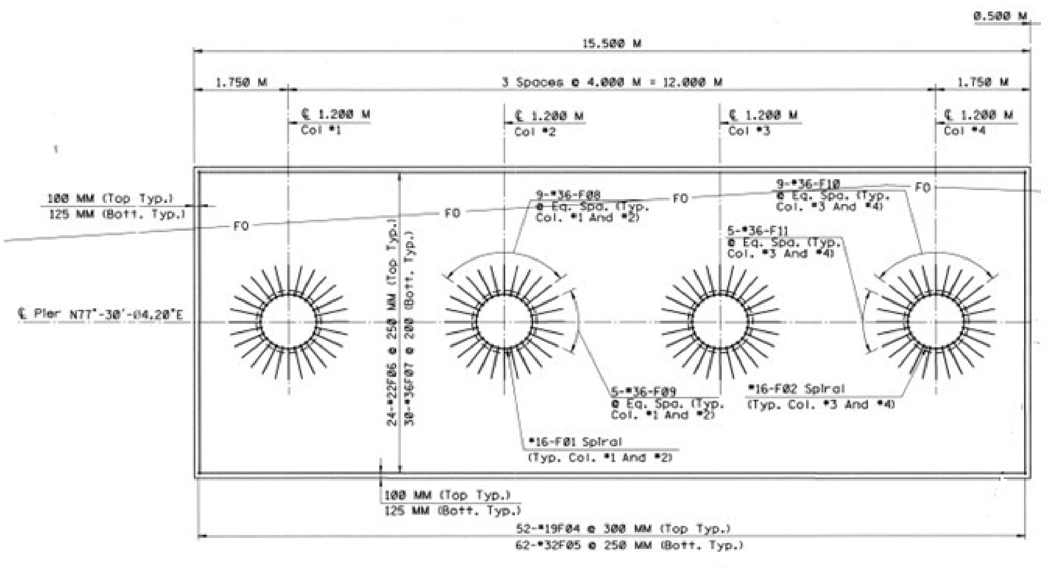


Figure 5.41 Partial Footing Plan (Left Shown, Right Similar)

Total Not Including Abutment Diaphragm: 9160-1176 = 7984 Kips

$\frac{1}{2}$ Columns: 113 Kips

Footing (See Figure 5.39 and 5.41): 5.8×19.7×50.9×0.15 = 873 Kips

For Continuous Girder, Consider $\frac{5}{8}$ factor for DL Distribution of continuous spans at center bent location.

Load on Columns: $\frac{5}{8}(7984+113)=5061\text{Kips}$

Load per Column: 5061/8 = 633 Kips

Calculate Abutment Pile Stiffness: (See Figure 5.40 Abutment Section for More Details)

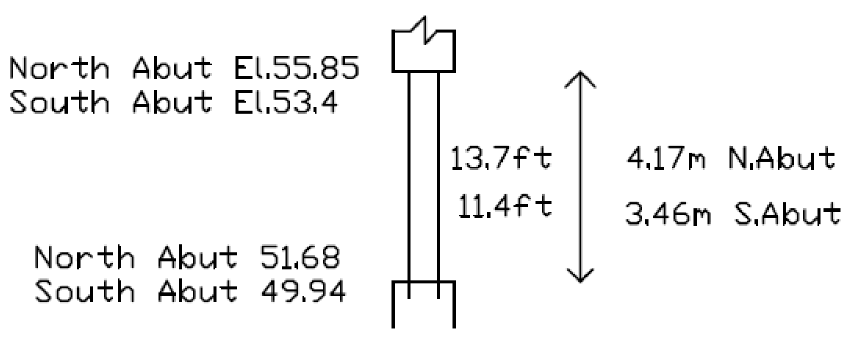


Figure 5.42 Effective Pile Length at Abutments.

Steel H Piles HP 360 mm by 152 KG/M Equivalent to HP 14×102 lb/ft

X_X Axis

I_x = 1053 in^4

S_x = 150.3 in^3

Z_x = 168.6 in^3

Y_Y Axis

I_{yy} = 380.2 in^4

S_y = 51.4 in^3

Z_y = 78.77 in^3

North Abutment Pile Stiffness: $K = \frac{12EI}{L^3}$ (No. of piles 29)

The Effective Pile Length (L) is shown in Figure 5.42

$$K_{xx} = \frac{12 \times 29000 \times 1053}{(13.7)^3 (12)^3} = 82.5 K/in$$

$$K_{yy} = \frac{12 \times 29000 \times 380.2}{(13.7)^3 (12)^3} = 29.8 K/in$$

South Abutment Pile Stiffness: $K = \frac{12EI}{L^3}$ (No. of piles 28)

$$K_{xx} = \frac{12 \times 29000 \times 1053}{(11.4)^3 (12)^3} = 143.1 \text{ K/in}$$

$$K_{yy} = \frac{12 \times 29000 \times 380.2}{(11.4)^3 (12)^3} = 51.7 \text{ K/in}$$

Calculate Abutment Passive Pressure: (AASHTO-SGS 5.2.3.3):

- For cohesionless, non-plastic backfill (fines content less than 30%), the passive pressure pp may be assumed equal to 2Hw/3 ksf per foot of wall length.
- For cohesive backfill (clay fraction > 15%), the passive pressure pp may be assumed to be equal to 5 ksf provided the estimated undrained shear strength is greater than 4 ksf.

Conservatively, Consider Cohesive backfill @ 5 Ksf

Consider Conservative $\frac{1}{2}''$ MR for Gapping

Width of Abutment: 108’

Height of Diaphragm: North Abutment $\frac{3.6}{0.304} = 11.8'$

(See calculation below Fig. 5.39)

South Abutment $\frac{3.7}{0.304} = 12.1'$

Following AASHTO-SGS 5.2.3.3, the total passive force may be determined as:

$$P_p = p_p H_w W_w$$

where:

p_p	=	passive lateral earth pressure behind backwall (ksf)
H_w	=	height of backwall (ft.)
W_w	=	width of backwall (ft.)

The total passive force capacity P_p is calculated as: $P_p = 5\ Ksf \times 11.8 \times 108 = 6372$ Kips.

(Total passive force capacity for the south abutment isn't calculated since north abutment is assumed to push against the north abutment).

Abutment Soil Stiffness Calculation:

An equivalent linear secant stiffness, Keff in kip/ft., is required for analyses. For integral or diaphragm type abutments, an initial secant stiffness (Figure 5.43) may be determined as follows:

$$K_{eff1} = \frac{P_p}{(F_w H_w)}$$

where:

P_p	=	passive lateral earth pressure capacity (kip)
H_w	=	height of backwall (ft.)
F_w	=	factor taken as between 0.01 to 0.05 for soils ranging from dense sand to compacted clays

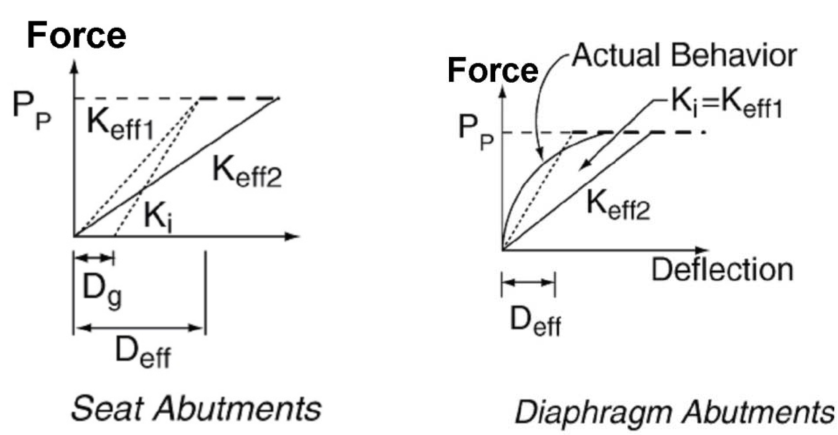


Figure 5.43 Characterization of Abutment Capacity and Stiffness.

If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively (K_{eff1} to K_{eff2}) until abutment displacements are consistent (within 30%) with the assumed stiffness. For seat type abutments, the expansion gap should be included in the initial estimate of the secant stiffness as follows:

$$K_{eff1} = \frac{P_p}{\left(F_w H_w + D_g\right)}$$

where:

D_g=width of gap between backwall and superstructure (ft.)

Calculate Soil Stiffness (Include the effect of $\frac{1}{2}$ in. M.R. Temperature Gapping):

$$K_{eff} = \frac{6372}{\left(0.01 \times 12.1 + \frac{0.5}{12}\right)} = \frac{6372}{0.121 + 0.04} = 39172 \text{ K/ft}$$

$$D_{eff} = (0.121 + 0.04) \times 12 \text{ in/ft} = 1.93'' \qquad \text{Say 2"}$$

Calculate Bent Stiffness by adding up the stiffness of individual columns:

Calculate Column Stiffness:

$$I_g = \frac{\pi D^4}{64} = \frac{\pi \times 4^4}{64} = 12.6 \text{ ft}^4$$

$$E = 57\sqrt{5000} = 580{,}000 \text{ Ksf}$$

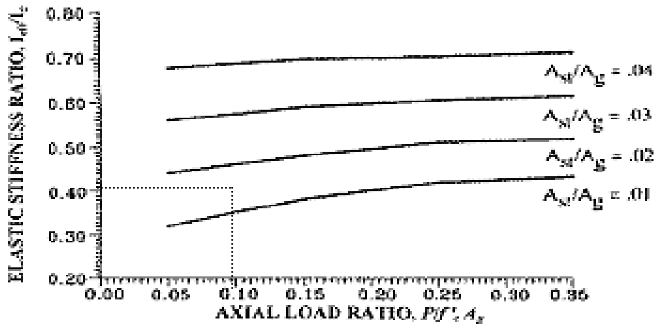
$$EI_g = 580{,}000 \times 12.6$$

Using AASHTO-SGS 5.6.2, calculate the Elastic Stiffness Ratio I_{eff} / I_g as shown in Figure 5.44

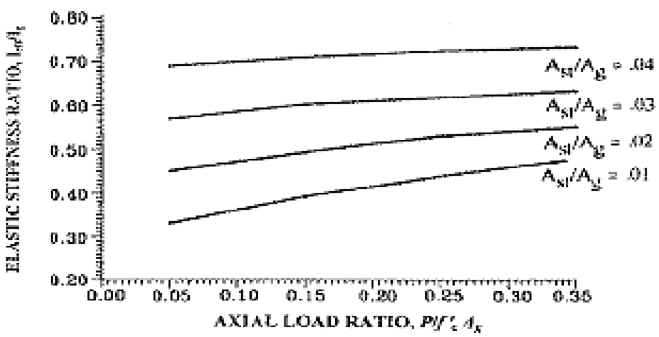
Calculate $\frac{A_{st}}{A_g}$ Column Reinforcement Ratio:

$$A_g = \frac{\pi D^2}{4} = 12.6 \text{ ft}^2$$

$$\frac{A_{st}}{A_g} = \frac{28 \times 1}{12.6 \times 144} = 0.015$$



(a) Circular Sections



(b) Rectangular Sections

Figure 5.44 Effective Flexural Stiffness of Cracked Reinforced Concrete Sections

Calculate $\frac{P}{f_c' A_g}$, $f_c' = 4$ Ksi

$$\frac{633}{4 \times 12.6 \times 144} = 0.087 \text{ or } 8.7\%$$

I_{eff}/I_g = 0.4 (See Fig. 5.44).

I_{eff} = 0.4×12.3 = 5.04 ft^4

EI_{eff} = 5.04×580,000 K·ft^2 or 4.21×10^8 K·in^2

From Sap Results (See Appendix V):

$$EI_{eff\,SAP} = \frac{M_y}{\phi_y} = \frac{33026.9}{0.0000857} = 4.04 \times 10^8 \text{ K·in}^2 \qquad \text{(For verification)}$$

Column M_p = 43023 K·in = 3585 K·ft

Examine Rocking of Bent in Longitudinal direction (Appendix A of the AASHTO-SGS).

Ultimate Bearing Pressure:

q_n = 1100 KPa or 23 Ksf See Note on As-built Sheet B4

8. Foundation Design Criteria

Pier:
(A) Allowable Bearing Pressure = 300 KPa
(B) Strength Bearing Pressure = 400 KPa
(C) Ultimate Bearing Pressure = 1100 KPa
Abutment:
(A) Pile Foundations: HP360x152
(B) Ultimate Pile Capacity: See Pile and Footing Plans
(C) All Test Piles shall be installed and tested prior to the development of Production Pile lengths and driving criteria. The Contractor shall submit for engineers approval Wave Equation Analysis Program (WEAP) results for hammer selection for driving piles. PDA with CAPWAP shall be utilized for the test piles.

Load at Bottom of Footing (2 Footing Total, see calculation below Fig. 5.41):

(5061 Kips +873 Kips) = 5943 Kips

Load per footing not including soil cover:

5943/2 = 2967 Kips

Footing Dimension: 19.7 ft × 50.9 ft

Width of Compression block “a” for soil bearing is calculated using AASHTO-SGS Equation A-2

$$a = \frac{W_T}{B_r q_n}$$

$$a = \frac{2967}{50.9 \times 23}$$

a = 2.5 ft

Restoring Moment for footing M_r is calculated using AASHTO-SGS Equation A-7

$$M_r = W_T \left(\frac{L_F - a}{2} \right)$$

$$2967 \text{ Kips} \times \left(\frac{19.7 - 2.5}{2} \right) = 25516 \text{ Kips·ft}$$

Moment demand at bottom of footing (4 columns):

$4M_p + 4V_p \times 5.8$ ft where 5.8 ft is the depth of the footing

Calculate $V_p = \frac{M_p}{L} = \frac{3585}{15} = 239 \text{ Kips}$

$$4M_p + 4V_p \times 5.8 = 4 \times 3585 + 4 \times 239 \times 5.8$$

$$= 14340+5545$$

$$= 19885 \text{ Kips·ft}$$

Since Restoring Moment > Plastic Moment Demand, calculate bent stiffness in longitudinal direction based on column flexural stiffness. Column Stiffness can be taken at $\frac{3EI}{L^3}$ for longitudinal period calculation, displacement, and force distribution.

Longitudinal Direction Total Stiffness Calculation:

North Abutment Stiffness of Piles:

Piles Total K_{yy}: $29.8 \text{ K/in} \times 29 = 864 \text{ K/in}$

South Abutment Stiffness of Piles:

Piles Total K_{yy}: $51.7 \text{ K/in} \times 28 = 1448 \text{ K/in}$

North Abutment Stiffness including Abutment Soil Stiffness:

$864 \text{ K/in} \times 12 \text{ in/ft} + 39172 \text{ K/ft}$

$= 10368 \text{ K/ft} + 39172 \text{ K/ft} = 49540 \text{ K/ft}$

Equivalent $K_{yy} = 49540 \text{ K/ft}$

South Abutment Stiffness including Abutment Soil Stiffness:

Consider Superstructure pushing against North Abutment; Therefore, only Pile Stiffness of the South Abutment is considered (i.e., abutment soil stiffness from south abutment is ignored).

Equivalent $K_{yy} = 1448 \text{ K/in} \times 12 \text{ in/ft} = 17376 \text{ K/ft}$

Individual Column Stiffness:

$$K_{C-yy} = \frac{3EI}{L^3} = 3 \times \frac{580000 \times 5.04}{15^3} = 2598 \text{ K/ft}$$

Total Column Stiffness (8 columns):

$8 \times 2598 \text{ K/ft} = 20784 \text{ K/ft}$

Summary of Longitudinal Stiffness (Demand Analysis Model):

		Stiffness Ratio (wrt total stiffness)
North Abutment:	49540 K/ft	0.56
South Abutment:	17376 K/ft	0.2
Bent:	20784 K/ft	0.24
Total:	87700 K/ft	1.00

Calculate Longitudinal Period:

Total Mass Participation: $\dfrac{9160 \text{ Kips}}{32.2 \text{ ft/sec}^2}$

Total Stiffness in Longitudinal direction: 87700 K/ft

$$\omega^2 = \frac{K}{M} = \frac{87700 \times 32.2}{9160} = 308$$

$$\omega = \sqrt{308} = 17.6 \text{ rad/sec}$$

$$T = \frac{2\pi}{\omega} = \frac{2\pi}{17.6} = 0.36 \text{sec}$$

The total force demand can be conservatively calculated based on Short Period response using a Spectral Acceleration of 0.45g (S_{DS} calculated below Fig. 5.30)

Total Force demand: 9160×0.45 = 4122 Kips

Force Distribution	Stiffness Ratio	Force Magnitude (Kips)
North Abutment	0.56	2308
South Abutment	0.2	825
Bent	0.24	990

Spectral Longitudinal Displacement

$$S_d = \frac{S_a}{\omega^2} = \frac{0.45 \times 32.2}{308} \times 12 \text{ in/ft} = 0.6 \text{ in}$$

Calculate Transverse Direction Total Stiffness:

North Abutment Stiffness of Piles:

Piles Total $K_{xx} = 82.5 \text{ K/in} \times 29 \times 12 \text{ in/ft} = 28710 \text{ K/ft}$

South Abutment Stiffness of Piles:

Piles Total $K_{xx} = 143.1 \text{ K/in} \times 28 \times 12 \text{ in/ft} = 48082 \text{ K/ft}$

Individual Column Stiffness

$$K_C \frac{12EI}{L^3} = \frac{12 \times 580000 \times 5.04}{15^3} = 10394 \text{ K/ft}$$

Total Column Stiffness:

$8 \times K_C$ = 83149 K/ft

Summary of Transverse Stiffness (Demand Analysis Model):

		Stiffness Ratio (wrt total stiffness)
North Abutment:	28710	0.18
South Abutment:	48082	0.30
Bent:	83149	0.52
Total:	159941	1.00

$$\omega^2 = \frac{K}{M}\frac{159941 \times 32.2}{9160} = 562$$

$$\omega = \sqrt{562} = 23.7$$

$$T = \frac{2\pi}{\omega} = \frac{2\pi}{23.7} = 0.27 \text{ sec}$$

Based on a Short Period Response, the Spectral Acceleration is equal to 0.45g (see SDS calculated below Fig. 5.30).

Total Force Demand: 9160×0.45 = 4122 Kips

Spectral Displacement (in translation mode):

$$\Delta = \frac{4122 \text{ K}}{159941} \times 12 \text{ in/ft} = 0.31 \text{ in}$$

Find additional displacement demand due to eccentricity between center of mass and center of rigidity:

Find center of rigidity (Refer to Figure 5.45):

$$X = \frac{0.18 \times 2 + 0.52 \times 1 + 0.3 \times 0}{(0.18 + 0.52 + 0.3)} = 0.18 \times 2 + 0.52 \times 1 = 0.88$$

Distance between Center of Mass and Center of Rigidity: 1- 0.88 =0.12

$$\sum M = 0$$

$$28710 \times \Delta(1 + 0.12) + 83149 \times 0.107\Delta(0.12) + 48082(0.79\Delta)(0.88)$$

$$= 4122 \times (0.12)$$

$$32155\Delta + 1067\Delta + 33427\Delta = 495$$

$$66649\Delta = 495$$

$$\Delta = \frac{495}{66649} \times 12 \text{ in/ft} = 0.1 \text{ in}$$

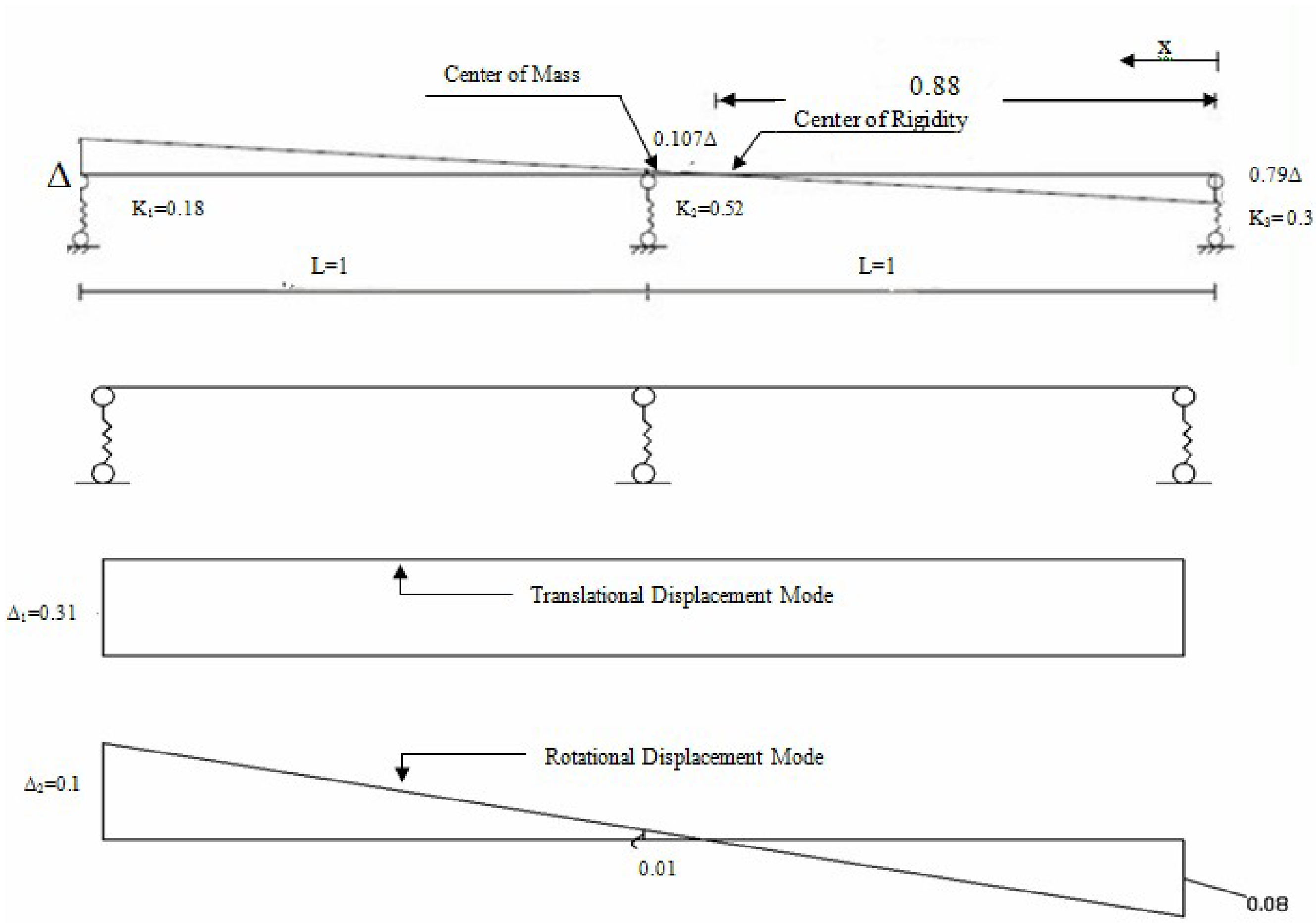


Figure 5.45 Superstructure Displacement Modes

Calculate Displacement Magnification for short period structures according to AASHTO-SGS 4.3.3

$$R_d = \left(1 - \frac{1}{\mu_D}\right)\frac{T^*}{T} + \frac{1}{\mu_D} \geq 1.0$$

$T^* = 1.25T_s$ (See Figure 5.30 for T_s)

$T^* = 1.25 \times 0.31 = 0.39$

$\mu_D = 2$ for SDC B

In the longitudinal direction, the translational mode period T is equal to 0.36 sec, the displacement magnification factor is:

$$R_d = \left(1 - \frac{1}{2}\right)\frac{0.39}{0.36} + \frac{1}{2} \geq 1.0$$

$R_d = 1.05$

In the transverse direction, the translational mode period T is equal to 0.27 sec, the displacement magnification factor is:

$$R_d = \left(1 - \frac{1}{2}\right)\frac{0.39}{0.27} + \frac{1}{2} \geq 1.0$$

$$R_d = 1.22$$

	Abutment	Bent	Abutment
Transverse Displacement	0.41 in	0.32 in	0.23in
Transverse Magnified Displacement	0.50 in	0.39 in	0.28in
Longitudinal Displacement	0.6 in	0.6 in	0.6in
Longitudinal Magnified Displacement	0.63 in	0.63 in	0.63in

Perform Combination of Orthogonal Seismic Displacement Demands following AASHTO-SGS Section 4.4:

Transv + 30%Long ↑ 0.39 in
→ 0.19 in

Long + 30%Transv ↑ 0.12 in
→ 0.63 in

Calculate Yield Displacement of Column Δ_y in the Longitudinal and Transverse direction:

Column Stiffness longitudinal direction:

$$\frac{3EI}{L^3} = 2598 \text{ K/ft}$$

Column Stiffness transverse direction:

$$\frac{12EI}{L^3} = 10394 \text{ K/ft}$$

Calculate the plastic shear V_p in the Longitudinal Direction:

$$V_p \ (Long \ direction) = \frac{3585}{15} = 239 \text{ Kips}$$

Calculate the plastic shear V_p in the Transverse Direction:

$$V_p \ (Transv \ direction) = \frac{2 \times 3585}{15} = 478 \text{ Kips}$$

$$\Delta_y \ (Long \ direction) = \frac{239}{2598} \times 12 \text{ in} = 1.1 \text{ in}$$

$$\Delta_y \ (Transverse \ direction) = \frac{478}{10394} \times 12 \text{ in} = 0.55 \text{ in}$$

In comparing the column displacement demands to the yield displacement in the transverse and longitudinal directions, the column is found to respond in the elastic range; therefore satisfy minimal requirements of SDC B. Calculate Local Displacement Capacity for SDC B according to AASHTO-SGS 4.8.1

For Type 1 structures, comprised of reinforced concrete columns in SDC B, the displacement capacity, Δ_C^L in., of each bent may be determined from the following approximation:

$$\Delta_C^L = 0.12H_o\left(-1.27\ln(x) - 0.32\right) \geq 0.12H_o$$

in which:

$$x = \frac{\Lambda B_o}{H_o}$$

`

where:

Ho	=	clear height of column (ft.)
Bo	=	column diameter or width measured parallel to the direction of displacement under consideration (ft.)
Λ	=	factor for column end restraint condition
	=	1 for fixed-free (pinned on one end)
	=	2 for fixed top and bottom

For a partially fixed connection on one end, interpolation between 1 and 2 is permitted for Λ. Alternatively, Ho may be taken as the shortest distance between the point of maximum moment and point of contra-flexure and Λ may be taken as 1.0 when determining x using the equation above.

Calculate local displacement capacity in longitudinal and transverse direction:

$$x = \frac{\Lambda B}{H}$$ where:

Λ = 2 for fixed top and bottom connections as in transverse direction

Λ = 1 for fixed free connection as in the longitudinal direction.

In the longitudinal direction, the bent has a partial fixity due to the deck restraint at the abutment and the eccentricity between the c.g. of the superstructure and the bearing location, therefore Λ can be reasonably taken as 1.5. Establish capacity in longitudinal direction based on Λ = 1.5

In transverse direction: $x = 2 \times \frac{4}{15} = 0.53$

In longitudinal direction: $x = 1.5 \times \frac{4}{15} = 0.40$

Transverse direction:

$$\Delta_C = 1.8(-1.27\ln(0.53) - 0.32) \geq 1.8''$$

$$= 1.8(0.486) \geq 1.8''$$

$$= 1.8 \text{ in}$$

Longitudinal direction:

$$\Delta_C = 1.8(-1.27\ln(0.4) - 0.32) \geq 1.8''$$

$$= 1.8(0.84) \geq 1.8''$$

$$= 1.8 \text{ in}$$

According to AASHTO-SGS Section 4.8

$$\Delta_D^L < \Delta_C^L$$

where:

Δ_D^L = displacement demand taken along the local principal axis of the ductile member

Δ_C^L = displacement capacity taken along the local principal axis corresponding to Δ_D^L of the ductile member as determined in accordance with Article 4.8.1 for SDC B and C.

Eq. 1 shall be satisfied in each of the local axis of every bent. The local axis of a bent typically coincides with the principal axis of the columns in that bent.

Displacement Demand in Longitudinal direction 0.63''

Displacement Demand in Transverse direction 0.39''

Displacement Demand ≤ Displacement Capacity in both Local Axes

Abutment Response

According to the AASHTO-SGS 5.2.3.1, abutments for bridges in SDC B are expected to resist earthquake loads with minimal damage. However, bridge superstructure displacement demands may be 4 in. or more and could potentially increase the soil mobilization. Comparing the displacement demand to the 4 in. threshold capacity, the abutments are deemed adequate for minimal damage requirement.

Column Shear Demand and Capacity

According to AASHTO-SGS 8.6.1, the shear demand for a column, V_u, in SDC B shall be determined based on the lesser of:

- The force obtained from a linear elastic seismic analysis
- The force, V_{po}, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_u, in SDC C or D shall be determined based on the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.

Given the uncertainty in the hazard and the consequence of column shear failure, it is deemed important to attempt to satisfy the capacity protection requirement for column shear.

The column shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated based on the nominal material strength properties and shall satisfy:

$$\phi_s V_n \geq V_u$$

in which:

$$V_n = V_c + V_s$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

Calculate Shear demand in longitudinal direction (Elastic Model)

Total Elastic Force Demand: 4122 Kips

Bent Stiffness Ratio: 0.24

Bent Elastic Force: 4122×0.24 = 990 Kips

Column Shear Force: $\frac{990}{8} = 124$ Kips

According to AASHTO-SGS Eq. 8.5.1

Column Plastic Shear Demand: $M_{po} = \lambda_{mo} M_p$

M_{po} = 1.4×3585 = 5019 K·ft

The column plastic shear demand in the longitudinal direction is:

$$V_{po} = \frac{M_{po}}{L} = \frac{5019}{15} = 335 \text{ Kips}$$

Shear Demand in Transverse direction (Elastic Model):

Total Elastic Force Demand: 4122 Kips

Bent Stiffness Ratio: 0.52

Force Demand: $4122 \times 0.52 = 2144$ Kips

Column Shear Force Demand: $\frac{2144}{8} = 268\,\text{K}$

$$V_{po} = \frac{M_{po}}{L} = \frac{2 \times 5019}{15} = 670 \text{ Kips}$$

The concrete shear capacity, V_c, of members designed for SDC B, C and D shall be taken as:

$$V_c = v_c A_e$$

in which:

$$A_e = 0.8 A_g$$

if P_u is compressive:

$$v_c = 0.032\alpha' \left(1 + \frac{P_u}{2A_g}\right)\sqrt{f_c'} \le \min \begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases}$$

otherwise:

$v_c = 0$

for circular columns with spiral or hoop reinforcing:

$$0.3 \le \alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \le 3$$

$$f_s = \rho_s f_{yh} \le 0.35$$

$$\rho_s = \frac{4A_{sp}}{sD'}$$

where:

A_g = gross area of member cross section (in.2)

P_u = ultimate compressive force acting on section (kip)

A_{sp} = area of spiral or hoop reinforcing bar (in.2)

s = pitch of spiral or spacing of hoops or ties (in.)

D' = diameter of spiral or hoop for circular column (in.)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f_c' = nominal concrete compressive strength (ksi)

μ_D = maximum local displacement ductility ratio of member

For SDC B, the concrete shear capacity, V_c, of a section within the plastic hinge region shall be determined using:

$$\mu_D = 2$$

$$\rho_s = \frac{4 \times 0.2}{5 \times 44} = 0.36\%$$

$$f_s = \frac{0.36 \times 60}{100} = 0.22 < 0.35$$

$$\alpha' = \frac{0.22}{0.15} + 3.67 - 2 = 3.12 < 3$$

$$\alpha' = 3$$

The Axial Force P_u can be conservatively taken from Plastic Capacity distribution (See Figure 5.46), or directly from elastic analysis.

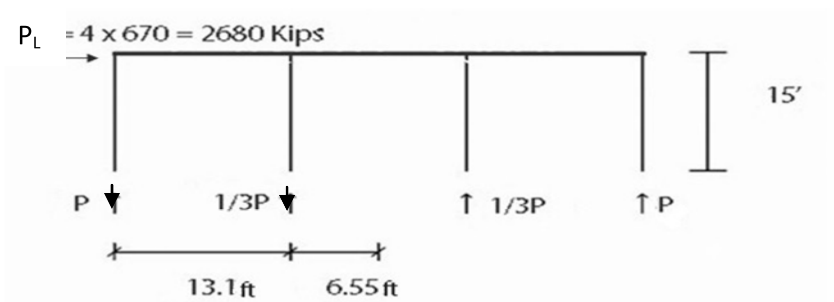


Figure 5.46 Column Axial Force Distribution

$$\sum M = \phi$$

$$2P \times (13.1 + 6.55) + 2 \times 1/3P(6.55) = 2680 \text{ Kips} \times 15 \text{ ft}$$

$$39.9P + 4.4P = 40,200 \text{ K-ft}$$

P = 920 Kips

P = 920 Kips is quite conservative and results in a net Tension Force on column since DL=633k

$$\therefore V_c = 0$$

Calculate Column Shear Reinforcement Capacity

According to AASHTO-SGS 8.6.3, members that are reinforced with circular hoops, spirals or interlocking hoops or spirals as specified in Article 8.6.6, the nominal shear reinforcement strength, Vs, shall be taken as:

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

where:

n = number of individual interlocking spiral or hoop core sections

A_{sp} = area of spiral or hoop reinforcing bar (in.2)

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

The pitch s is taken equal to 5" since shear demand is constant and governs the design outside the plastic hinge region.

$$V_s = \frac{\pi}{2}\left(1\times 0.2\times 60\times\frac{44}{5}\right) = 166\text{ K}$$

Capacity $\phi_s\left(V_s + V_c\right) = 0.9\left(166+0\right) = 150\text{ K} < 670\text{ K}$

Revise V_c based on more refined results obtained from the elastic linear analysis or from increase shear reinforcement. Elastic Demand in Transverse direction was found equal to 268K (i.e. it is expected that axial force can be reduced proportionally).

$$P_{refined} = \frac{268\text{ K}}{670\text{ K}}\times 920\text{ K} = 368\text{ K}$$

$$A_g = \frac{\pi D^2}{4} = \frac{\pi}{4}\times 48^2 = 1810\text{ in}^2$$

$$A_e = 0.8A_g = 1448\text{ in}^2$$

$$v_c = 0.032\times 3\left(1+\frac{633-368}{2\times 1810}\right)\sqrt{4}$$

$$= 0.096\left(1+0.07\right)\sqrt{4} = .1\sqrt{4} = .20 \le \min\begin{Bmatrix}0.11\sqrt{4}\\ 0.047\times 3\sqrt{4}\end{Bmatrix}$$

$$V_c = 0.20\times 1448 = 290\text{ Kips}$$

$$=0.9\left(290+166\right)=410\text{ Kips} > 268\text{ Kips}$$

$\phi_s V_n > V_u$ OK

As Column height decreases, capacity protection for this column is not easily obtained, however is acceptable for SDC B but not preferable. The design is inappropriate for SDC C or D where capacity protection is required.

The following requirements need to be satisfied for SDC B:

AASHTO-SGS 8.6.4 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, Vs, shall not be taken greater than:

$$V_s \leq 0.25\sqrt{f'_c} A_e$$

where:

A_e = eff.ective area of the cross section for shear resistance by Eq. 8.6.2-2 (in^2)

f'_c = compressive strength of concrete (ksi)

$V_s \leq 0.25\sqrt{f'_c} A_e$

$< 0.25\sqrt{4} \times 1448 = 724$ Kips

166<724 Kips OK

AASHTO-SGS 8.6.5 Minimum Shear Reinforcement

The area of column spiral reinforcement, Asp, shall be used to determine the reinforcement ratio, ρs as given by Eq. 8.6.2-7. For SDC B, the spiral reinforcement ratio, ρs, for each individual circular core of a column shall satisfy:

$$\rho_s \geq 0.003$$

$$\rho_s = 0.36\% > 0.3\% \quad \text{OK}$$

AASHTO-SGS 8.7.1 Minimum Lateral Strength

The minimum lateral flexural capacity of each column shall be taken as:

$$M_{ne} \geq 0.1 P_{trib} \frac{(H_h + 0.5 D_s)}{\Lambda}$$

where:

M_{ne} = Nominal moment capacity of the column based upon expected material properties as shown in Figure 8.5-1(kip-ft.)

P_{trib} = Greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kips)

H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft.)

D_s = depth of superstructure (ft.)

Λ = fixity factor for the column defined in Article 4.8.1

$M_{ne} = M_p$ for SDC B (See AASHTO-SGS 8.5) = 3585 K·ft

$H_n = 15$ ft

$D_s = 1.5 + 0.26 + 1.58 + 0.1/.304 = 11.3$ ft

$0.5D_s = 5.7$ ft

$\Lambda=1$ in the Longitudinal Direction

$P_{trib} = 633$ Kips

$$0.1 \times 633\left(\frac{15+5.7}{1}\right) = 1310 \text{ K·ft}$$

$M_{ne} > 1310$ K·ft OK

AASHTO-SGS 8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall satisfy:

$$A_l \leq 0.04A_g$$

where:

A_g = gross area of member cross section (in^2)

A_l = area of longitudinal reinforcement in member (in^2)

$$\frac{A_l}{A_g} = \frac{28 \times 1}{1810} = 0.015 \quad \text{Considering 28\#9}$$

$$\frac{A_l}{A_g} \leq 0.04 \quad \text{OK}$$

AASHTO-SGS 8.8.2 Minimum Longitudinal Reinforcement

For columns in SDC B and C,the minimum area of longitudinal reinforcement for compression members shall not be less than:

$$A_l \geq 0.007A_g$$

where:

A_g = gross area of member cross section (in^2)

A_l = area of longitudinal reinforcement in member (in^2)

$$\frac{A_l}{A_g} = 0.015 \geq 0.007 \quad \text{OK}$$

Example 4: Design of a Three Span Steel Bridge in SDC A Category

Bridge Description:

This example is based on a bridge carrying Dormeus Avenue, Structure No. 0751-160. The bridge is a nine span with expansion joints at piers 3 and 6 in addition to the joints South and North Abutments. The abutments are seat type. Figures 5.47, 5.48, and 5.49 show the General Plan and Elevation of the bridge. Figures 5.50, 5.51, and 5.52 show a typical section at various piers that include the superstructure and substructure. Appendix VI.B contains superstructure details. Appendix VI.C contains substructure details.

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum Shown in Figure 5.53. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

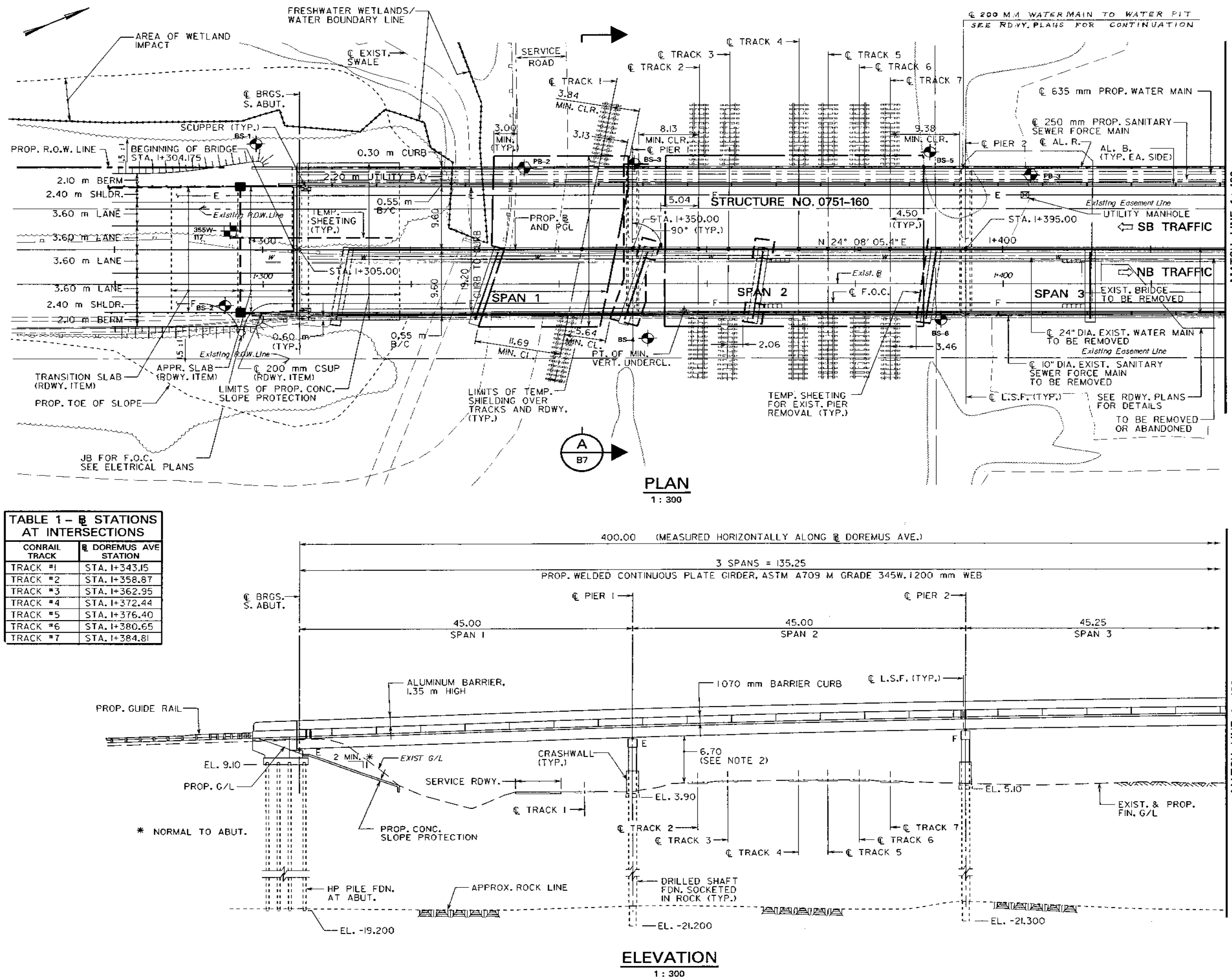


TABLE 1 - ℄ STATIONS AT INTERSECTIONS	
CONRAIL TRACK	℄ DOREMUS AVE STATION
TRACK #1	STA. 1+343.15
TRACK #2	STA. 1+358.87
TRACK #3	STA. 1+362.95
TRACK #4	STA. 1+372.44
TRACK #5	STA. 1+376.40
TRACK #6	STA. 1+380.65
TRACK #7	STA. 1+384.81

Figure 5.47 Dormeus Avenue Plan and Elevation (1 of 3)

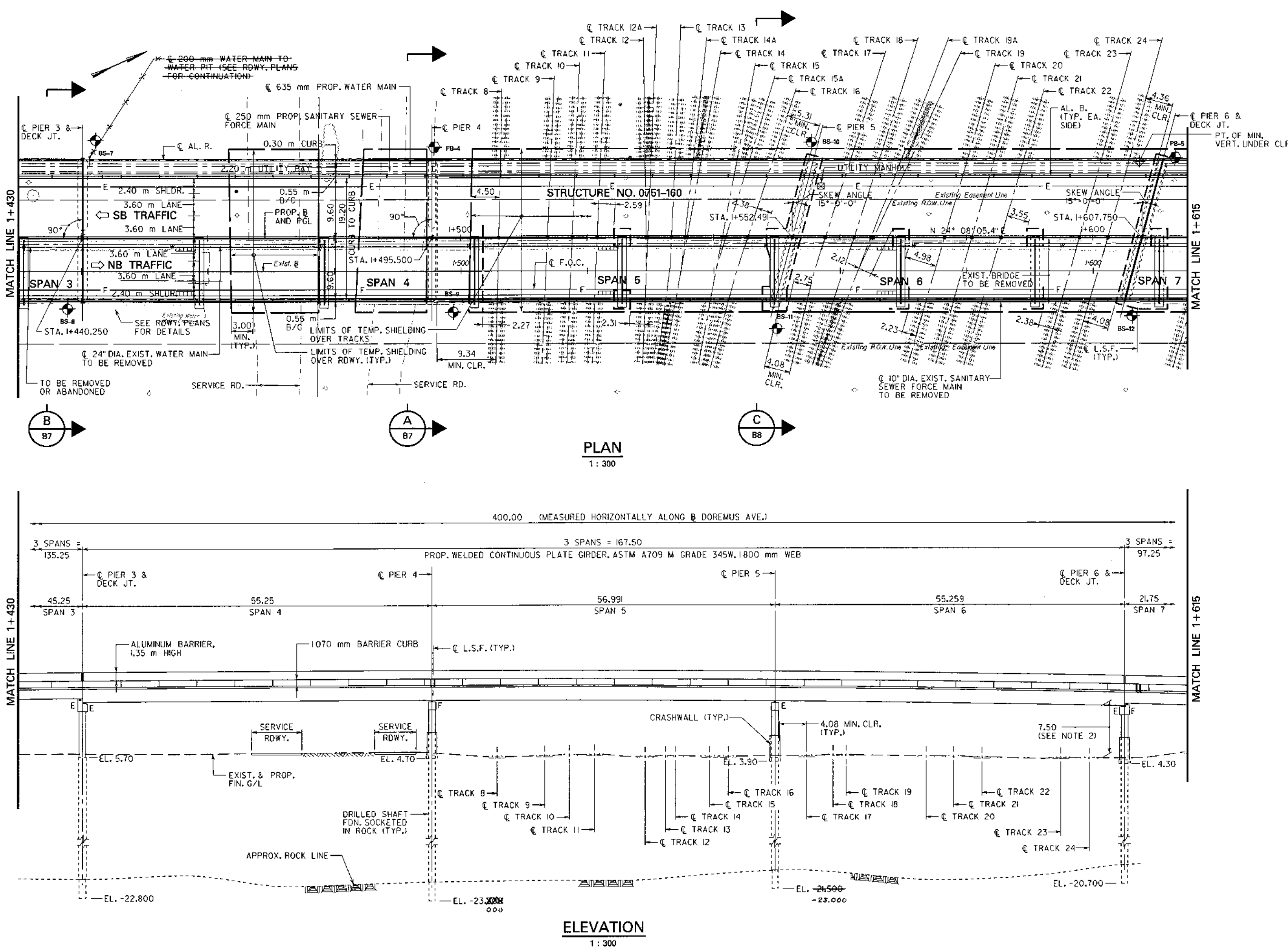


Figure 5.48 Dormeus Avenue Plan and Elevation (2 of 3)

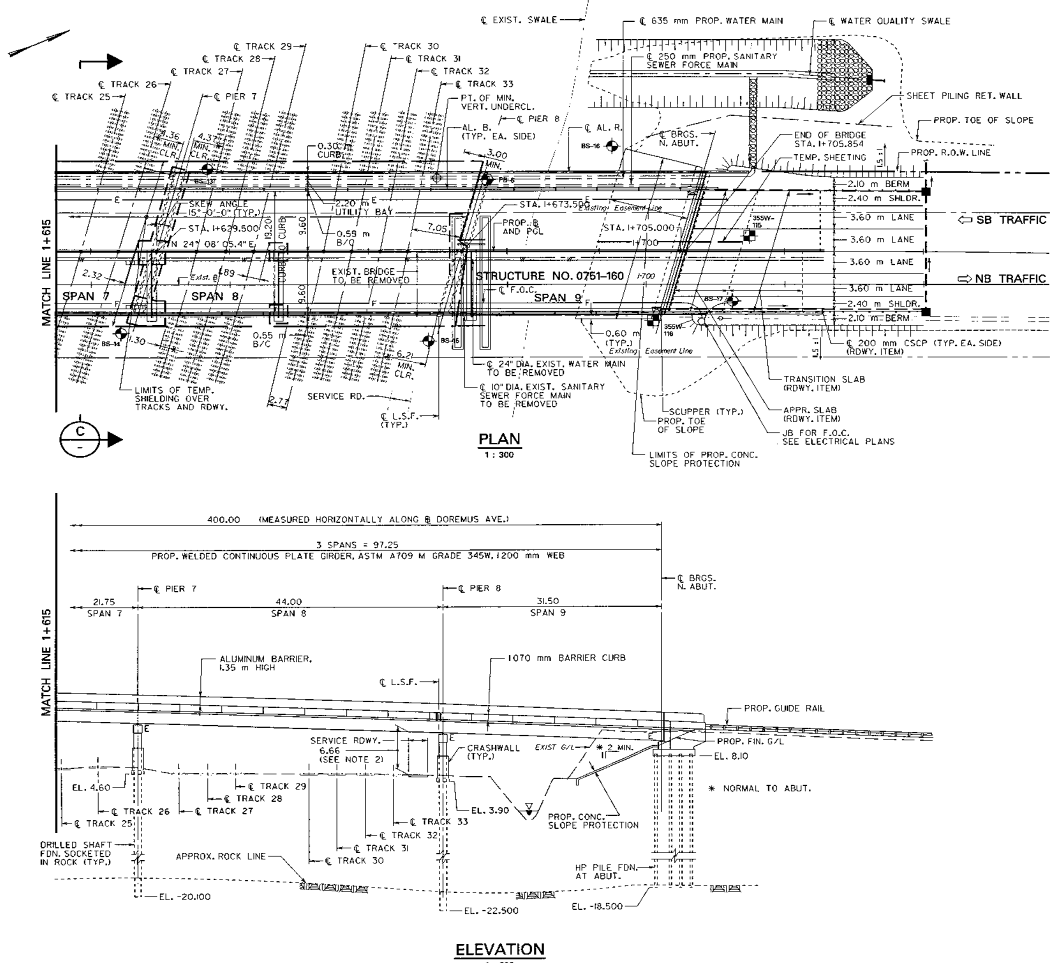


Figure 5.49: Dormeus Avenue Plan and Elevation (3of 3)

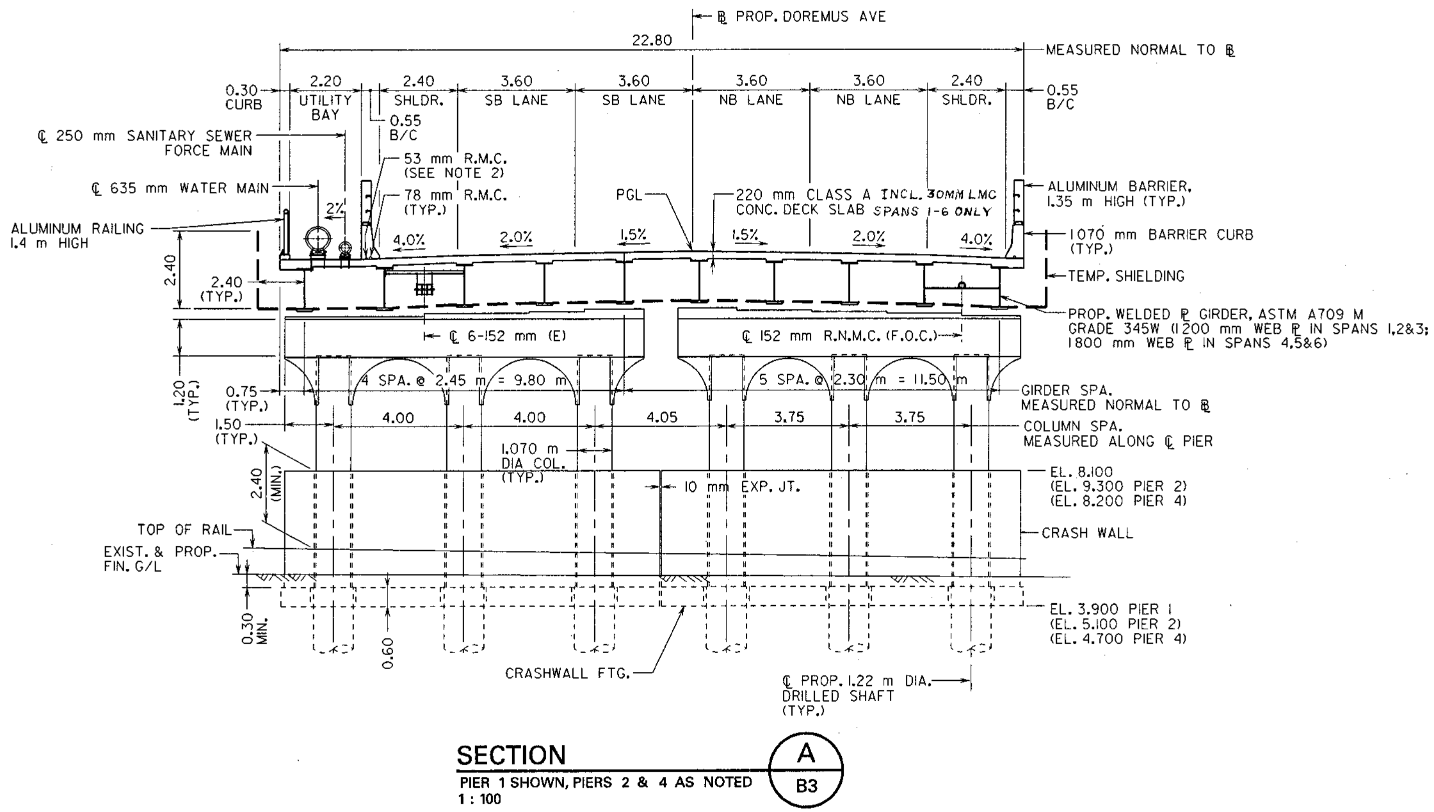


Figure 5.50 Piers 1, 2, and 4 Section

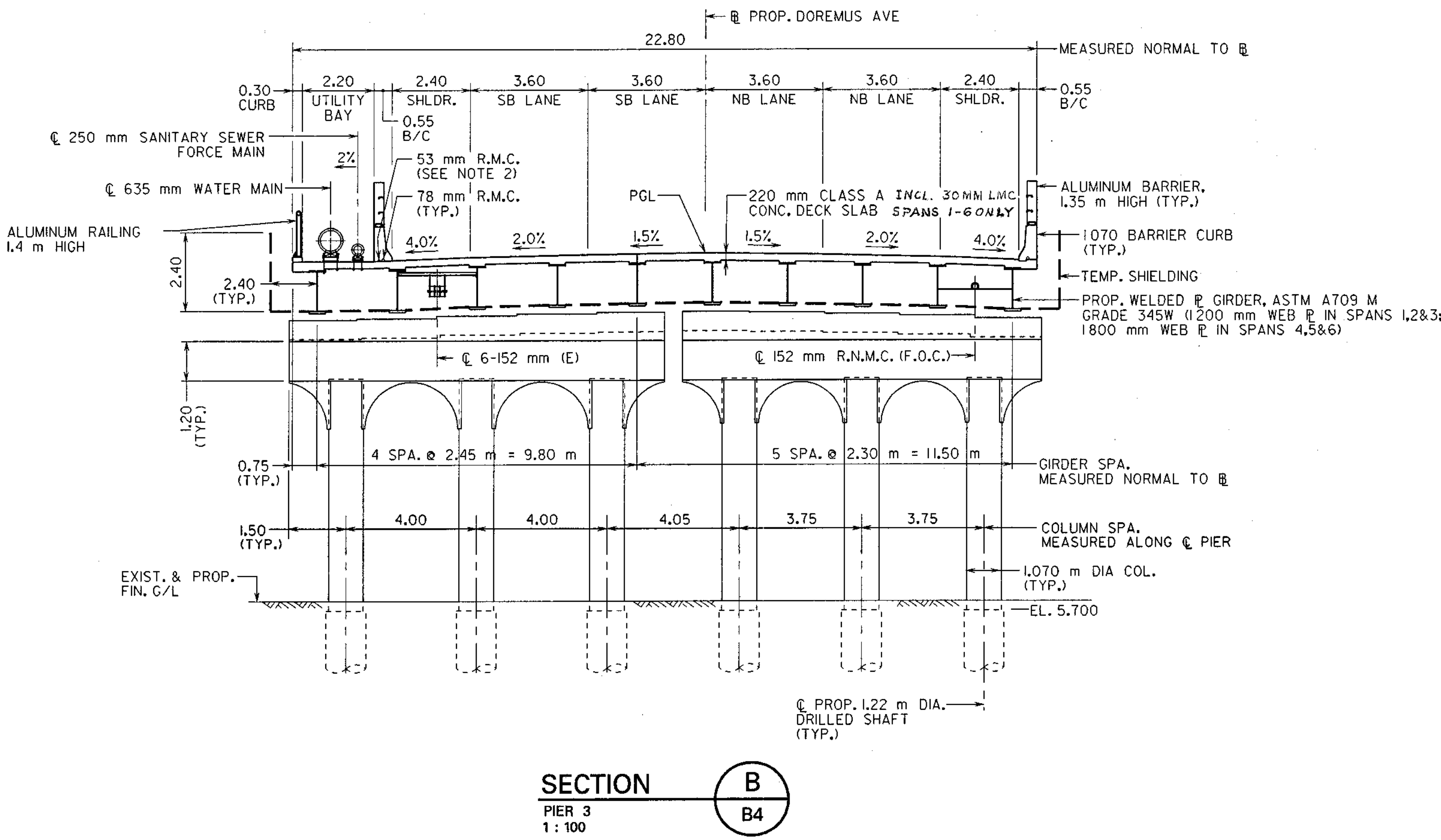
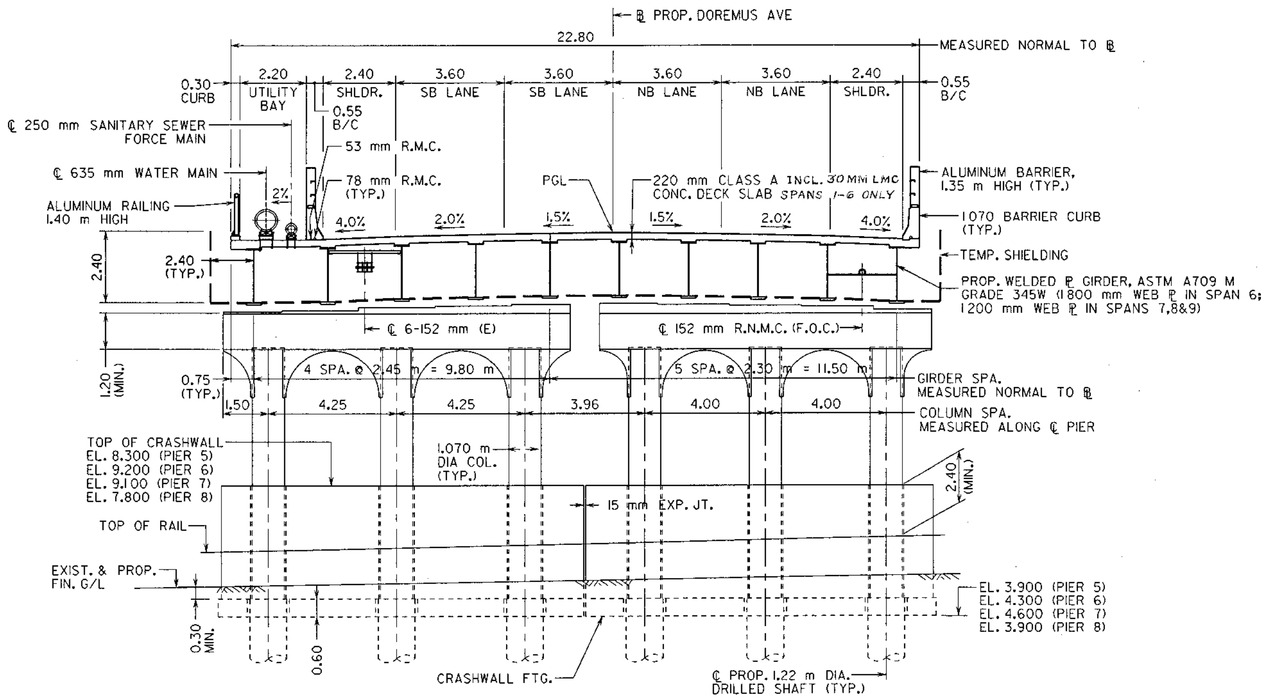


Figure 5.51 Pier 3 Section

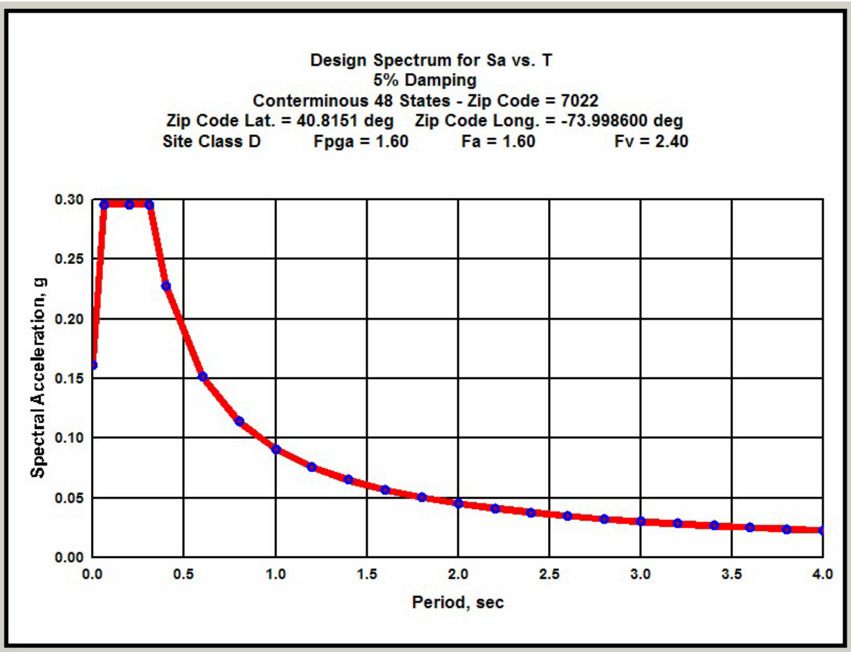


SECTION

PIER 5 SHOWN, PIERS 6,7&8 AS NOTED
1 : 100

C
B4, B5

Figure 5.52 Piers 5, 6, 7, and 8 Section



Graph Data	
Period, sec	Sa, g
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0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.53 AASHTO-USGS Site Class D Unfactored Design Spectrum

Flow charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 4 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path. However, this level of examination of the load path to the substructure is not applicable to SDC A.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.54 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a multi-span bridge.

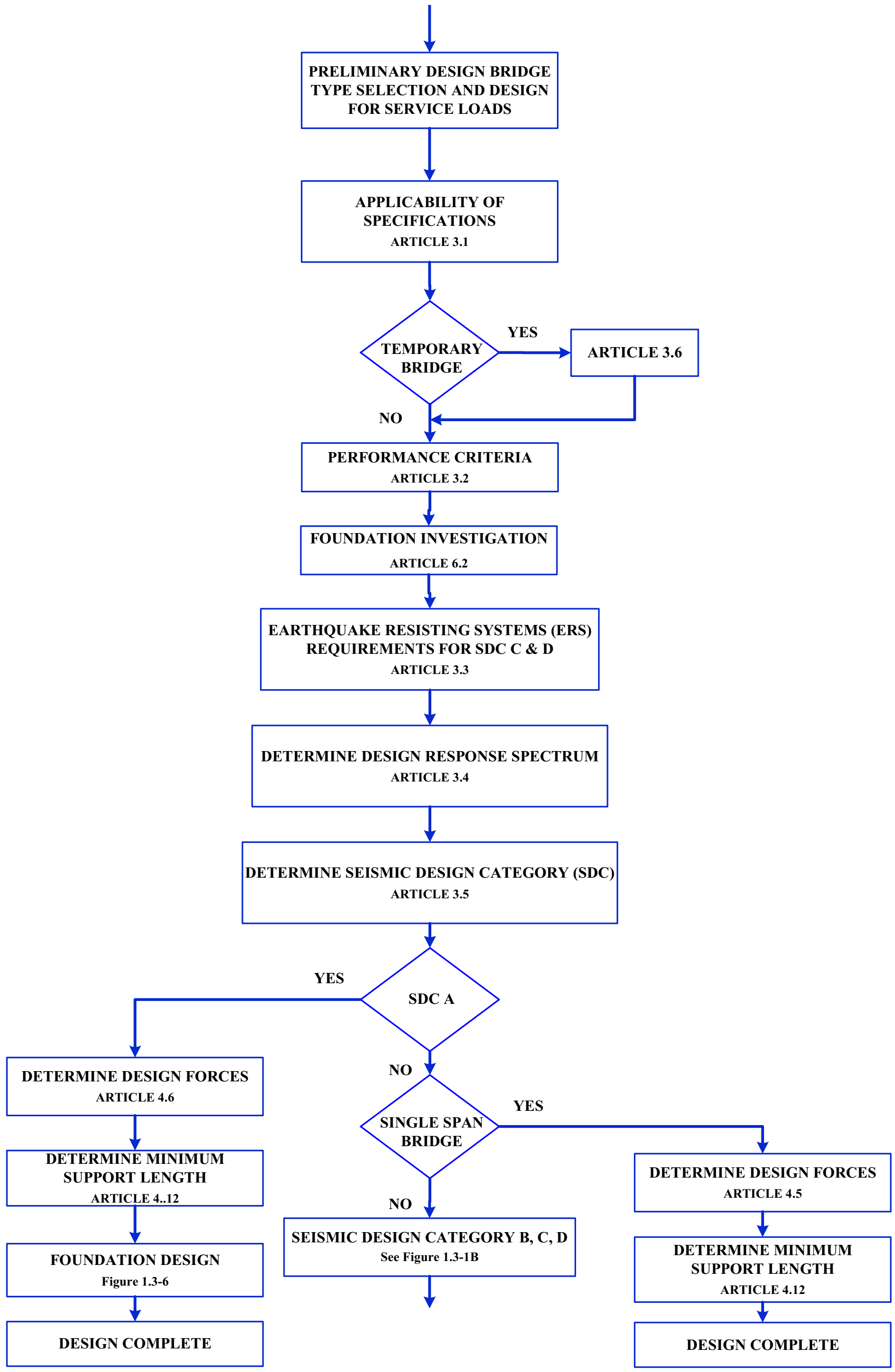


Figure 5.54 Seismic Design Procedure Flow Chart

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.6.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}

Table 5.6 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.55 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} = 0.14 the example bridge is treated in SDC A with the following basic requirements:

- No Identification of ERS according to Article 3.3
- No Demand Analysis
- No Implicit Capacity Check Needed
- No Capacity Design Required
- Minimum detailing requirements for support length, superstructure/substructure connection design force, and column transverse steel
- No Liquefaction Evaluation Required

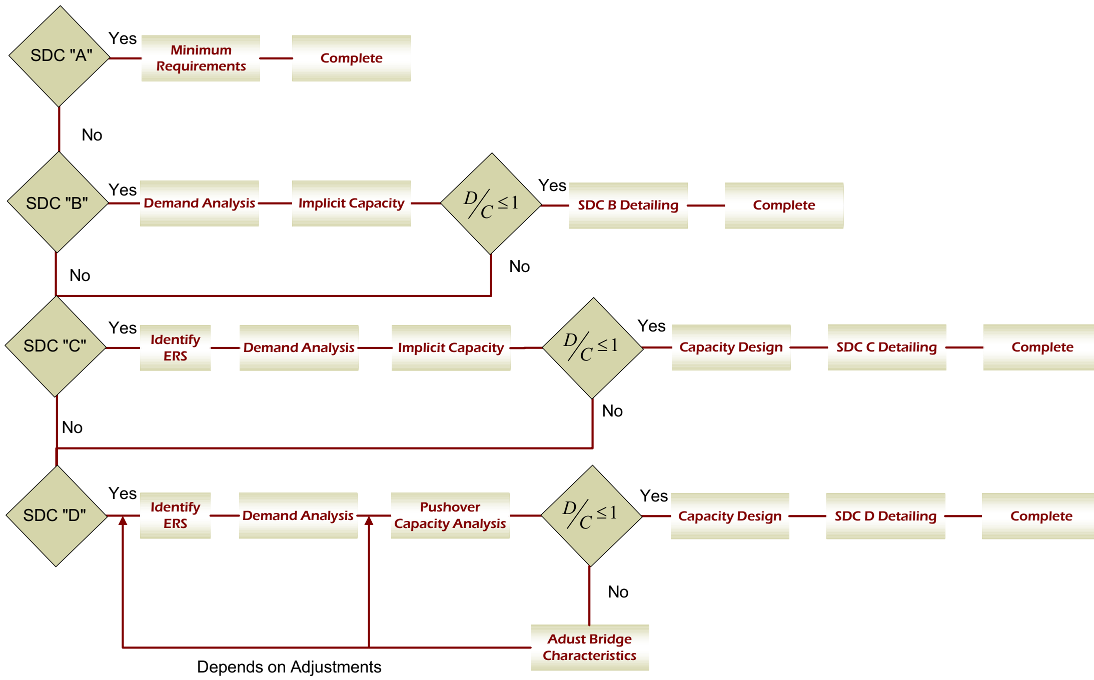


Figure 5.55 Seismic Design Category (SDC) Core Flowchart

Bridge Bearing Connections

According to Section 4.6 of the AASHTO-SGS, for bridges in SDC A, where the acceleration coefficient, As, as specified in Article 3.4., is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, assumed to exist during an earthquake.

The NJ PGA calculated in the Site Seismicity Section is shown equal to 0.24g. Therefore, the horizontal design connection force is considered at the minimum of 0.25g mentioned above.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Frame 2 consisting of spans 4, 5,and 6, is examined in detail given that it includes the largest span length of 55.25, 56.99, and 55.26m (184, 190, and 184ft) with one single pier 4 having fixed bearings and all other piers having expansion PTFE bearings. Lubricated PTFE has a coefficient of friction range between 0.08 and 0.03 while un-lubricated PTFE has a coefficient of friction range between 0.16 and 0.06 depending on the pressure exerted on the confined PTFE. For purpose of simplifying the seismic analysis, and given that there are no longitudinal devices or keys to resist any significant force at Piers 3, 5, and 6, the tributary mass of spans 4, 5, and 6 is applied at Pier 6 in the longitudinal direction in contrast to all piers sharing the resistance in the transverse direction.

The bearing loads are shown in table 5.7 below and used to compute the dead load at Piers 3, 4, 5, and 6 as shown in tables 5.8, 5.9, 5.10, and 5.11

Table 5.7 Bearing Service Loads

BEARING DESIGNATION	TYPE FIX./EXP.	QUANTITY REQUIRED	DL (KN)	LL + IM		LONGITUDINAL (KN)	TRANSVERSE (KN)	MOVEMENT (mm)
				MAX. (KN)	MIN. (KN)			
E1.EA1	EXP.	20	600	550	-40	0	400	40
E2.EA2	EXP.	10	600	550	-155	0	350	75
E3.EA3	EXP.	20	1200	850	0	0	250	45
E4.EA4	EXP.	20	1900	1100	0	0	350	40
E5.EA5	EXP.	20	600	550	-155	0	350	65
F1	FIX.	20	1900	1100	0	475	125	0
F2	FIX.	10	105	445	-70	425	25	0

Table 5.8 Expansion Bearings at Pier 3 Supporting Span 4

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E1	600	135
PG12	E1	600	135
PG13	EA1	600	135
PG14	E1	600	135
PG15	E1	600	135
PG16	E1	600	135
PG17	E1	600	135
PG18	EA1	600	135
PG19	E1	600	135
PG20	E1	600	135

Total 10 Girders x 135 Kips/each = 1350 Kips

Table 5.9 Fixed Bearings at Pier 4 Supporting Spans 4 and 5

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11 Through PG20	F1	1900	427

Span Total 10 girders x 427 Kips/each=4270 Kips

Table 5.10 Expansion Bearings at Pier 5 Supporting Spans 5 and 6

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E4	1900	427
PG 12	EA4	1900	427
PG 13	E4	1900	427
PG 14	E4	1900	427
PG 15	E4	1900	427
PG 16	E4	1900	427
PG 17	E4	1900	427
PG 18	E4	1900	427
PG 19	E4	1900	427
PG 20	E4	1900	427

Total 10 girders x 427 Kips/each =4270 Kips

Table 5.11 Expansion Bearings at Pier 6 Supporting Span 6

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E2	600	135
PG 12	E2	600	135
PG 13	EA2	600	135
PG 14	E2	600	135
PG 15	E2	600	135
PG 16	E2	600	135
PG 17	E2	600	135
PG 18	EA2	600	135
PG 19	E2	600	135
PG 20	E2	600	135

Total 10 girders x 135 Kips/each =1350 Kips

Longitudinal Mass Tributary to one girder line for fixed bearing at Pier 4:

135+427+427+135 = 1124 Kips

Longitudinal Load:

$$\frac{427}{g} \times 0.25g = 281 \text{ Kips}$$

Transverse Load:

$$\frac{427}{g} \times 0.25g = 107 \text{ Kips}$$

Considering Loading Combination:

$1.0 \times$ Longitudinal +0.3 $\times$ Transverse

The vector sum of Transverse and Longitudinal is calculated as:

$$\sqrt{281^2 + (0.3 \times 107)^2} = \sqrt{281^2 + 32^2} = 283 \text{ Kips}$$

The 283 Kips is applied to the fixed bearing at Pier 4.

Consider $1\frac{1}{2}''$ ϕ bolt:

According to AASHTO-SGS section 6.13: $R_n = 0.48 A_b F_{ub} N_s$

R_n0.48×1.77×60=51 Kips

$\phi_s R_n = 0.65 \times 51 = 33$ Kips (A307 bolts in shear $\phi_s = 0.65$)

For 1" and 2" ϕ bolts (See Experimental Testing of Anchor Bolts Appendix IV.A)

P_{crack} = 13.7 Kips @ $\Delta_{crack} = 0.96''$ for 1" ϕ bolts

P_{crack} = 16.8 Kips @ $\Delta_{crack} = 0.04''$ for 2" ϕ bolts

Connection Capacity Considering 4 bolts: 4×33 Kips = 132 < 283 Kips

where 283 kips is the connection lateral load demand.

A longitudinal external shear key is required to provide a load sharing mechanism to other bents if minimal damage requirement is to be satisfied.

Transverse Load demand @ expansion bearings is 107 Kips compared to a capacity of 132 Kips.

Consider West bent at Pier4 (See Figure VI.C.4 and VI.C.5):

Average Pedestal Elevation

$$\frac{14.36+14.596}{2}=14.5 \text{ m}$$

Depth of West Cap:

(14.5-13.1)×3.33 ft/m = 4.66 ft

West Cap Weight:

$11\times1.6\times(3.33)^2\times4.66\times0.15\ K/ft^3 = 137$ Kips

Consider 10% added weight for flares, total weight is calculated as:

1.1×137 = 151 Kips

Calculate Column Height as shown in table 5.12 below:

Table 5.12 Piers 3, 4, 5, and 6 Column Height

Pier	Elevation A	Bottom "Cap"	Height(m)	Height (ft)
3	5.7	12.9	7.2	24.0
4	5.3	13.1	7.8	26.0
5	4.5	12.8	8.3	27.6
6	4.9	11.95	7.1	23.6

Elevation A refers to bottom of column as shown in Figure VI.C.12 and Table 5.13 below:

Table 5.13 Pier 1 to 8 Elevations

PIER NO.	ELEVATION			DRILLED SHAFT LENGTH D
	"A"	"B"	"C"	
1	4.500	-18.200	-21.200	25.7
2	5.700	-18.300	-21.300	27.0
3	5.700	-19.800	-22.800	28.5
4	5.300	-20.000	-23.000	28.3
5	4.500	-20.000	-23.000	27.5
6	4.900	-17.700	-20.700	25.6
7	5.200	-17.100	-20.100	25.3
8	4.500	-19.500	-22.500	27.0

Check minimum support length

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.14 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; For hinges within a span, L shall be the sum of the distances to either side of the hinge; For single-span bridges, L equals the length of the bridge deck (ft.)

H = for abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.) for columns and/or piers, column, or pier height (ft.) for hinges within a span, average height of the adjacent two columns or piers (ft.) 0.0 for single-span bridges (ft.)

S = angle of skew of support measured from a line normal to span (°)

Table 5.14 Percentage N by SDC and Effective Peak Ground Acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percent N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC A:

$N = 1.0(8+0.02L+0.08H)(1+0.000125S^2)$

L = 558 ft calculated based on the total length of three continuous spans 4, 5, and 6 from Pier 3 to Pier 6.

H = 68 ft (Including length to point of fixitity)

H = 28 ft for column only

$S = 15^\circ$ at pier 6

$N = 1.0\ (8+0.02\times558+0.08\times68)(1+.000125\times15^2)$

= (8+11.16+5.4)(1+0.028)

= 25.3 in

Cap Width 1.6m or 5.3 ft (See Figure VI.C.15 and VI.C.16)

Half Cap Width 32 in.

Expansion Joint 210 min or 4” (See Figure VI.B.10)

Available Cap Width 32 in – 2 in = 30 in

Calculate N based on H = 28 ft

N = (8+0.02×558+0.08×28)(1+0.028)

= (8+11.16+2.2)(1.028)

= 22 in

Available support length slightly more than the required support length.

Example 5: Design of a Three Span Steel Bridge in SDC B Category

Bridge Description:

This example is based on a bridge carrying Dormeus Avenue, Structure No. 0751-160. The bridge is a nine span with expansion joints at piers 3 and 6 in addition to the joints South and North Abutments. The abutments are seat type. Figures 5.56, 5.57, and 5.58 show the General Plan and Elevation of the bridge. Figures 5.59, 5.60, and 5.61 show a typical section at various piers that include the superstructure and substructure. Appendix VI.A contains pier analysis. Appendix VI.B contains superstructure details. Appendix VI.C contains substructure details.

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum Shown in Figure 5.62. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.

- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.

- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.

- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

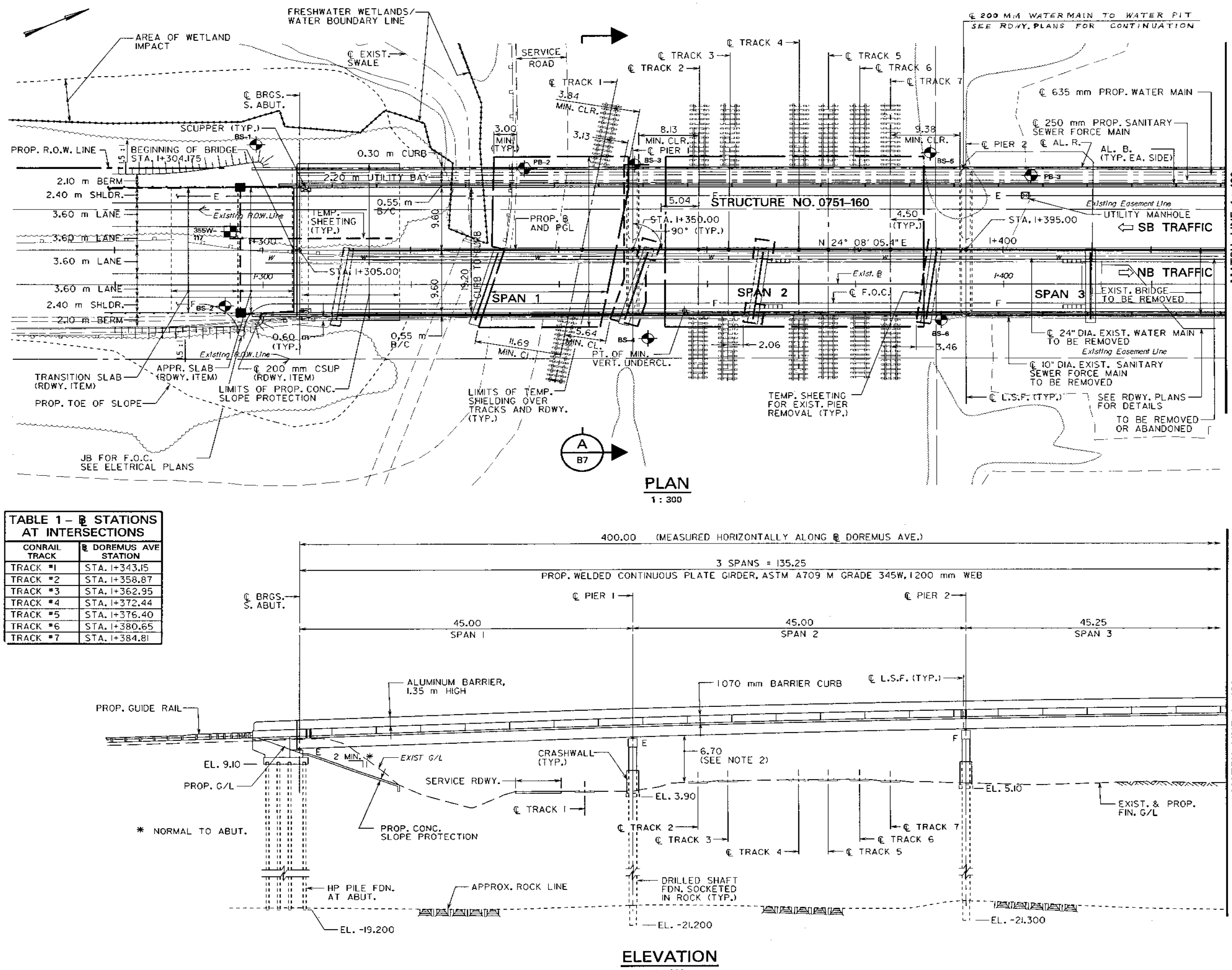


TABLE 1 – ℄ STATIONS AT INTERSECTIONS

CONRAIL TRACK	℄ DOREMUS AVE STATION
TRACK #1	STA. 1+343.15
TRACK #2	STA. 1+358.87
TRACK #3	STA. 1+362.95
TRACK #4	STA. 1+372.44
TRACK #5	STA. 1+376.40
TRACK #6	STA. 1+380.65
TRACK #7	STA. 1+384.81

Figure 5.56 Dormeus Avenue Plan and Elevation (1 of 3)

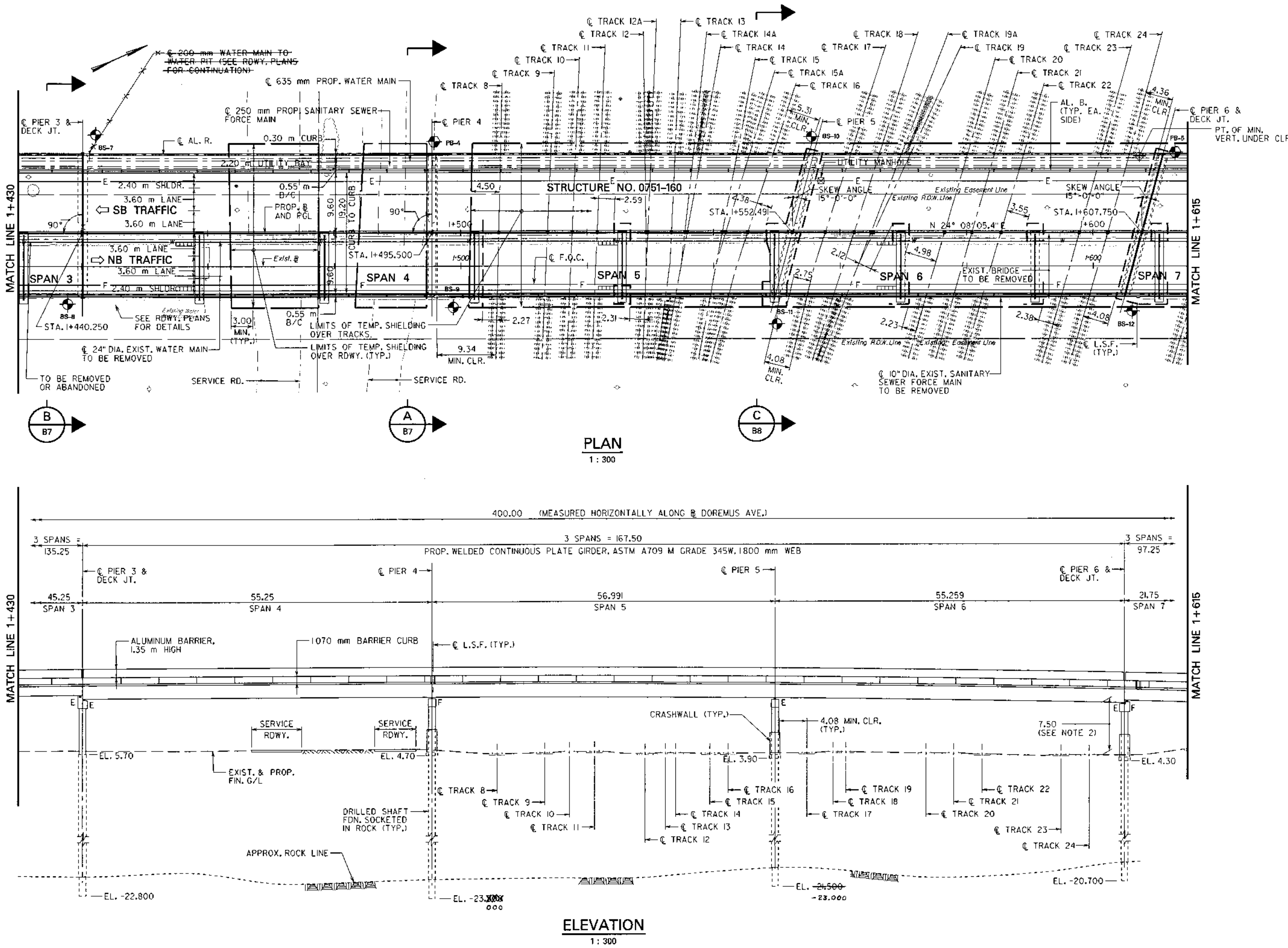


Figure 5.57 Dormeus Avenue Plan and Elevation (2 of 3)

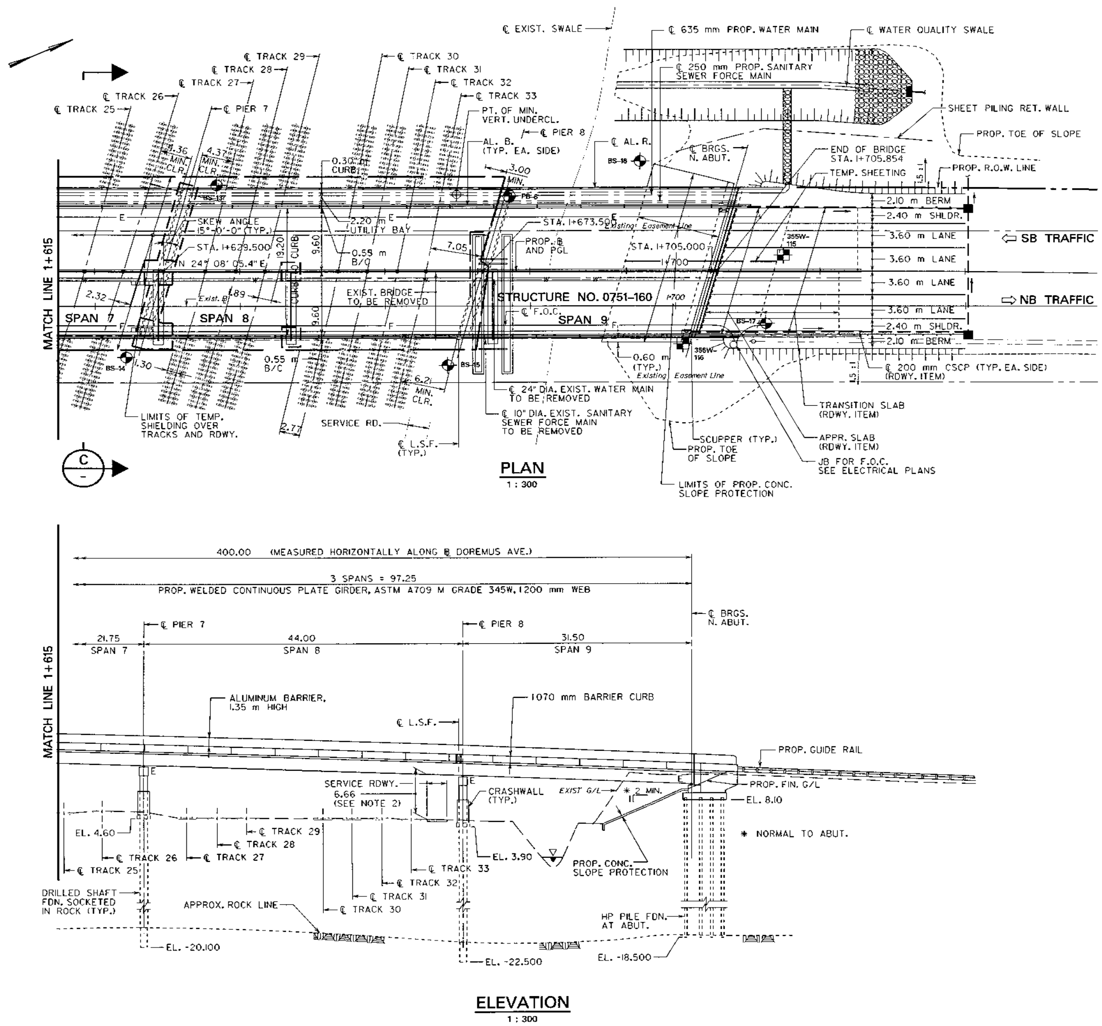


Figure 5.58 Dormeus Avenue Plan and Elevation (3 of 3)

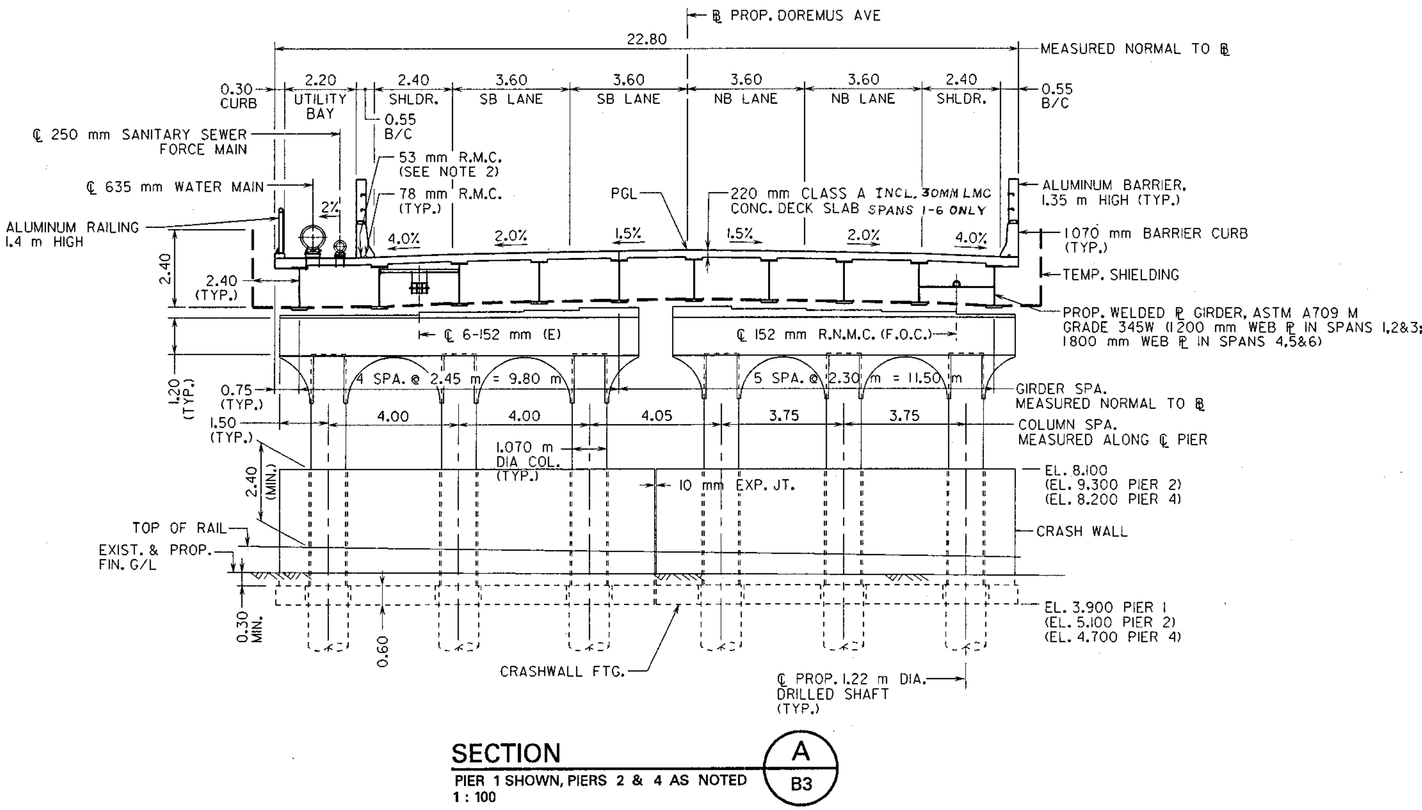


Figure 5.59 Piers 1, 2, and 4 Section

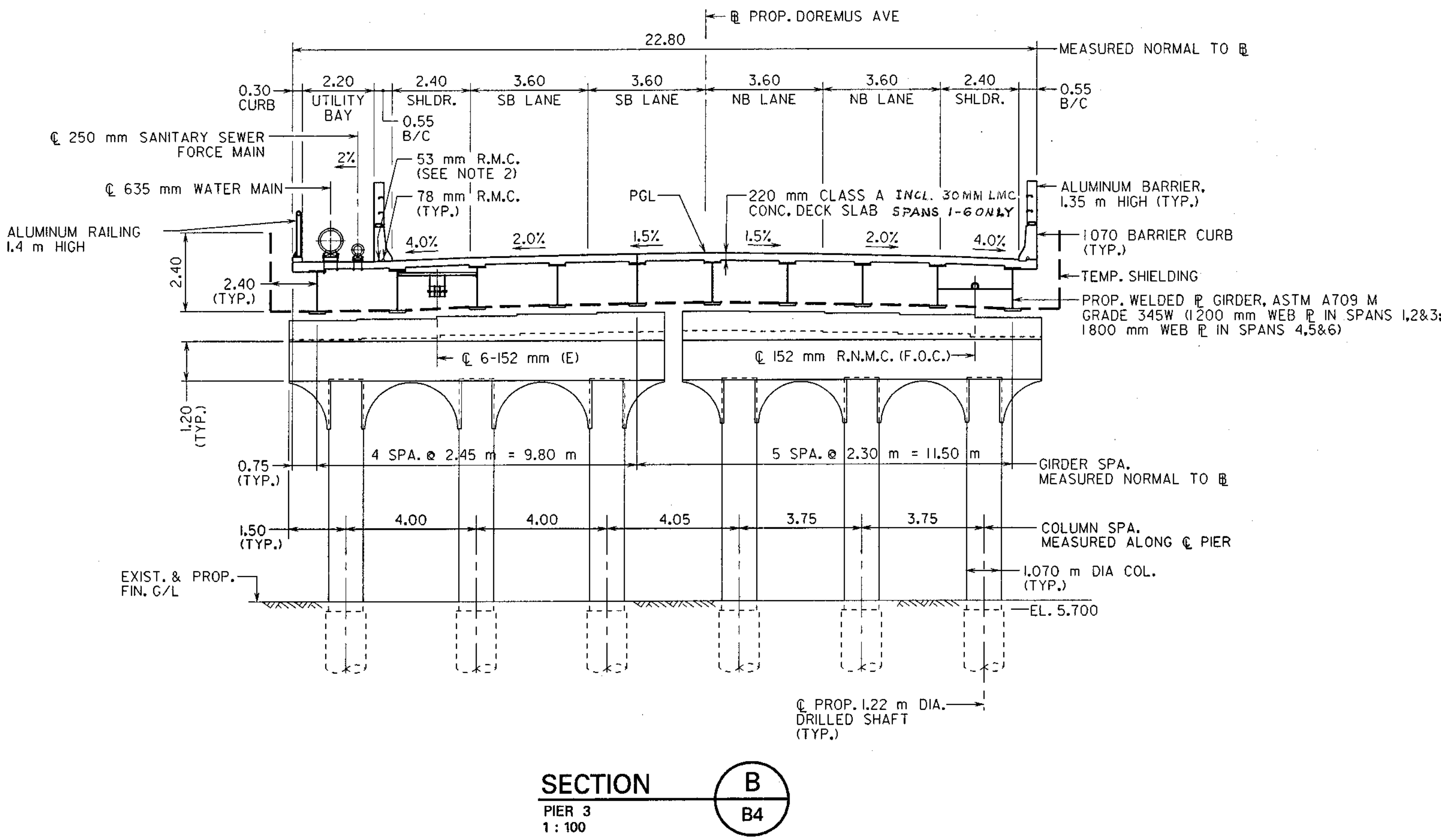


Figure 5.60 Pier 3 Section

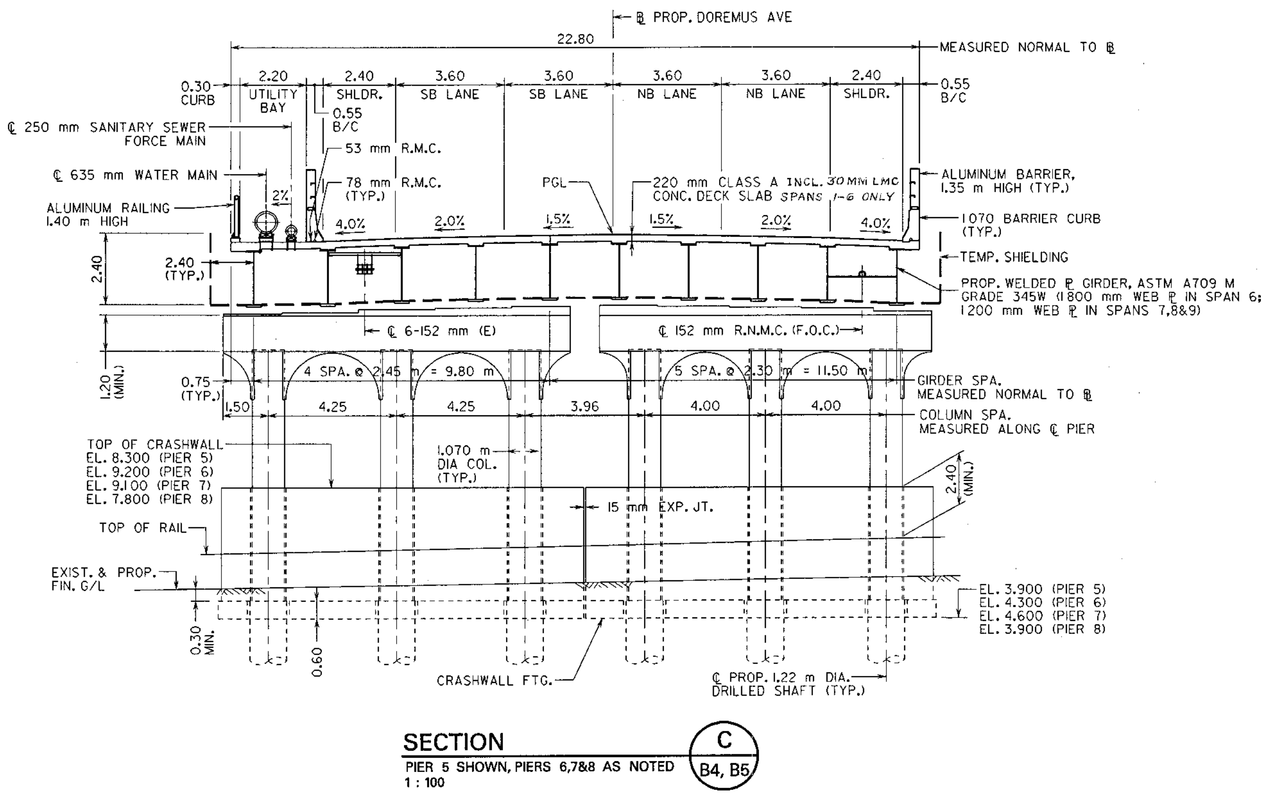
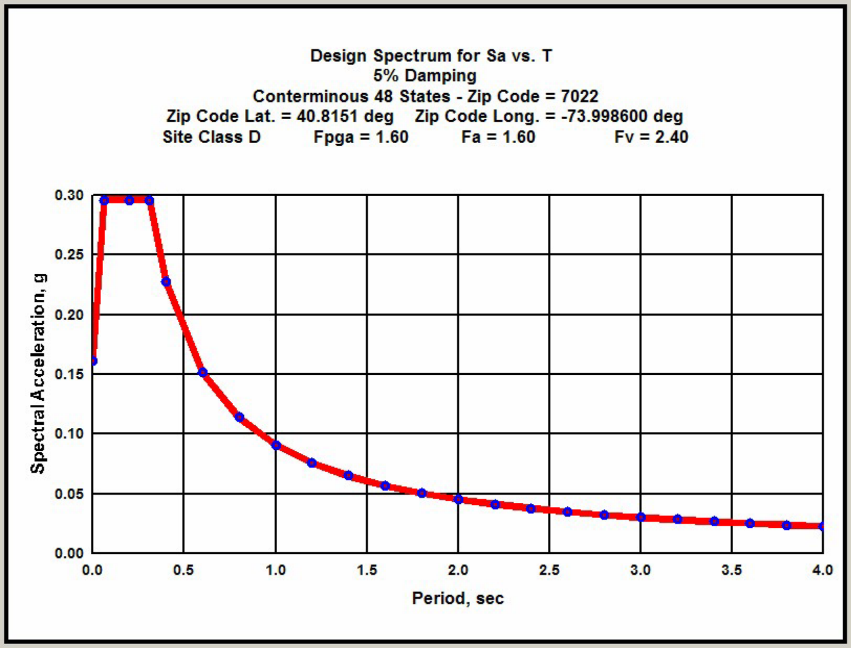


Figure 5.61 Piers 5, 6, 7, and 8 Section



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.62 AASHTO-USGS Site Class D Unfactored Design Spectrum

Flow charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 5 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.63 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a multi-span bridge. Figure 5.64 shows the core flow chart of procedures outlined for bridges in SDC B, C, and D. Figure 5.65 outlines the demand analysis. Figure 5.66 directs the designer to determine displacement capacity. Figure 5.67 shows the modeling procedure. Figure 5.68 shows the foundation and abutment design applicable mainly for SDC C and D.

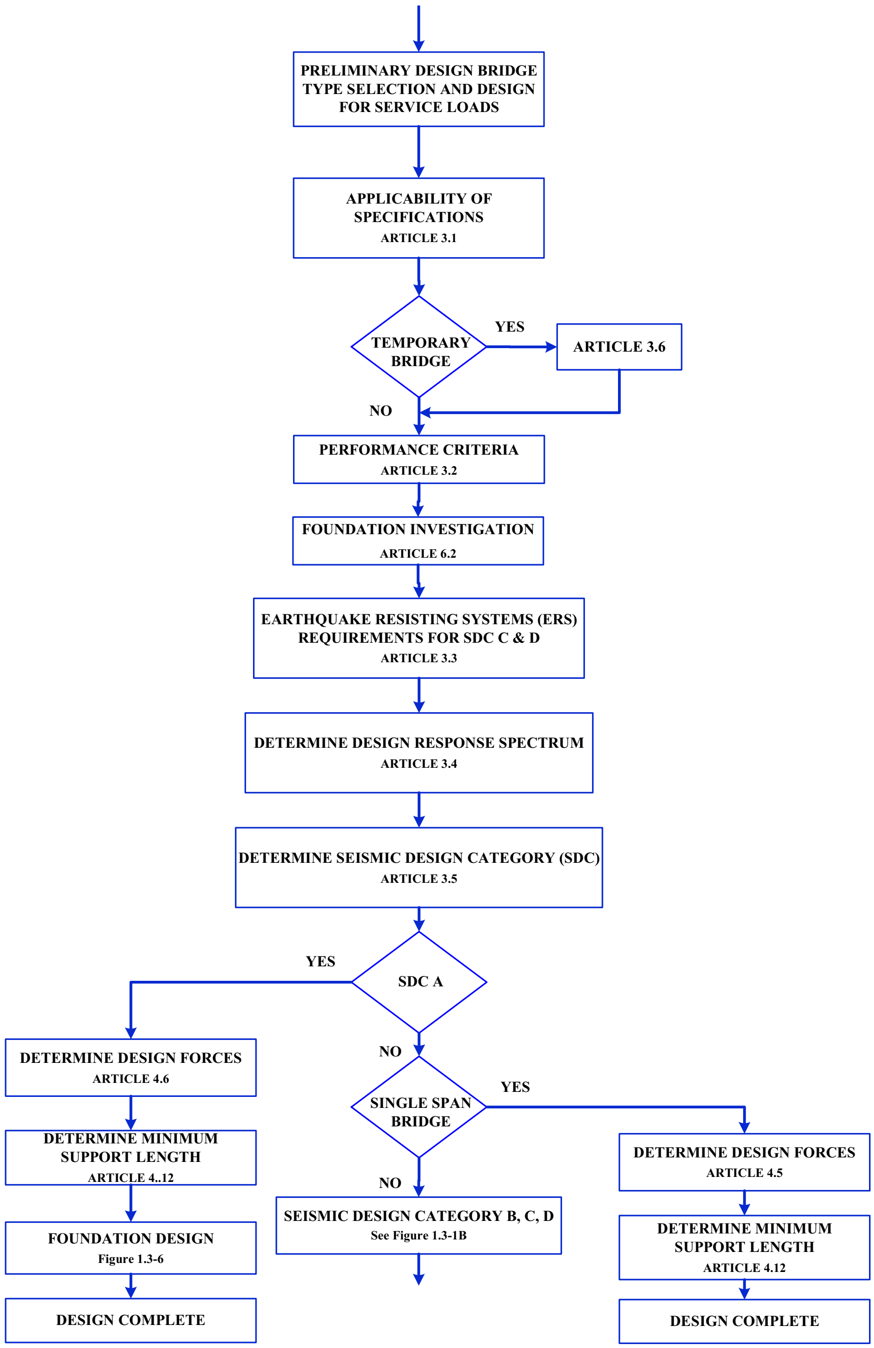


Figure 5.63 Seismic Design Procedure Flow Chart 1a

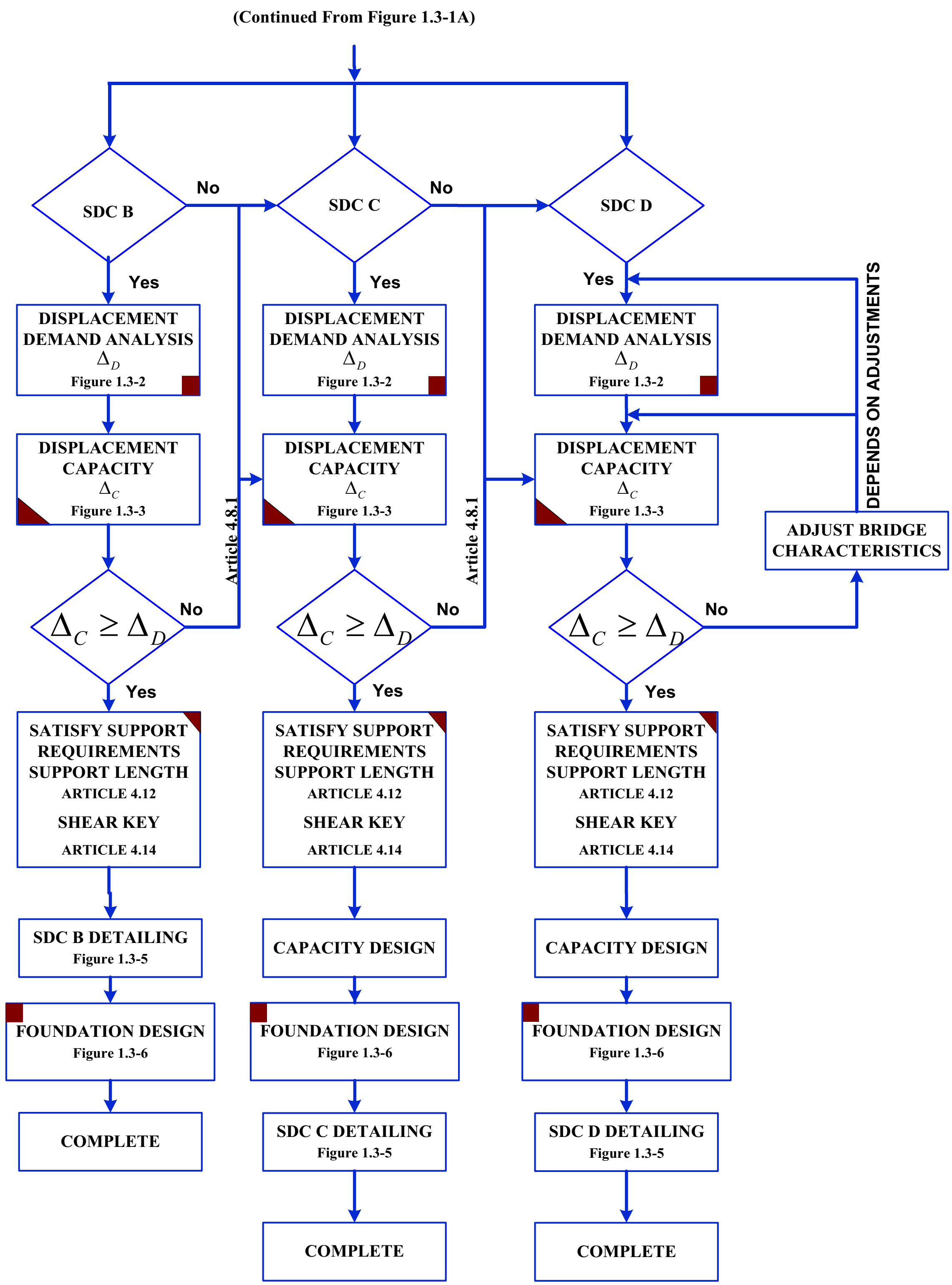


Figure 5.64 Seismic Design Procedure Flow Chart 1b

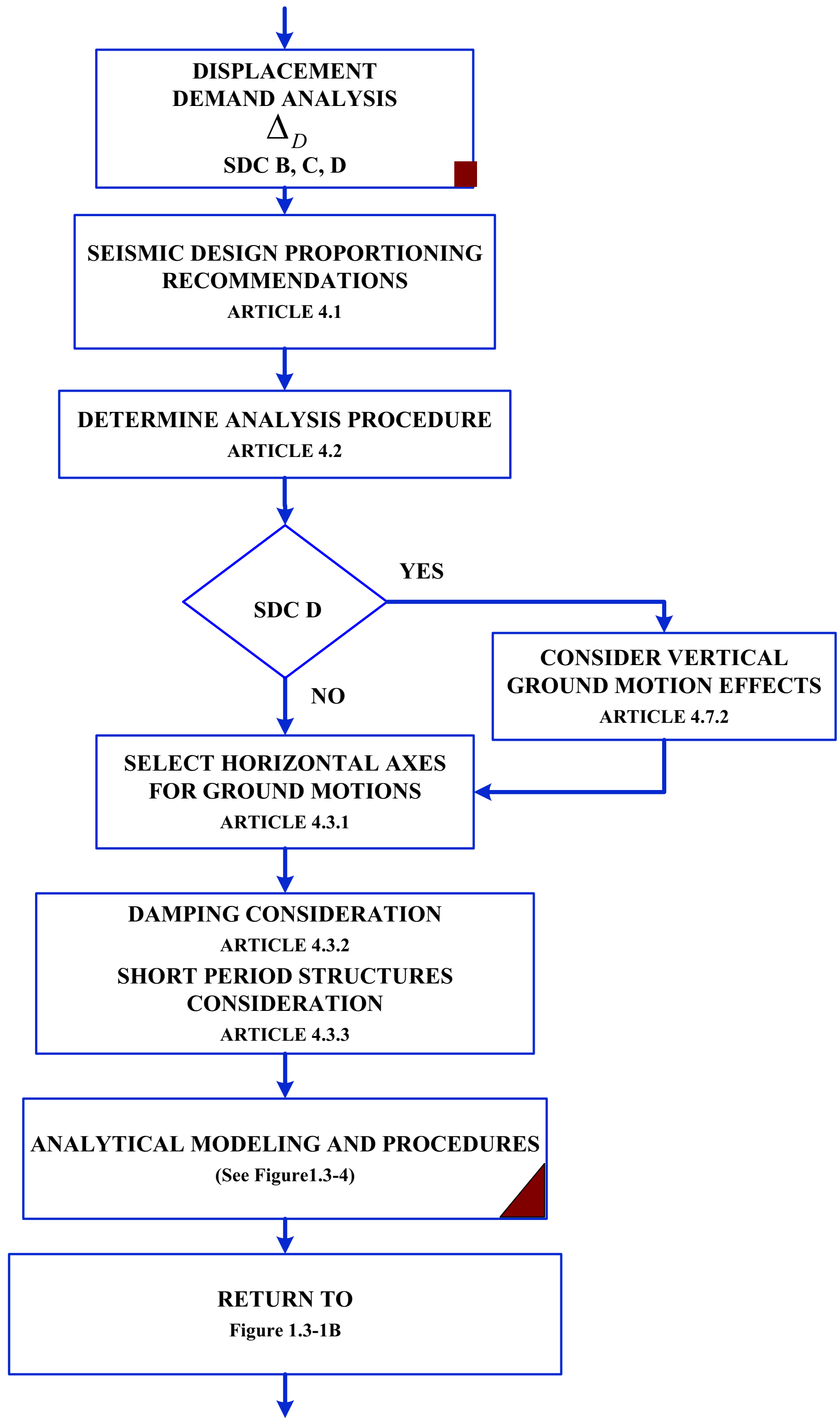


Figure 5.65 Demand Analysis Flow Chart 2

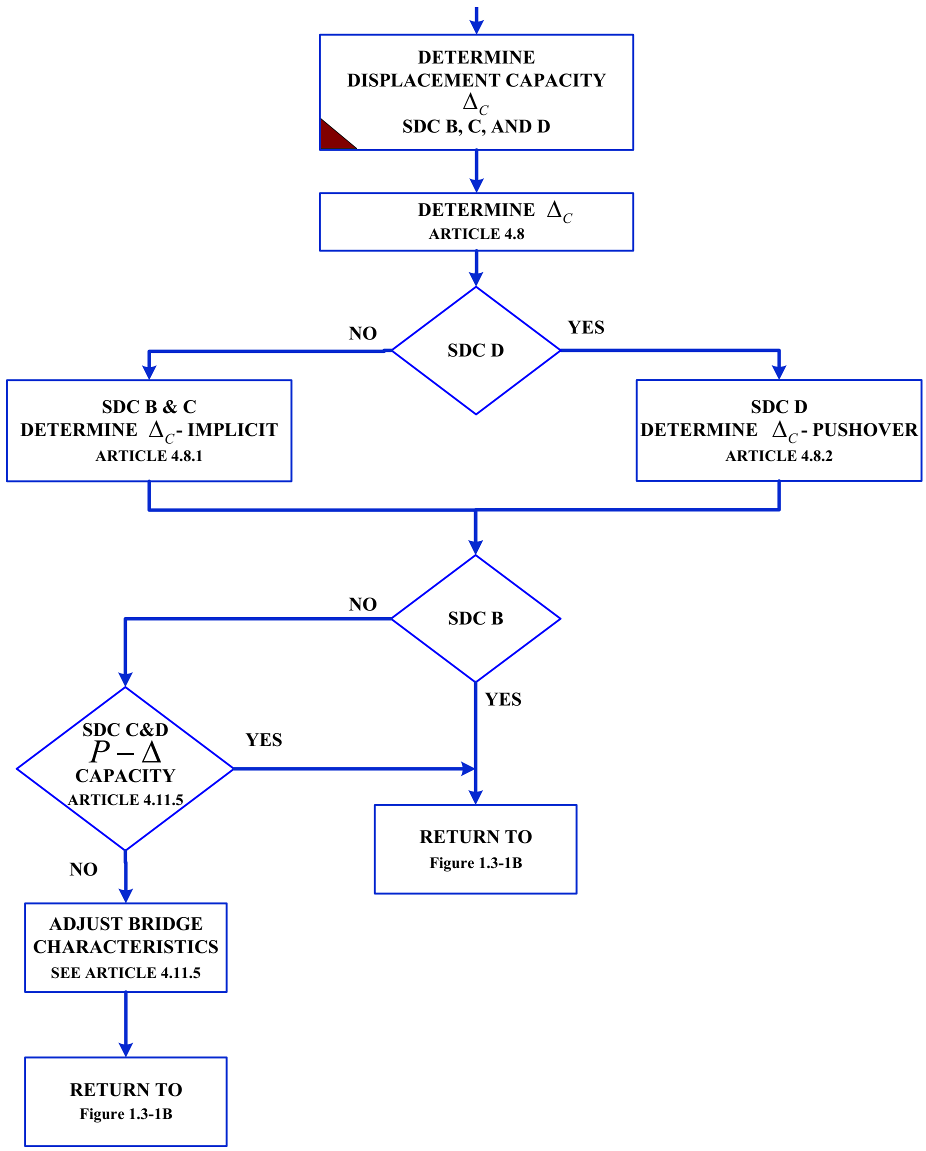


Figure 5.66 Displacement Capacity Flow Chart 3

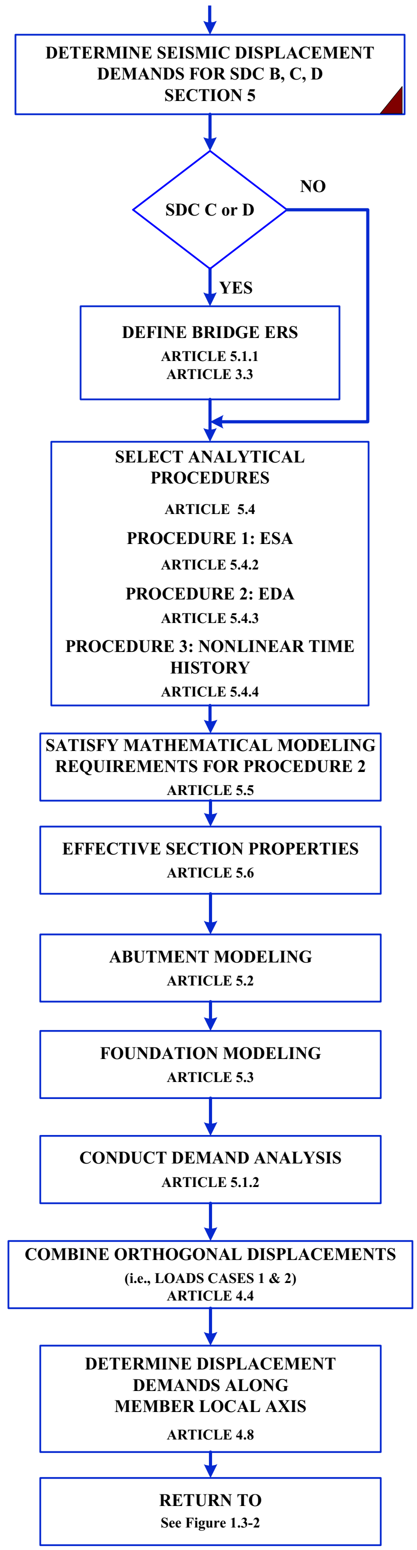


Figure 5.67 Modeling Procedure Flowchart 4

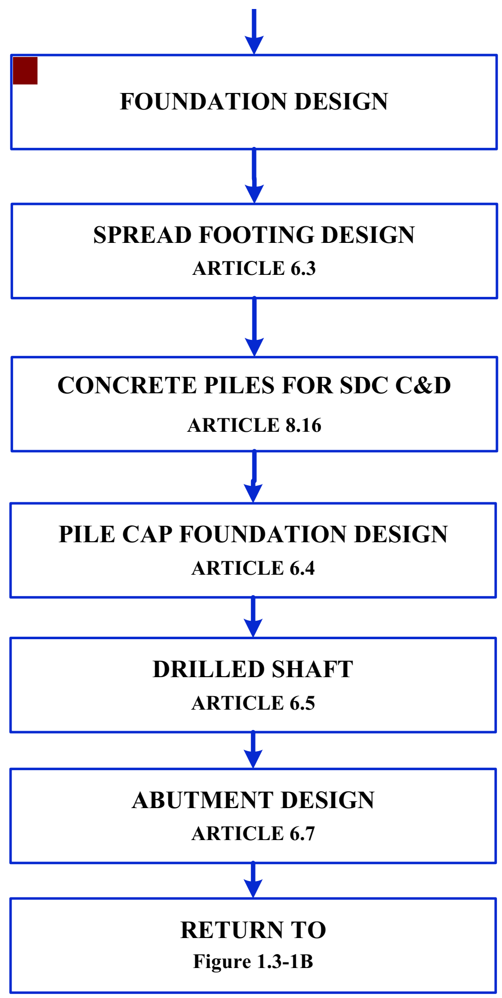


Figure 5.68 Foundation Design Flowchart 6

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.4.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1} .

Table 5.15 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.69 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} =0.14, the example bridge is treated in SDC B with the following basic requirements:

- No Identification of ERS according to Article 3.3
- Demand Analysis
- Implicit Capacity Check Required (displacement, P–Δ support length)
- No Capacity Design Required except for column shear requirement
- SDC B Level of Detailing

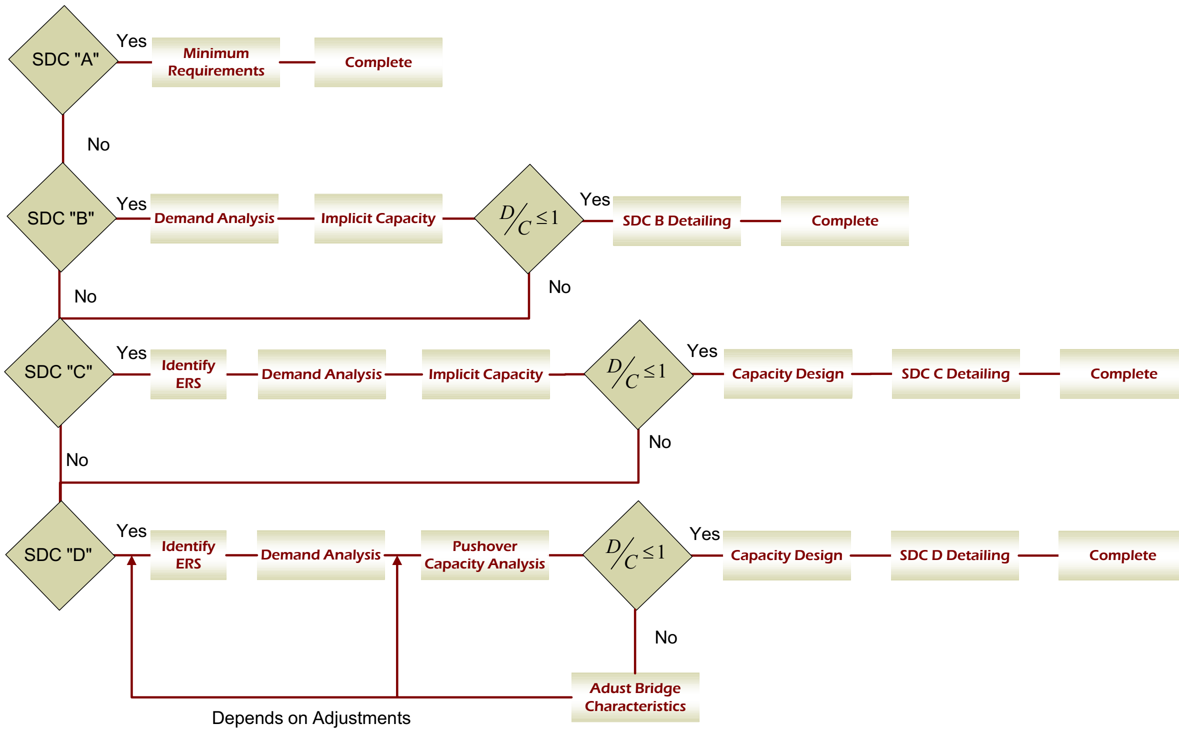


Figure 5.69 Seismic Design Category (SDC) Core Flowchart

Selection of Analysis Procedure

Minimum requirements for the selection of an analysis method to determine seismic demands for a particular bridge type shall be taken as specified in Tables 5.16 and 5.17. Applicability shall be determined by the “regularity” of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges shall be taken as those having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and which satisfy the requirements in Table 5.18. Any bridge not satisfying the requirements of Table 5.17 shall be considered “not regular”.

Table 5.16 Analysis Procedures.

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or more Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

Table 5.17 Description of Analysis Procedures.

Procedure Number	Description	Article
1	Equivalent Static	5.4.2
2	Elastic Dynamic Analysis	5.4.3
3	Nonlinear Time History	5.4.4

Procedure 3 is generally not required unless:

- P-Δ effects are too large to be neglected,
- damping provided by a base isolation system is large,
- requested by the owner per Article 4.2.2

Table 5.18 Regular Bridge Requirements.

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30º	30º	30º	30º	30º
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

Note: All ratios expressed in terms of the smaller value.

According to the AASHTO-SGS 5.3.1, the Foundation Modeling Methods (FMM) defined in Table 5.8 should be used as appropriate. The requirements for estimating foundation springs for spread footings, pile foundations, and the depth to fixity for drilled shafts shall be as specified in AASHTO-SGS Articles 5.3.2, 5.3.3 and 5.3.4, respectively. For a foundation which is considered as rigid, the mass of the foundation should be ignored in the analytical model. The Engineer shall assess the merits of including the foundation mass in the analytical model where appropriate taking into account the recommendations in this Article.

The required FMM depends on the SDC:

- FMM I is permitted for SDCs B and C provided the foundation is located in Site Class A, B, C, or D. Otherwise FMM II is required.
- FMM II is required for SDC D.

For sites identified as susceptible to liquefaction or lateral spread, the ERS global model shall consider the non-liquefied and liquefied conditions using the procedures specified in AASHTO-SGS Article 6.8.

Table 5.19 Definition of Foundation Modeling Method (FMM).

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Rigid for Site Classes A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

Considering that the subject bridge is in SDC B, FMM I is permitted. The estimated depth of fixity method is illustrated in Figure 5.70. Figures 5.71 and 5.72 show the depth to fixity in sand and clay consecutively with respect to the standard penetration index N (blows/ft). This method is deemed adequate given that the bridge is in SDC B with piers having pile shaft foundation type. Based on the Boring at the site shown in Figures 5.73 and 5.74, a 25 ft of fill is considered below ground elevation.

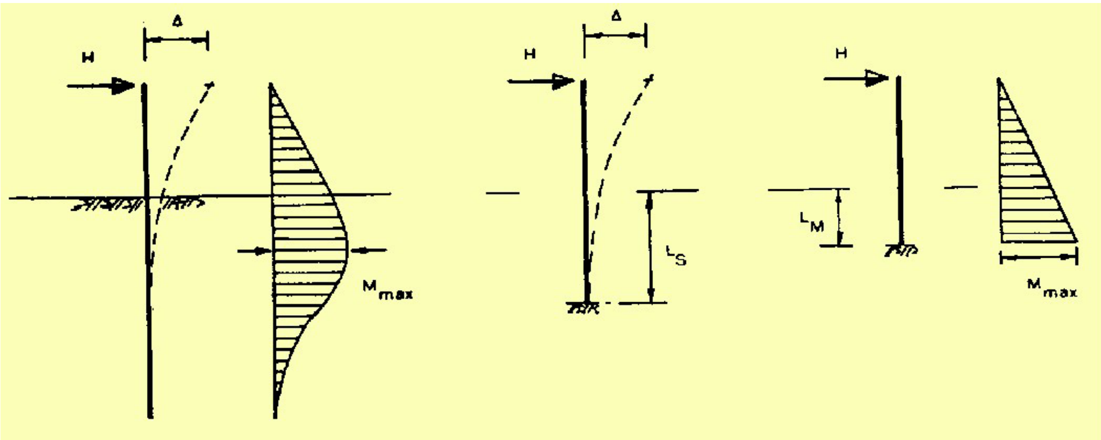


Figure 5.70 Estimated Depth to Fixity Model

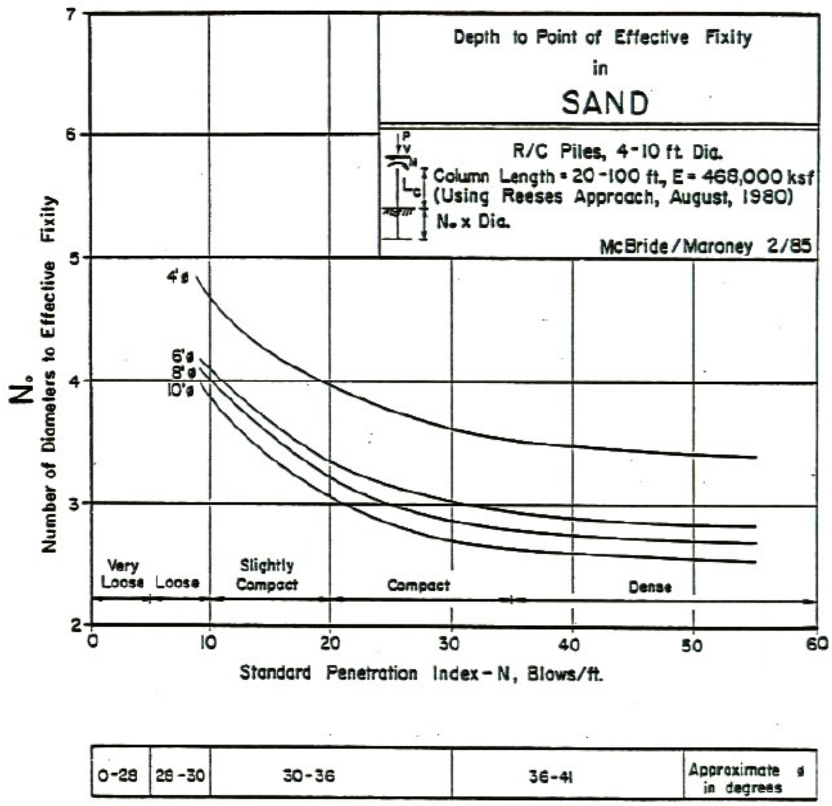


Figure 5.71 Depth to Fixity in Sand

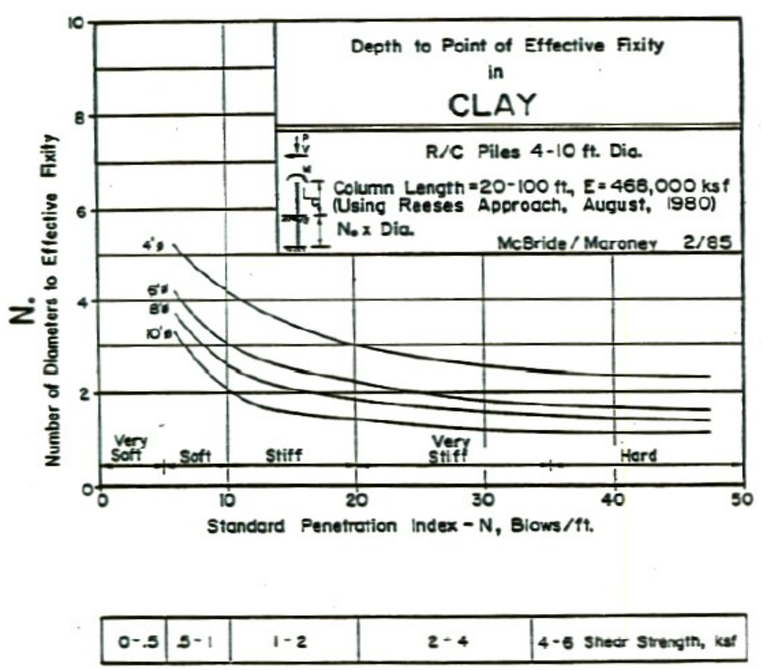


Figure 5.72 Depth to Fixity in Clay

PARSONS BRINCKERHOFF-FG, INC.

BORING NUMBER: PB-1

BORING LOG

PROJECT: Doremus Avenue Bridge Replacement
LOCATION: Newark, NJ
CONTRACTOR: Jersey Drilling & Boring Inc.
DRILLER: Frank Carroza
TYPE RIG: Tripod
INSPECTOR: Bob Sidorski

GROUND ELEVATION: +3.5 m
STATION: 1+139
DRILLING START TIME: 7:30 AM
DRILLING FINISH TIME: 12:30 PM

BASELINE: Doremus Avenue
OFFSET: 15.0 m LT
DATE: 11 / 9 / 94
DATE: 11 / 10 / 94

DEPTHS (meters)	METHOD(S) OF DRILLING
0.0 -10.7	Rotary Drilling

BOREHOLE WATER LEVEL DATA			
DEPTH	HOUR	DATE	REMARKS
		11 / 9 / 94	Installed 7.3 m
			Well
1.5 m	12:00 PM	11 / 10 / 94	Initial reading

TYPE OF SAMPLE

	UNDISTURBED	
SS *SPLIT SPOON* Nominal I.D.: 35 mm LENGTH: 610 mm HAMMER WEIGHT: 63.5 kg HAMMER FALL: 760 mm	U *SHELBY TUBE* O.D.: 76 mm I.D.: 73 mm I.L.: 610 mm	D *DENISON* O.D.: I.D.: I.L.:
C *CORE* BARREL TYPE: NX O.D.: 76 mm I.D.: 54 mm	P *PISTON* O.D.: I.D.: I.L.:	

CASING

O.D.: 108 mm — N.I.D.: 102 mm
WEIGHT OF HAMMER: 63.5 kg — HAMMER FALL: 610 mm

Notes:
1. The subsurface information shown hereon was obtained for the design and estimate purposes for our Client. It is made available to authorized users only that they may have access to the same information available to our Client. It is presented in good faith, but is not intended as a substitute for investigations, interpretations or judgment of such authorized users.
2. Field identification of soil samples is based on Burmister Soil Identification System.
3. pp = Unconfined compression strength from Pocket Penetrometer (kg per square centimeter)
4. WOH = Weight of Hammer; WOR = Weight of Rod

DEPTH BELOW GROUND SURFACE (meters)	BLOWS ON CASING	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE DEPTH (meters)	RUN (mm) / 0-150	REC. (mm) / 150-300	REC. (%) / 300-450	L>102 (mm) / 450-600	RQD (%) / REC (mm)	FIELD IDENTIFICATION OF SOIL / ROCK
					ROCK CORING INFORMATION / SOIL SAMPLING (Blows per 150 mm)					
	129	SS	1	0.0 - 0.6	6	18	39	39	440	Brown cf SAND, little (+) cf Gravel, little Silt, trace Glass, Stone & Brick fragments (Fill)
	155									
	132									
	62									
1.5	33									
	23	SS	2	1.5 - 2.1	3	3	3	4	205	Black Organic Silty CLAY, and Peat (Fill)
	32									
	41									
	96									
3.0	74									
	30	SS	3	3.1 - 3.7	19	13	10	6	305	Black cf SAND, little cf Gravel, little Silt, trace Glass, Plastic & Wood fragments, w/ Cinder (Fill)
	62									
	69									
	78									
4.5	89									

BORING NO. PB-1 SHEET 1 OF 2

Figure 5.73 Boring Log (1 of 2)

PARSONS BRINCKERHOFF-FG, INC.

BORING NUMBER: PB-1

BORING LOG (continued)

PROJECT: Doremus Avenue Bridge Replacement
LOCATION: Newark, NJ
INSPECTOR: Bob Sidorski

DEPTH BELOW GROUND SURFACE (meters)	BLOWS ON CASING	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE DEPTH (meters)	ROCK CORING: RUN (mm) / SOIL SAMPLING (Blows per 150 mm): 0-150	REC. (mm) / 150-300	REC. (%) / 300-450	L>102 (mm) / 450-600	RQD (%) / REC. (mm)	FIELD IDENTIFICATION OF SOIL / ROCK
	46	SS	4	4.6 - 5.2	8	1	2	1	330	Black Organic Silty CLAY, little Peat
	39									
	51									
	115	U	1	5.5 - 6.1					610	Same as SS-4
6.0	72									
		SS	5	6.1 - 6.7	WOR	3	3	3	560	Same as SS-4
7.5										
		SS	6	7.6 - 8.2	10	12	20	21	305	Red/Brown Clayey SILT, trace (-) f Sand (pp = 1.25)
9.0										
		SS	7	9.1 - 9.7	25	20	21	28	460	Same as SS-6 (pp = 1.25)
10.5										
		SS	8	10.7-11.3	21	40	48	61	510	Same as SS-6 (pp = 1.25)
										End of Boring at 11.3 m
12.0										
13.5										

BORING NO. PB-1 SHEET 2 OF 2

Figure 5.74 Boring Log (2 of 2)

Frame 2 consisting of spans 4, 5,and 6, is examined in detail given that it includes the largest span length of 55.25, 56.99, and 55.26m (184, 190, and 184ft) with one single pier 4 having fixed bearings and all other piers having expansion PTFE bearings. Lubricated PTFE has a coefficient of friction range between 0.08 and 0.03 while un-lubricated PTFE has a coefficient of friction range between 0.16 and 0.06 depending on the pressure exerted on the confined PTFE. For purpose of simplifying the seismic analysis, and given that there are no longitudinal devices or keys to resist any significant force at Piers 3, 5, and 6, the tributary mass of spans 4, 5, and 6 is applied at Pier 6 in the longitudinal direction in contrast to all piers sharing the resistance in the transverse direction.

The bearing loads are shown in table 5.20 below and used to compute the dead load at Piers 3, 4, 5, and 6 as shown in tables 5.21, 5.22, 5.23, and 5.24.

Table 5.20 Bearing Service Loads

BEARING DESIGNATION	TYPE FIX./EXP.	QUANTITY REQUIRED	DL (KN)	LL + IM		LONGITUDINAL (KN)	TRANSVERSE (KN)	MOVEMENT (mm)
				MAX. (KN)	MIN. (KN)			
E1.EA1	EXP.	20	600	550	-40	0	400	40
E2.EA2	EXP.	10	600	550	-155	0	350	75
E3.EA3	EXP.	20	1200	850	0	0	250	45
E4.EA4	EXP.	20	1900	1100	0	0	350	40
E5.EA5	EXP.	20	600	550	-155	0	350	65
F1	FIX.	20	1900	1100	0	475	125	0
F2	FIX.	10	105	445	-70	425	25	0

Table 5.21 Expansion Bearings at Pier 3 Supporting Span 4

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E1	600	135
PG12	E1	600	135
PG13	EA1	600	135
PG14	E1	600	135
PG15	E1	600	135
PG16	E1	600	135
PG17	E1	600	135
PG18	EA1	600	135
PG19	E1	600	135
PG20	E1	600	135

Total 10 Girders x 135 Kips/each = 1350 Kips

Table 5.22 Fixed Bearings at Pier 4 Supporting Spans 4 and 5

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11 Through PG20	F1	1900	427

Span Total 10 girders x 427 Kips/each=4270 Kips

Table 5.23 Expansion Bearings at Pier 5 Supporting Spans 5 and 6

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E4	1900	427
PG 12	EA4	1900	427
PG 13	E4	1900	427
PG 14	E4	1900	427
PG 15	E4	1900	427
PG 16	E4	1900	427
PG 17	E4	1900	427
PG 18	E4	1900	427
PG 19	E4	1900	427
PG 20	E4	1900	427

Total 10 girders x 427 Kips/each =4270 Kips

Table 5.24 Expansion Bearings at Pier 6 Supporting Span 6

Girder	Bearing	D.L. (KN)	D.L. (Kips)
PG11	E2	600	135
PG 12	E2	600	135
PG 13	EA2	600	135
PG 14	E2	600	135
PG 15	E2	600	135
PG 16	E2	600	135
PG 17	E2	600	135
PG 18	EA2	600	135
PG 19	E2	600	135
PG 20	E2	600	135

Total 10 girders x 135 Kips/each =1350 Kips

Consider West Bent at pier 3 (See Figures VI.C.6 and VI.C.7)

Average Top Pedestal Elevation: $\frac{14.74+14.96}{2}$=14.9 m

Average Bottom Pedestal Elevation: $\frac{14.15+14.37}{2}$=14.3 m

Top Pedestal X-Section Area: $(14.9-12.9)\times 0.75\times 3.33^2$ = 16.6 ft^2

Bottom Pedestal X-Section Area: $(14.3-12.9)\times 0.75\times 3.33^2$ = 11.64 ft^2

Bent Cap X-Section Area at Pier 3:

11.64+16.6 = 28.3 ft^2

Bent Cap (West) Weight:

$28.3\ ft^2 \times 11.5\ m \times 3.33\ ft/m \times 0.15\ K/ft^3 = 163$ Kips

Total Bent Cap (West) Weight add 10% for flares:

1.1×163 = 180 Kips

Consider West bent at Pier4 (See Figure VI.C.4 and VI.C.5):

Average Pedestal Elevation

$$\frac{14.36+14.596}{2}=14.5\ m$$

Depth of West Cap:

(14.5-13.1)×3.33 ft/m = 4.66 ft

West Cap Weight:

$11 \times 1.6 \times (3.33)^2 \times 4.66 \times 0.15\ K/ft^3 = 137$ Kips

Consider 10% added weight for flares, total weight is calculated as:

1.1×137 = 151 Kips

Calculate Column Height as shown in table 5.25 below:

Table 5.25 Piers 3, 4, 5, and 6 Column Height

Pier	Elevation A	Bottom “Cap”	Height(m)	Height (ft)
3	5.7	12.9	7.2	24.0
4	5.3	13.1	7.8	26.0
5	4.5	12.8	8.3	27.6
6	4.9	11.95	7.1	23.6

Elevation A refers to bottom of column as shown in Figure VI.C.12 and Table 5.26 below:

Table 5.26 Pier 1 to 8 Elevations

PIER NO.	ELEVATION			DRILLED SHAFT LENGTH D
	“A”	“B”	“C”	
1	4.500	-18.200	-21.200	25.7
2	5.700	-18.300	-21.300	27.0
3	5.700	-19.800	-22.800	28.5
4	5.300	-20.000	-23.000	28.3
5	4.500	-20.000	-23.000	27.5
6	4.900	-17.700	-20.700	25.6
7	5.200	-17.100	-20.100	25.3
8	4.500	-19.500	-22.500	27.0

Column X-Section Area:

$$\frac{\pi D^2}{4} = \pi \times \frac{3.5^2}{4} = 9.6 \text{ ft}^2$$

Dead Load Corresponding to Minimum Column Length: 23.6×9.6×0.15 K/ft^3 = 34 Kips

Dead Load Corresponding to Maximum Column Length: 27.6×9.6×0.15 K/ft^3 = 40 Kips

Pier 4 Dead Load (West Side)

Bearings Loading 4270/2 for West side only	2135 Kips
Bent Caps	151 Kips
Top Columns [Loading (West Pier 4)]	2286 Kips
Top Column Loading Pier 4	762 Kips
Bottom Column Loading Pier 4	796 Kips

Consider 24 #9 Vertical Reinforcement:

$$\rho = \frac{24 \times 1}{9.6 \times 144} = 1.7\%$$

Consider 16 #9 Vertical Reinforcement

$$\rho_e = \frac{16 \times 1}{9.6 \times 144} = 1.16\%$$

$$\text{I}_\text{g} = \frac{\pi D^4}{64} = \pi \times \frac{3.5^4}{64} 7.4 \text{ ft}^4$$

$$\text{EI}_\text{g} = 580{,}000 \times 7.4 \times 144 = 6.2 \times 10^8 \text{ K-in}^2$$

$$\text{I}_\text{casing} = \frac{\pi}{64}\left(48^4 - 47^4\right) = 21046 \text{ in}^4$$

$$\text{E}_\text{s}\text{I}_\text{casing} = 29{,}000 \times 21{,}046 = 6.1 \times 10^8 \text{ K-in}^2 \text{ or } 4236111 \text{ K-ft}^2$$

$$\text{I}_\text{casing/evq} \frac{4236111}{580000} = 7.3 \text{ ft}^4$$

Column I_{crack} = 3.6 ft^4, M_p = 2696 K-ft (See Figure VI.A.6)

Column with Casing (See Figure VI.A.8):	M_n = 6804 K-ft
I_{crack} = 13.83 ft^4	M_p = 8572 K-ft
Casing (See Figure VI.A.10):	M_n = 5370 K-ft
I_{crack} = 11.82 ft^4	M_p = 6909 K-ft

A summary of member properties for model 1 is shown in Table 5.27.

Table 5.27 Model 1 Member Properties

Model 1				
	Column	Column Casing	Casing	Cap
I_g	7.3	12.4	12.4	28.4
I_{crack}	3.6	13.8	11.8	8.9
Ratio	0.5	1.1	0.95	0.31

$$\frac{I_{crack\text{-}casing}}{I_{crack\text{-}col}} = \frac{11.82}{3.6} = 3.3$$

Calculate Equivalent Diameter D_{eqv} :

$$D_{eqv} = \sqrt[4]{3.3} \times D = 1.35 \times 3.5 = 4.7 \text{ ft}$$

$$\frac{M_{crack\text{-}casing}}{M_{crack\text{-}col}} = \frac{5370}{2696} = 2$$

$$D_{eqv} = \sqrt[3]{2} \times D = 1.26 \times 3.5 = 4.4 \text{ ft}$$

Consider fixity at $3 \times D_{eqv} = 3 \times 4.7 = 14$ ft

Calculate Bent Stiffness in Longitudinal direction based on casing properties (3 columns shaft):

$$K_l = 3 \times \frac{3EI}{L^3}$$

$$K_l = 3 \times \frac{3 \times 580{,}000 \times 11.82}{68^3} = 3 \times 65.4 \text{ K/ft} = 196 \text{ K/ft}$$

Based on Model, Bent Stiffness (see figure VI.A.16) is calculated as follows:

$$\frac{1000}{5.7} = 176 \text{ K/ft}$$

Longitudinal Mass (5 girders tributary to West bent):

Pier 3	1350 K/2	675 K
Pier 4	780 K/col x3	2340 K
Pier 5	4270 K/2	2135 K
Pier 6	1350 K/2	675 K
Total		5825 K

$$\omega^2 = \frac{K}{M} = \frac{176 \text{ K/ft}}{5825 \text{ K}} \times 32.2 \text{ ft/sec}^2 = 1.0$$

$\omega = 1.0$ rad/sec

$T = \frac{2\pi}{\omega} = 6.3$ sec (greater than the maximum period of 4 sec in the AASHTO-SGS response spectrum)

Consider casing 0.75 in. in thickness as shown in model 2 (See Figure VI.A.22):

$I_{crack\text{-}casing} = 15.5 \text{ ft}^4$

M_n = 7272 K-ft

M_p = 9948 K-ft

A summary of member properties for Model 2 is shown in Table 5.28

Table 5.28 Model 2 Member Properties.

Model 2				
	Column	Column Casing	Casing	Cap
I_g	7.3	12.4	12.4	28.4
I_{crack}	3.6	17.2	15.5	8.9
Ratio	0.5	1.4	1.25	0.31

$$\frac{I_{crack\text{-}pile}}{I_{crack\text{-}col}} = \frac{15.5}{3.6} = 4.3$$

$$D_{eqv} = \sqrt[4]{4.3}D = 1.44D = 1.44 \times 3.5 = 5 \text{ ft}$$

$$\frac{M_{crack\text{-}casing}}{M_{crack\text{-}col}} = \frac{7272}{2696} = 2.7$$

$$D_{eqv} = \sqrt[3]{2.7} \times D = 1.39 \times 3.5 = 4.9 \text{ ft}$$

Consider Fixity at $3 \times D_{eqv} = 3 \times 5 = 15$ ft

The results of analysis are documented in Appendix VI.A

According to the AASHTO-SGS 8.7.1, the minimum lateral flexural capacity of each column shall be taken as:

$$M_{ne} \geq 0.1 P_{trib} \frac{(H_h + 0.5D_s)}{\Lambda}$$

where:

M_{ne} = nominal moment capacity of the column based upon expected material properties as shown in Figure 8.5-1(kip-ft.)

P_{trib} = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kip)

H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft.)

D_s = depth of superstructure (ft.)

Λ = fixity factor for the column defined in Article 4.8.1

$0.1P_{trib}\left(\frac{H_s+0.5D_s}{\Lambda}\right)=0.1\times\frac{5825}{3}\times\frac{69\text{ ft}}{1}=13{,}398\text{ K-ft}>M_{n\text{-casing}}=7272\text{ K-ft}$ calculated for Model 2

Consider 30#11 for column reinforcement (4.50 ft column with a 5ft shaft and a 1" casing).

A summary of member properties for Model 3 is shown in Table 5.29

Table 5.29 Model 3 Member Properties.

5 ft Shaft 1" casing				
	Column	Column Casing	Shaft	Cap
I_g	19.9	30.3	30.3	28.4
I_{crack}	10.4	44.4	39.3	8.9
Ratio	0.52	1.47	1.3	0.31

$$\frac{I_{crack\text{-shaft}}}{I_{crack\text{-col}}}=\frac{39.3}{10.3}=3.8$$

$$D_{eqv}=\sqrt[4]{3.8}D=1.44D=1.44\times4.5=6.3\text{ ft}$$

$$\frac{M_{n\text{-pile}}}{M_{n\text{-col}}}=\frac{14402}{5752}=2.5$$

$$D_{eqv}=\sqrt[3]{2.5}\times D=1.39\times4.5=6.1\text{ ft}$$

Consider Fixity at $3\times D_{eqv}$ = 3×3.63 = 19.0 ft

Total height to fixity: 28 ft + 25 ft (fill) + 19 ft (embedment) = 72 ft

Calculated Longitudinal Period for Model 3 (see figure VI.A.32):

$$K = \frac{1000}{2.15} = 465 \text{ K/ft}$$

$$\omega^2 = \frac{K}{M} = \frac{465}{5825} \times 32.2 \frac{\text{ft}}{\text{sec}^2} = 2.57$$

ω=1.60 rad/sec

$$T = \frac{2\pi}{\omega} = 3.9 \text{ sec}$$

S_a = .0228g

Apply 1.5 N.J. Factor

$$S_a = \omega^2 S_d$$

$$S_d = \frac{1.5 \times .0228 \times 32.2}{2.37} = 0.46 \text{ ft} = 5.6 \text{ in.}$$

Calculate yield deflection corresponding to reaching Nominal Moment of the Shaft 14,402 K-ft

$$K_{col} = \frac{465}{3} = 155 \text{ K/ft}$$

Force applied at bent caps centroid corresponding to Nominal Moment of the Pile.

72 ft × F_{yield} = 14402 K-ft

F_{yield} = 200 Kips

$$\Delta_y = \frac{200}{155 \text{ K/ft}} = 1.3 \text{ ft}$$

Calculate Column Nominal Moment Capacity based on AASHTO-SGS 8.7.1.

Bot of Bent Cap: 13.1m

Elevation A –Bottom of Column: 5.3

Clear Height: 7.8m or 26 ft

Total Height (including Bent Cap Depth):

26 ft + 4 ft = 30 ft

$M_{n\text{-}col} = 0.1 \times \frac{5825}{3} \times 30 = 5825 \text{ K-ft}$ Compared to $M_n = 5752 \text{ K-ft}$ (See Figure VI.A.26 considered adequate)

Calculate Transverse Period for Model 3, applicable to pier 4 (See Figure VI.A.31):

$$K_T = \frac{1000}{0.268} = 3731 \text{ K/ft}$$

$\omega^2 = \frac{K}{M}$

The transverse inertia is calculated based on:

800 Kips × 3 + wall weight

Wall weight 10 × 5 × 38 × 0.15 = 285 K

Total Weight = 2400 + 285 = 2685 Kips

$\omega^2 = \frac{3731}{2685} \times 32.2 = 44.7$

$\omega = 6.7$ rad/sec

$T = \frac{2\pi}{\omega} = 0.94$ sec

Spectral Acceleration from Figure 5.41 is 0.1g

Apply N.J. 1.5 Factor

$S_a = 0.1 \times 1.5 = 0.15g$

$S_a = \omega^2 d$

$S_d = \frac{0.15 \times 32.2}{44.7} = 0.1$ ft or 1.3 in

Calculating transverse direction seismic force:

$S_a = 0.15g$

Applied force as bent cap:

0.15 × 2685 = 403 Kips

The moment distribution for Bent subject 1000 Kips of transverse loading is shown in Figure V.A.33.

Model 3 D/C ratios are shown in Table 5.30.

Table 5.30 Model 3 D/C Ratios

Moment (K-ft)	Model 3	0.4×1000	M_n	D/C
Column	6820	2728	5752	0.48
Shaft	9141	3657	14402	0.25

Calculate Transverse Period (Model 1) applicable to Piers 3 and 5 (See Figure V.A.15):

$K_T = \frac{1000}{0.55} = 1818$ K/ft

$$\omega^2 = \frac{K}{M_T} \text{ When } M_T = 2685/32.2$$

$$\omega^2 = \frac{1818}{2685} \times 32.2 = 21.8$$

ω=4.66 rad/sec

$$T = \frac{2\pi}{\omega} = 1.3 \text{ sec}$$

Spectral Acceleration from Figure 5.41 is 0.07; Apply N.J. 1.5 factor

$$S_a = 1.5 \times 0.07 = 0.11g$$

$$S_d = \frac{0.11 \times 32.2}{21.8} = 0.16 \text{ ft or 2 in}$$

Calculating transverse direction seismic force:

S_a = 0.11g

Applied force at bent cap:

0.11 × 2685 = 295 Kips

The moment distribution for Bent subject 1000 Kips of transverse loading is shown in Figure V.A.17.

Model 1 D/C Ratios are shown in Table 5.31.

Table 5.31 Model 1 D/C Ratios

Moment (K-ft)	Model 1	0.3 x Model1	M_n	D/C
Column	6605	1982	2636	0.75
Shaft	7886	2366	5370	0.44

Considering 16#9 instead of 24#9 $M_n = 2150$ K-ft yielding a $D/C = \frac{1982}{2150} = 0.92$

Calculate Transverse Period, for model 4 (see Figure VI.A.37):

$$K_T = \frac{1000}{2.38} = 420 \text{ K/ft}$$

$$\omega^2 = \frac{K}{M_T} \text{ When } M_T = 2400 \text{ Kips}$$

$$\omega^2 = \frac{420}{2400} \times 32.2 = 5.6$$

ω=2.4 rad/sec

$$T = \frac{2\pi}{\omega} = 2.6 \text{ sec}$$

Spectral Acceleration from Figure 5.41 is 0.035 Apply N.J. factor of 1.5

S_a = 1.5 × 0.035 = 0.053g

$$S_d = \frac{0.053 \times 32.2}{5.6} = 0.3 \text{ ft or } 3.6 \text{ in}$$

Specified by AASHTO-SGS 8.7.1

Therefore, applied force is 0.05 × 2400 Kips = 127 Kips

Model 4 D/C ratios are shown in Table 5.32.

Table 5.32 Model 4 D/C Ratios

Moment	Model 4	$0.13 \times$ Model 4	M_n	D/C
Column	8549	1111	2636	0.42
Shaft	14972	1946	5370	0.36

In comparing the column displacement demands to the yield displacement in the transverse and longitudinal directions, the column is found to respond in the elastic range; therefore satisfy minimal requirements of SDC B. Calculate Local Displacement Capacity for SDC B according to AASHTO-SGS 4.8.1

The most critical column response is considered in the transverse direction on piers where the crash wall inhibits column displacement. Therefore, we consider the following two models:

a.) Model 3 is representative of Pier 4, the transverse displacement demand of the bent is calculated as 1.3 in.

b.) Model 1 is representative of piers 5 and 6, the transverse displacement demand of the bent is calculate as 2 in.

The displacement magnification for short period structures of AASHTO-SGS 4.3.3 does not apply considering that responses of Models 1 and 3 are elastic. The transverse period of Model 3 and Model 1 0.94 sec and 1.3 sec, respectively

For Type 1 structures, comprised of reinforced concrete columns in SDC B, the displacement capacity, Δ_C^L in., of each bent may be determined from the following approximation:

$$\Delta_C^L = 0.12H_o\left(-1.27\ln(x) - 0.32\right) \geq 0.12H_o$$

in which:

$$x = \frac{\Lambda B_o}{H_o}$$

`

where:

H_o = clear height of column (ft.)

B_o = column diameter or width measured parallel to the direction of displacement under consideration (ft.)

Λ = factor for column end restraint condition

= 1 for fixed-free (pinned on one end)

= 2 for fixed top and bottom

For a partially fixed connection on one end, interpolation between 1 and 2 is permitted for Λ. Alternatively, Ho may be taken as the shortest distance between the point of maximum moment and point of contra-flexure and Λ may be taken as 1.0 when determining x using the equation above.

Calculate local displacement capacity in the transverse direction:

$$x = \frac{\Lambda B}{H}$$

where:

Λ = 2 for fixed top and bottom connections as in transverse direction

Λ = 1 for fixed free connection as in the longitudinal direction.

Establish capacity in transverse direction based on Λ= 2 considering full flexural constraint at bottom of the cap and top of the wall. For Model 3, the column has 4.5 ft diameter and the clear distance between bottom of cap and top of wall is 16.3 ft.

$$x = 2 \times \frac{4.5}{16.3} = 0.55$$

$$\Delta_C = 2.0\left(-1.27\ln\left(0.55\right) - 0.32\right) \geq 2''$$

$$= 2''\left(0.44\right) \geq 2''$$

$$= 2 \text{ in}$$

According to AASHTO-SGS Section 4.8

$$\Delta_D^L < \Delta_C^L$$

where:

Δ_D^L = displacement demand taken along the local principal axis of the ductile member

Δ_C^L = displacement capacity taken along the local principal axis corresponding to Δ_D^L of ductile member as determined in accordance with Article 4.8.1 for SDC B and C.

Eq. 1 shall be satisfied in each of the local axis of every bent. The local axis of a bent typically coincides with the principal axis of the columns in that bent.

Displacement Demand in Transverse direction 1.3 in

Displacement Demand (1.3) ≤ Displacement Capacity (2 in)

This is important to mention that this displacement capacity check is conservative and ignore flexibility of the shaft in the fill material. All piers in the longitudinal direction are slender and have adequate displacement capacity.

Abutment Response

According to the AASHTO-SGS 5.2.3.1, abutments for bridges in SDC B are expected to resist earthquake loads with minimal damage. However, bridge superstructure displacement demands may be 4 in. or more and could potentially increase the soil mobilization. Comparing the displacement demand to the 4 in. threshold capacity, the abutments are deemed adequate for minimal damage requirement

Column Shear Demand and Capacity

According to AASHTO-SGS 8.6.1 The shear demand for a column, Vu, in SDC B shall be determined based on the lesser of:

- The force obtained from a linear elastic seismic analysis
- The force, V_{po}, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_u, in SDC C or D shall be determined based on the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.

Given the uncertainty in the hazard and the consequence of column shear failure, it is deemed important to attempt to satisfy the capacity protection requirement for column shear.

The column shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated based on the nominal material strength properties and shall satisfy:

$$\phi_s V_n \geq V_u$$

in which:

$$V_n = V_c + V_s$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

Shear demand in Transverse direction (Elastic Model)

Model 1 0.3 × 389 Kips = 117 Kips (See Figure VI.A.18)

Model 3 0.4 × 416 Kips = 116 Kips (See Figure VI.A.34)

According to AASHTO-SGS Eq. 8.5.1

Column Plastic Shear Demand: $M_{po} = \lambda_{mo} M_p$

Model 1 M_p = 2696 K-ft for 3.5 ft dia. column

Model 3 M_p = 6085 K-ft for 4.5 ft dia. column

For Model 1:

M_{po} = 1.4 × 2696 = 3774 K-ft

For Model 3:

M_{po} = 1.4 × 6085 = 8519 K-ft

Maximum Shear Demand in Transverse direction

For Model 1:

$$V_{po} = \frac{2M_{po}}{L} = \frac{2 \times 3774}{16.3} = 463 \text{ Kips}$$

For Model 3:

$$V_{po} = \frac{2 \times 8519}{16.3} = 1045 \text{ Kips}$$

The concrete shear capacity, V_c, of members designed for SDC B, C and D shall be taken as:

$$V_c = v_c A_e$$

in which:

$A_e = 0.8A_g$

A_e = 0.8 × 1385 = 1108 in^2 for 3.5 ft column

Ae = 0.8 × 2290 = 1832 in^2 for 4.5 ft column

if Pu is compressive:

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c'} \le \min\begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases}$$

otherwise:

$v_c = 0$

$$0.3 \le \alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \le 3$$

$$f_s = \rho_s f_{yh} \le 0.35$$

$$\rho_s = \frac{4A_{sp}}{sD'}$$

where:

A_g = gross area of member cross section (in^2)

P_u = ultimate compressive force acting on section (kip)

A_{sp} = area of spiral or hoop reinforcing bar (in^2)

s = pitch of spiral or spacing of hoops or ties (in.)

D' = diameter of spiral or hoop for circular column (in.)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f_c' = nominal concrete compressive strength (ksi)

μ_D = maximum local displacement ductility ratio of member

For SDC B, the concrete shear capacity, Vc, of a section within the plastic hinge region shall be determined using:

μ_D = 2

For Model 1:

$$\rho_s = \frac{4 \times 0.31}{3 \times 38} = 1\%$$

$$f_s = 0.01 \times 60 = 0.60 < 0.35$$

$$\alpha' = \frac{0.35}{0.15} + 3.67 - 2 = 4 < 3$$

$$\alpha' = 3$$

For Model 3:

$$\rho_s = \frac{4 \times 0.31}{3 \times 50} = 0.83\%$$

$$f_s = 0.0083 \times 60 = 0.50 < 0.35$$

$$\alpha' = \frac{0.35}{0.15} + 3.67 - 2 = 4 < 3$$

$$\alpha' = 3$$

for circular columns with spiral or hoop reinforcing:

$$v_c = 0.032 \times 3\left(1 + \frac{800}{2 \times 1385}\right)\sqrt{4} = 0.25 \le \begin{cases} 0.22 \\ \\ 0.282 \end{cases}$$

V_c = 0.22 × 1108 = 244 Kips for a 3.5 ft column

V_c = 0.22 × 1832 = 403 Kips for a 4.5 ft column

Calculate Column Shear Reinforcement Capacity

According to AASHTO-SGS 8.6.3, members that are reinforced with circular hoops, spirals or interlocking hoops or spirals as specified in Article 8.6.6, the nominal shear reinforcement strength, Vs, shall be taken as per Eq.(8.6.3-1):

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

where:

n = number of individual interlocking spiral or hoop core sections.

A_{sp} = area of spiral or hoop reinforcing bar (in^2)

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

The pitch s is taken equal to 3" since shear demand is constant and governs the design outside the plastic hinge region.

For model 1:

$$V_s = \frac{\pi}{2}\left(1 \times 0.31 \times 60 \times \frac{38}{3}\right) = 370\ \text{K}$$

Capacity $\phi_s\left(V_s + V_c\right) = 0.9\left(370 + 244\right) = 533\ \text{K} > 463$ Kips (plastic demand)

For model 3:

$$V_s = \frac{\pi}{2}\left(1 \times 0.31 \times 60 \times \frac{50}{3}\right) = 487\ \text{K}$$

Capacity $\phi_s(V_s + V_c) = 0.9(487 + 403) = 801\,K > 116$ Kips (elastic demand)

< 1045 Kips (plastic demand)

The following requirements need to be satisfied for SDC B

AASHTO-SGS 8.6.4 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, Vs, shall not be taken greater than:

$$V_s \leq 0.25\sqrt{f'_c}A_e$$

where:

A_e = effective area of the cross section for shear resistance as defined by AASHTO-SGS Eq. 8.6.2-2 (in^2)

f'_c = compressive strength of concrete (ksi)

For Model 1:

$0.25\sqrt{4} \times 1108 = 554$ Kips $> V_s$ equal to 370 Kips O.K.

For Model 3:

$0.25\sqrt{4} \times 1832 = 916$ Kips $> V_s$ equal to 487 Kips

AASHTO-SGS 8.6.5 Minimum Shear Reinforcement

The area of column spiral reinforcement, A_{sp}, shall be used to determine the reinforcement ratio, ρ_s as given by AASHTO-SGS Eq. 8.6.2-7. For SDC B, the spiral reinforcement ratio, ρ_s, for each individual circular core of a column shall satisfy:

$\rho_s \geq 0.003$

$\rho_s = 1\% > 0.3\%$ OK for Model 1

$\rho_s = 0.83\% > 0.3\%$ OK for Model 3

AASHTO-SGS 8.7.1 Minimum Lateral Strength

The minimum lateral flexural capacity of each column shall be taken as:

$$M_{ne} \geq 0.1 P_{trib} \frac{(H_h + 0.5D_s)}{\Lambda}$$

Where:

P_{trib} = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kip).

H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft.)

D_s = depth of superstructure (ft.)

Λ = fixity factor for the column defined in Article 4.8.1

This requirement was used to enlarge the column/shaft for Pier 4 acting as seismic collector in the longitudinal direction.

AASHTO-SGS 8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall satisfy:

$$A_l \leq 0.04A_g$$

where:

A_g = gross area of member cross section (in^2)

A_l = area of longitudinal reinforcement in member (in^2)

$$\frac{A_l}{A_g} = \frac{24\times 1}{1385} = 1.7\% \text{ Considering 24\#9 For Model 1}$$

$$\frac{A_l}{A_g} = \frac{30\times 1.56}{2290} = 2\% \text{ Considering 30\#11\#9 for Model 3}$$

$$\frac{A_l}{A_g} \leq 0.04 \text{ OK}$$

AASHTO-SGS 8.8.2 Minimum Longitudinal Reinforcement

For columns in SDC B and C, the minimum area of longitudinal reinforcement for compression members shall not be less than:

$$A_l \geq 0.007A_g$$

where:

A_g = gross area of member cross section (in^2)

A_l = area of longitudinal reinforcement in member (in^2)

$$\frac{A_l}{A_g} = 0.017 \geq 0.007 \quad \text{O.K. For Model 1}$$

$$\frac{A_l}{A_g} = 0.02 \geq 0.007 \quad \text{O.K. For Model 3}$$

Check minimum support length.

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.33 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; For hinges within a span, L shall be the sum of the distances to either side of the hinge; For single-span bridges, L equals the length of the bridge deck (ft.)

H = For abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.) for columns and/or piers, column, or pier height (ft.) for hinges within a span, average height of the adjacent two columns or piers (ft.) 0.0 for single-span bridges (ft.)

S = Angle of skew of support measured from a line normal to span (°)

Table 5.33 Percentage N by SDC and Effective Peak Ground Acceleration, A_s.

SDC	Effective peak ground acceleration, A_s	Percent N
A	< 0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC B:

$N = 1.5(8+0.02L+0.08H)(1+0.000125S^2)$

L = 558 ft calculated based on the total length of three continuous spans 4, 5, and 6 from Pier 3 to Pier 6.

H = 68 ft (Including length to point of fixity)

H = 28 ft for column only

S = 15° at pier 6

$N = 1.5(8+0.02\times558+0.08\times68)(1+.000125\times15^2)$

$= 1.5(8+11.6+5.4)(1+0.028)$

= 38 in

Cap Width 1.6m or 5.3 ft (See Figure VI.C.15 and VI.C.16)

Half Cap Width 32 in.

Expansion Joint 210 min or 4” (See Figure VI.B.10)

Available Cap Width 32 in – 2 in = 30 in

Calculate N based on H = 28 ft

$N = 1.5(8+.02\times558+0.8\times28)(1+0.028)$

$= 1.5(8+11.16+2.2)(1.028) = 33$ in

Available support length slightly less than required support length; however, considered satisfactory based on conservative N values in AASHTO-SGS.

Summary:

For critical performance, two aspects of seismic design related to this bridge need to be highlighted:

1.) The increase in size of pier 4 column/shaft is intended to satisfy requirements of SDC B where the target ductility is expected at a magnitude equal to 2. This increase in size may be ignored or discontinued given the elastic response of the structure.

2.) The use of PTFE spherical bridge bearings with High-Temperature Adhesives may be considered to ensure functionality during a seismic event [Konstantinidis et al. (2008)].

Example 6: Design of a Single Span Concrete Bridge in SDC A Category

Bridge Description

This example is based on a bridge carrying Route 101 over Ramp D, Morris County, New Jersey. The bridge is a precast superstructure single span supported by seat abutments. Following figures show relevant drawings needed for calculations.

Figure 5.75: Plan and Elevation
Figure 5.76: Typical Bridge Section (Westbound)
Figure 5.77: Abutments 1 Eastbound Plan and Elevation
Figure 5.78: Abutment Typical Section
Figure 5.79: Anchor Bolts Location at Abutment Pedestals

Standard drawing shown on the Index Table below were not provided. Therefore, information about precast beam properties was not taken directly from these standard drawings, but deemed close enough to evaluate the subject bridge for the AASHTO-SGS.

INDEX	
TITLE	SHEET No.
General Plan	1
Abutment #1 - Westbound	2
Abutment #1 - Eastbound	3
Abutment #2 - Westbound	4
Abutment #2 - Eastbound	5
Abutment Details & Sections	6
Wingwalls	7
Wingwall Sections	8
Superstructure	9
Reinforcing Bar Details	10
Bearings - 45"&54" Beams	SD-2
Beam Details	SD-3
54" Prestressed Concrete Beams	SD-6
Typical Architectural Details	SD-7
End Post & Miscellaneous Details	SD-8

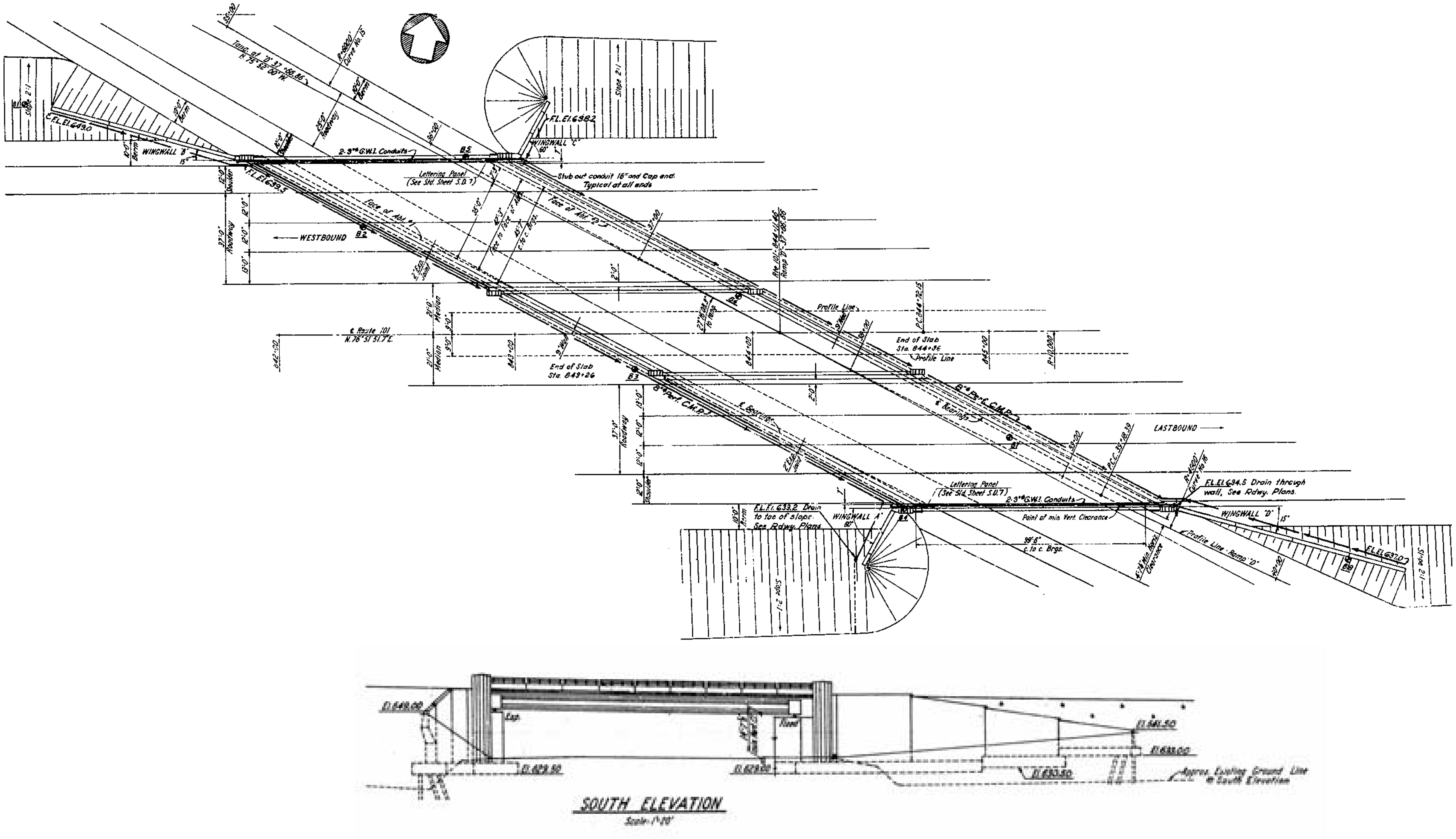


Figure 5.75 Plan and Elevation

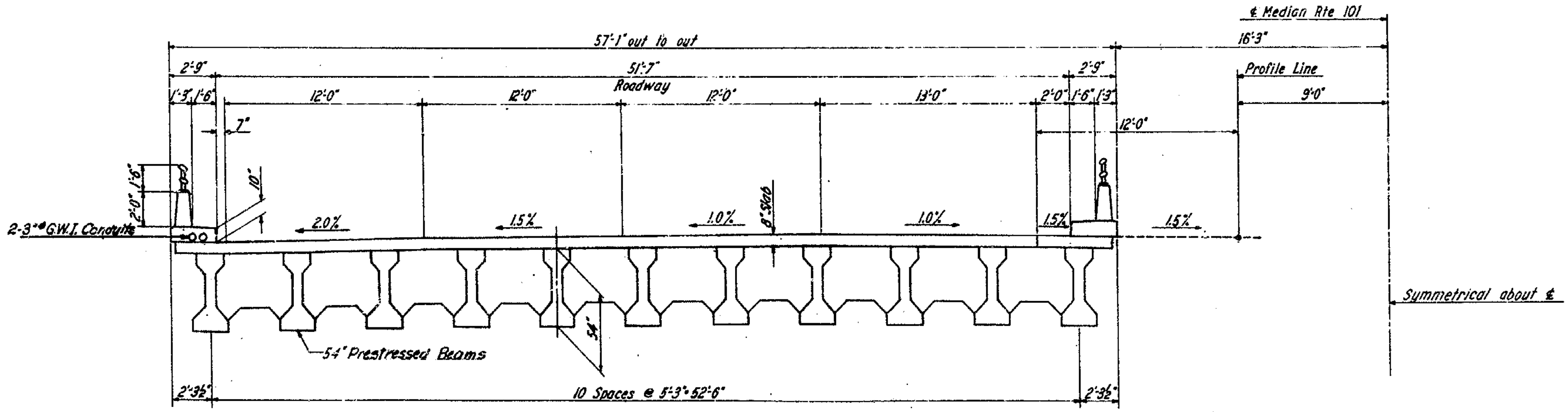


Figure 5.76 Typical Bridge Section (Westbound shown, Eastbound Similar)

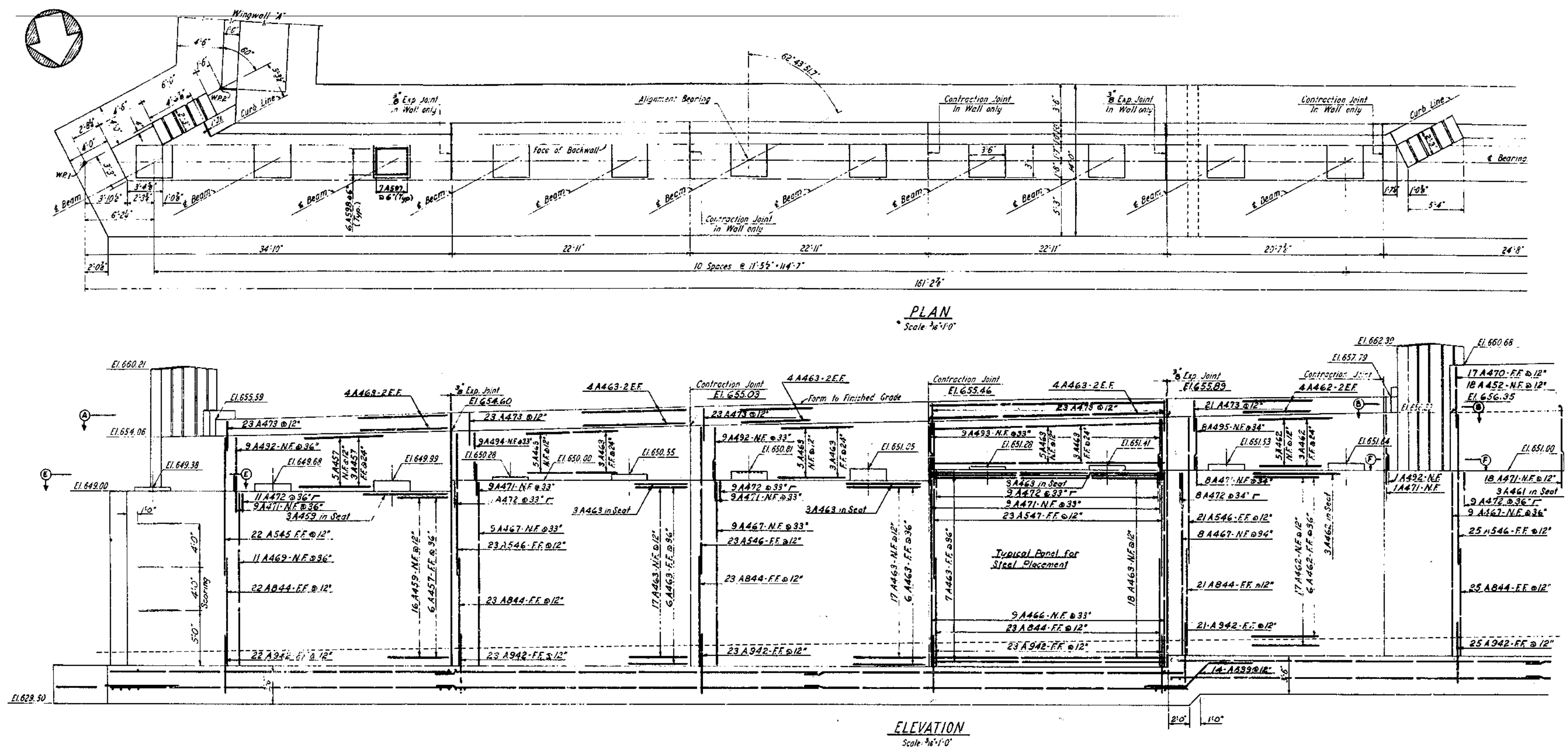


Figure 5.77 Abutment 1 Eastbound Plan and Elevation.

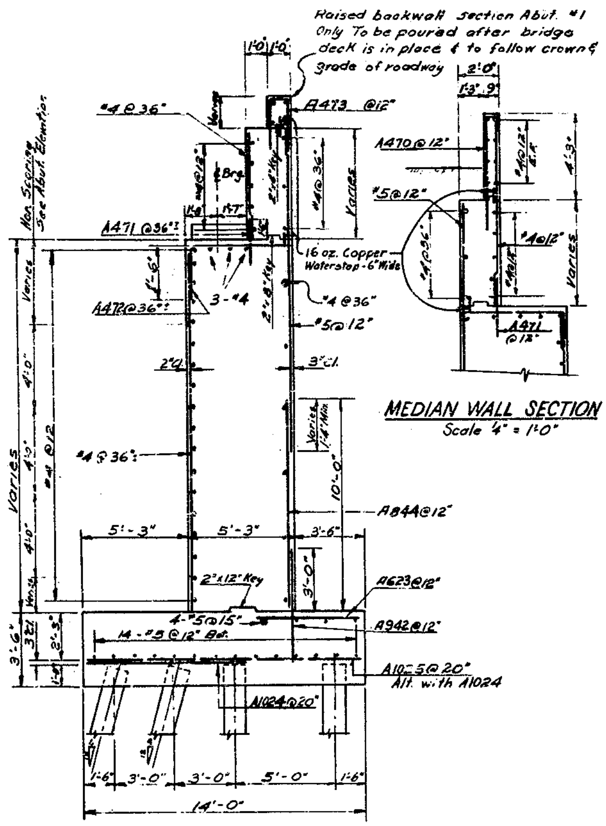


Figure 5.78 Abutment Typical Section

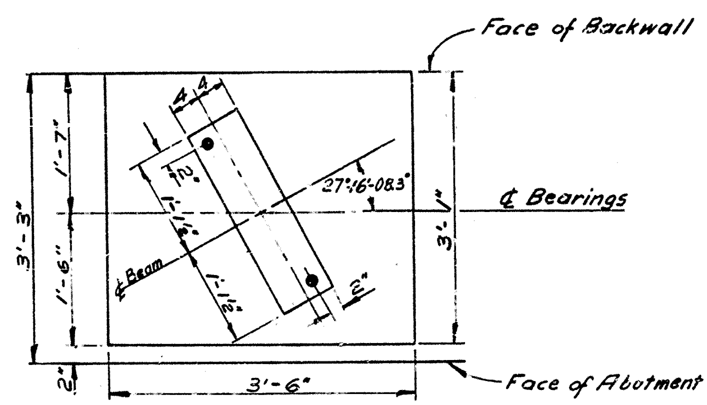
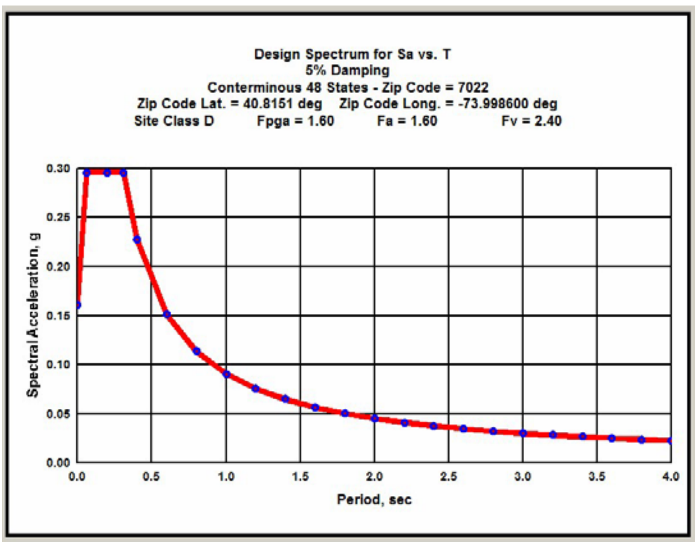


Figure 5.79 Anchor Bolts Location at Abutment Pedestals

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.80. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.80 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D
PGA = 1.5×0.16 = 0.24
S_{DS} = 1.5×0.3 = 0.45
S_{D1} = 1.5×0.09 = 0.14

Flow Charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 6 reflects a Type 3 bridge system with the bearing connections considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.81 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a single span bridge versus a multi-span bridge.

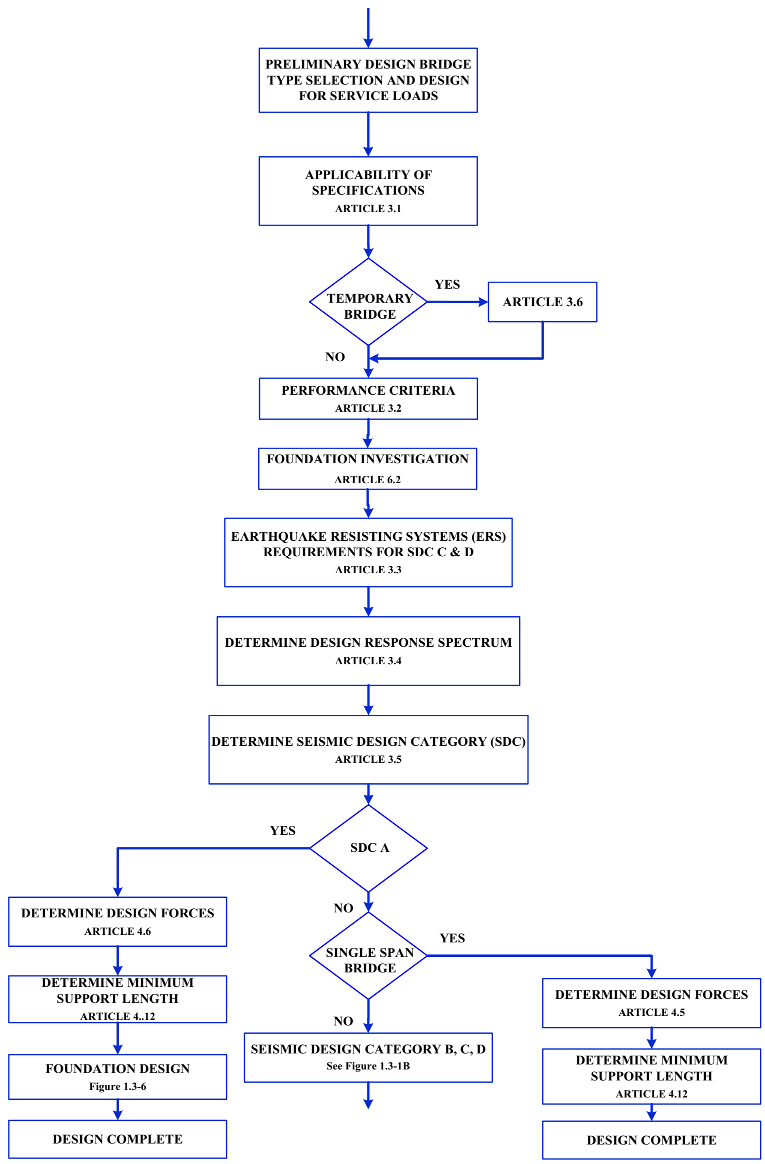


Figure 5.81 Seismic Design Procedure Flow Chart

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.34.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

Table 5.34 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.82 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that $S_{D1} = 0.14$ the example bridge is treated in SDC A with the following basic requirements:

- No Identification of ERS according to Article 3.3
- No Demand Analysis
- No Implicit Capacity Check Needed
- No Capacity Design Required
- Minimum detailing requirements for support length, superstructure/substructure connection design force, and column transverse steel
- No Liquefaction Evaluation Required

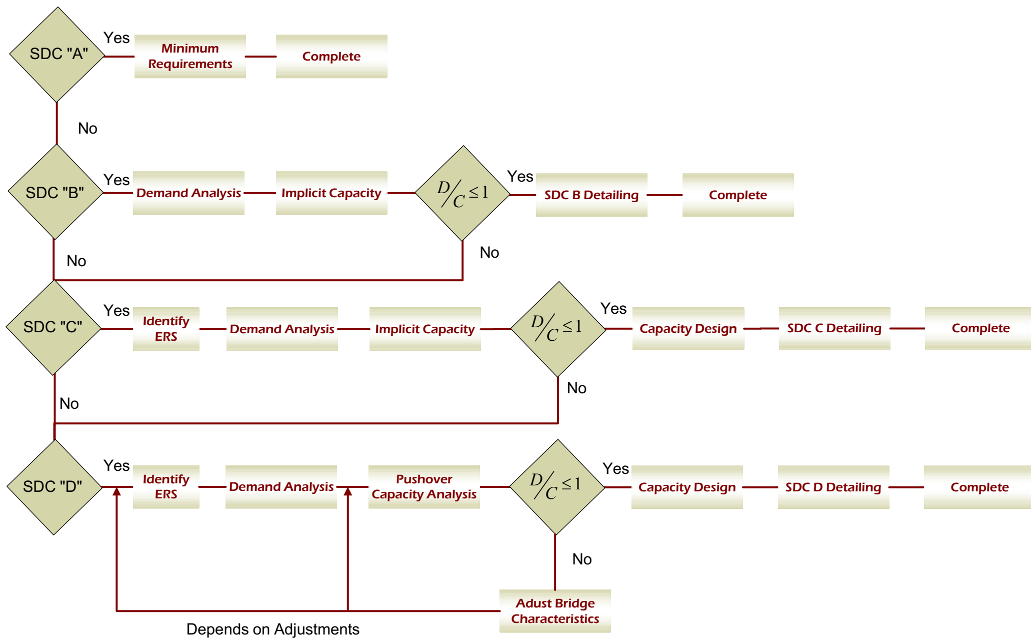


Figure 5.82 Seismic Design Category (SDC) Core Flowchart.

Seismic Analysis

Dead Load Calculation

Stringer Weight Calculation:

The cross-section area of 54"Prestressed Beam Typical Dimension sheet (See Figure 5.83) showing a value of 789 in^2/ft or 5.5 ft^2/ft

Total Beam Weight:

$$101\text{ ft}\times 11\text{ beams}\times 5.5\ \frac{\text{ft}^2}{\text{ft}}\times 0.15\text{ K/ft}^3 = 557\text{ Kips}$$

Total Deck Weight:

$$\frac{8}{12}\times 101\text{ ft}\times 57.1\text{ ft}\times 0.15\text{ K/ft}^3 = 577\text{ Kips}$$

Total Diaphragm Weight:
(2 intermediate diaphragm and 1 @ each abutment)

$$\frac{1}{\sin 27^\circ}\times 51.6\text{ ft}\times\left(\frac{9}{12}\times\frac{46}{12}\right)\times 0.15\text{ K/ft}^3\times 4 = 196\text{ Kips}$$

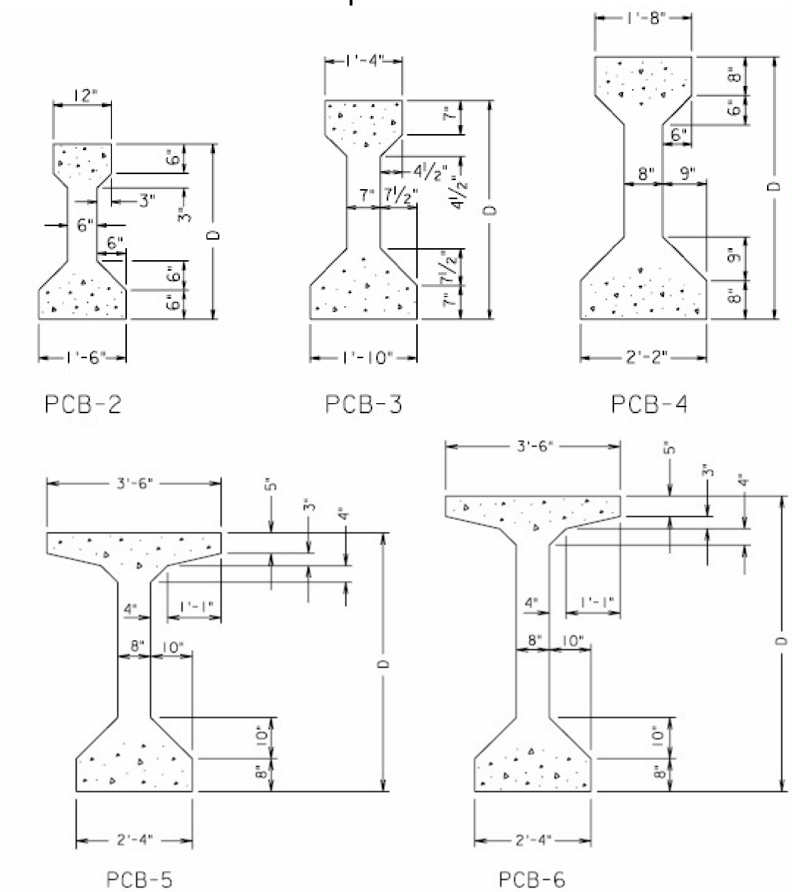
Barrier 1K/ft each side:

$$2\text{ K/ft}\times 101 = 202\text{ Kips}$$

Overlay 25psf:

$$0.025\text{ Ksf}\times 57.1\text{ ft}\times 101\text{ ft} = 144\text{ Kips}$$

Consider total weight of Superstructure as follows:
Beams: 917 Kips
Deck: 577 Kips
Diaphragms: 196 Kips
Barriers: 202 Kips
Overlay: 144 Kips
Total: 917+577+196+202+144= 2036 Kips



Beam Type	Depth D (in)	Area A (in^2)	Centroid to Bottom y_b (in)	Moment of Inertia I ($x\ 10^3\ in^4$)	Section Modulus		Weight @ 150 pcf (lbs/lin. ft.)
					S_{top} (in^3)	S_{bott} (in^3)	
PCB-2	36	369	15.83	50.98	2528	3220	384
PCB-3	45	560	20.27	125.39	6186	5070	583
PCB-4	54	789	24.73	260.73	8908	10543	822
PCB-5	63	1013	31.96	521.18	16791	16307	1055
PCB-6	72	1085	36.38	733.32	20587	20157	1130

Figure 5.83 Prestressed Concrete I-Beams Section Properties.

Design Requirements for Single Span Bridges, SDC A

According to section 4.5 of AASHTO-SGS

- A detailed seismic analysis shall not be deemed to be required for single span bridges regardless of SDC as specified in Article 4.1.
- The connections between the bridge span and the abutments shall be designed in both longitudinal and transverse directions to resist a horizontal seismic force not less than the effective peak ground acceleration coefficient, A_s, as specified in Article 3.4, times the tributary permanent load except as modified for SDC A in Article 4.6.
- The minimum support lengths shall be as specified in Article 4.12.

Bridge Bearing Connections

According to Section 4.6 of the AASHTO-SGS, for bridges in SDC A, where the acceleration coefficient, A_s, as specified in Article 3.4., is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, assumed to exist during an earthquake.

The NJ PGA calculated in the Site Seismicity Section is shown equal to 0.24g. Therefore, the horizontal design connection force is considered at the minimum of 0.25g mentioned above.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Considering 11 beams simply supported, the tributary permanent load per connection is calculated as:

$$\left(\frac{2036}{2}\right)/11 = 93 \text{ Kips}$$

According to AASHTO-SGS Section 8.13.3, the principal tension stress specified as $0.11\sqrt{f_c'}$ is used

where:

f_c' = nominal concrete compressive strength (ksi)

The principal tension stress of $0.11\sqrt{f_c'}$ corresponds to minimal concrete cracking and no yielding of reinforcement associated with the crack opening of concrete in the anchorage connection of the bearing.

Connection Lateral Load Demand (As described above according to AASHTO-SGS Sections 4.5 and 4.6): $93 \times 0.25 = 23\,\text{Kips}$

Tensile stress in concrete (Corresponding to minimal damage of the bearing connection): $0.11 \times \sqrt{4} = 0.22$ Ksi

Shear Failure Plane for Seat Pull-out is considered based on minimum 3 in. edge distance (as shown in Figure 5.84), the depth of the shear failure plan is considered equal to the pedestal dimension between the bearing centerline and the pedestal exterior face (1' – 6"): $\left[(5.25 \times 2) \times \sqrt{2}\right] \times 18'' = 267\ \text{in}^2$

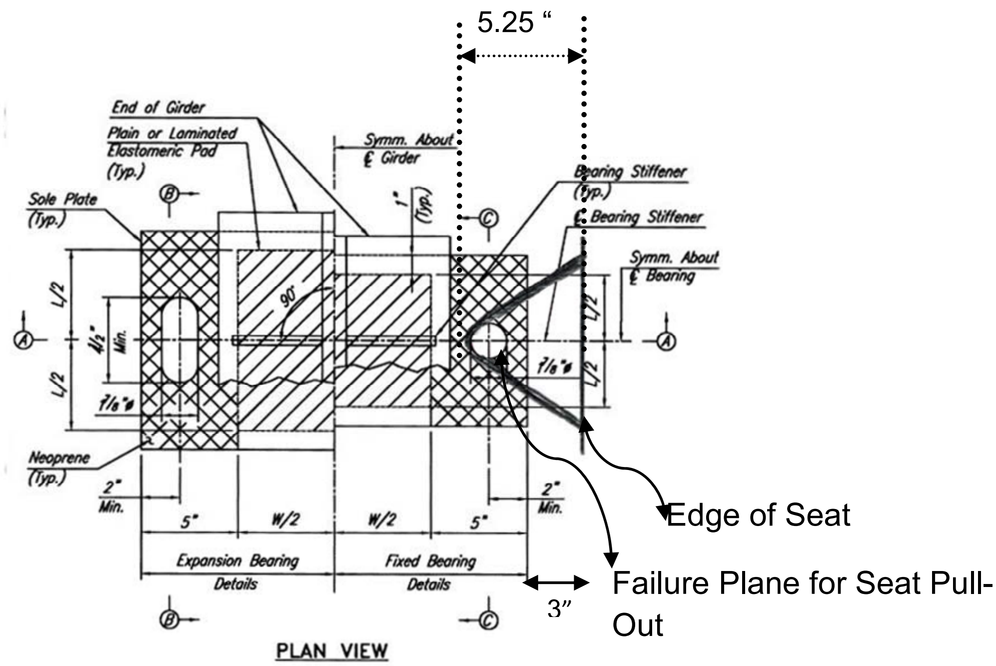


Figure 5.84 Anchor Bolt Shear Failure Plane

In calculating the seat pull out area, 18" is the embedment length of the bolt. This calculation is performed to show that concrete pull out doesn't govern. It is just a check to confirm that the bolt capacity is the focus in determining the strength of the connection.

Pull-out Capacity per Bolt:

$267 \times 0.22 = 59$ Kips

Consider $1'' \phi$ bolt:

According to AASHTO-SGS section 6.13: $R_n = 0.48 A_b F_{ub} N_s$

$R_n = 0.48 \times 0.785 \times 60 = 22.6$ Kips

$\phi_s R_n = 0.65 \times 22.6 = 14.7$ Kips (A307 bolts in shear $\phi_s = 0.65$)

For 1" ϕ bolt (See Experimental Testing of Anchor Bolts Appendix IV.A)

$P_{crack} = 13.7 \text{ Kips} @ \Delta_{crack} = 0.96''$

Consider Capacity @ 13.7 Kips based on Testing, considering Minimal Damage Requirement

Connection Capacity Considering 2 bolts: 2 × 13.7 Kips = 27.4 Kips $>$ 23 Kips O.K.,

where 23 kips is the connection lateral load demand.

Examine bolt anchor capacity based on ACI318 "Appendix D Anchoring to Concrete" section.

The basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall not exceed:

$$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda\sqrt{f_c'}\left(C_{a1}\right)^{1.5}$$

Where l_e is the load-bearing length of the anchor for shears equal to the embedment depth, and in no case shall exceeds $8d_a$

C_{a1} is the edge distance as shown in Figure 5.85, d_a is the anchor diameter.

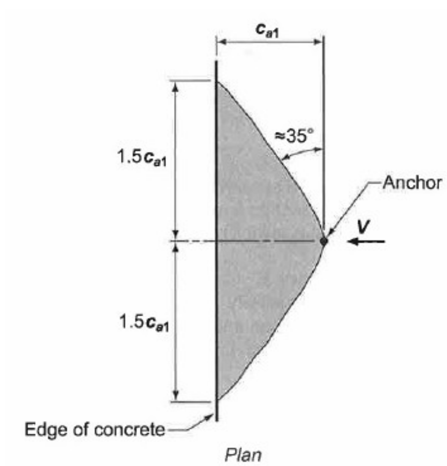


Figure 5.85 Break-Out Cone for Shear.

Modification factor λ is for light weight concrete. The value of $f_c^{'}$ shall not exceed 10,000 psi for cast-in anchors.

Based on 3" minimum edge distance to the sole plate (See Figure 5.84):

$$V_b = 7(8)^{0.2}\sqrt{1''}\sqrt{5000}(5)^{1.5}$$
$$= 7 \times 1.5 \times 1 \times \frac{70}{1000} \times 11.2$$
$$= 8.2 \text{ Kips}$$

Providing 5 in edge distance, the shear capacity of the 1" ϕ bolt is equal to:

$$V_b = 7(8)^{0.2}\sqrt{1}\sqrt{5000}(7)^{1.5}$$
$$= 7 \times 1.5 \times 1.0 \times \frac{70}{1000}(18.5) = 13.5 \text{ Kips}$$

It is deemed that the 3 in. minimum edge distance is adequate considering that ACI values are conservative relative to experimental values.

Check Minimum Support Length.

Figures 5.78 and 5.79 show a typical abutment section and the corresponding seat detail.

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.35 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; For hinges within a span, L shall be the sum of the distances to either side of the hinge; For single-span bridges, L equals the length of the bridge deck (ft.)

H = for abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.); For columns and/or piers, column, or pier height (ft.); For hinges within a span, average height of the adjacent two columns or piers (ft.); 0.0 for single-span bridges (ft.)

S = angle of skew of support measured from a line normal to span (°)

Table 5.35 Percentage N by SDC and Effective Peak Ground Acceleration, A_s.

SDC	Effective peak ground acceleration, A_s	Percent N
A	< 0.05	≥ 75
A	≥ 0.05	100
B	All applicable	150
C	All applicable	150

For SDC A:

$N = 1.0(8+0.02L+0.08H)(1+0.000125S^2)$

Elevations at abutment 1 and 2 Westbound and Eastbound are considered to determine the height "H" used in calculating the support "N"; Figure 5.56 shows Abutment 1 Eastbound Plan and Elevation, others are comparable as shown in the table below:

	Top of Pedestal	Bottom of Footing	H
Abutment 1 Westbound	653.5ft	631.5ft	22ft
Abutment 1 Eastbound	651.7ft	629.5ft	22.2ft
Abutment 2 Westbound	651.8ft	630.0ft	21.8ft
Abutment 2 Eastbound	649.8ft	629ft	20.8ft

Based on table above, consider "H" equal to 22.2ft conservatively.

Length of Bridge Deck: L = 101'

For SDC A:

Angle of Skew of Support: S = 62.7°

$$N = 1.0(8+0.02\times101'+1.8\times22.2')(1+0.000125\times62.7^2) = 17.6"$$

$$N = 1.0(8+2+1.8)(1+0.49) = 17.6"$$

Available Seat Length: (See Figure 5.79)

(1'-6") + 13.5cos27.3° = 18 + 12 = 30"

Available Support Length: 30"-1" joint = 29"

Available Support Length O.K. greater than required support length N.

Example 7: Design of a Single Span Concrete bridge in SDC B Category

Bridge Description

This example is based on a bridge carrying Route 101 over Ramp D, Morris County, New Jersey. The bridge is a precast superstructure single span supported by seat abutments. Following figures show relevant drawings needed for calculations.

Figure 5.86: Plan and Elevation
Figure 5.87: Typical Bridge Section (Westbound)
Figure 5.88: Abutments 1 Eastbound Plan and Elevation
Figure 5.89: Abutment Typical Section
Figure 5.90: Anchor Bolts Location at Abutment Pedestals

Standard drawing shown on the Index Table below were not provided. Therefore, information about precast beam properties was not taken directly from these standard drawings, but deemed close enough to evaluate the subject bridge for the AASHTO-SGS.

INDEX	
TITLE	SHEET No.
General Plan	1
Abutment #1 - Westbound	2
Abutment #1 - Eastbound	3
Abutment #2 - Westbound	4
Abutment #2 - Eastbound	5
Abutment Details & Sections	6
Wingwalls	7
Wingwall Sections	8
Superstructure	9
Reinforcing Bar Details	10
Bearings - 45" & 54" Beams	SD-2
Beam Details	SD-3
54" Prestressed Concrete Beams	SD-6
Typical Architectural Details	SD-7
End Post & Miscellaneous Details	SD-8

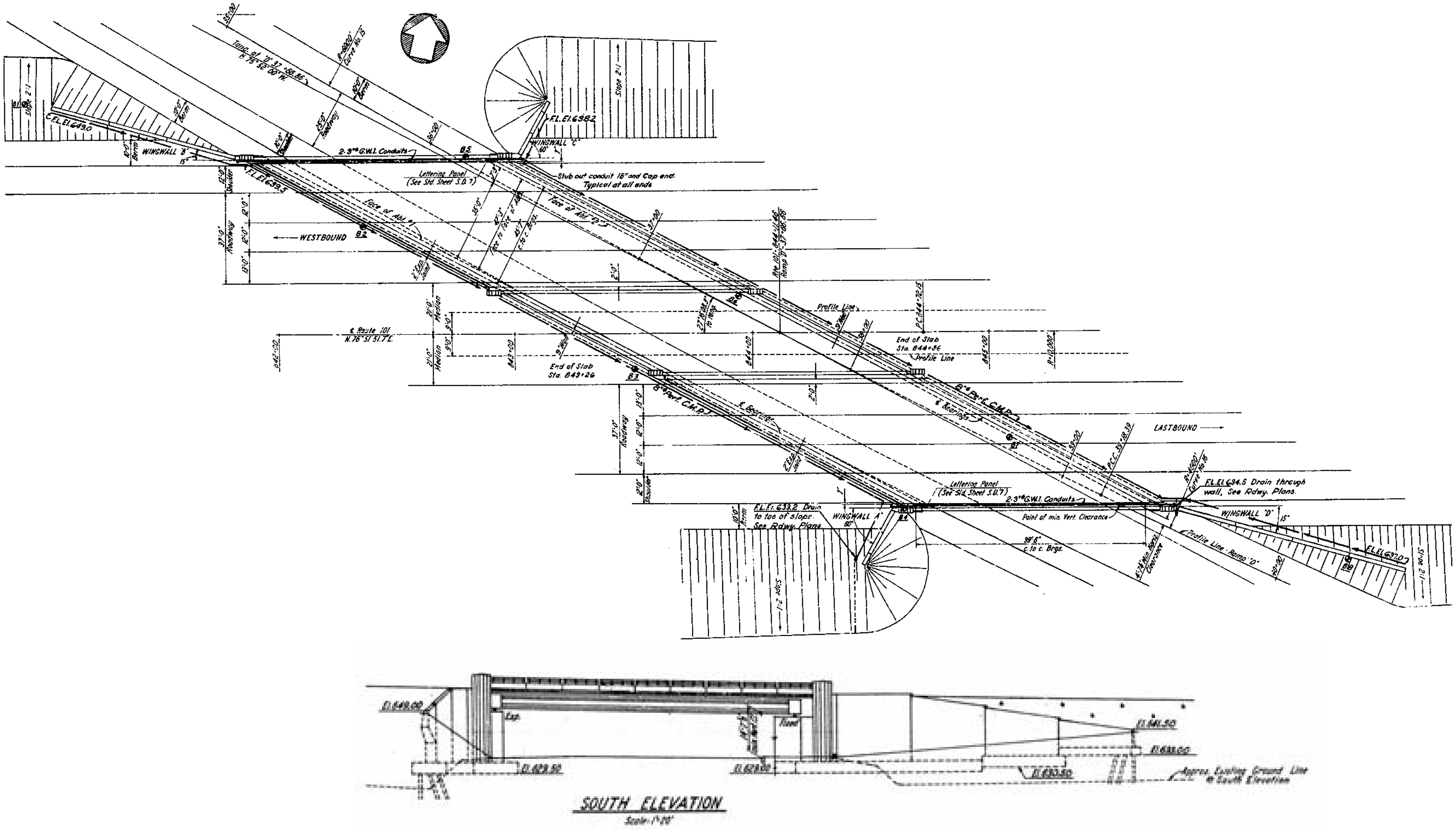


Figure 5.86 Plan and Elevation

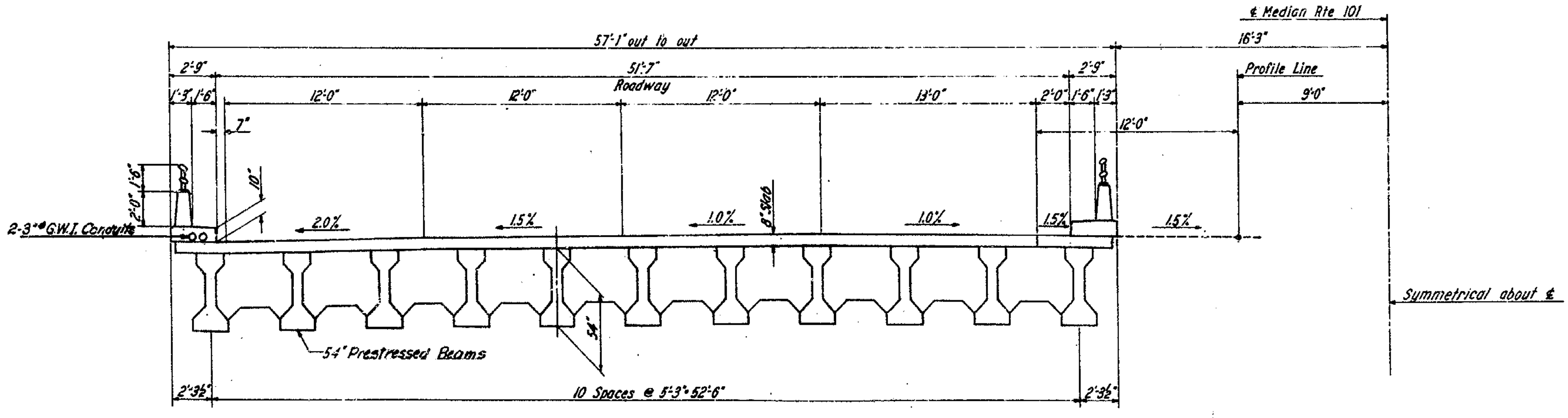


Figure 5.87 Typical Bridge Section (Westbound shown, Eastbound Similar)

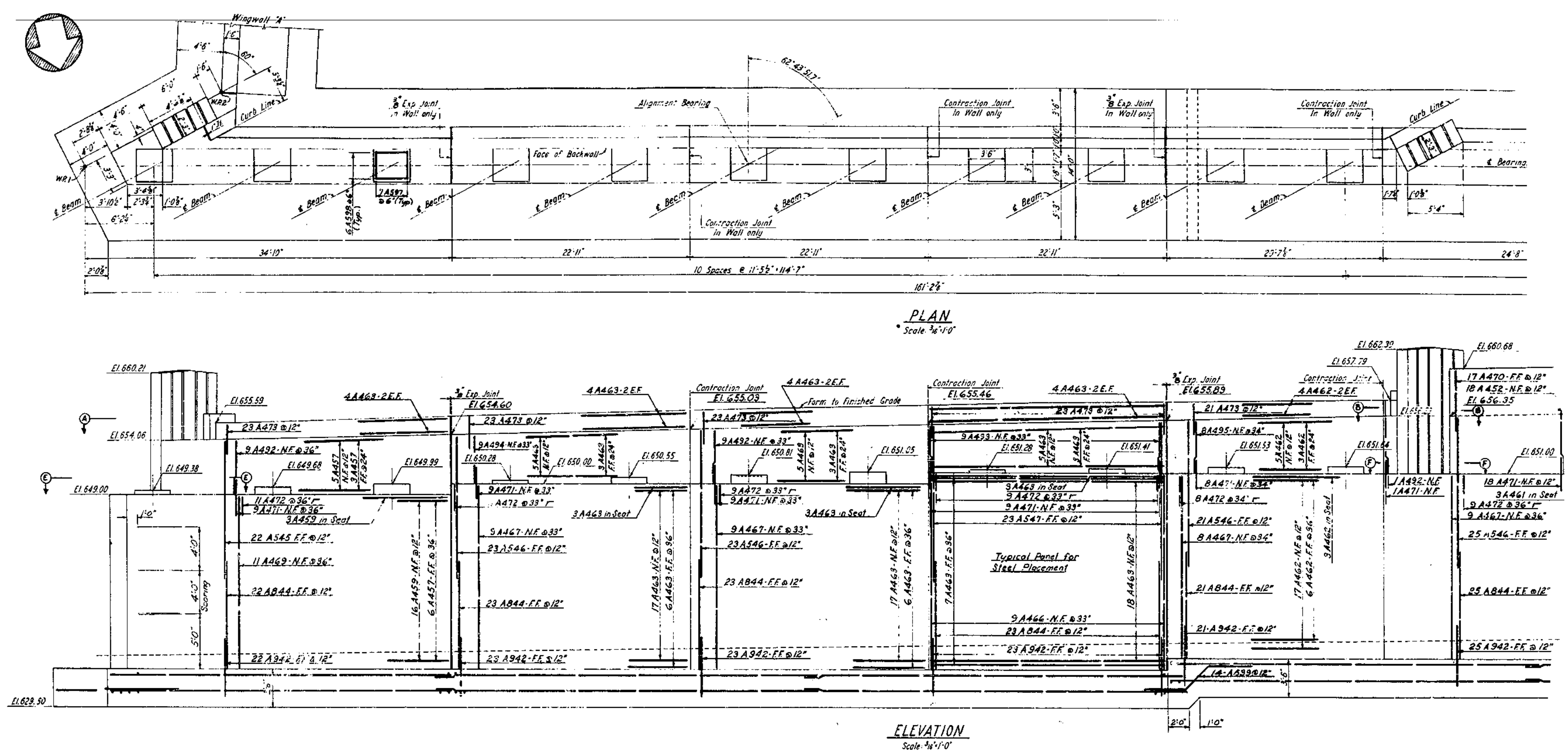


Figure 5.88 Abutment 1 Eastbound Plan and Elevation

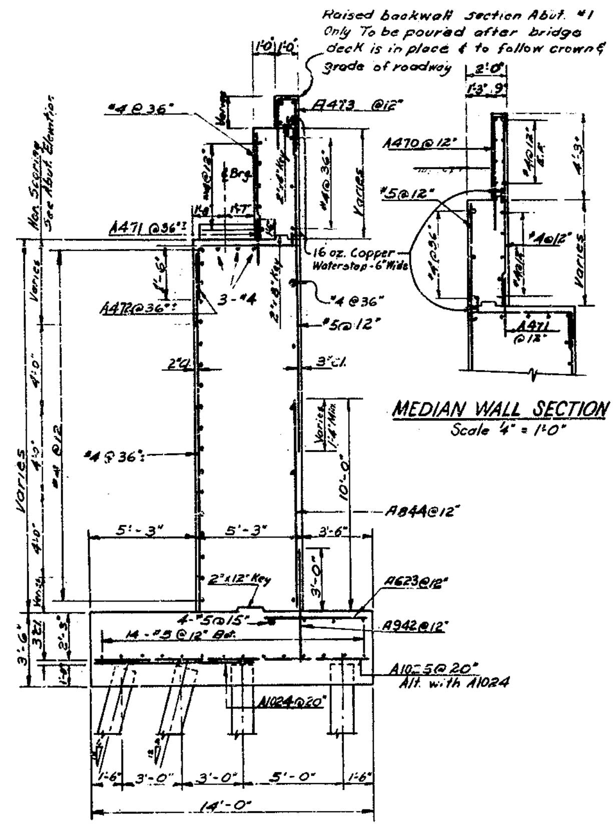


Figure 5.89 Abutment Typical Section

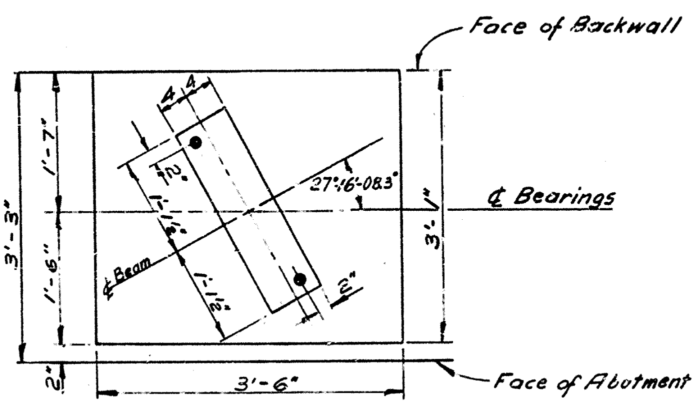
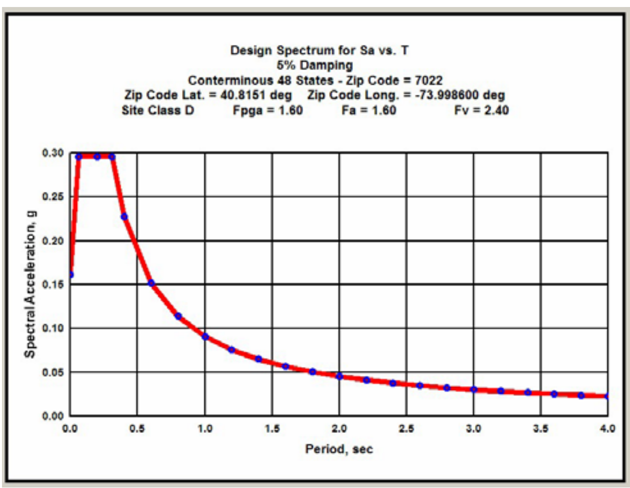


Figure 5.90 Anchor Bolts Location at Abutment Pedestals

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.91. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.91 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D

PGA = 1.5 × 0.16 = 0.24

S_{DS} = 1.5 × 0.3 = 0.45

S_{D1} = 1.5 × 0.09 = 0.14

Flow Charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.

- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 7 reflects a Type 3 bridge system with the bearing connections considered to be the critical locations to the seismic load path.
- Flowchart 1a of section of section 1.3 of the AASHTO-SGS shown in Figure 5.92 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a single span bridge versus a multi-span bridge.

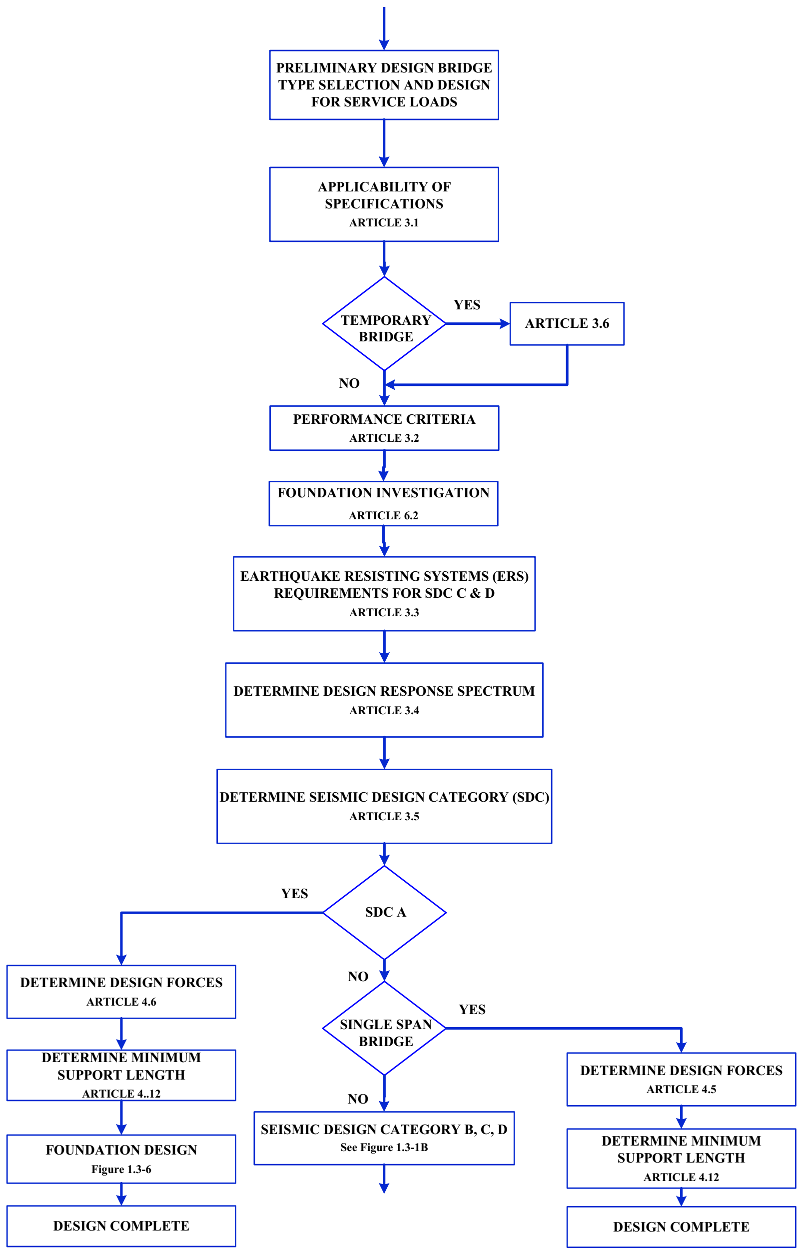


Figure 5.92 Seismic Design Procedure Flow Chart

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.36.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}

Table 5.36 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.93 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} =0.14, the example bridge is treated in SDC B with the following basic requirements:

- No Identification of ERS according to Article 3.3
- Demand Analysis
- Implicit Capacity Check Required (displacement, P-Δ□ support length)
- No Capacity Design Required except for column shear requirement
- SDC B Level of Detailing

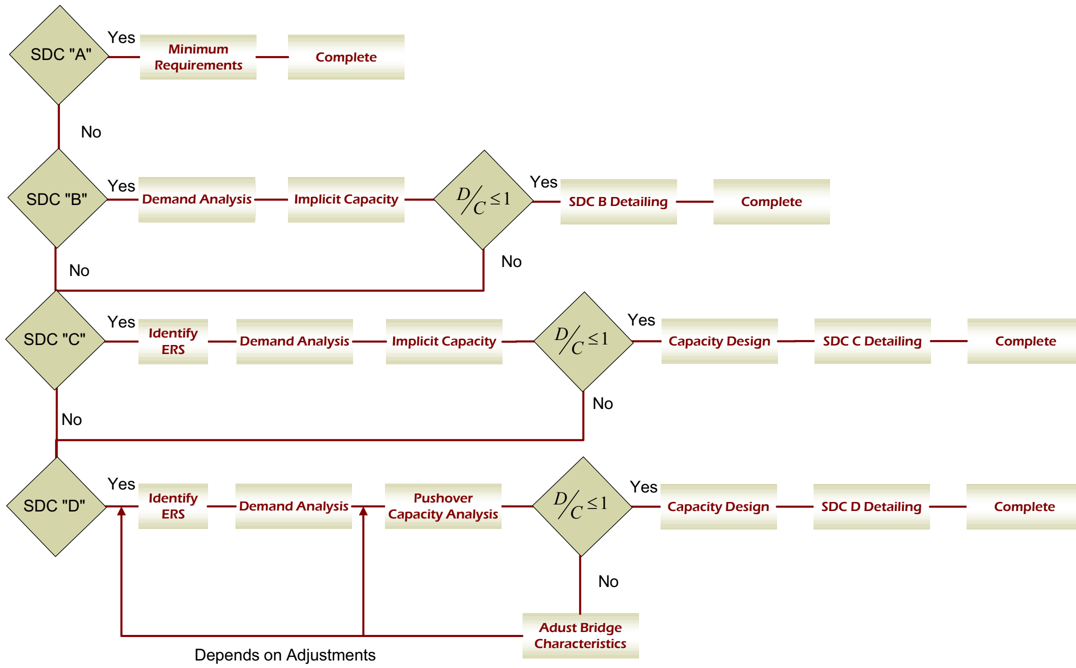


Figure 5.93 Seismic Design Category (SDC) Core Flowchart.

Seismic Analysis

Dead Load Calculation

Stringer Weight Calculation:

The cross-section area of 54"Prestressed Beam Typical Dimension sheet (See Figure 5.94) showing a value of 789 in^2/ft or 5.5 ft^2/ft

Total Beam Weight:

$$101\text{ ft} \times 11\text{ beams} \times 5.5\ \frac{\text{ft}^2}{\text{ft}} \times 0.15\text{ K/ft}^3 = 557\text{ Kips}$$

Total Deck Weight:

$$\frac{8}{12} \times 101\text{ ft} \times 57.1\text{ ft} \times 0.15\text{ K/ft}^3 = 577\text{ Kips}$$

Total Diaphragm Weight:

(2 intermediate diaphragm and 1 @ each abutment)

$$\frac{1}{\sin 27^\circ} \times 51.6\text{ ft} \times \left(\frac{9}{12} \times \frac{46}{12}\right) \times 0.15\text{ K/ft}^3 \times 4 = 196\text{ Kips}$$

Barrier 1K/ft each side:

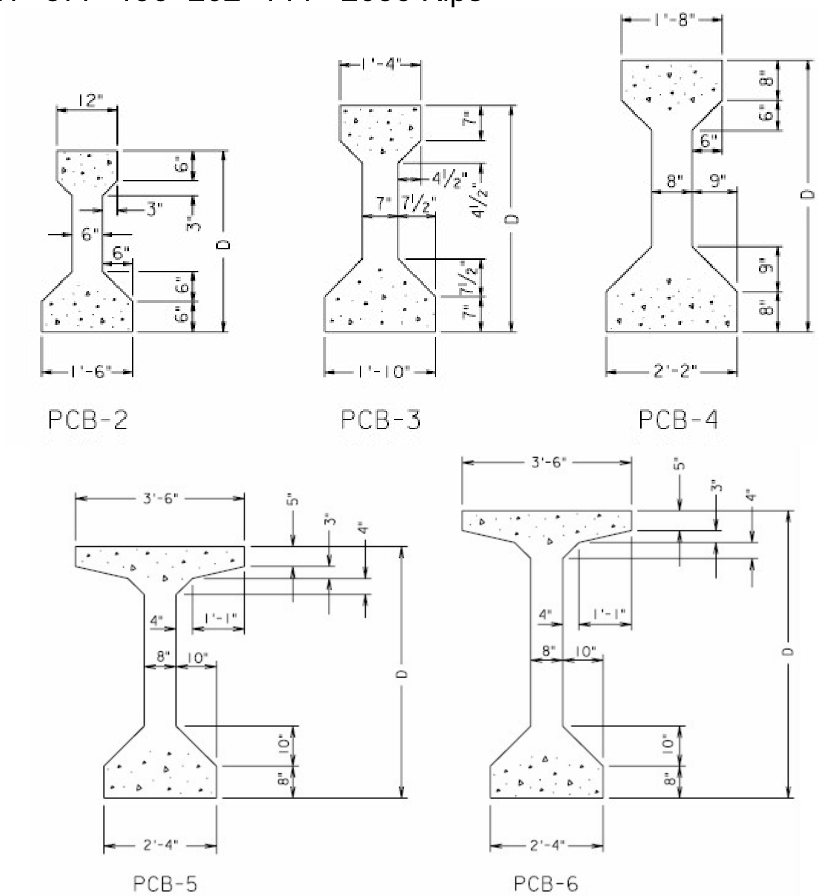
$$2\text{ K/ft} \times 101 = 202\text{ Kips}$$

Overlay 25psf:

$$0.025\text{ Ksf} \times 57.1\text{ ft} \times 101\text{ ft} = 144\text{ Kips}$$

Consider total weight of Superstructure as follows:

Beams:	917 Kips
Deck:	577 Kips
Diaphragms:	196 Kips
Barriers:	202 Kips
Overlay:	144 Kips
Total: 917+577+196+202+144=	2036 Kips



Beam Type	Depth D (in)	Area A (in^2)	Centroid to Bottom y_b (in)	Moment of Inertia I ($x\ 10^3\ in^4$)	Section Modulus S_{top} (in^3)	Section Modulus S_{bott} (in^3)	Weight @ 150 pcf (lbs/lin. ft.)
PCB-2	36	369	15.83	50.98	2528	3220	384
PCB-3	45	560	20.27	125.39	6186	5070	583
PCB-4	54	789	24.73	260.73	8908	10543	822
PCB-5	63	1013	31.96	521.18	16791	16307	1055
PCB-6	72	1085	36.38	733.32	20587	20157	1130

Figure 5.94 Prestressed Concrete I-Beams Section Properties

Design Requirements for Single Span Bridges

According to section 4.5 of AASHTO-SGS

- A detailed seismic analysis shall not be deemed to be required for single span bridges regardless of SDC as specified in Article 4.1.

- The connections between the bridge span and the abutments shall be designed both longitudinally and transversely to resist a horizontal seismic force not less than the effective peak ground acceleration coefficient, A_s, as specified in Article 3.4, times the tributary permanent load except as modified for SDC A in Article 4.6.
- The lateral force shall be carried into the foundation in accordance with Articles 5.2 and 6.7.
- The minimum support lengths shall be as specified in Article 4.12.

Bridge Bearing Connections

According to Section 4.6 of the AASHTO-SGS, for bridges in SDC A, where the acceleration coefficient, A_s, as specified in Article 3.4., is less than 0.05, the horizontal design connection force in the restrained directions shall not be less than 0.15 times the vertical reaction due to the tributary permanent load.

For all other sites in SDC A, the horizontal design connection force in the restrained directions shall not be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads, if applicable, assumed to exist during an earthquake.

The NJ PGA calculated in the Site Seismicity Section is shown equal to 0.24g. Therefore, the horizontal design connection force is considered at the minimum of 0.25g mentioned above.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDC A and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

Considering 11 beams simply supported, the tributary permanent load per connection is calculated as:

$$\left(\frac{2036}{2}\right)/11 = 93 \text{ Kips}$$

According to AASHTO-SGS Section 8.13.3, the principal tension stress specified as $0.11\sqrt{f_c'}$ is used

Where $\sqrt{f_c'}$ = nominal concrete compressive strength (ksi)

The principal tension stress of $0.11\sqrt{f_c^{'}}$ corresponds to minimal concrete cracking and no yielding of reinforcement associated with the crack opening of concrete in the anchorage connection of the bearing.

Connection Lateral Load Demand (As described above according to AASHTO-SGS Sections 4.5 and 4.6):

93 × 0.25 = 23 Kips

Tensile stress in concrete (Corresponding to minimal damage of the bearing connection):

$0.11 \times \sqrt{4} = 0.22$ Ksi

Shear Failure Plane Area for Seat Pull-out is considered based on minimum 3 in. edge distance (as shown in Figure 5.63), the depth of the shear failure plan is considered equal to the pedestal dimension between the bearing centerline and the pedestal exterior face (1' – 6"): $\left[(5.25 \times 2) \times \sqrt{2}\right] \times 18'' = 267 \text{ in}^2$.

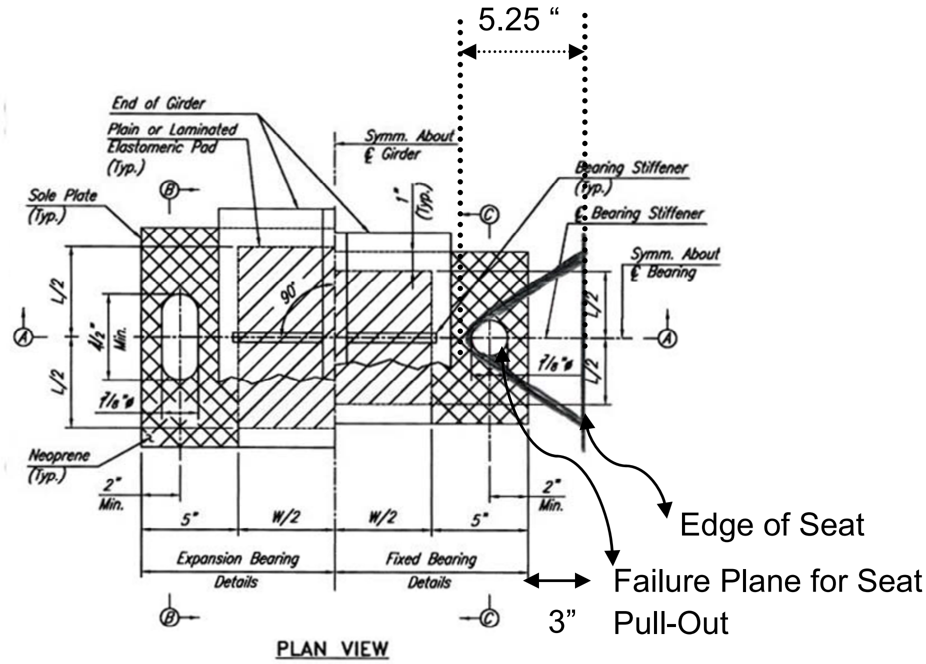


Figure 5.95 Anchor Bolt Shear Failure Plane

In calculating the seat pull out area, 18" is the embedment length of the bolt. This calculation is performed to show that concrete pull out doesn't govern. It is just a check to confirm that the bolt capacity is the focus in determining the strength of the connection.

Pull-out Capacity per Bolt = Shear Failure Plane Area ×Tensile Stress in Concrete = 267 × 0.22 = 59 Kips

Consider 1" ϕ bolt:

According to AASHTO-SGS section 6.13: $R_n = 0.48 A_b F_{ub} N_s$

$$R_n = 0.48 \times 0.785 \times 60 = 22.6 \text{ Kips}$$

$$\phi_s R_n = 0.65 \times 22.6 = 14.7 \text{ Kips} \qquad \text{(A307 bolts in shear } \phi_s = 0.65\text{)}$$

For 1" ϕ bolt (See Experimental Testing of Anchor Bolts Appendix VII.A)

$$P_{crack} = 13.7 \text{ Kips} @ \Delta_{crack} = 0.96''$$

Consider Capacity @ 13.7 Kips based on Testing, considering Minimal Damage Requirement

Connection Capacity Considering 2 bolts:

= 2 × 13.7 Kips = 27.4 Kips > 23 Kips, where 23 kips is the connection lateral load demand.

Examine bolt anchor capacity based on ACI318 "Appendix D Anchoring to Concrete" section.

The basic concrete breakout strength in shear of a single anchor in cracked concrete, V_b, shall not exceed:

$$V_b = \left(7 \left(\frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda \sqrt{f'_c} \left(C_{a1} \right)^{1.5}$$

Where l_e is the load-bearing length of the anchor for shears equal to the embedment depth, and in no case shall exceeds $8d_a$, C_{a1} is the edge distance as shown in Figure 5.96, d_a is the anchor diameter and λ is the modification factor for light weight concrete.

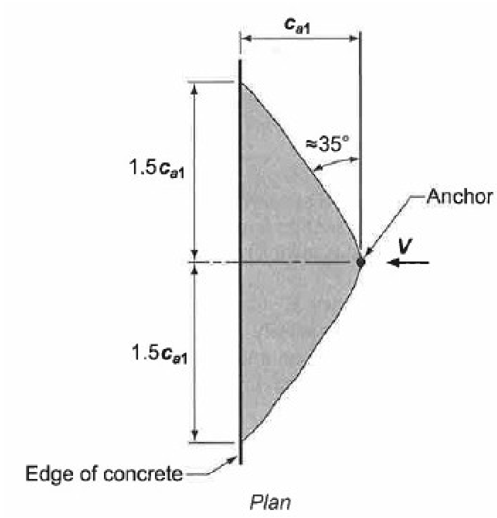


Figure 5.96 Break out cone for shear

The value of f'_c shall not exceed 10,000 psi for cast-in anchors. Based on 3" minimum edge distance to the sole plate (See Figure 5.95):

$$V_b = 7(8)^{0.2} \sqrt{1''} \sqrt{5000} (5)^{1.5}$$

$$= 7 \times 1.5 \times 1 \times \frac{70}{1000} \times 11.2 \ = 8.2 \text{ Kips}$$

Providing 5 in edge distance, the shear capacity of the 1” ϕ bolt is equal to:

$$V_b = 7\left(8\right)^{0.2}\sqrt{1}\sqrt{5000}\left(7\right)^{1.5}$$

$$= 7 \times 1.5 \times 1.0 \times \frac{70}{1000}\left(18.5\right) = 13.5 \text{ Kips}$$

It is deemed that the 3 in. minimum edge distance is adequate considering that ACI values are conservative relative to experimental values.

Abutment Lateral Load Path into the Foundation

According to AASHTO-SGS Sections 5.2 and 6.7, abutments in SDC B are expected to resist earthquake loads with minimal damage. For seat-type abutments, minimal abutment movement could be expected under dynamic passive pressure conditions. Testing at UCLA Report 2007/02 summarized in Appendix 1B show that friction contribution is sufficient for satisfying SDC B requirement for lateral load path into the abutment foundation.

Check Minimum Support Length

Figures 5.89 and 5.90 show a typical abutment section and the corresponding seat detail.

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU’s or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.37 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = Minimum support length measured normal to the centerline of bearing (in.)

L = Length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; For hinges within a span, L shall be the sum of the distances to either side of the hinge; For single-span bridges, L equals the length of the bridge deck (ft.)

H = For abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.); For columns and/or piers, column, or pier height (ft.); For hinges within a span, average height of the adjacent two columns or piers (ft.); 0.0 for single-span bridges (ft.)

S = angle of skew of support measured from a line normal to span (°)

Table 5.37 Percentage N by SDC and effective peak ground acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percent N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC B:

$N = 1.5(8+0.02L+0.08H)(1+0.000125S^2)$

Elevations at abutment 1 and 2 Westbound and Eastbound are considered to determine the height "H" used in calculating the support "N"; Figure 5.56 shows Abutment 1 Eastbound Plan and Elevation, others are comparable as shown in the table below:

	Top of Pedestal	Bottom of Footing	H
Abutment 1 Westbound	653.5ft	631.5ft	22ft
Abutment 1 Eastbound	651.7ft	629.5ft	22.2ft
Abutment 2 Westbound	651.8ft	630.0ft	21.8ft
Abutment 2 Eastbound	649.8ft	629ft	20.8ft

Consider "H" equal to 22.2ft conservatively.

Length of Bridge Deck: L = 101'

Angle of Skew of Support: S = 62.7°

$N = 1.5(8+0.02\times101'+1.8\times22.2)(1+0.000125\times62.7°) = 26.4"$

$N = 1.5(8+2+1.8)(1+0.49) = 26.4"$

Available Seat Length (See Figure 5.90) =

(1'-6")+13.5cos27.3° = 18+12 = 30"

Available Support Length: 30"-1" joint = 29"

Available Support Length O.K., greater than required support length N.

Example 8: Design of a Six Span Concrete Bridge in SDC B Category

Bridge Description

This example is based on a bridge carrying Route 70 over Manasquian River, Structure No. 1511-150. The bridge is a six span with continuous superstructure over pier 1, 2, 4, and 5 with expansion joints at West Abutment 1, Pier 3, and East Abutment. The abutments are seat type. Figure 5.97 shows the General Plan and Elevation of the bridge. Figure 5.98 shows a typical section that includes the superstructure and substructure. Figure 5.99 shows the photo of the bridge during construction. Appendix VII.B contains superstructure details. Appendix VII.C contains substructure details.

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.100. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data, the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user clicking on a map name from the list to display the map.

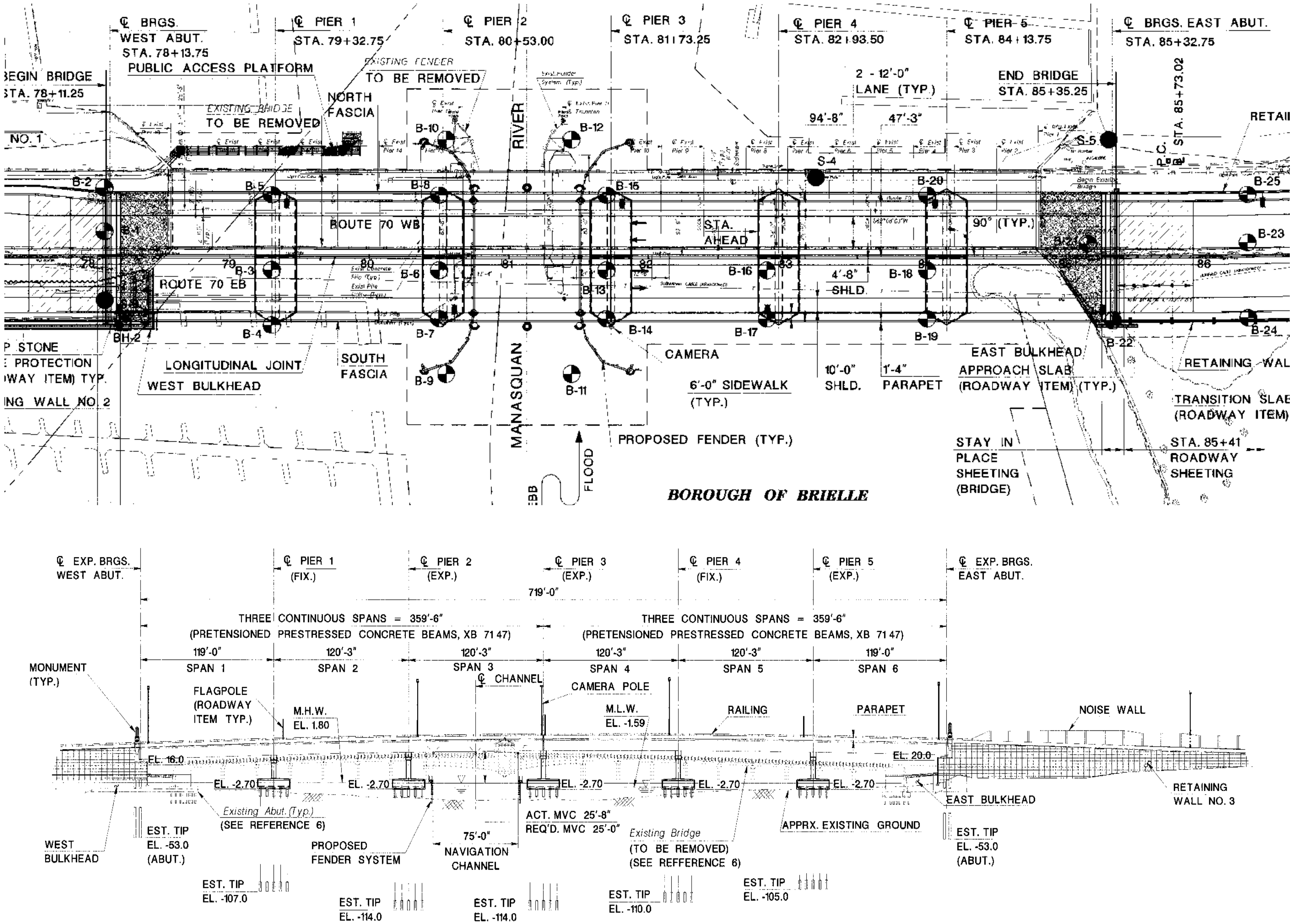


Figure 5.97 Plan and Elevation for Route 70 Over Manasquian River Bridge

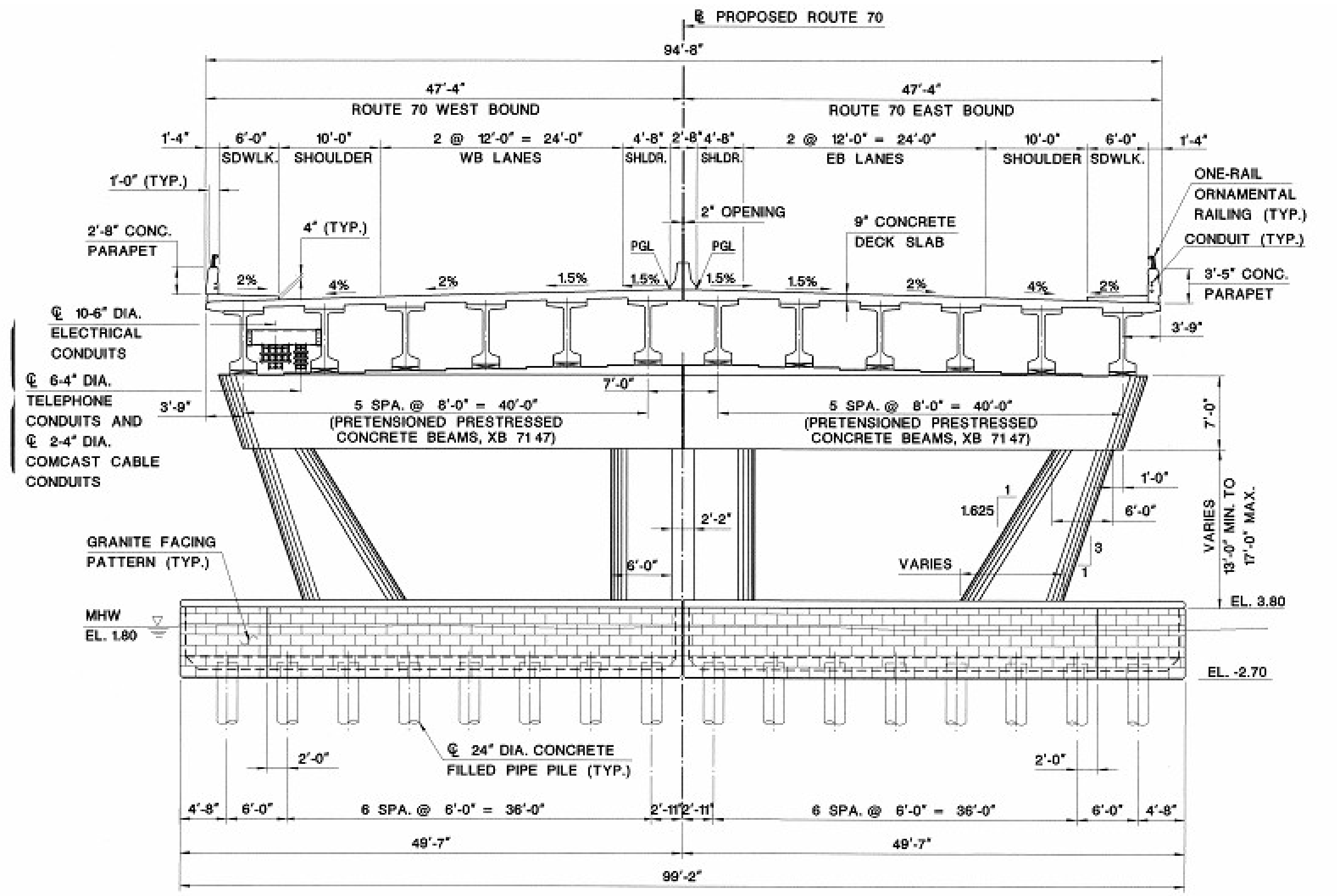
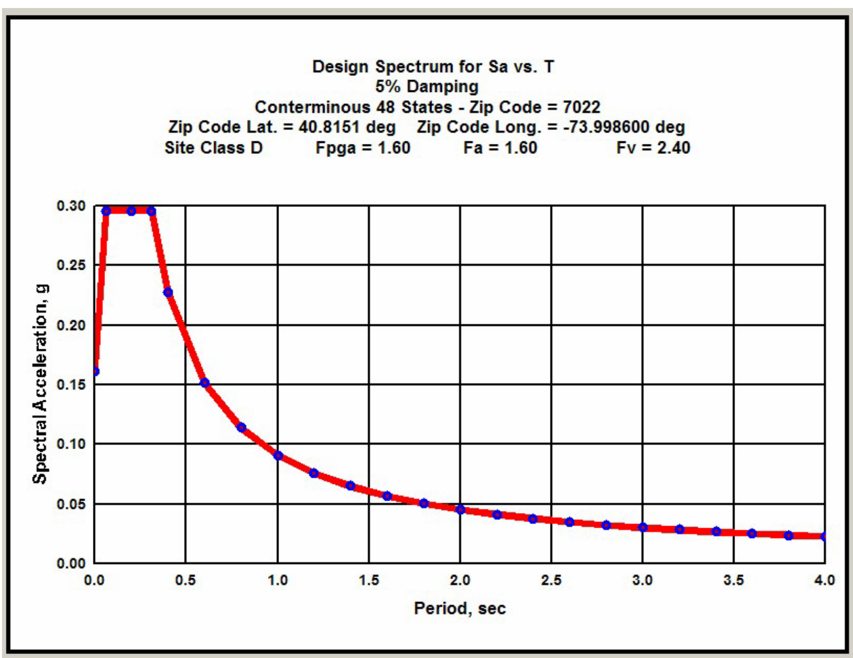


Figure 5.98 Typical Section

Figure 5.99 Route 70 Over Manasquian River Bridge during construction



Graph Data	
Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.100 AASHTO-USGS Site Class D Unfactored Design Spectrum

Calculate NJ Factored Design Spectrum parameters developed for site class D:

PGA = 1.5×0.16 = 0.24

S_{DS} = 1.5×0.3 = 0.45

S_{D1} = 1.5×0.09 = 0.14

Flow charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 3 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.101 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a multi-span bridge. Figure 5.102 shows the core flow chart of procedures outlined for bridges in SDC B, C, and D. Figure 5.103 outlines the demand analysis. Figure 5.104 directs the designer to determine displacement capacity. Figure 5.105 shows the modeling procedure. Figure 5.106 shows the foundation and abutment design applicable mainly for SDC C and D.

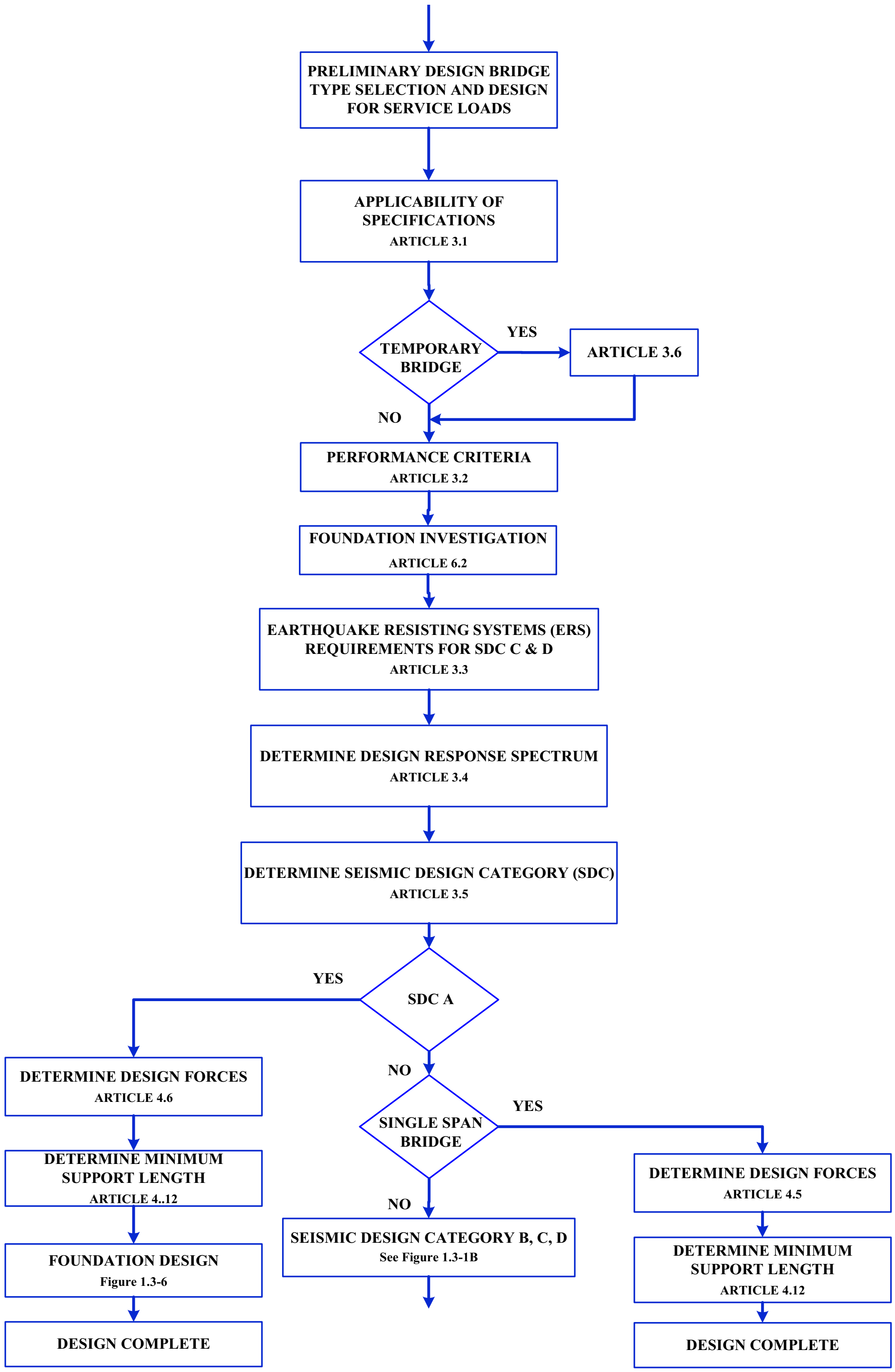


Figure 5.101 Seismic Design Procedure Flow Chart 1a

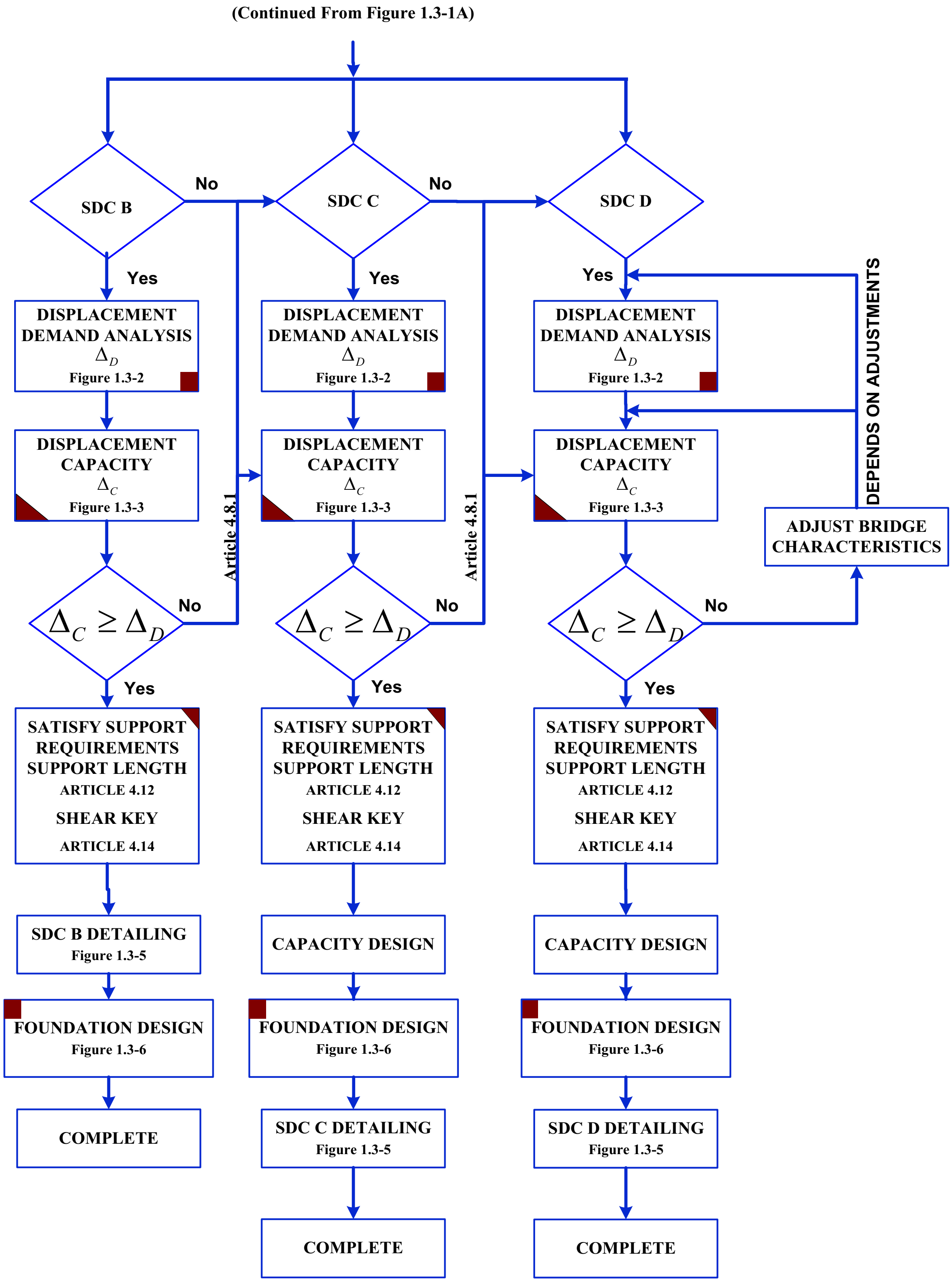


Figure 5.102 Seismic Design Procedure Flow Chart 1b

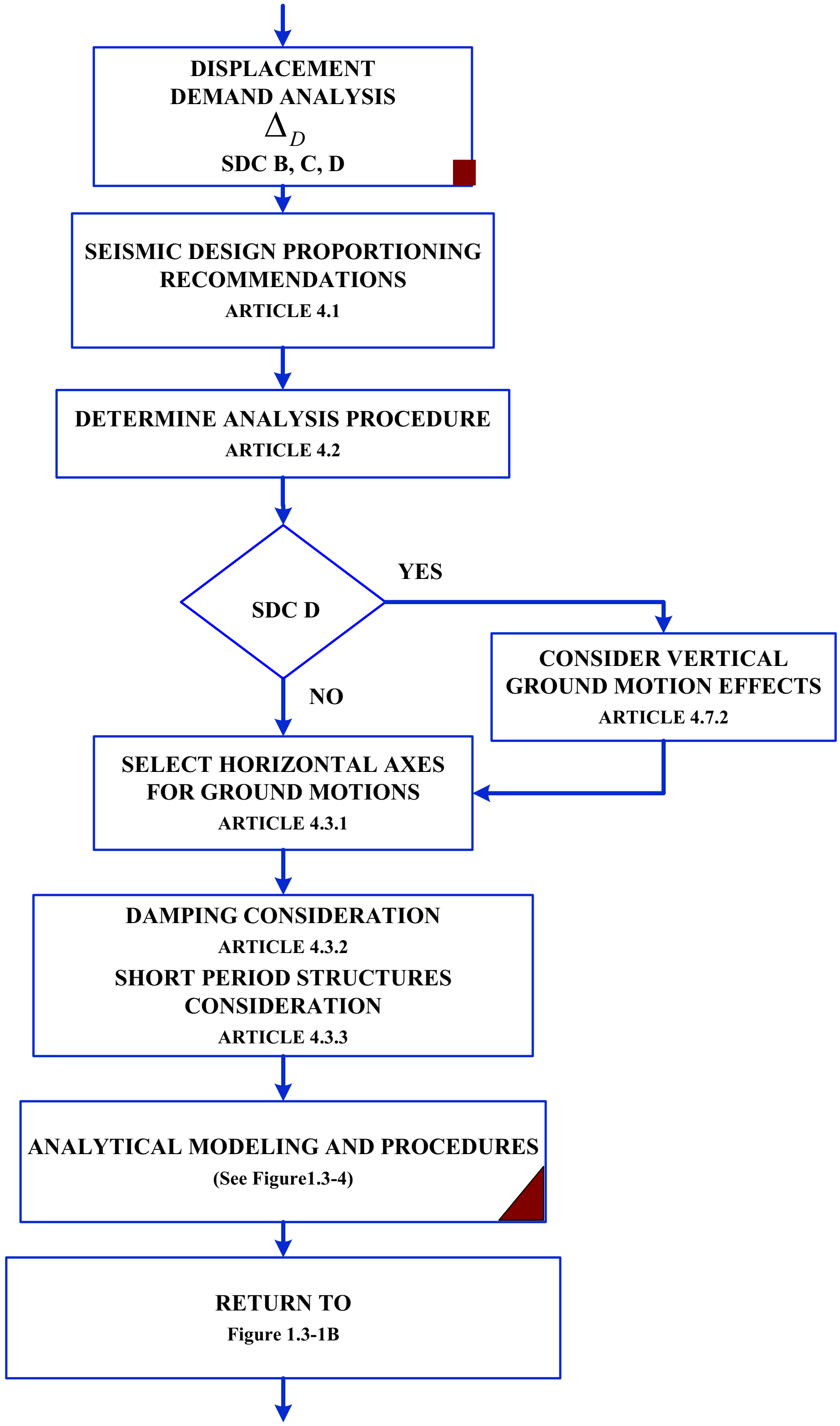


Figure 5.103 Demand Analysis Flow Chart 2

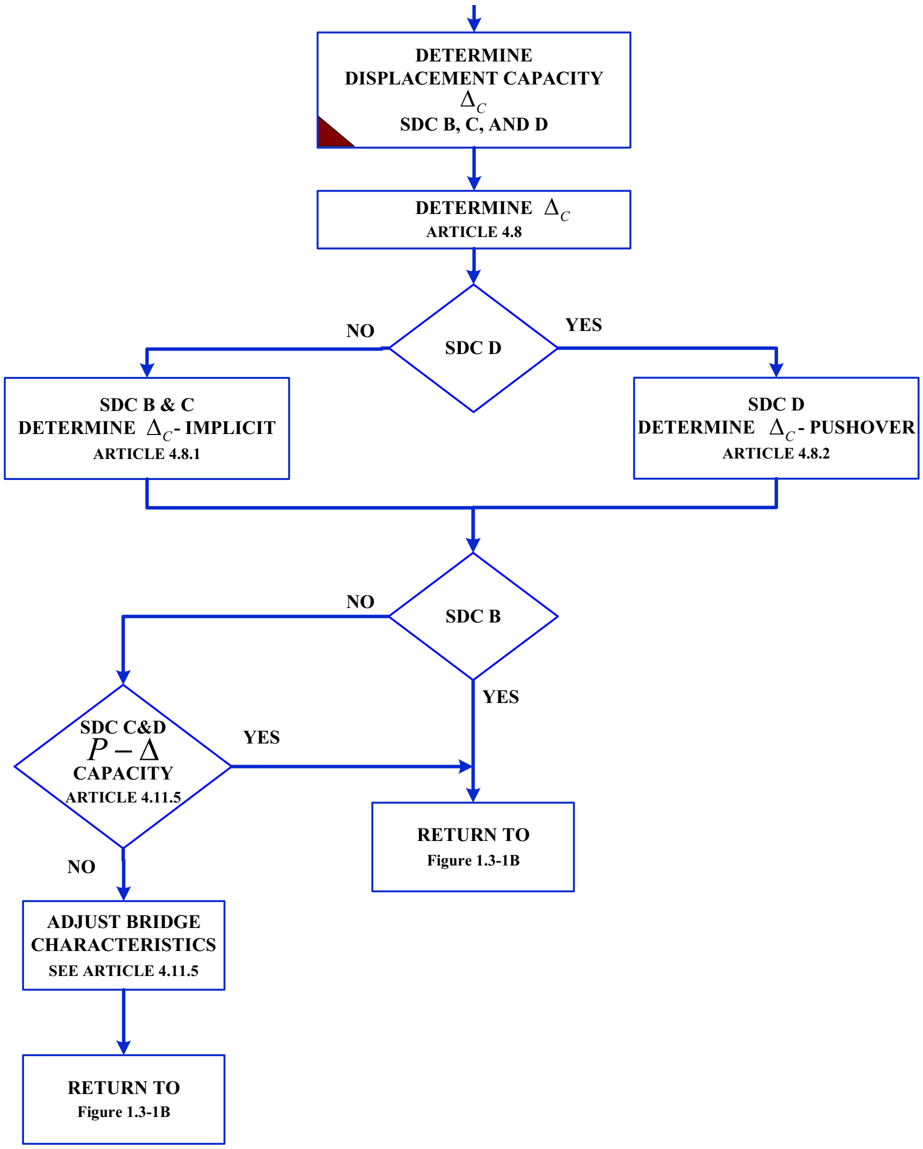


Figure 5.104 Displacement Capacity Flow Chart 3

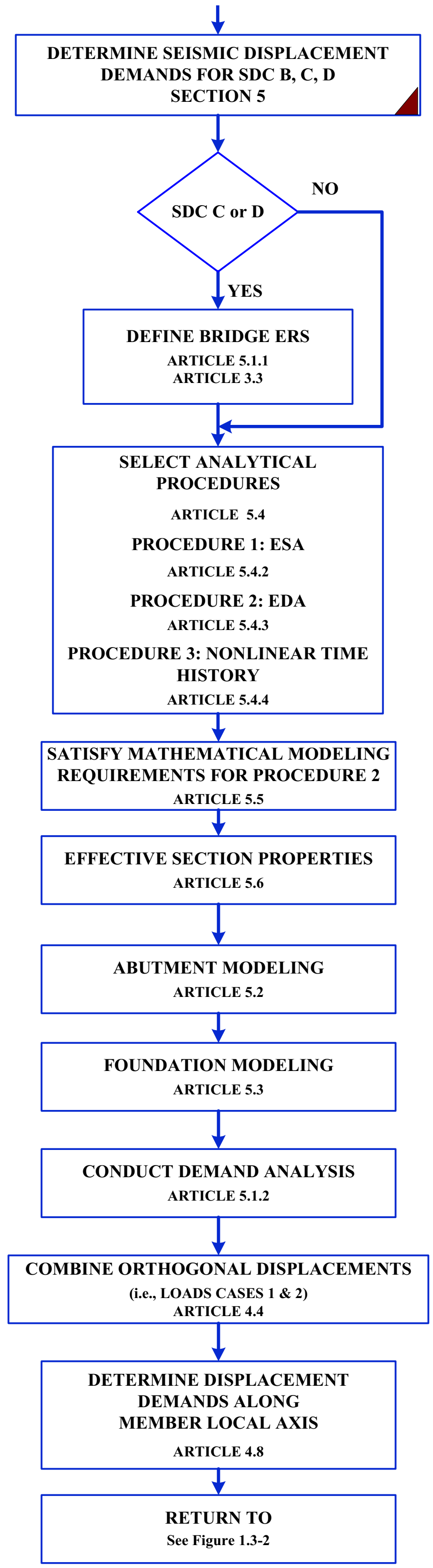


Figure 5.105 Modeling Procedure Flowchart 4

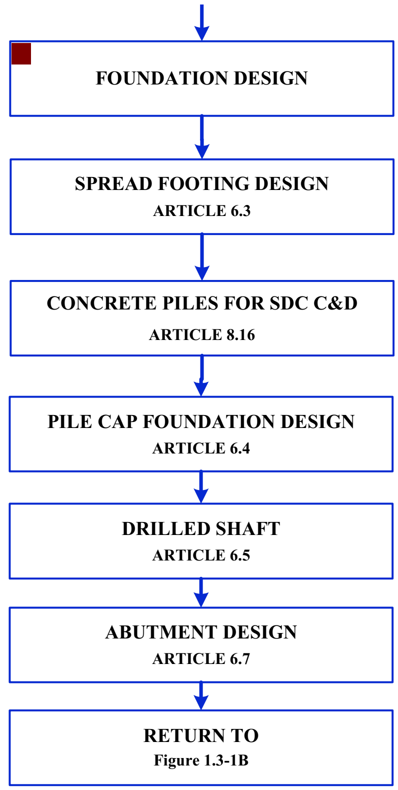


Figure 5.106 Foundation Design Flowchart 6

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.38.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}.

Table 5.38 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.107 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that S_{D1} =0.14, the bridge is designed as per SDC B with the following basic requirements:

- No Identification of ERS according to Article 3.3
- Demand Analysis
- Implicit Capacity Check Required (displacement, P-Δ support length)
- No Capacity Design Required except for column shear requirement
- SDC B Level of Detailing

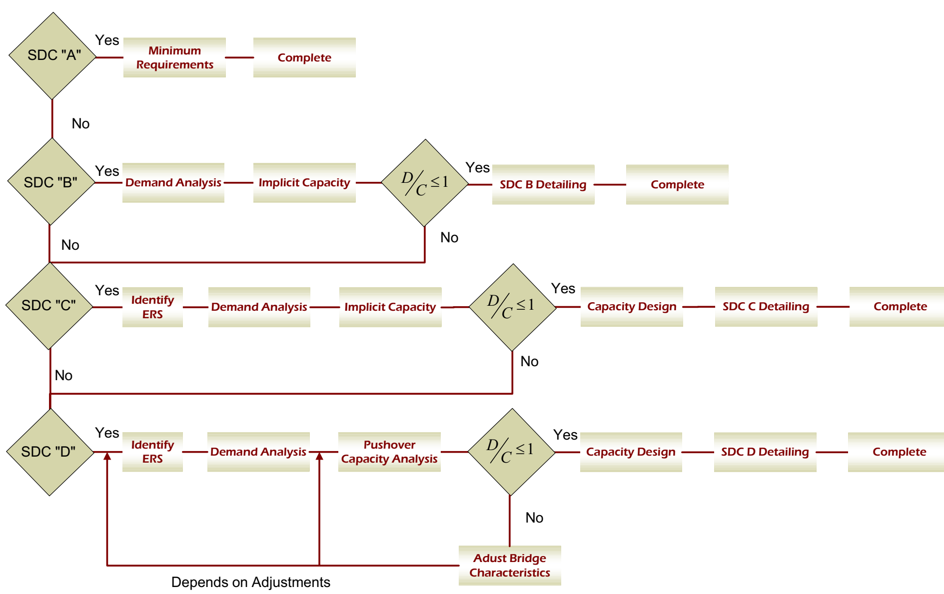


Figure 5.107 Seismic Design Category (SDC) Core Flowchart

For SDC B, identification of an ERS is recommended to be considered. The articulation of example 8 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path. The seismic behavior of segmental precast columns in the inelastic range is not the focus of this example. Therefore, these elements are treated as elastic elements as the AASHTO-SGS permissible Earthquake-Resistant Elements (EREs) do not cover segmental precast columns.

Selection of Analysis Procedure

Minimum requirements for the selection of an analysis method to determine seismic demands for a particular bridge type shall be taken as specified in Tables 5.39 and 5.40. Applicability shall be determined by the "regularity" of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges shall be taken as those having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and which satisfy the requirements in Table 5.41. Any bridge not satisfying the requirements of Table 5.40 shall be considered “not regular”.

Table 5.39 Analysis Procedures.

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or more Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

Table 5.40 Description of Analysis Procedures.

Procedure Number	Description	Article
1	Equivalent Static	5.4.2
2	Elastic Dynamic Analysis	5.4.3
3	Nonlinear Time History	5.4.4

Procedure 3 is generally not required unless:

- P-Δ effects are too large to be neglected,
- damping provided by a base isolation system is large,
- requested by the owner per Article 4.2.2

Table 5.41 Regular Bridge Requirements.

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30º	30º	30º	30º	30º
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

Note: All ratios expressed in terms of the smaller value.

According to the AASHTO-SGS 5.3.1, the Foundation Modeling Methods (FMM) defined in Table 5.42 should be used as appropriate. The requirements for estimating foundation springs for spread footings, pile foundations, and the depth to fixity for drilled shafts shall be as specified in AASHTO-SGS Articles 5.3.2, 5.3.3 and 5.3.4, respectively. For a foundation which is considered as rigid, the mass of the foundation should be ignored in the analytical model. The Engineer shall assess the merits of including the foundation mass in the analytical model where appropriate, taking into account the recommendations in this Article.

The required FMM depends on the SDC:

- FMM I is permitted for SDCs B and C provided the foundation is located in Site Class A, B, C, or D. Otherwise FMM II is required.
- FMM II is required for SDC D.

For sites identified as susceptible to liquefaction or lateral spread, the ERS global model shall consider the non-liquefied and liquefied conditions using the procedures specified in AASHTO-SGS Article 6.8.

Table 5.42 Definition of Foundation Modeling Method (FMM).

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Rigid for Site Classes A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

Considering that the subject bridge is in SDC B, FMM I is permitted. Furthermore, the use of 24 in. diameter concrete filled pipe pile is considered relatively the best practice for foundation type in challenging cases of soft soil sites subjected to high ground motion. Therefore, the adoption of FMM I is deemed appropriate.

Seismic Analysis:

Calculate Girder Cross Section Area (See Figure VII.B.4, reference sheet VII.B.33)

$$A_1 = \left(\frac{47+11}{2}\right)\times 5.5 = 159.5$$

$$A_2 = \left(\frac{11+7}{2}\right)\times 2 = 18.0$$

$$A_3 = 7\times 48 = 336$$

$$A_4 = \left(\frac{7+14}{2}\right)\times 3.5 = 36.75$$

$$A_5 = \left(\frac{14+32}{2}\right)\times 12 = \frac{276.0}{826.3 \text{ in}^2/\text{ft}}$$

$$\text{or } 5.75 \text{ ft}^2/\text{ft}$$

Deck Cross Section:

$$\left(\frac{9}{12}\times 2\times 47.25\right) = 70.9 \text{ ft}^2/\text{ft}$$

Total Girders Weight Per ft Length (includes 12 girders, see Figure VII.B.1)

$$12\times 5.75\times 0.15 = 10.4 \text{ K/ft}$$

Deck Weight per ft Length

$$70.9\times 0.15 \text{ Kips/ft}^3 = 10.7 \text{ K/ft}$$

Concrete in Sidewalk: (See reference sheet VII.B.44)

ESTIMATE OF QUANTITIES					
PAY ITEM NO.	STANDARD ITEM NO.	DESCRIPTION	UNIT	CONTRACT QUANTITY	
				EASTBOUND	WESTBOUND
329	5A31E	REINFORCEMENT STEEL IN STRUCTURES, EPOXY-COATED	LBS	298654	279139
332	5A21L	SAWCUT GROOVED DECK SURFACE	SF	26300	26300
334	N5A01	CONCRETE IN SUPERSTRUCTURE, DECK SLABS, HPC	CY	1265	1255
335	N5A02	CONCRETE IN SUPERSTRUCTURE, SIDEWALKS, HPC	CY	82	108
336	N5A03	CONCRETE IN SUPERSTRUCTURES, PARAPET, 2'-8" HIGH, HPC	LF	-	724
337	N5A04	CONCRETE IN SUPERSTRUCTURES, PARAPET, 3'-5" HIGH, HPC	LF	724	-
367	N6E01	15" X 32" CONCRETE BARRIER CURB, BRIDGE, HPC	LF	724	724
370	7A26C	1.5" RIGID METALLIC CONDUIT, TYPE CUG	LF	80	70
371	7A31C	2" RIGID METALLIC CONDUIT, TYPE CUG	LF	590	240
372	7A41C	3" RIGID METALLIC CONDUIT, TYPE CUG	LF	724	724
373	7A75J	JUNCTION BOX FRAMES AND COVERS	U	16	14
374	N7A01	1 1/4" RIGID METALLIC CONDUIT, TYPE CUG	LF	62	62
375	N7A02	4 - 1 1/4" RIGID METALLIC CONDUIT, TYPE CUG	LF	724	-
377	6C73H	NEW MANHOLE CASTINGS, SQUARE FRAME, CIRCULAR COVER	U	-	1

Eastbound 82 Cubic Yard

Westbound 108 Cubic Yard

Total $\quad 190 \text{ CY}\times 27 \text{ ft}^3/\text{CY} = 5130 \text{ ft}^3$

Total Bridge Length: 719 ft

Concrete sidewalk per foot of bridge length:

$$5130 \text{ ft}^3 / 719 \text{ ft} = 7.13 \text{ ft}^2$$

Unit weight of concrete sidewalk:

$$0.15 \text{ Kips/ft}^3 \times 7.13 \text{ ft}^2 = 1.1 \text{ K/ft}$$

Concrete in Barrier (See Figures VII.B.6 and VII.B.7, reference sheet VII.B.46)

2'- 8" High Parapet Area:

$$\left(\frac{1.33+1}{2}\right) \times 2.66 = 3.1 \text{ ft}^2\text{/ft}$$

3'- 5" High Parapet Area:

$$\left(\frac{1.33+1}{2}\right) \times 2.66 + 0.75 \times 1.33 = 4.1 \text{ ft}^2\text{/ft}$$

Typical Median Barrier Section Area (See Figure VII.B.8, reference sheet VII.B.47)

$$\frac{1}{144}\left[\left(\frac{8+10}{2}\right) \times 19 + \left(\frac{10+17}{2}\right) \times 10 + 10 \times 3\right]$$

$$\frac{1}{144}(171 + 135 + 30) = 2.33 \text{ ft}^2\text{/ft}$$

Westbound Barrier Total:

(3.1+2.33)×0.15 = 0.82 K/ft

Eastbound Barrier Total:

(4.1+2.33)×0.15 = 0.97 K/ft

Consider 1 K/ft each Westbound and Eastbound; total 2 K/ft

Consider 10% for Fillets and Intermediate Diaphragms

0.1×21 K/ft = 2.1 K/ft

Consider 25psf Added Dead Load (reference sheet B4, note shown below)

25×(2×47.25 ft)/1000 = 2.4 K/ft

SUPERSTRUCTURE:
(A) DEAD LOAD INCLUDES A 25 PSF PROVISION FOR A FUTURE 2" THICK CONCRETE OVERLAY PROTECTIVE SYSTEM ON THE BRIDGE DECK.

Consider End Diaphragm at Pier 3 and Abutments (See Figure VII.B.11, reference sheet VII.B.38)

Weight =

1'×(2×47.25)×4×0.15 Kips/ft^3 = 57 Kips

Diaphragms at Piers 1 and 4 (See Figure VII.B.12, reference sheet B39):

$2.5 \times (2 \times 47.25) \times 4 \times 0.15 \text{ Kips/ft}^3 = 142 \text{ Kips}$

Diaphragm at Piers 2 and 5 (See Figure VII.B.13, reference sheet B40):

Similar to Piers 1 and 4

$2.5 \times (2 \times 47.25) \times 4 \times 0.15 \text{ Kips/ft}^3 = 142 \text{ Kips}$

Summary of Dead Load Items:

Girder	10.4 K/ft
Deck	10.7 K/ft
Concrete Sidewalk	1.1 K/ft
Barrier	2.0 K/ft
Fillets and Intermediate Diaphragms	2.1 K/ft
Added Dead Load	2.4 K/ft
Total	28.7 K/ft

Consider 28.7 K/ft Dead Load on Simply Supported Spans.

Dead Load Distribution on Abutments and Piers:

West Abutment: $119/2 \times 28.6 \text{ K/ft} + 57 = 1760 \text{ Kips}$

Pier 1: $\left(\dfrac{119 + 120.25}{2}\right) \times 28.6 + 142 = 3564 \text{ Kips}$

Pier 2: $\left(\dfrac{120.25 + 120.25}{2}\right) \times 28.6 + 142 = 3581 \text{ Kips}$

Pier 3: $\left(\dfrac{120.25 + 120.25}{2}\right) \times 28.6 + 2 \times 57 = 3554 \text{ Kips}$

Pier 4: $\left(\dfrac{120.25 + 120.25}{2}\right) \times 28.6 + 142 = 3581 \text{ Kips}$

Pier 5: $\left(\dfrac{119 + 120.25}{2}\right) \times 28.6 + 142 = 3564 \text{ Kips}$

Calculate Dead Load per Bearing:

Total Bearings per Pier: 12 Girders x 2 Sides = 24 Bearings

Total Bearings per Abutment: 12 Girder x 1 Side = 12 Bearings

W. Abutment Bearing DL: 1760 Kips/12 = 147 Kips

Pier 1 Bearing DL: 3564/24 = 149 Kips

Pier 2 Bearing DL: 3581/24 = 149 Kips

Pier 3 Bearing DL: 3554/24 = 148 Kips

Bent Cap Weight (See Figures VII.C.14, VII.C.15, and VII.C.16, reference sheets VII.B.28 and VII.B.29):

X_Section AA Area

Flanges: $\left(\frac{5.5\times 2}{12}\right)\times 5 = 4.6\ \text{ft}^2$

Fillets: $2\times\left(\frac{3\times 3}{144}\right) = 0.13\ \text{ft}^2$

Webs: $\left(7-\frac{11}{12}\right)\times\left(\frac{5.5\times 2}{12}\right) = 5.58\ \text{ft}^2$

Total: $4.6+0.13+5.58=10.3\ \text{ft}^2$

X_Section B_B Area: $5\times 7 = 35\ \text{ft}^2$

X_Section B_B Length along Cap = (9’-11”)+(7’-0”) = 16’-11”

For Pier 3 EB Only = (14’-4”)+(7’-0”) = 21’-5”

X_Section A_A Length along Cap = (45’-11”)-(16’-11”) = 29’

Pier 3 EB Only = (50’-5”)-(21’-5”) = 29’

Typical Pier Cap Weight = [(16’-11”)×35+29×10.3]×2×.15 = 268 Kips

Pier EB Cap Weight = [(21’-5”)×35+29×10.3]×2×.15 = 312 Kips

Calculate Column Weight: (See Figure VII.C.4):

Pier 3 Column Height 17’- 2”

X_Section Area Top of Column: 1584 in^2

X_Section Area Bottom of Column: 2627 in^2

Sloping Column Weight:

$$17.2\times\left(\frac{1584+2627}{2}\right)\frac{1}{144}\times 0.15\ \text{lb/ft}^3 = 38\ \text{Kips}$$

Vertical Column Weight:

$$17.2\times\left(\frac{1946}{144}\right)\times 0.15 \text{ lb/ft}^3 = 35 \text{ Kips}$$

Calculate Approximate Dead Load in each of the four columns:

$$\frac{(3554+268)}{4} = 955 \text{ Kips}$$

Consider D.L of 950 Kips Top of Column

1000 Kips Bottom of Column

The dead load in columns is used to generate Moment_Curvature of Columns X_sections at different column elevation using CSI-SAP software.

Calculate Transverse Direction Period

Transverse direction lateral 1: Applied load 1000 Kips

Top of column displacement 0.75in (See Appendix VII.A, Figure VII.A.50)

Transverse direction lateral 2: Applied lateral load 1000 Kips

Top of column displacement 0.76 in. (See Appendix VII.A, Figure VII.A.55)

Total Bent Stiffness $K_T = \frac{1000}{0.75} + \frac{1000}{0.76} = 2649$ K/in or 31790 K/ft

Bent Tributary mass is taken @ 4000 Kips

$$\omega^2 = \frac{K}{M} = \frac{31790}{4000}\times 32.2 = 256$$

$$\omega = 16 \text{ rad/sec}$$

$$T = \frac{2\pi}{\omega} = \frac{2\pi}{16} = 0.40 \text{ sec}$$

Spectral Acceleration is equal to 0.23g from Figure 5.100.

Calculate NJ Factored Design Spectrum

S_a = 1.5×0.23g = 0.35g

Calculate Longitudinal Direction Period:

By imposing a rigid body constraint on outer and center columns, the pier 3 longitudinal displacement is calculated as the average displacement of outer and center columns. The pier 3 longitudinal stiffness may be calculated as:

$$K_{long} = \frac{2000}{(13.98 + 13.1)/2} = 296.3 \text{ K/in or 3556 K/ft}$$

Bent total tributary mass is considered @ 4000 Kips:

$$\omega^2 = \frac{K}{M} = \frac{3556}{4000} \times 32.2 \ ft/\sec^2 = 28.6$$

$$\omega = 5.35 \text{ rad/sec}$$

Longitudinal Period:

$$T = \frac{2\pi}{5.35} = 1.17 \text{ sec}$$

Spectral Acceleration is equal to 0.079g from Figure 5.100.

Calculate N.J. Factored Design Spectrum Acceleration:

S_a = 1.5×0.079 = 0.12g

Calculate Displacement Magnification for short period structures according to AASHTO-SGS 4.3.3

$$R_d = \left(1 - \frac{1}{\mu_D}\right)\frac{T^*}{T} + \frac{1}{\mu_D} \geq 1.0$$

$T^* = 1.25T_s$ (See Figure 5.100 for T_s)

$T^* = 1.25 \times 0.31 = 0.39$

$\mu_D = 2$ for SDC B

Since longitudinal and transverse periods are calculated greater than 0.39 sec, the short period Displacement Magnification does not apply.

Transverse Direction Earthquake Demand on Pier 3: S_a = 0.35g

Total Force Demand on Pier 3: 0.35g×4000 kips = 1400 kips

Force Demand for Left and Right Bents considering that lateral 1 and lateral 2 push analysis led approximately equal displacement of 0.75 in. and 0.76 in. (See Appendix VII.A, Figures VII.A.46 through VII.A.58)

1400 Kips/2 = 700 Kips

Displacement Demand:

$$S_d = \frac{1400\ \text{K}}{2649\ \text{K/in}} = 0.53\ \text{in}$$

Figures VII.A.49 through VII.A.58 show Transverse Reactions, Members Shears and Members Moments corresponding to an applied lateral load of 1000 Kips for Left and Right bents of Pier 3. Therefore, apply a 0.7 factor (since the force demand for left and right bents calculated above is 700 Kips) to results shown in these figures to get demands corresponding to Site Seismicity described earlier for the subject bridge. Table 5.43 and 5.44 show the flexural and shear demands for transverse loading lateral 1 and 2. Table 5.44 also shows the shear demand for an applied load of 700 Kips obtained from 0.7×1000 Kips. The flexural capacity of different column cross-sections is obtained in Appendix VII.A and the summarized results are shown in Table 5.45. Tables 5.46 and 5.47 show the flexural demand and capacity for Top and Bottom of outer and center columns under transverse loading lateral 1 and 2.

Table 5.43 Flexural Demands for Transverse Loading of 1,000 Kips.

Flexural Demand (K-in)	Left Column	Center Column	Center Column	Right Column
1000K Applied	Lateral 1	Lateral 1	Lateral 2	Lateral 2
Top of Column	59800	57470	58745	58975
Bottom of Column	98760	57470	58745	97224

Table 5.44 Shear Demand for Transverse Loading of 1000 Kips and 700 Kips

Shear Demand: (Kips)	Loading	Applied Load (1000Kips)	Applied Load (700 Kips)
Left Column	Lateral 1	603	422
Center Column	Lateral 1	433	303
Center Column	Lateral 2	443	310
Right Column	Lateral 2	594	416 Kips

Table 5.45 Nominal Flexural Capacity of Column Sections.

Flexural Capacity	Weak Axis	Strong Axis 1	Strong Axis 2
CS1-PS	86068	108993	116008
CS1-PS-HS	96471	119132	132022
CS 3A-PS	85216	103640	112819
CS 3A-PS-HS	96081	113642	121370
CS 4B-PS	91436	157523	172315
CS 4B-PS-HS	103163	174783	189088
CS 13B-PS	95600	221000	243038
CS 13B-PS-HS	107980	248263	269733

Table 5.46 Flexural Demand Capacity for Top of Columns

Flexural Demand and Capacity	Left Column Lateral 1	Center Column Lateral 1	Center Column Lateral 2	Right Column Lateral 2
Demand	41860	40229	41122	41283
Capacity	103640	108993	108993	103640
D/C Ratio	0.4	0.37	0.38	0.4

Table 5.47 Flexural Demand Capacity for Bottom of Columns

Flexural Demand and Capacity	Left Column Lateral 1	Center Column Lateral 1	Center Column Lateral 2	Right Column Lateral 2
Demand	69132	40229	41122	68057
Capacity	248263	119132	119132	248263
D/C Ratio	0.28	0.34	0.35	0.28

Longitudinal Direction Earthquake Demand on Pier 3: S_a = 0.12g

Total Force Demand on Pier = 0.12 × 4000 = 480 Kips

Displacement Demand on Pier 3 = $\frac{480}{296.3 \text{ K/in}}$ Kips $= 1.6$ in

Force Demand Per Column = $\frac{480}{4}$=120 Kips

Height between C.G. of Superstructure and top of footing =

$$24.3 + \left(\frac{5' - 11'' + 9''}{2}\right) = 27.6 \text{ ft}$$

Moment Demand at base of column = 120 Kips × 27.6 ft = 3312 Kips-ft = 39744 Kips.in

Moment Capacity at base of Center Column:

Section CS1-PS-HS: 96471 K-in

D/C Ratio of center column = $\frac{39744}{96471} = 0.41$ O.K.

Moment Capacity at the base of Outer Column:

Section CS13B-PS-HS: 107980 K-in

D/C Ration of Outer Column = $\frac{39744}{107980} = 0.37$ O.K.

Column Shear Demand and Capacity

According to AASHTO-SGS 8.6.1, the shear demand for a column, V_u, in SDC B shall be determined based on the lesser of:

- The force obtained from a linear elastic seismic analysis
- The force,V_{po}, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_u, in SDC C or D shall be determined based on the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.

Given the uncertainty in the hazard and the consequence of column shear failure, it is deemed important to attempt to satisfy the capacity protection requirement for column shear.

The column shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated based on the nominal material strength properties and shall satisfy:

in which: $\phi_s V_n \geq V_u$

where: $V_n = V_c + V_s$

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

The equations above are not applicable for the Precast Post-Tensioned column of the subject bridge.

Shear demand in longitudinal direction (Elastic Model): 120 Kips

Maximum Shear Demand in Transverse direction (Elastic Model): 422 Kips (See Table 5.44)

Following AASHTO-SGS 5.8.4.1 the nominal resistance of the shear interface plane is taken as:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} f_y + P_c\right)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of :

$V_{ni} \leq K_1 f_c' A_{cv}$ or

$V_{ni} \leq K_2 A_{cv}$

in which:

$A_{cv} = b_{vi} L_{vi}$

Where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in^2)

A_{cf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in^2)

b_{vi} = interface width considered to be engaged in shear transfer (in.)

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in AASHTO-SGS Article 5.8.4.3 (ksi)

μ = friction factor specified in AASHTO-SGS Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

P_c = Permanent net compressive force normal to the shear plane; if force is tensile, $P_c = 0.0$ (kip)

f_c' = Specified 28 day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear, as specified in AASHTO-SGS Article 5.8.4.3.

K_2 = limiting interface shear resistance specified in AASHTO-SGS Article 5.8.4.3. (ksi)

The interface shear strength equations are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

According to AASHTO-SGS 5.8.4.3., most conservative values considered for a clean concrete interface surface, free of laitance, but not intentionally roughened.

$c = 0.075$ Ksi

$\mu = 0.6$

$K_1 = 0.2$

$K_2 = 0.8$ Ksi

The value of cohesion "c" is typically taken as zero for extreme load event where structural member is subjected to post-elastic demands. For the subject bridge, the column behaves elastically and considering "c" equal to zero is deemed very conservative.

Relying on externally applied post-tensioning and the magnitude of dead load, the value P_c can be calculated as: Dead Load: 950 Kips

Post-tensioning for exterior column is based on jacking force in Table5.48 below and considering 10% of losses: $0.9\left(2\times289+2\times496\right)=1413$ Kips

$$V_{ni}=0.6\left(950+1413\right)=1418 \text{ Kips}$$

Table 5.48 Column Post-tensioning Schedule

PIER	EXTERIOR/INTERIOR COLUMN	TENDON SIZE	NO. TENDONS	TENDON LENGTH (FT.)	TENDON WEIGHT (LBS.)	TOTAL WEIGHT PER PIER (LBS.)	JACKING FORCE (KIPS)	ELONGATION (INCH)
1 WB & EB	EXTERIOR, T1	14 X 0.5" Dia.	2	25.434	377	1616	289	1.32
	EXTERIOR, T2	16 X 0.5" Dia.	2	28.937	491		496	2.27
	INTERIOR	14 X 0.5" Dia.	4	25.208	748		434	1.95
2 WB & EB	EXTERIOR, T1	14 X 0.5" Dia.	2	28.245	419	1790	289	1.47
	EXTERIOR, T2	16 X 0.5" Dia.	2	32.068	544		496	2.54
	INTERIOR	14 X 0.5" Dia.	4	27.875	827		434	2.18
3 WB & EB	EXTERIOR, T1	14 X 0.5" Dia.	2	29.651	440	1877	289	1.55
	EXTERIOR, T2	16 X 0.5" Dia.	2	33.634	570		496	2.67
	INTERIOR	14 X 0.5" Dia.	4	29.208	867		434	2.29
4 WB & EB	EXTERIOR, T1	14 X 0.5" Dia.	2	29.739	441	1883	289	1.56
	EXTERIOR, T2	16 X 0.5" Dia.	2	33.732	572		496	2.68
	INTERIOR	14 X 0.5" Dia.	4	29.292	869		434	2.30
5 WB & EB	EXTERIOR, T1	14 X 0.5" Dia.	2	28.421	422	1801	289	1.48
	EXTERIOR, T2	16 X 0.5" Dia.	2	32.264	547		496	2.55
	INTERIOR	14 X 0.5" Dia.	4	28.042	832		434	2.19

Equivalent Shear Stress Capacity Considering A_{cv} equal to 1152 in^2 (See Figure VII.A.12)

$$\frac{V_{ni}}{A_{cv}}=\frac{1418}{1152}=1.2 \text{ Ksi}>K_2=0.8 \text{ Ksi}$$

Therefore:

$$V_{ni}=0.8 \text{ Ksi}\times1152 \text{ in}^2=922 \text{ Kips}$$

$$\phi V_{ni}=0.9\times922=829 \text{ Kips}>422 \text{ Kips}$$

Considering 422 Kips as the maximum shear demand in the transverse direction

Shear D/C Ratio: $\frac{422}{829}=0.5$ O.K.

Check Minimum Support Length.

According to AASHTO-SGS Section 4.12.2, Seismic Design Categories A, B, and C support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.49 below.

$N=(8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = minimum support length measured normal to the centerline of bearing (in.)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; For hinges within a span, L shall be the sum of the distances to either side of the hinge; For single-span bridges, L equals the length of the bridge deck (ft.)

H = for abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.); For columns and/or piers, column, or pier height (ft.); For hinges within a span, average height of the adjacent two columns or piers (ft.); 0.0 for single-span bridges (ft.)

S = angle of skew of support measured from a line normal to span (°)

Table 5.49 Percentage N by SDC and Effective Peak Ground Acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percent N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC B:

$N=1.5(8+0.02L+0.08H)(1+0.000125S^2)$

L=359.5 ft

Calculated based on the total length of three continuous spams from West Abutment to Pier 3 or equally from East Abutment to Pier 3.

H=24.3 ft

Conservatively taken as the height of Pier 3 from top of footing to top of cap.

S=0

$N = 1.5(8+0.02\times359.5+0.08\times24.3)$

$= 1.5(8+7.2+2)$

= 25.8 in say 26 in

For Abutment Section A-A, see Figures VII.C.20, VII.B.5, VII.B.9, and VII.B.10, reference sheets VII.B.14, VII.B.50, and VII.B.33).

Abutment available support length = (1’-3”)+9” = 24 in < Required 26 in

(Note: AASHTO-SGS “N Equation is conservative”, refining heights of column would lead a slightly reduced N value)

For Available support length at Pier 3 (see Figures VII.C.15, and VII.B.11, reference sheets VII.B.38 and VII.B.29)

Cap Width not including 3/4 in. chamfers:

$5\times12-1.5 = 58.5$ in

Cap Available Support Length:

$$\frac{58.5}{2}-6''=23.25 \text{ in}$$

Available support length slightly less than required support length; however, considered adequate based on conservative N values.

Not Applicable Provisions:

AASHTO-SGS 8.6.5 Minimum Shear Reinforcement
Not Applicable

AASHTO-SGS 8.6.4 Maximum Shear Reinforcement
Not Applicable

AASHTO-SGS 8.7.1 Minimum Lateral Strength
Not Applicable

AASHTO-SGS 8.8.2 Minimum Longitudinal Reinforcement
Not Applicable

AASHTO-SGS 8.8.1 Maximum Longitudinal Reinforcement
Not Applicable

Example 9: Design of a Nine Span Concrete Bridge in SDC B Category

Bridge Description:

This example is based on a bridge carrying Route 35 over the Navesink River, Structure No. 1312-254. The bridge is a nine span with expansion joints at bents 3 and 6 in addition to the joints at South and North Abutments. The abutments are seat type. Figure 5.108 shows the General Plan and Elevation of the bridge. Figure 5.109 shows a typical section at various bents that include the superstructure and substructure. Appendix VIII.A contains pier analysis. Appendix VIII.B contains superstructure details. Appendix VIII.C contains substructure details.

Site Seismicity

The ground motion software tool packaged with the AASHTO-SGS was used to obtain the AASHTO-USGS Site Class D Unfactored Design Spectrum shown in Figure 5.110. A site class D is considered for this example bridge for illustration. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, S_s, and S_1: Determination of the parameters PGA, S_s, and S_1 by latitude-longitude or zip code from the USGS data.
- Design values of PGA, S_s, and S_1: Modification of PGA, S_s, and S_1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.
- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, S_s, and S_1. In addition to the numerical data, the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also includes the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

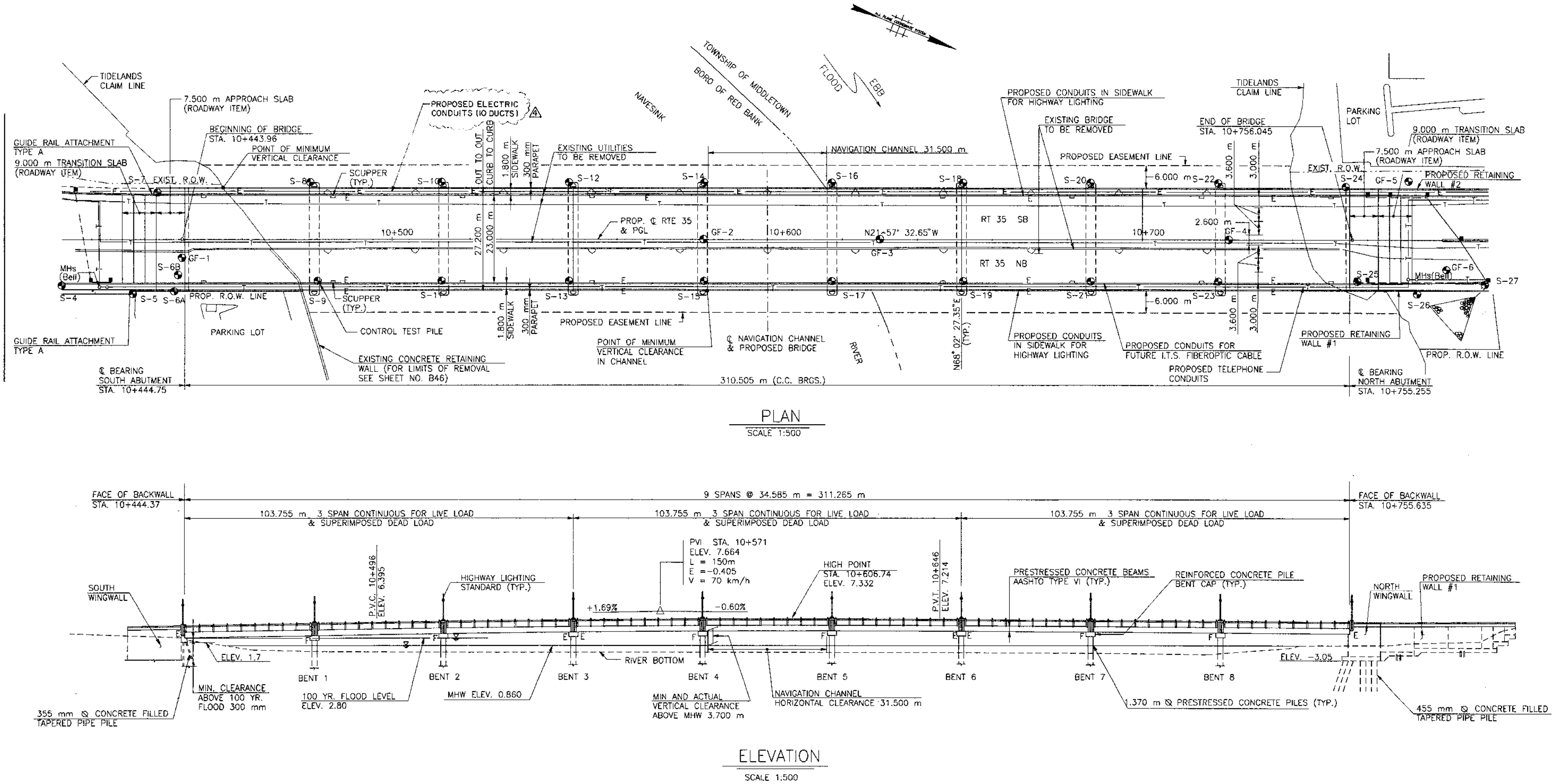


Figure 5.108 Route 35 Over the Navesink River Plan and Elevation

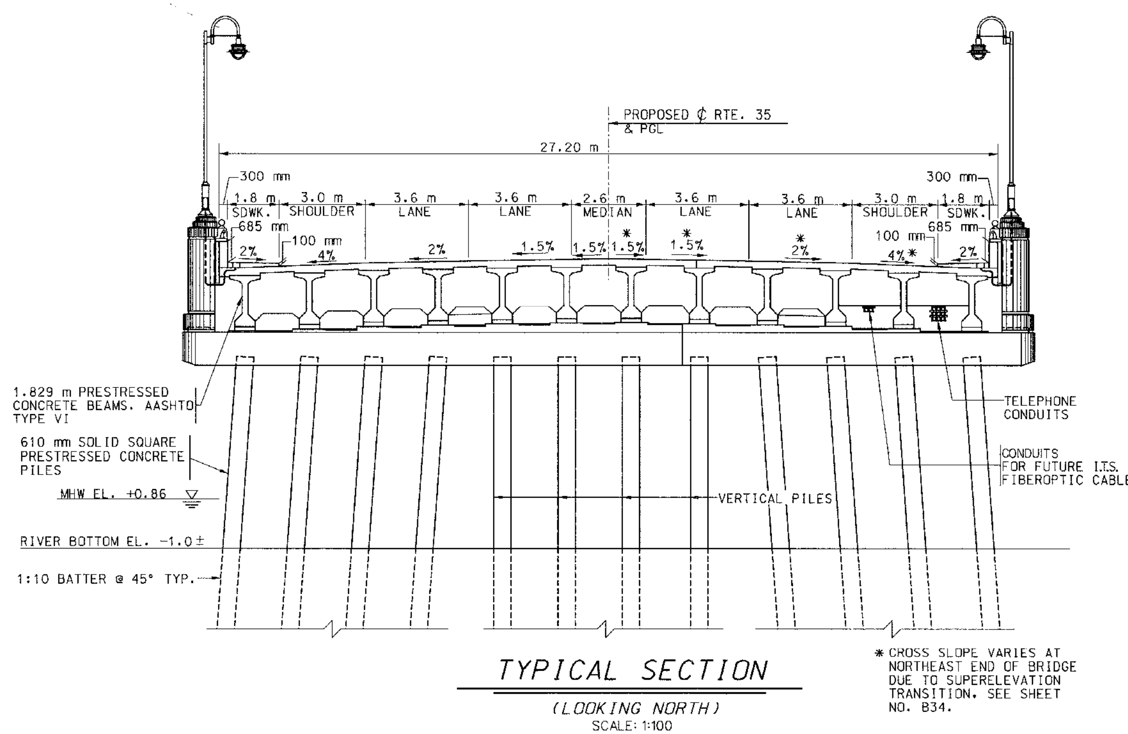
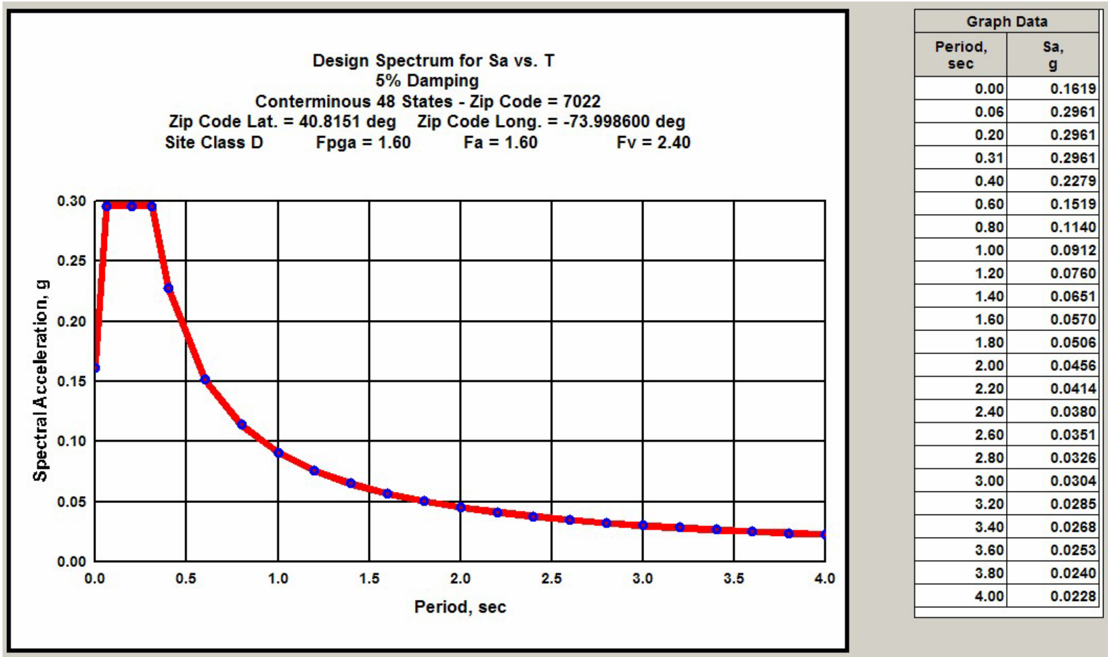


Figure 5.109 Typical Section



Period, sec	Sa, g
0.00	0.1619
0.06	0.2961
0.20	0.2961
0.31	0.2961
0.40	0.2279
0.60	0.1519
0.80	0.1140
1.00	0.0912
1.20	0.0760
1.40	0.0651
1.60	0.0570
1.80	0.0506
2.00	0.0456
2.20	0.0414
2.40	0.0380
2.60	0.0351
2.80	0.0326
3.00	0.0304
3.20	0.0285
3.40	0.0268
3.60	0.0253
3.80	0.0240
4.00	0.0228

Figure 5.110 AASHTO-USGS Site Class D Unfactored Design Spectrum

Flow charts

The Guide Specifications were developed to allow three Global Seismic Design Strategies based on the characteristics of the bridge system, which include:

- Type 1 - Design a ductile substructure with an essentially elastic superstructure.
- Type 2 - Design an essentially elastic sub-structure with a ductile superstructure.
- Type 3 - Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
- The articulation of Example 6 reflects a Type 1 bridge system with the substructure elements at the bent and abutment considered to be the critical locations to the seismic load path.
- Flowchart 1a of section 1.3 of the AASHTO-SGS shown in Figure 5.111 guides the designer on the applicability of the specifications and the breadth of the design procedure dealing with a multi-span bridge. Figure 5.112 shows the core flow chart of procedures outlined for bridges in SDC B, C, and D. Figure 5.113 outlines the demand analysis. Figure 5.114 directs the designer to determine displacement capacity. Figure 5.15 shows the modeling procedure. Figure 5.116 shows the foundation and abutment design applicable mainly for SDC C and D.

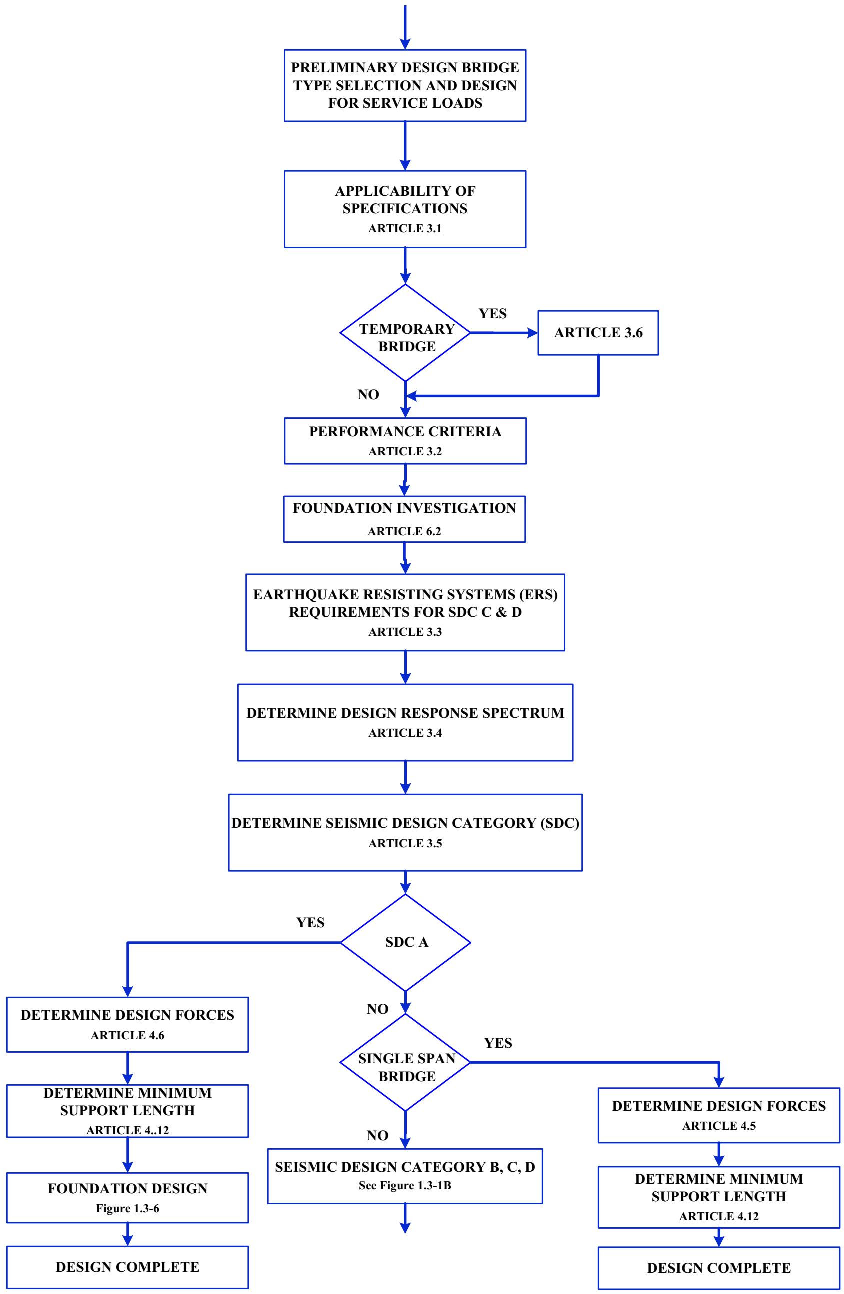


Figure 5.111 Seismic Design Procedure Flow Chart 1a

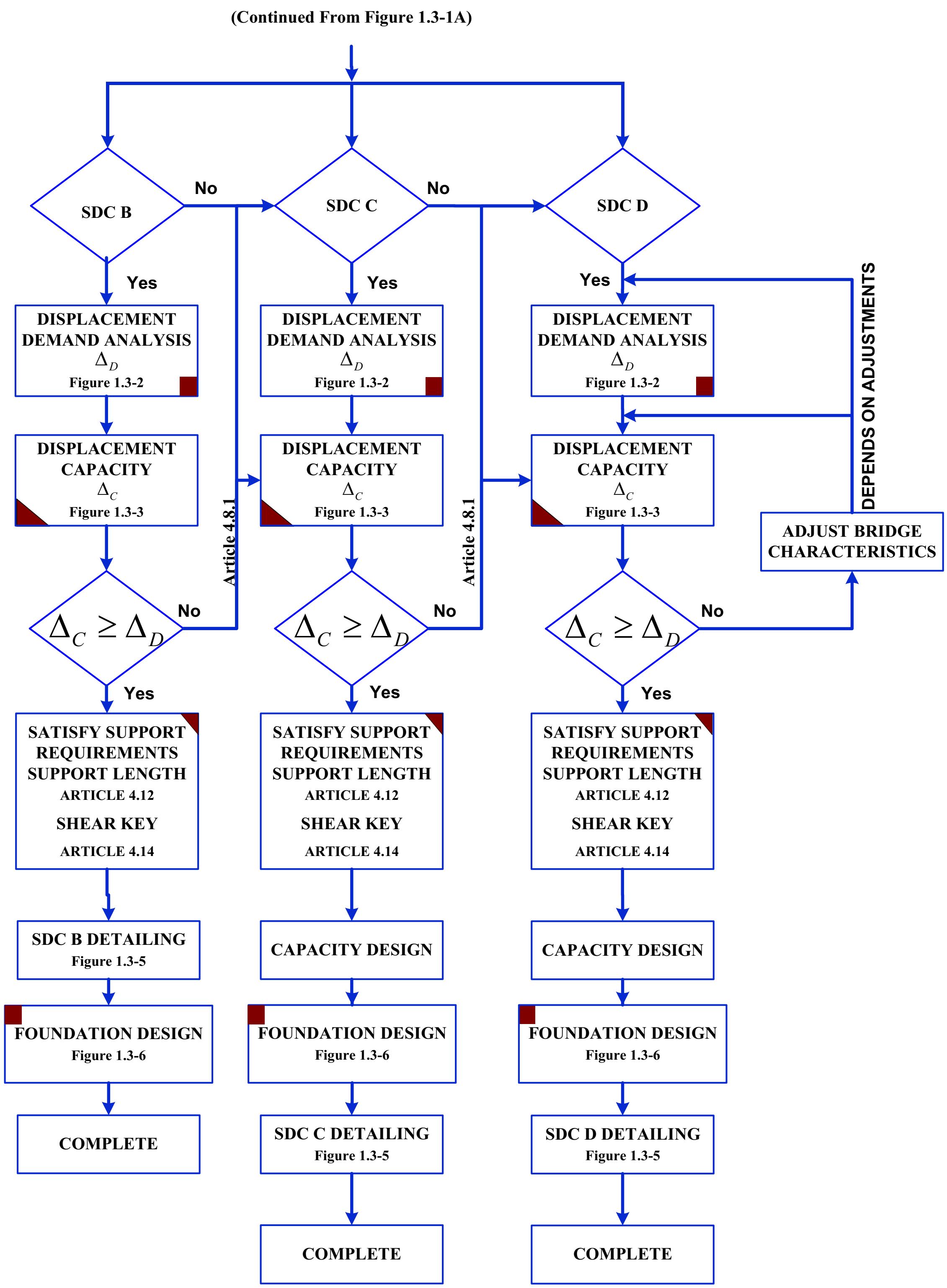


Figure 5.112 Seismic Design Procedure Flow Chart 1b

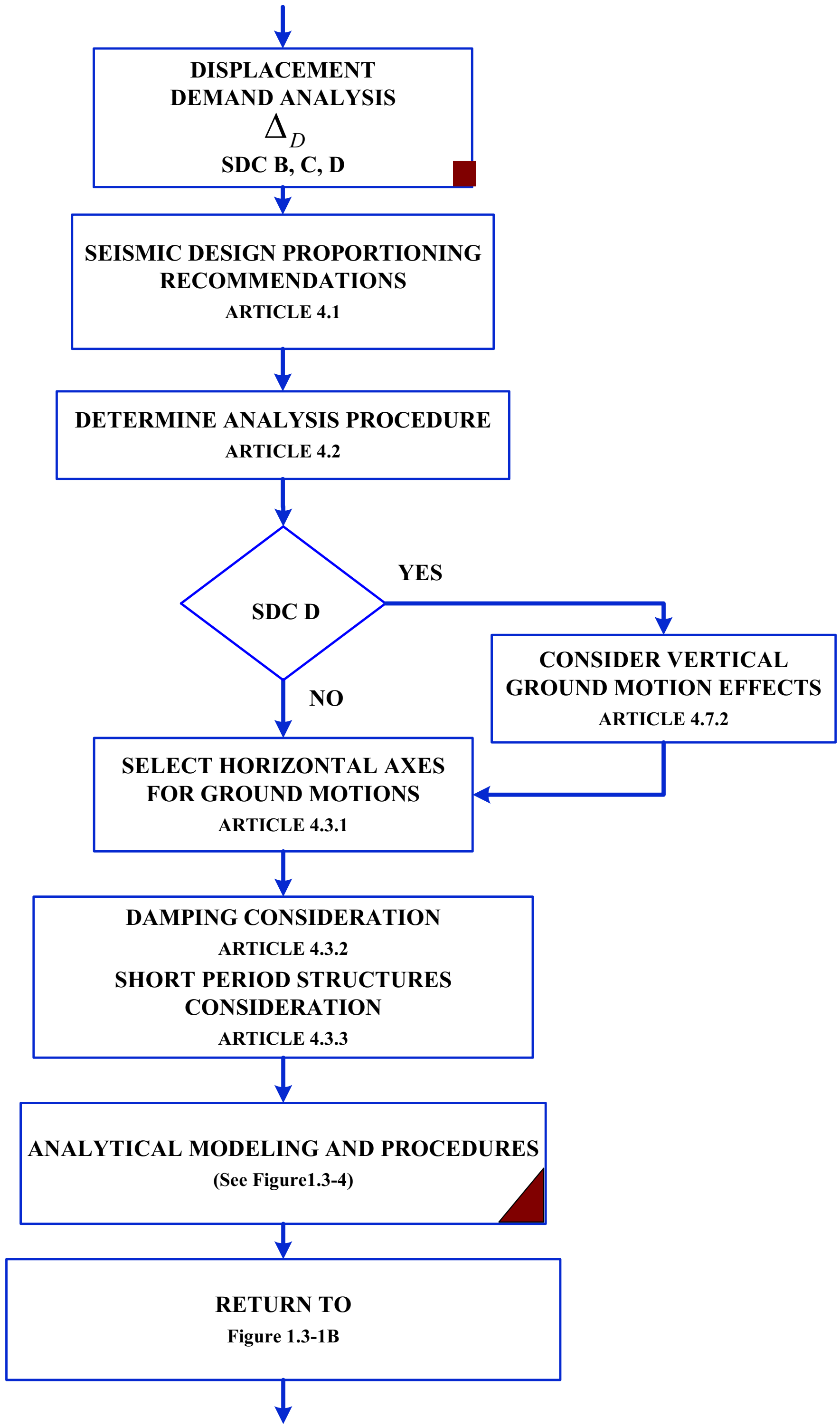


Figure 5.113 Demand Analysis Flow Chart 2

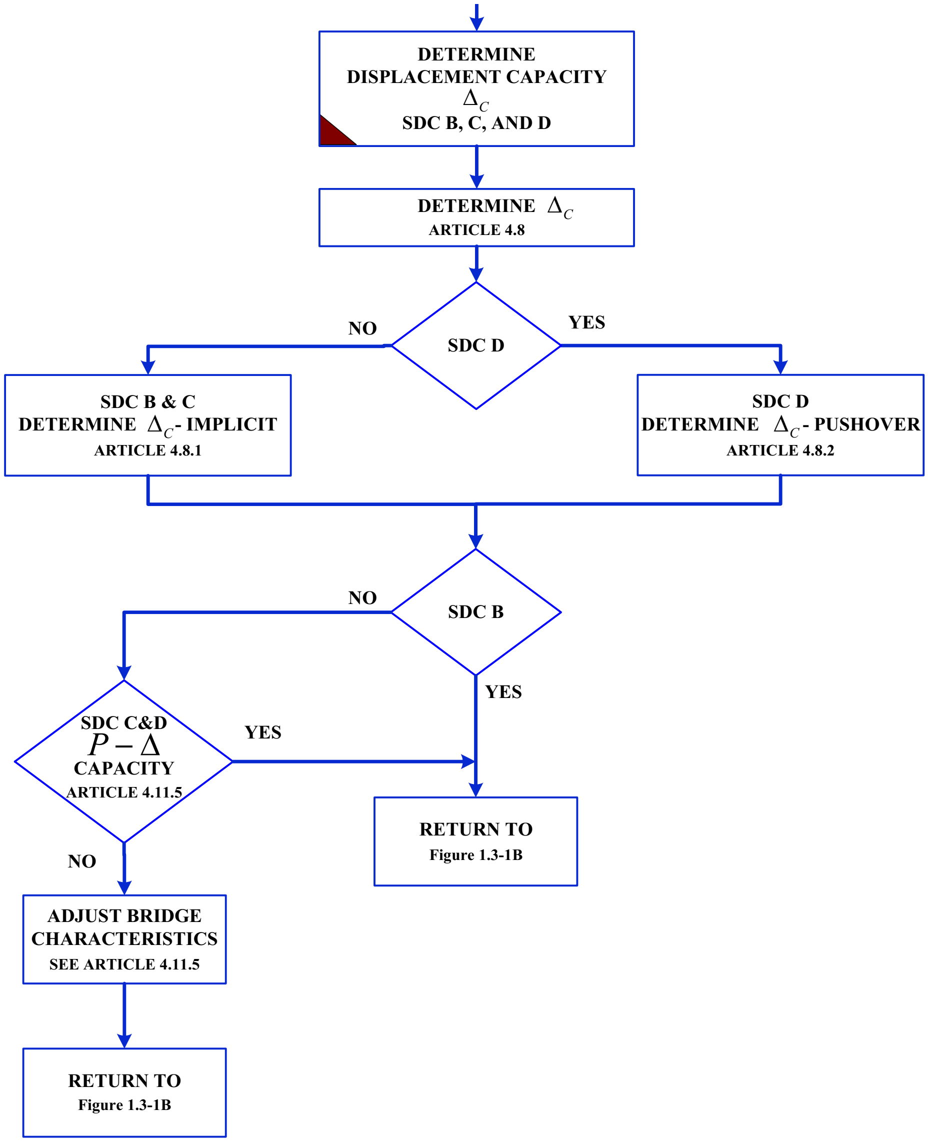


Figure 5.114 Displacement Capacity Flow Chart 3

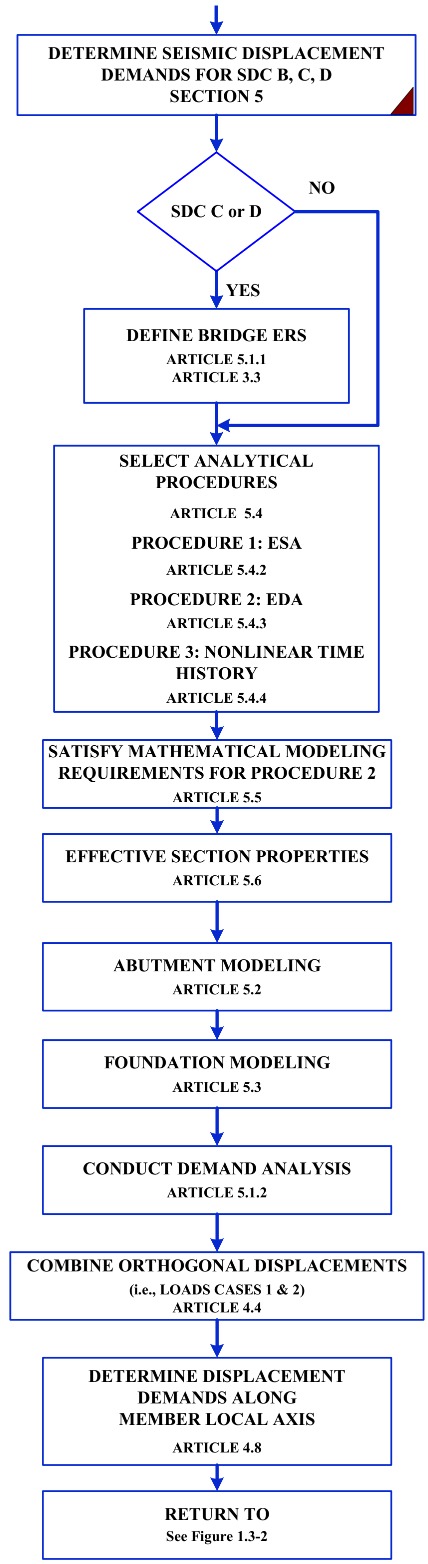


Figure 5.115 Modeling Procedure Flowchart 4

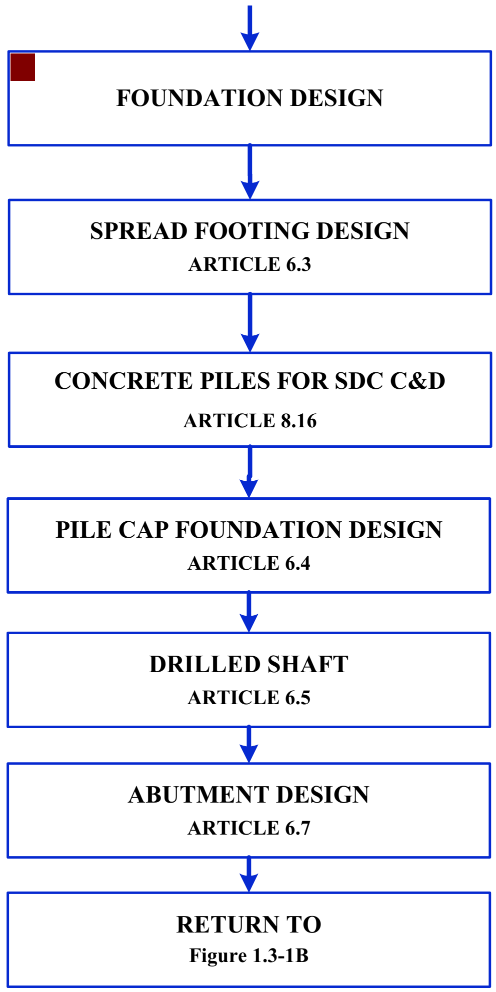


Figure 5.116 Foundation Design Flowchart 6

Selection of Seismic Design Category (SDC)

According to AASHTO-SGS Section 3.5, each bridge is assigned to one of four Seismic Design Categories (SDCs), A through D, based on the one second period design spectral acceleration for the design earthquake (S_{D1}) as shown in Table 5.50.

If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed in accordance with SDC D, regardless of the magnitude of S_{D1}

Table 5.50 Partitions for Seismic Design Categories A, B, C and D.

Value of $S_{D1} = F_vS_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

The requirements for each of the proposed SDCs shall be taken as shown in Figure 5.117 and described in Section 3.5 of the AASHTO-SGS. For both single-span bridges and bridges classified as SDC A, the connections shall be designed for specified forces in Article 4.5 and Article 4.6 respectively, and shall also meet minimum support length requirements of Article 4.12.

Given that $S_{D1} = 0.14$, the example bridge is treated in SDC B with the following basic requirements:

No Identification of ERS according to Article 3.3

- Demand Analysis
- Implicit Capacity Check Required (displacement, p-Δ support length)
- No Capacity Design Required except for column shear requirement
- SDC B Level of Detailing

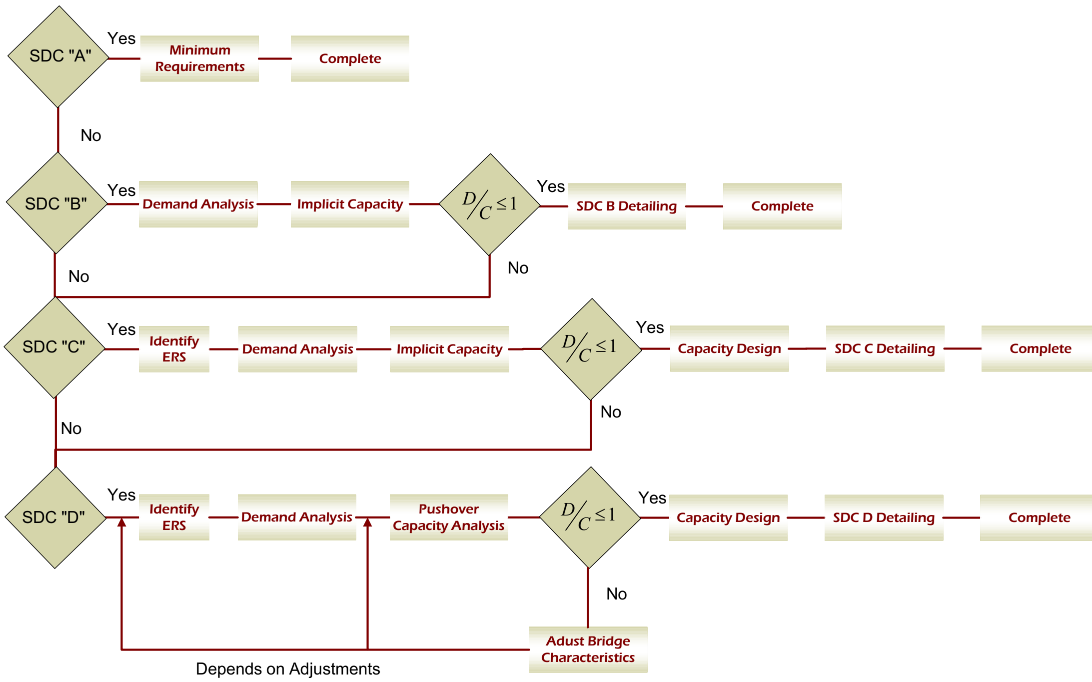


Figure 5.117 Seismic Design Category (SDC) Core Flowchart

Selection of Analysis Procedure

Minimum requirements for the selection of an analysis method to determine seismic demands for a particular bridge type shall be taken as specified in Tables 5.51 and 5.52. Applicability shall be determined by the "regularity" of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges shall be taken as those having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and which satisfy the requirements in Table 5.53. Any bridge not satisfying the requirements of Table 5.53 shall be considered "not regular".

Table 5.51 Analysis Procedures.

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or more Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

Table 5.52 Description of Analysis Procedures.

Procedure Number	Description	Article
1	Equivalent Static	5.4.2
2	Elastic Dynamic Analysis	5.4.3
3	Nonlinear Time History	5.4.4

Procedure 3 is generally not required unless:

- P-Δ effects are too large to be neglected,
- damping provided by a base isolation system is large,
- requested by the owner per Article 4.2.2

Table 5.53 Regular Bridge Requirements.

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle (curved bridge)	30º	30º	30º	30º	30º
Maximum span length ratio from span-to-span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span-to-span (excluding abutments)	-	4	4	3	2

Note: All ratios expressed in terms of the smaller value.

According to the AASHTO-SGS 5.3.1, the Foundation Modeling Methods (FMM) defined in Table 5.54 should be used as appropriate. The requirements for estimating foundation springs for spread footings, pile foundations, and the depth to fixity for drilled shafts shall be as specified in AASHTO-SGS Articles 5.3.2, 5.3.3 and 5.3.4, respectively. For a foundation which is considered as rigid, the mass of the foundation should be ignored in the analytical model. The Engineer shall assess the merits of including the foundation mass in the analytical model where appropriate taking into account the recommendations in this Article.

The required FMM depends on the SDC:

- FMM I is permitted for SDCs B and C provided the foundation is located in Site Class A, B, C, or D. Otherwise FMM II is required.
- FMM II is required for SDC D.

For sites identified as susceptible to liquefaction or lateral spread, the ERS global model shall consider the non-liquefied and liquefied conditions using the procedures specified in AASHTO-SGS Article 6.8.

Table 5.54 Definition of Foundation Modeling Method (FMM).

Foundation Type	Modeling Method I	Modeling Method II
Spread Footing	Rigid	Rigid for Site Classes A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

Considering that the subject bridge is in SDC B, FMM I is permitted. The estimated depth of fixity method is illustrated in Figure 5.118. Figures 5.119 and 5.120 show the depth to fixity in sand and clay consecutively with respect to the standard penetration index N (blows/ft). This method is deemed adequate given that the bridge is in SDC B with piers having pile extension foundation type. Based on the Profile at the site shown in Figure 5.121, the river bottom at -1.0m elevation is considered as ground elevation. The estimated depth to fixity is estimated below the (-1.0m) elevation.

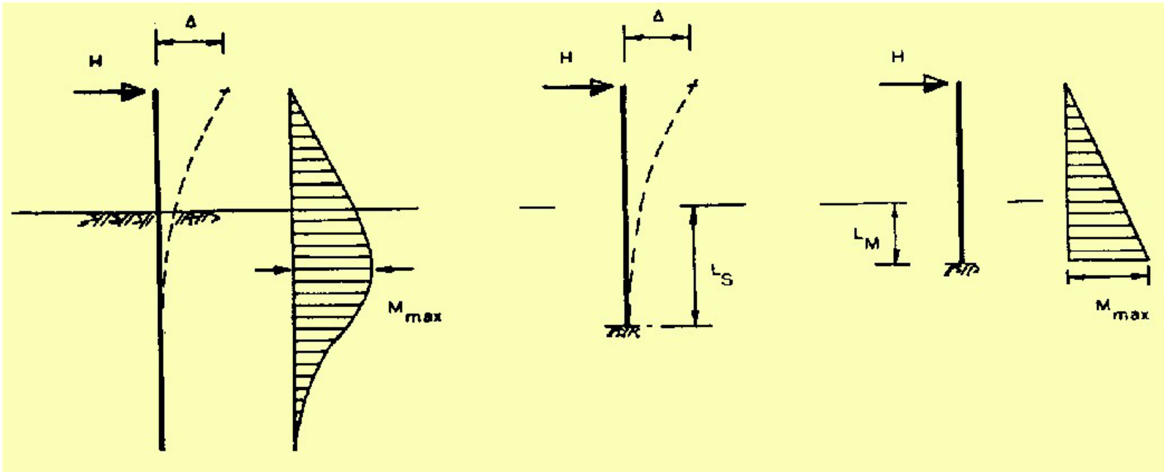


Figure 5.118 Estimated Depth to Fixity Model

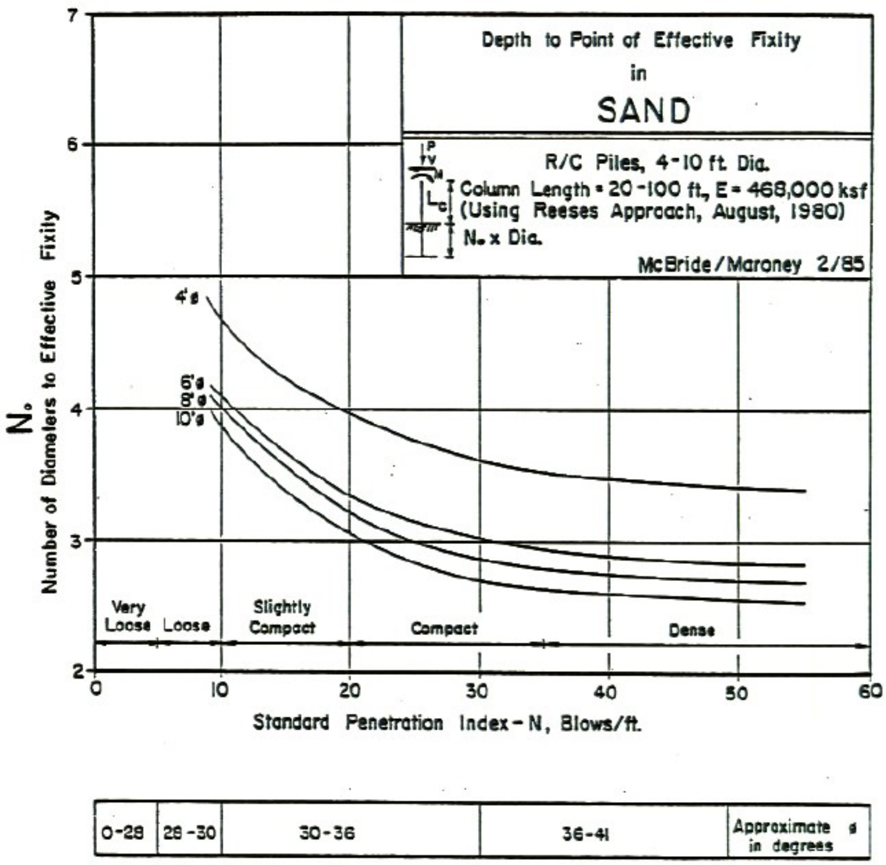


Figure 5.119 Depth to Fixity in Sand

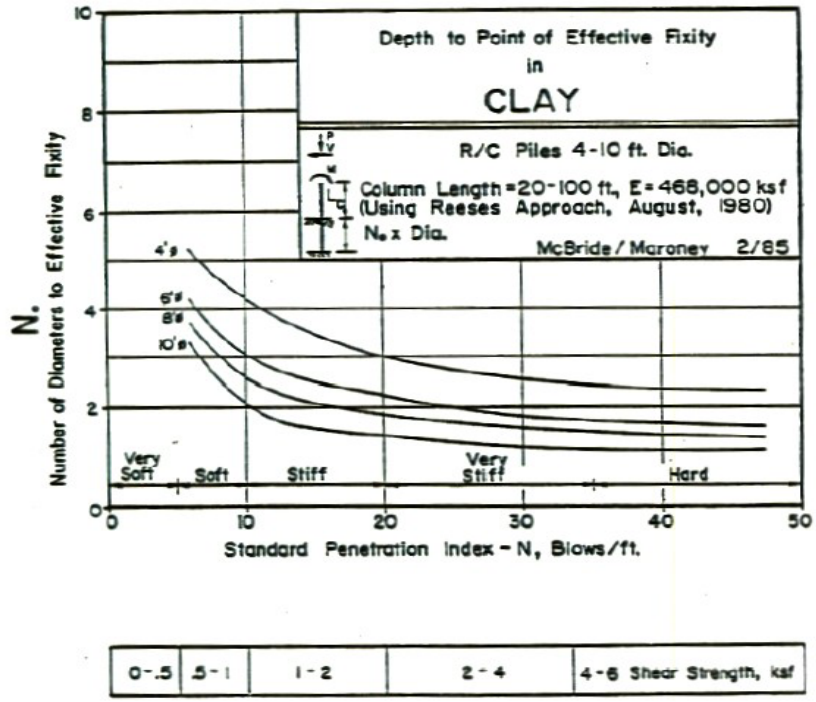


Figure 5.120 Depth to Fixity in Clay

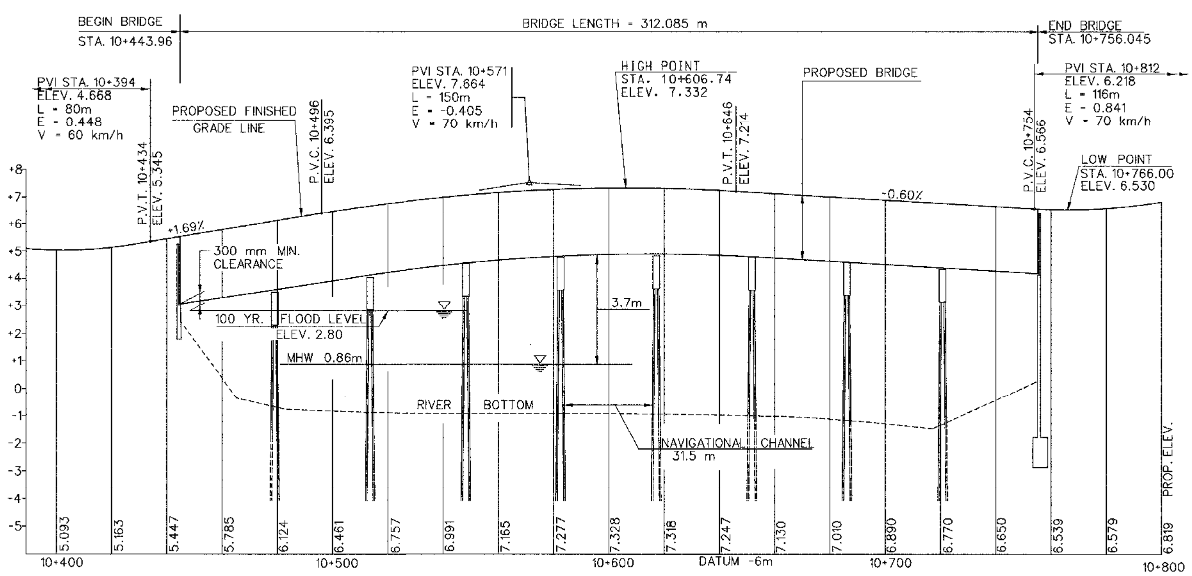


Figure 5.121 River Bottom Profile

The minimum tip elevation at pile bents is duplicated from as-built plans and shown below:

THE ORDER LENGTHS FOR THE FOUNDATION PILES AT EACH PILE BENT SHALL BE DETERMINED BASED ON THE RESULTS OF DRIVING THE TEST PILE AT EACH PILE BENT LOCATION. TEST PILES SHALL BE DRIVEN TO **BOTH** THE MINIMUM TIP ELEVATION **AND** THE REQUIRED ULTIMATE DRIVING RESISTANCE SPECIFIED BELOW. DRIVING CRITERIA SHALL BE BASED ON WAVE EQUATION ANALYSIS USING PARAMETERS CALIBRATED BY THE CONTROL TEST PILE DATA.

PILE BENT	MINIMUM TIP ELEVATION	REQUIRED ULTIMATE DRIVING RESISTANCE	ESTIMATED TIP ELEVATION
1	−14.5 m	2 890 kN	−24.0 m
2	−17.7 m	3 470 kN	−24.8 m
3	−20.0 m	3 480 kN	−27.0 m
4	−20.3 m	3 500 kN	−20.3 m
5	−20.0 m	3 225 kN	−28.3 m
6	−18.4 m	3 200 kN	−27.7 m
7	−17.7 m	2 760 kN	−27.7 m
8	−17.5 m	2 760 kN	−27.5 m

The pile bent cap elevations are shown in Table 5.55 below:

Table 5.55 Pile Bent Cap Elevations

PILE BENT CAP ELEVATIONS IN METERS (m)								
LOCATION	BENT NO.							
	1	2	3	4	5	6	7	8
TOP OF PILE CAP	3.482	4.045	4.448	4.668	4.705	4.558	4.351	4.143
BOTTOM OF PILE CAP	2.232	2.795	3.198	3.418	3.455	3.308	3.101	2.893
TOP OF PILES	2.537	3.100	3.503	3.723	3.760	3.613	3.406	3.198

Frame 2 consisting of spans 4, 5, and 6, is similar to frames 1 and 3. This frame is examined in detail. Bent 5 has fixed bearings while all other bents have expansion bearings.

For seismic analysis in the longitudinal direction, equivalent stiffness of bents 3, 4, and 6 is considered by taking into account the flexibility of the bearing pad stiffness. For the transverse direction, given the presence of transverse shear keys, all piers are sharing the loading in the transverse direction equally.

Calculate x-section girder area:

Span 34.585m or 115.25 ft

Top of flange $$\left(\frac{42+16}{2}\right)\times 2+42\times 5=268$$

Top flange haunch $$\left(\frac{8+16}{2}\right)\times 4=48$$

Web	$42\times8=336$
Bottom flange	$\left(\frac{8+28}{2}\right)\times10+28\times8=\frac{404}{1056}$ in^2 or $7.33\ ft^2$
Deck slab	$\frac{8.5}{12}\times91=64.5\ ft^2/ft$

Total length of bridge: 1037.5 ft

Concrete in Superstructure Deck = 64.5×1037.5 = 66915 ft^3

Increase 10% for fillets = 1.1×66915 ft^3 = 73607 ft^3 or 1994 m^3

Calculate concrete weight for parapets = 4 ft^2×2 = 8 ft^2

Concrete in parapet Total: 8×1037.5 = 8300 ft^3

Weight of parapet per linear foot: 8×0.15 K/ft^3 = 1.2 K/ft^3

Consider quantities as shown in the plans

Concrete in deck superstructure:	2096 m^3 or 77552 ft^3
Concrete in sidewalk:	170 m^3 or 6290 ft^3
Concrete in parapets:	8300 ft^3
Concrete in diaphragms:	256 m^3 or 9472 ft^3
Total:	101614 ft^3

Considering 1037.5 ft of bridge length:

Concrete superstructure per ft is = $\frac{101614\ ft^3}{1037.5\ ft}=98\ ft^2/ft$

Concrete superstructure weight per ft = 0.15 K/ft^3×98 ft^2 = 14.7 K/ft

Total superstructure weight per linear foot:

P.S. Girders Total: 12×7.33 ft^2×0.15 K/ft^3 = 13.2 K/ft

Concrete Superstructure weight per ft: 14.7 K/ft

Consider Total: 28 K/ft

Pile Cap Dimensions: 4.2 ft×8.76 ft

Length: 99.1 ft

	Bottom Pile Cap	River Bottom	Height
Bent 3	3.2m	-1	4.2
Bent 4	3.4m	-1	4.4
Bent 5	3.46m	-1	4.5
Bent 6	3.31m	-1	4.3

Estimated depth of Fixity 3×D below river bottom or 6 ft

Equivalent Height of Piles at Bent 5:

15 ft+6 ft = 21 ft

Pile Cap Weight:

$0.15\ K/ft^3 \times (4.2 \times 8.7 \times 99.1) = 543$ Kips

Calculate D.L. applied on piles at Bent 5:

= 115.3 ft×28 K/ft+543 Kips = 3228+543 = 3772 Kips

Total Piles: 24

Consider D.L. on pile:

$$\frac{3772}{24} = 157 \text{ Kips}$$

Based on the CSI-SAP analysis results Appendix VIII.A, Figures VIII.A.4, VIII.A.5, VIII.A.6, and VIII.A.7

Pile Top: $I_g = 1.3\ ft^4$

$I_{eff} = 0.40\ ft^4$ $I_{eff}/I_g = 0.31$

$M_n = 492$ K-ft

Based on the CSI-SAP analysis results Appendix VIII.A, Figures VIII.A.10, and VIII.A.11

Pile Bottom: $I_g = 1.3$

$I_{eff} = 0.2$ $I_{eff}/I_g = 0.15$

$M_n = 579$ K-ft

Model Stiffness in Transverse direction:

$$K_T = \frac{1200}{0.28} = 4286 \text{ K/ft}$$

Model Stiffness in Longitudinal direction:

$$K_L = \frac{1200}{0.15} = 8000 \text{ K/ft}$$

Stiffness of Bearing Pad in Longitudinal Direction

Bearing Pad Pressure = $\frac{3228/24}{28\times14}=343$ psi

Elastomer thickness = 4×75 in = 3 in

Based on Konstantinidis et al. (2008),

$F_b = GA\gamma(1-\lambda_y)$

$$\lambda_y = \frac{3}{2\times14} = 0.11$$

$F_b = GA\gamma(.89)$

Consider G = 90 psi

$$F_b = \frac{9}{1000}\times14\times28(0.89) = 31.4 \text{ Kips}$$

$$K_b = \frac{31.4 \text{ K}}{3 \text{ in}}\times12\frac{\text{in}}{\text{ft}} = 126 \text{ K/ft}$$

Total bearing stiffness: 126×24 = 3024 K/ft

Bridge Response in Transverse Direction:

Considering bearings restrained in transverse direction.

$$\omega^2 = \frac{4286}{3772}\times32.2 = 36.6$$

ω=6.1 rad/sec

$$T=\frac{2\pi}{\omega}=1 \text{ sec}$$

S_a = 0.09g

Considering NJ Factor of 1.5

S_a = 0.09×1.5 = 0.14g

$$S_d = \frac{0.14\times32.2}{36.6}\times12 = 1.5 \text{ in}$$

Transverse Force 0.14×3772 = 528 Kips. This load is equivalent to (0.44×3772 Kips)

Bridge Response in Longitudinal direction

$$\frac{1}{K_{LE}} = \frac{1}{3024}+\frac{1}{8000}$$

K_{LE}=2195 K/ft

Consider stiffness of one bent with fixed bearing and two other bents with expansion bearings:

Total Stiffness: 8000+2×2195 = 12390 K/ft

$$\omega^2 = \frac{12390}{3 \times 3772} \times 32.2 = 35$$

$\omega = 5.9$ rad/sec

$$T = \frac{2\pi}{5.9} = 1 \text{ sec}$$

S_a = 0.09g

Considering N.J. Factor 1.5

S_a = 0.09×1.5 = 0.14g

$$S_d = \frac{0.14 \times 32.2}{35} \times 12 = 1.5 \text{ in}$$

Longitudinal force at bent 5 is obtained based on the stiffness of bent 5:

$$= \frac{8000}{12390}(3228 \times 3 + 543) \times 0.14 \text{ ft}$$

= 925 Kips. This load is equivalent to (0.77×1200 Kips)

Based on the CSI-SAP analysis results Appendix VIII.A, Figures VIII.A.12 to VIII.A.15:

Transverse Demand		Model	Model x 0.44
	Top of Pile	554 K-ft	244
	Bottom of Pile	454 K-ft	200
	Shear Pile	46 Kips	21
	Axial Force	344 Kips	152

Longitudinal Demand		Model	Model x 0.77
	Top of Pile	353 K-ft	272
	Bottom of Pile	287 K-ft	221
	Shear Pile	31 Kips	24
	Axial Force	524 Kips	404

Transverse Demand:

Compression 152+157 = 309 Kips

Tension 157-152 = 5 Kips

Longitudinal Demand:

Compression 404+157 = 561 Kips

Tension -404+157 = -247 Kips

The nominal moment M_n (Top of pile and bottom of pile) is obtained at various axial loads:

Axial (Kips)	Top of Pile M_n (K-ft)	Bottom Pile M_n (K-ft)
-247	232	434
0	392	539
309	585	605
561	701	646

The flexural D/C ratio is calculated for the demand moment and corresponding axial force at Bottom & Top of Pile:

Transverse Demand + D.L.:

Top of Pile	Moment	Axial	M_n	D/C
	244	309	585	0.44
	244	5	392	0.62
Bottom of Pile	200	309	605	0.33
	200	5	539	0.37

Longitudinal Demand + D.L.:

Top of Pile	Moment	Axial	M_n	D/C
	272	561	701	0.39
	272	-247	232	1.17
Bottom of Pile	221	561	646	0.34
	221	-247	434	0.51

Calculate Local Displacement Capacity for SDC B

The displacement magnification for short period structures of AASHTO-SGS 4.3.3 does not apply considering that both longitudinal and transverse models have a period of 1.0 sec.

For Type 1 structures, comprised of reinforced concrete columns in SDC B, the displacement capacity, Δ_C^L in., of each bent may be determined from the following approximation:

$$\Delta_C^L = 0.12H_o\left(-1.27\ln(x) - 0.32\right) \geq 0.12H_o$$

in which:

$$x = \frac{\Lambda B_o}{H_o}$$

`

where:

H_o = Clear height of column (ft.)

B_o = Column diameter or width measured parallel to the direction of displacement under consideration (ft.)

Λ = factor for column end restraint condition

= 1 for fixed-free (pinned on one end)

= 2 for fixed top and bottom

For a partially fixed connection at one end, interpolation between 1 and 2 is permitted for Λ. Alternatively, H_o may be taken as the shortest distance between the point of maximum moment and point of contra-flexure and Λ may be taken as 1.0 when determining x using the equation above.

Calculating local displacement capacity in the transverse direction:

$$x = \frac{\Lambda B}{H}$$

where:

Λ = 2 for fixed top and bottom connections as in transverse direction

Λ = 1 for fixed free connection as in the longitudinal direction.

Establish capacity in both longitudinal and transverse direction based on Λ = 2, considering full flexural constraint at bottom of the pile cap and the river bottom. The pile has a 2 ft diameter and the clear distance between bottom of the cap and the bottom of the river is 15 ft.

$$x = 2 \times \frac{2}{15} = 0.27$$

$$\Delta_c = 0.12 \times 15(-1.27 \ln(0.27) - 0.32) \geq 0.12 \times 15$$

= 1.8(1.34) ≥ 0.12×15

= 2.4 in Compared to 1.5 in. displacement demand in Longitudinal or Transverse direction.

According to AASHTO-SGS Section 4.8

$$\Delta_D^L < \Delta_C^L$$

where:

Δ_D^L = displacement demand taken along the local principal axis of the ductile member

Δ_C^L = displacement capacity taken along the local principal axis corresponding to Δ_D^L of ductile member as determined in accordance with Article 4.8.1 for SDC B and C.

Eq. 1 shall be satisfied in each of the local axis of every bent. The local axis of a bent typically coincides with the principal axis of the columns in that bent.

Displacement Demand in Transverse and longitudinal directions 1.5 in

Displacement Demand (1.5in) ≤ Displacement Capacity (2.4 in)

It is important to mention that the displacement capacity check in the longitudinal direction is conservative and ignore flexibility between the pile cap and the superstructure. In the transverse direction, the flexibility of the pile cap is much less significant.

Response of the Abutment

According to the AASHTO-SGS 5.2.3.1, abutments for bridges in SDC B are expected to resist earthquake loads with minimal damage. However, bridge superstructure displacement demands may be 4 in. or more before the soil mobilization may potentially be increased. Comparing the displacement demand to the 4 in. threshold capacity, the abutments are deemed adequate for minimal damage requirement

Column Shear Demand and Capacity

According to AASHTO-SGS 8.6.1, the shear demand for a column, V_u, in SDC B shall be determined based on the lesser of:

- The force obtained from a linear elastic seismic analysis
- The force,V_{po}, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_u, in SDC C or D shall be determined based on the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.

Given the uncertainty in the hazard and the consequence of column shear failure, it is deemed important to attempt to satisfy the capacity protection requirement for column shear.

The column shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated based on the nominal material strength properties and shall satisfy:

$$\phi_s V_n \geq V_u$$

in which:

$$V_n = V_c + V_s$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

Calculate Pile Shear Demand and Capacity:

Maximum Elastic Shear demand in longitudinal direction is 24 Kips. Plastic shear corresponding to over strength moment and compression force of 561 kips is equal to

$$V_{po} = 1.4\left(\frac{701+646}{15}\right) = 126 \text{ Kips}$$

The concrete shear capacity,Vc, of members designed for SDC B, C and D shall be taken as:

$V_c = v_c A_e$, in which

$A_e = 0.8 A_g$

$A_e = 0.8 \times 576 = 461 \text{ in}^2$

if P_u is compressive:

$$v_c = 0.032\alpha'\left(1 + \frac{P_u}{2A_g}\right)\sqrt{f_c'} \le \min\begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases}$$

otherwise:

$v_c = 0$

$$0.3 \le \alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \le 3$$

$$f_s = \rho_s f_{yh} \le 0.35$$

$$\rho_s = \frac{4A_{sp}}{sD'}$$

where:

A_g = gross area of member cross section (in^2)

P_u = ultimate compressive force acting on section (kip)

A_{sp} = area of spiral or hoop reinforcing bar (in^2)

s = pitch of spiral or spacing of hoops or ties (in.)

D' = diameter of spiral or hoop for circular column (in.)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f_c' = nominal concrete compressive strength (ksi)

μ_D = maximum local displacement ductility ratio of member

For SDC B, the concrete shear capacity, V_c, of a section within the plastic hinge region shall be determined using:

μ_D = 2

$$\rho_s = \frac{4 \times 0.2}{6 \times 18} = 0.74\%$$

$$f_s = 0.0074 \times 60 = 0.44 < 0.35$$

$$\alpha' = \frac{0.35}{0.15} + 3.67 - 2 = 4 < 3$$

$$\alpha' = 3$$

For circular columns with spiral or hoop reinforcing:

$$v_c = 0.032 \times 3\left(1 + \frac{561}{2 \times 576}\right)\sqrt{4} = 0.14 \le \begin{cases} 0.22 \\ \\ 0.282 \end{cases}$$

V_c = 0.14×461 = 65 Kips

Calculate Column Shear Reinforcement Capacity

According to AASHTO-SGS 8.6.3, members that are reinforced with circular hoops, spirals or interlocking hoops or spirals as specified in Article 8.6.6, the nominal shear reinforcement strength, Vs, shall be taken as per Eq.(8.6.3-1) as:

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

where:

n = number of individual interlocking spiral or hoop core sections

A_{sp} = area of spiral or hoop reinforcing bar (in.2)

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

The pitch s is taken equal to 6" since shear demand is constant and governs the design outside the plastic hinge region.

$$V_s = \frac{\pi}{2}\left(1 \times 0.20 \times 60 \times \frac{18}{6}\right) = 57$$

Capacity $\phi_s(V_s + V_c) = 0.9(57 + 65) = 110$ Kips Close enough to plastic demand V_{po} equal to 126 Kips

The following requirements need to be satisfied for SDC B:

Check Minimum Support Length

According to AASHTO-SGS Section 4.12.2 for Seismic Design Categories A, B, and C, support lengths at expansion bearings without STU's or dampers shall be designed to either accommodate the greater of the maximum calculated displacement, except for bridges in SDC A, or a percentage of the empirical support length, N, specified below The percentage of N, applicable to each SDC, shall be as specified in Table 5.56 below.

$N = (8+0.02L+0.08H)(1+0.000125S^2)$

where:

N = Minimum support length measured normal to the centerline of bearing (in.)

L = Length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (ft.)

H = For abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft.); For columns and/or piers, column, or pier height (ft.); For hinges within a span, average height of the adjacent two columns or piers (ft.); 0.0 for single-span bridges (ft.)

S = Angle of skew of support measured from a line normal to span (°)

Table 5.56 Percentage N by SDC and effective peak ground acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percent N
A	<0.05	≥75
A	≥0.05	100
B	All applicable	150
C	All applicable	150

For SDC B:

$N = 1.5(8+0.02L+0.08H)(1+0.000125S^2)$

L = 345.5 ft calculated based on the total length of three continuous spans 4, 5, and 6 from Bent 3 to Bent 6.

H = 21 ft (Including length to point of fixitity)

$N = 1.5(8+0.02\times34.5+0.08\times21)$

$= 1.5(8+6.9+1.7) = 25$ in

Support length at abutment (See VIII.B.5, VIII.C.3, VIII.C.4)

(230 mm + 460 mm) = 27 in > 25 O.K.

Support length at Bent (See VIII.B.5, VIII.C.14)

(230 + 380 mm) = 24 compared to N requirement of 25

Available support length slightly less than required support length. However, considered satisfactory based on conservative N values in AASHTO-SGS.

CHAPTER 6: SIMPLIFIED CRITERIA FOR THE RETROFITTING ANALYSIS OF EXISTING BRIDGES IN NEW JERSEY

Currently, all bridges in New Jersey are retrofitted according to 2006 FHWA Manual on Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges" [FHWA (2006)]. Following the recommendation of NJDOT in February 2009, the research team has conducted an extensive review of the FHWA manual to present simplified guidelines for existing bridges that are consistent with provisions of AASHTO Guide Specification on Bridge Seismic Design (AASHTO-SGS), while meeting the level of retrofits required for New Jersey bridges.

This chapter presents simplified guidelines that are applicable to low seismicity regions like New Jersey. All bridges in New Jersey should be retrofitted as per guidelines presented in this chapter.

Anticipated Service Life (ASL)

Existing bridges should be categorized into the following three ASL classes (see Table 6.1), assuming a service life of 75 years for new bridges.

Table 6.1 Anticipated Service Life for Bridges

Service Life Category	Remaining Service Life
ASL1	0-15 Years
ASL2	16-50 Years
ASL3	>50 Years

For example, if a bridge with service life of less than 15 years (ASL1) is planned to undergo non-seismic rehabilitation to increase its remaining service life to 35 Years (i.e., ASL2), then seismic retrofits should be planned for ASL2 category.

Bridge Importance

All bridges in New Jersey undergoing retrofit are considered as "Standard" bridges unless New Jersey Department of Transportation decides to classify a bridge as critical based on criteria for importance classification presented in Chapter 3.

Exempt Bridges

A bridge is exempt from retrofitting for both levels of ground motion if it satisfies any one of the following criteria:

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

A critical bridge satisfying above criteria should not be exempt from retrofitting.

Earthquake Ground Motion Levels

Lower level ground motion prescribed in the 2006 FHWA Manual can be ignored in the analysis for seismic retrofit of bridges in New Jersey since it is very small and its requirement on bridge performance will automatically be satisfied if the bridge is retrofitted based on the higher level ground motion. This is because of the fact that a majority of bridges in New Jersey subject to the upper level ground motion behave essentially elastic.

Standard Bridges

All standard bridge retrofits in New Jersey should be designed for a single ground motion with a hazard with 7% probability of exceedance in 75 years, corresponding to a return period of 1000 years, as specified for new bridges in AASHTO-SGS.

Design response spectra should be constructed as per national ground motion maps described in AASHTO-SGS.

The construction of the response spectra should be using three-point method as per Figure 6.1 below. In Figure 6.1, S_1 = 1.0 second period spectral acceleration coefficient on Class B rock, S_s= 0.2 second period spectral acceleration coefficient on Class B rock, F_a = site coefficient for 0.2 second period spectral acceleration specified in Table 6.1 and F_v = site coefficient for 1.0 second period spectral acceleration specified in Table 6.2.

A detailed procedure for the construction of the spectra in Figure 6.1 is specified in article 3.4.1 of 2008 AASHTO-SGS.

Table 6.2 Values of F_{PGA} and F_a as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	PGA≤0.10 S_s≤0.25	PGA=0.20 S_s=0.50	PGA=0.30 S_s=0.75	PGA=0.40 S_s=1.00	PGA≥0.50 S_s≥1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	A	A	A	a	A

Note: Use straight line interpolation for intermediate values of PGA and S_s, where PGA is the peak ground acceleration and S_s is the spectral acceleration coefficient at 0.2 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

Table 6.3 Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	A	A	A	a	A

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

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

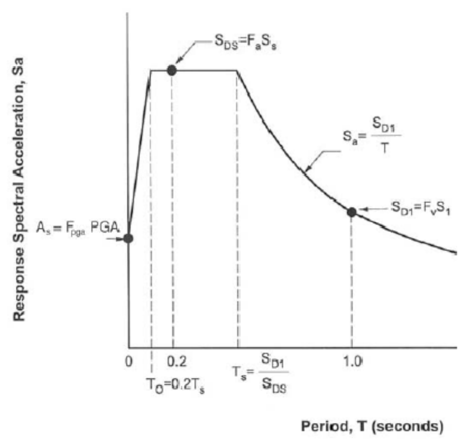


Figure 6.1 Design Spectra Using Thee-Point Method

Critical Bridges

Seismic ground motion hazard for existing bridges in New Jersey for critical bridges shall be the same as that for standard bridges in the preceding section.

Damage Performance Levels

All standard bridges should be retrofitted for life safety level.
All critical bridges shall be retrofitted for minimal damage to its components as per requirements in Chapter 3.

Seismic Retrofit Categories

Based on review of seismic hazard to bridges in New Jersey using soil site class maps and zip code based spectra in AASHTO-SGS, New Jersey Seismic Retrofit Categories (NJSRC) L and H have been proposed.

It should be noted that the FHWA Seismic Retrofit manual uses short period spectral period acceleration S_s for determining seismic hazard level. It has been observed from soil site class maps that this criterion affects only 8 zip codes in the state. For this hazard, bridges in these zip codes will be retrofitted as per SRC C (of FHWA Manual) for PL2 performance (operational level for critical bridges). Soil types in these regions is E. Short period component (S_s) of the design spectra will have insignificant contribution to the bridge response in these regions because of interaction of the bridge with surrounding soft soils. Hence, seismic retrofit categories can be determined based on long-period (1.0 sec) component S_{D1}, as shown in Table 6.4.

Standard Bridges

Table 6.4 Seismic Retrofit Categories for Standard Bridges

ASL	Hazard	NJ-SRC
1	-	Do Nothing
2, 3	$S_{D1} \leq 0.15$	NJ-SRC L
	$0.15 < S_{D1} \leq 0.30$	NJ-SRC H

Critical Bridges

Table 6.5 Seismic Retrofit Categories for Critical Bridges

ASL	Hazard	NJ-SRC
1, 2 or 3	$S_{D1} \leq 0.15$	NJ-SRC L
	$0.15 < S_{D1} \leq 0.30$	NJ-SRC H

New Jersey Vs. FHWA Seismic Retrofit Categories

A correlation between NJSRC and FHWA Seismic Design Categories is presented in the table below.

Table 6.6 New Jersey Seismic Retrofit Categories (NJ-SRC).

NJSRC	Requirement	FHWA SRC
L	• A1/A2 Analysis as per FHWA Seismic Retrofit Manual: No Analysis, Minimum Capacity Check • Check Seat Widths • Check Connections	B
H	• Elastic Component Capacity/Demand Analysis • Check Seat Widths • Check Connections • Retrofit of piers and footings for Demand Reduction/Capacity Protection.	C with Method C.

Figure 6.2 shows the flow chart for the selection of appropriate seismic Retrofit Category (SRC).

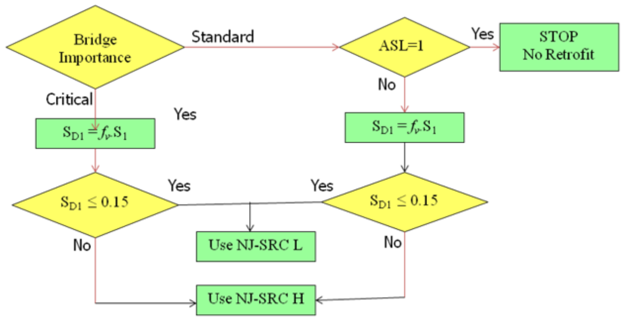


Figure 6.2 Flow Chart for Selection of NJSRC.

Geotechnical Hazards

As per 2006 FHWA seismic retrofit manual, liquefaction hazards analysis isn't required for seismic retrofit of bridges in New Jersey because of mean earthquake magnitude for New Jersey being smaller than 6.0.

Site Specific Analysis

Site specific analysis should be performed for critical bridges and bridges on soil site F. When site-specific spectra are determined from a site-specific study, the design spectra shouldn't be lower than 2/3rd of the zip-code based spectra provided by AASHTO if the peer review requirement is waived. For site class F, the generic response spectra should be on the basis of site class D.

Site specific analysis can be performed by the procedure described in Chapter 4.

Time Histories

Dynamic time-history analysis of a bridge isn't required in New Jersey. Generation of ground motion for site-specific analysis can be carried out automatically using SIMQKE based approach presented in Chapter 4 and Appendix III of this report.

Seismic Retrofit of Superstructure and Substructure

Selection and design of seismic retrofit measures for the superstructure and substructure should be carried out as per methods provided in Chapters 8 and 9 of the 2006 FHWA manual. New Jersey being a low seismic region, following two seismic retrofit measures should be sufficient to provide adequate safety against design earthquakes to prevent collapse of standard bridges and minimal damage (or essentially elastic behavior) in case of critical bridges:

- Retrofit of Bridge Piers by Carbon Fiber Reinforced Plastic (CFRP) Wrapping
- Elastomeric / Isolation Bearings to reduce seismic demand on columns and footings.

Recent research has shown that the wrapping of bridge piers by CFRP increases the ductility capability of bridge piers significantly [Pan et al. (2007)]. The method is very cost effective, doesn't require closure of the bridge and can be carried out within few days. For example, Figure 6.3 shows the seismic fragility (risk of failure) curves for a bridge pier in Figure 6.4 retrofitted by FRP wrapping [Pan et al. (2007)]. It is observed that the collapse in CFRP piers occurs by sudden fracture of CFRP wrapping at very high PGA. Hence, CFRP wrapping is recommended as preferred and cost-effective retrofit options, if piers require seismic retrofit.

Examples Illustrating Seismic Retrofit of Existing Bridges

Seismic retrofit categories NJSRC-L and NJSRC-H have similar requirements as those of SDC A and B for new bridges as proposed in the AASHTO-SGS. Hence, examples of bridges presented in Chapter 5 of this report can be used to train bridge engineers about the application of proposed guidelines for seismic retrofit design of existing bridges.

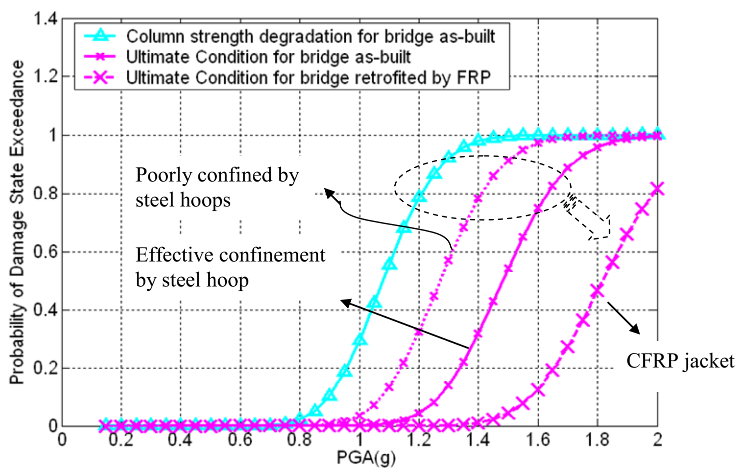


Figure 6.3 Fragility Curves of Piers versus PGA for a Multi-Span Steel Bridge Retrofitted by CFRP Jackets.

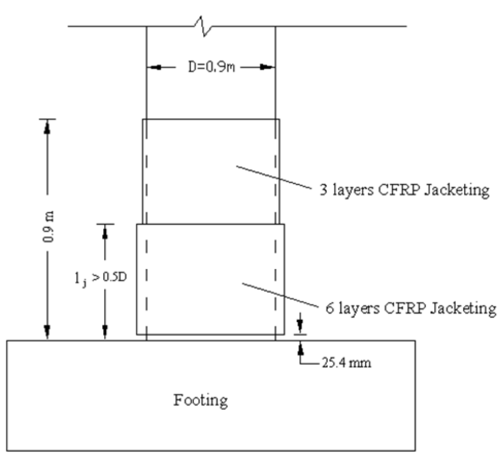


Figure 6.4 Seismic Retrofit Design of Bridge Pier by Using CFRP Wrapping

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

This report addresses and resolves numerous important issues towards practical implementation of AASHTO Guide Specifications on Bridge Seismic Design and the 2006 FHWA Seismic Retrofitting Manual for Highway Structures. The guideline can be used for seismic design of new and existing bridges in New Jersey and for the training of engineers about the provisions of these guidelines / manuals. Following are the main conclusions and recommendations of this report for implementations / future investigations.

Conclusions

1. AASHTO Guide Specifications on Bridge Seismic Design (AASHTO-SGS) doesn't provide design spectra for critical bridges. For the design of new critical bridges, a factor of 1.5 has been proposed to be multiplied to zip-code spectra corresponding to 1000 Yr return period earthquake recommended in AASHTO-SGS for standard bridges. All new critical bridges have been recommended to be designed for essentially elastic behavior using the 1000 Yr spectra multiplied by a factor of 1.5. This factor has been the basis of reducing the seismic demand from 2500 Yr return period to 1000 Yr return period in the AASHTO-SGS for standard bridges designed for life safety performance.
2. Existing critical bridges have been proposed to be designed for essentially elastic behavior for 1000 Yr return period spectra. Modified design criteria for existing bridges that align with guidelines presented in AASHTO-SGS for new bridges have been proposed. These proposed guidelines for existing bridges either meet or exceed guidelines recommended in the 2006 FHWA manual for seismic retrofitting of bridges.
3. Both generic and NJ specific approaches for the classification of bridges into standard and critical categories have been proposed. However, NJDOT has the discretion of classifying a bridge as standard or critical based on their risk management strategy.
4. NJDOT has an extensive electronic database of soil boring logs across the state. More than 12,000 selected boring logs from this database have been used to develop seismic site class map for the state of New Jersey. Seismic Design Category (SDC) maps for standard and critical bridges have been developed for the state of New Jersey based on this seismic site class map. Further extensive analysis using soil boring logs has been done to develop liquefaction hazard maps for the entire state of New Jersey. These maps can be used to determine the need for further detailed analysis for liquefaction, thereby further economizing any seismic design / retrofit project.
5. It has been observed from these maps that a majority of bridges in New Jersey are in SDC A, with some on soil class E falling into SDC B. It has been observed that areas with higher liquefaction hazard are mainly in the northeastern part of New Jersey. Liquefaction hazard maps can be used to determine the need for further detailed liquefaction hazard analysis of a bridge site.

6. Nine examples of bridges of different span lengths and material types (concrete and steel) have been developed to illustrate applications of provisions of the AASHTO-SGS for the design of new bridges. Six of these examples illustrate the design of bridges in seismic design category (SDC) B, while three examples illustrate the design in SDC A category.
7. AASHTO-SGS require the design of critical bridges using site specific spectra. This analysis is generally done by consultants, adding to costs of seismic design / retrofit projects in New Jersey. A semi-automatic computer tool and procedure using freely available software has been developed so that NJDOT engineers can carry out the development of site-specific spectra in-house. Usage of this tool and procedure is expected to result in significant cost savings in seismic design / retrofit projects, while improving the reliability and consistency of design of critical bridges in New Jersey.

Recommendations for Implementations / Future Investigations

1. The guideline doesn't include examples illustrating design of various approaches for seismic retrofit of bridges, including limitations, advantages and cost effectiveness of these approaches. These examples will provide training to engineers and standardize the seismic retrofit process, resulting in significant cost savings to NJDOT. Development of these examples can also incorporate recent advances in analysis for seismic retrofits and new retrofit approaches, such as FRP wrapping and the use of viscous dampers.

2. Seismic design guidelines for New Jersey for new and existing bridge structures are implemented through Section 38 of the New Jersey Department of Transportation Design Manual for Bridges and Structures, 5th Edition. Provisions in Section 38 of the Design Manual for Bridges and Structures need to be updated on the basis of this report for an effective implementation of research outcome of this project.

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Seismic Design Considerations
Volume II: Appendices II - VIII

FINAL REPORT
March 2012

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Division of Research and Technology
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16. Abstract

NJDOT has adopted "AASHTO Guide Specifications for LRFD Seismic Bridge Design" approved by the Highway Subcommittee on Bridges and Structures in 2007. The main objective of research presented in this report has been to resolve following issues for an effective implementation of AASHTO Guide Specifications: (i) AASHTO Guide Specifications don't provide any specific guidelines for classification and performance requirements for critical bridges. This issue is resolved by proposing performance requirements and classification criteria for critical bridges in New Jersey. (ii) Guide Specifications present displacement based approach, which is significantly different than the force-based approach in previous versions of seismic guidelines. Nine examples of reinforce concrete and steel bridges of different characteristics (spans, skew, etc.) illustrating the use of newly adopted seismic guide specifications have been developed for training of engineers in New Jersey. (iii) NJDOT maintains an extensive electronic database of soil boring logs for the State of New Jersey. A zip-code based soil site map for New Jersey has been developed by analyzing soil boring data and other available New Jersey soil information. This map can be used for a rapid seismic hazard evaluation for the entire state or for a network of bridges in the state. (iv) AASHTO Guide Specifications introduce seismic design categories based on local seismicity and soil properties. Using the seismic soil map and zip code based seismic spectra provided in the AAHSTO Guide Specifications, seismic design category maps for critical and standard bridges in New Jersey have been developed. A detailed analysis has also been carried out to develop liquefaction potential maps for the state of New Jersey. These maps can be used to determine the need for a detailed liquefaction analysis for a particular bridge site. A detailed guideline on developing site-specific spectra has also been developed, since AASHTO Guide Specifications recommend site-specific spectra for critical bridges. (v) Existing bridges in New Jersey are required to be retrofitted on the basis of 2006 Edition of the "Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges". Simplified guidelines for seismic retrofit of existing bridges, that are consistent with guidelines for the design of new bridges in AASHTO Guide Specifications, have been developed.

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Finally, dedicated work of all graduate and undergraduate students with assistance in various parts of the project is admirable and is sincerely appreciated.

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APPENDIX II: SEISMIC SOIL MAP FOR NEW JERSEY

AASHTO Site Class Definitions

ASSHTO site class definitions are presented in Table II.1.

Table II.1 Site class definitions according to AASHTO Guide Specifications

Site Class	Soil type and profile
Class A	Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec.
Class B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/sec.
Class C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/sec, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf.
Class D	Stiff soil with 600 ft/sec < $\bar{v}_s$ < 1,200 ft/sec, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf.
Class E	Soil profile with $\bar{v}_s$ < 600 ft/sec or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40 percent and $\bar{s}_u$ < 0.5 ksf.
Class F	Soils requiring site-specific evaluations, such as: Peats or highly organic clays (H > 10 ft of peat or highly organic clay where H = thickness of soil) Very high plasticity clays (H > 25 ft with PI > 75) Very thick soft/medium stiff clays (H >120 ft)
Where: $\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile $\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile $\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or D2850) for the upper 100 ft of the soil profile PI = plasticity index (ASTM D4318) w = moisture content (ASTM D2216)	

General Procedure for Soil Site Classification

The procedure is based on the site class definitions in Table II.1 using average SPT blow counts. However, considering the availability of data, some criteria were adjusted, including:

1) The criteria of Site Classes E & F based on plasticity (PI) were not used since PI is generally not available in the database;
2) Rock sites were based on the description in the boring log instead of wave velocity, unless available;
3) If specific description of “rock”, which is not highly weathered or fractured, was not found in the boring log, the material was considered as very dense soil;
4) If a site was classified as a rock site according to the definition in Table 1 as well as the boring log, but description of “hard rock” was not found in the boring log, the site was classified as Site Class B.

These adjustments are consistent with the AASHTO Specifications. According to the Specifications, Site Class E and F can only be considered when adequate information is available. And highly weathered or fractured rock shall be classified as very dense soil (Class C) unless shear wave velocity was measured. Besides, considering the precision of the site class map, the proposed map should take into account adequate conservativeness so that upper bound seismic response of a region can be estimated based on the map. It is noted that soil site classification based on PI for Class E and Class F was also not considered by New Jersey Geological Survey in generating the HAZUS soil maps.

Due to the large amount of data involve in the site classification (about 50,000 boreholes in the NJDOT soil boring database during the time of analysis in spring and summer of 2009), a system was established for data collection, grouping and analysis so that the relevant data can be analyzed according to their geological locations and conditions. Geographical Information System (GIS) was used to assist in the boreholes' collection and grouping according to their locations. For each zip code, a maximum of 30 boreholes were classified, and for the whole New Jersey, a total of about 10,000 boreholes were analyzed.

The site classification of each zip code followed the procedures described in the following, depending on the availability of soil data.

Site Classification of a specific site according to its boring log

As mentioned above, the site classification analysis is based on the Standard Penetration Test (SPT) following AASHTO Specifications. For a selected borehole, the following steps were conducted to determine its site class:

1. First of all check if more than 10ft of peat or highly organic soil layer, or more than 120ft of clayed soil layer exist; if yes, the site was classified as Site Class F.
2. For other soil sites, for the first 100ft of soil or rock below ground surface, the average SPT blow counts were calculated using the following equation:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(II.1)}$$

Where $\sum_{i=1}^{n} d_i$ is equal to 100ft, d_i is the thickness of a layer of soil, and N_i is the SPT blow count of that layer. The maximum number of N_i is 100. If SPT blow count more than 100 is encountered, 100 is used in Equation II.1, and if a layer of rock is encountered in the borehole, the N_i for the rock layer is also 100.

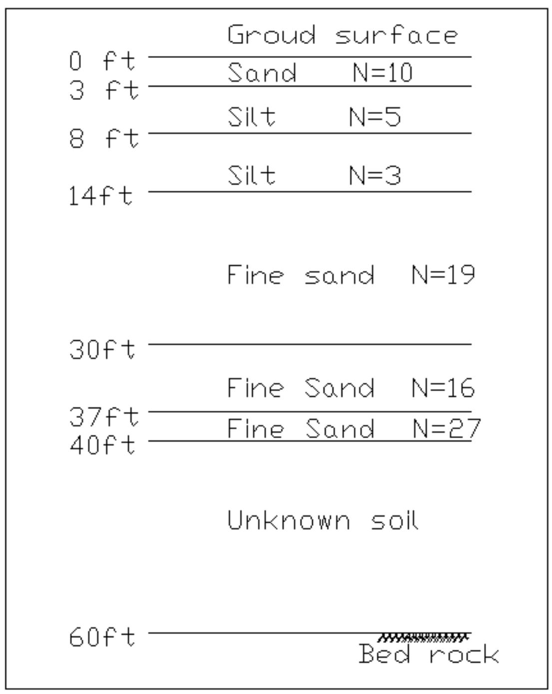


Figure II.1 Illustration of a soil profile (not to scale)

3. If a borehole is less than 100-ft deep and at the same time rock is not encountered at the bottom of the borehole, the soil beneath the end of borehole until bedrock surface, which can be estimated from the surficial geological map, was assumed to be the same as the bottom soil layer in the boring log. Figure II.1 illustrates the scenario. The borehole is only 40 ft deep, but according to the geological surficial

map, the depth of bedrock surface is about 60 ft. The 20 ft of unknown soil was then assumed to have a SPT blow count of 27, the same as the last layer of soil in the borehole.

4. The site was then classified based on the following criterion:

$\overline{N} > 50$	Class C
$15 \le \overline{N} \le 50$	Class D
$\overline{N} < 15$	Class E

5. If less than 10ft (3 m) of soil exit above bedrock surface, the site could be classified as a rock site. To be conservative, unless “hard rock” is specified in the boring log, the site is classified as Site Class B.

<u>Site classification of a zip code with adequate boring logs</u>

If a zip code had adequate boring logs from the database, i.e., the total number of boring logs in the data base is at least 30, the following procedure was employed to determine its site class.

1. If the total borehole number available in the database for one zip code was larger than 30, 30 of them were selected according to their specific locations in the zip code. The selected boreholes were distributed in the zip code region as evenly as possible. Figure II.2 compares the locations of all available boreholes in zip code 08648 in Mercer County and the selected ones.
2. If the total number of boreholes in the region was equal to 30, all of them were analyzed.
3. The site classes of the selected boreholes were determined based on the procedure outlined in previous subsection on “Site Classification of a specific site according to its boring log”.
4. The distribution of boreholes in the region was observed in conjunction with the surficial geological map. If the distribution was relatively even and the boreholes represented the main soil condition in the zip code, the site class was classified according to the majority of boreholes. Majority in this context means that it is larger than 50%. Figure II.3 shows the soil condition in zip code 07017 in Essex County. The main soil type in this zip code according to the surficial geological map is Qwtr, which represents Rahway Till, and most of the boreholes locate within this type of soil. The site class of zip code 07017 was then classified as Site Class C according to the site classes of these boreholes.

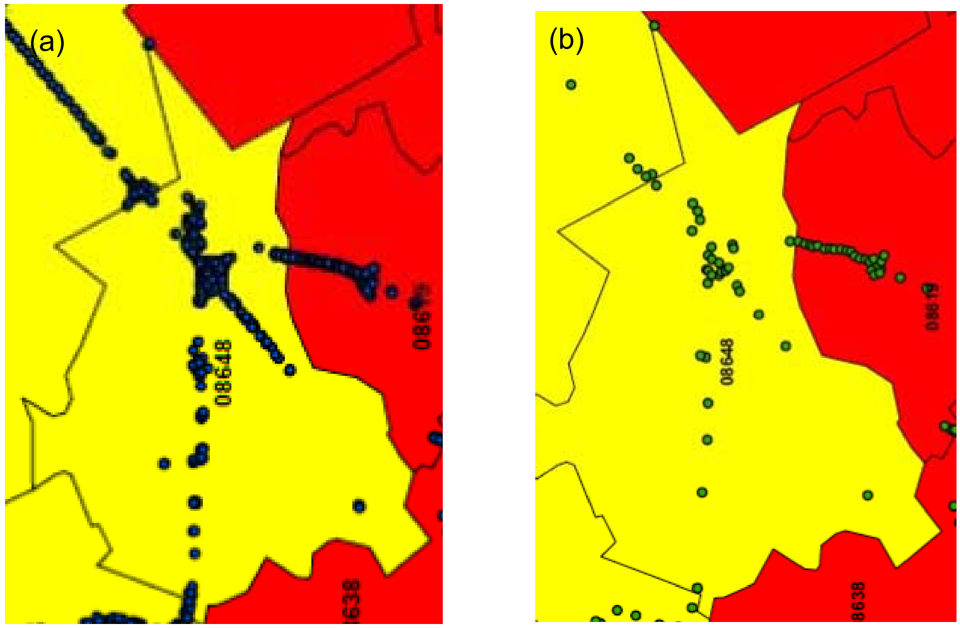


Figure II.2 (a) Available and (b) Selected Borehole Data for analysis in zip code 08648

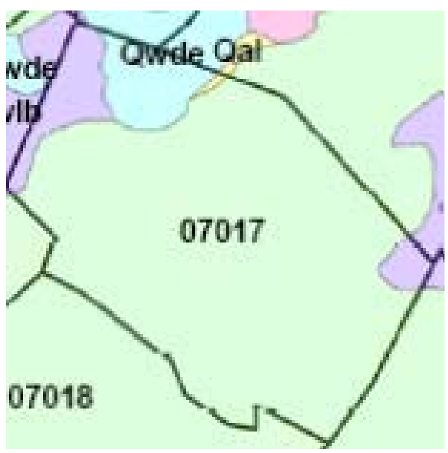


Figure II.3 Geological Soil Condition in Zip Code 07017
(light blue color represents geological soil type Qwtr)

5. If the available boreholes do not represent the soil condition in the zip code, the procedure outlined in later subsection on “Site classification of a zip code without adequate boring logs” was used instead.
6. Conservativeness was considered in the site classification of the zip code region. For example, if only limited region in the zip code was classified as rock sites according to the borehole data as well as the surficial geological map, the zip code

was classified as soil site according to the site class of the soils in the same region. Figure II.4 illustrates such a scenario. In zip code 08534 in Mercer County, the geological soil type is mainly Qws, which represents weathered shale, mudstone and sandstone. The thickness of soil ranges from less than 10 feet to 30 feet in the area. The results of borehole show a mixture of Site Class B and Site Class C. And to be conservative, the zip code was assigned as Site Class C in the map.

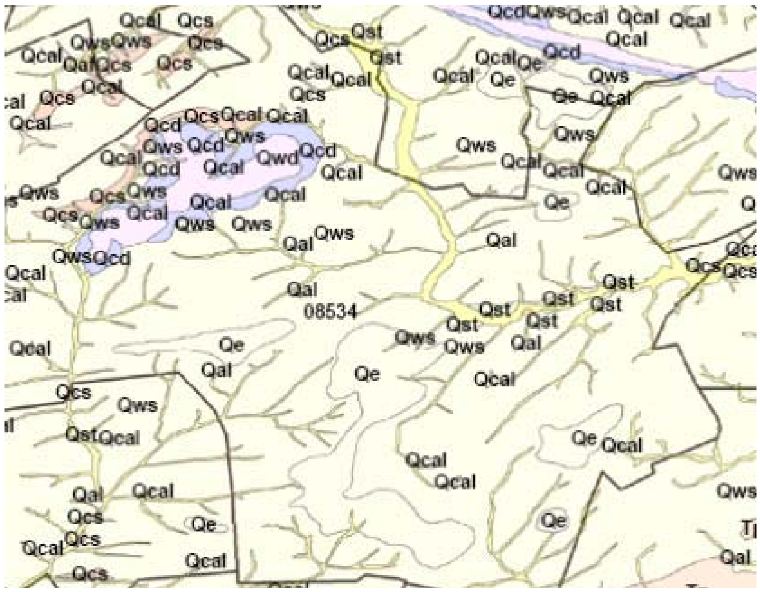


Figure II.4 Soil Condition in Zip Code 08534 in Mercer County

Site classification of a zip code with few boring logs

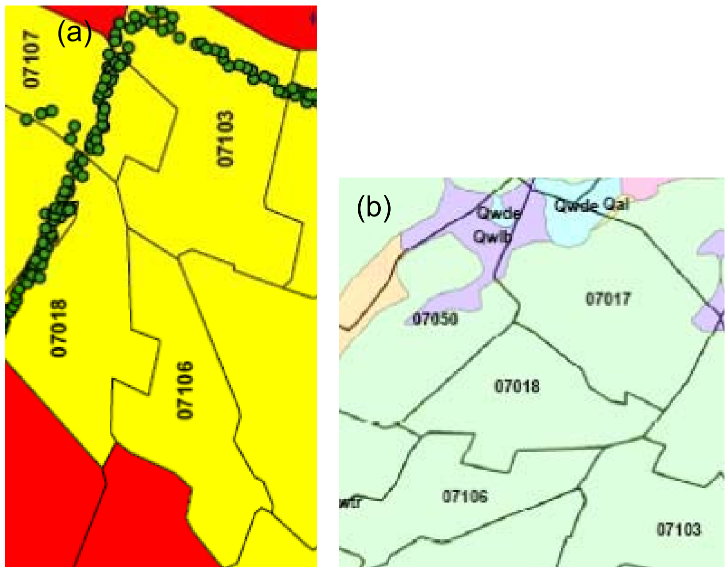
If the number of boring logs available in one zip code is smaller than 30, all of them were analyzed according to the procedure outlined in previous subsection on “Site Classification of a specific site according to its boring log”. Procedures identical to steps 4, 5 and 6 in the previous subsection on “Site classification of a zip code with adequate boring logs” were then followed to determine the site class of the zip code.

Site classification of a zip code without adequate boring logs

In this case, soil information from the surficial geological map as well as the boring log data in the adjacent regions were used to classify the site, the HAZUS soil maps for the nine counties in northern New Jersey, as indicated above, were also used. Specifically, the following procedures were undertaken for different scenarios.

1. If the zip code is in the nine counties with HAZUS soil maps, the classification of that region from the HAZUS maps was used. If the adjacent zip codes were already classified with adequate data according to the procedure in the previous subsection

on “Site classification of a zip code with adequate boring logs” and according to the surficial geological map the interested zip code has similar soil type with the adjacent zip codes, it was classified as the same site class as the adjacent zip codes. Figure II.5 illustrates such a scenario. The four zip codes, 07103, 07106, 07107 and 07108 share similar geological soil deposit, but 07106 does not have boreholes. This zip code was then classified as Site Class C based on the results of the other three zip codes.



(a) (b)

Figure II.5 Site classification of zip code 07106 based on the site classes of zip codes 07107, 07103 and 07108: (a) site classes; (b) geological soil deposit

2. For all the others, the procedure similar to the one used by New Jersey geological survey in generating the HAZUS maps was employed. Specifically,
   - a.) All the soil types according to the surficial geological map in the county, and in the adjacent counties if the boring data for the interested county were limited, were identified;
   - b.) The available boring logs in the county (and adjacent counties when necessary) were grouped according to their soil types in the surficial geological map;
   - c.) The site class of any identified soil type was determined based on the grouped boring logs of that soil type;
   - d.) The site class of a zip code was then determined based on the main soil type in the zip code and the corresponding site class from step c).

Soil Site Class Maps for New Jersey

The maps for the 21 counties are shown in Figure II.6 to Figure II.26. These maps also show county, zip code, soil site class and location of analyzed boring logs. Other information such as municipality can be overlapped on the maps using the digital file. The specific numbers of boreholes analyzed for a zip code are not shown on the map but they can be retrieved from the digital file enclosed on the CD.

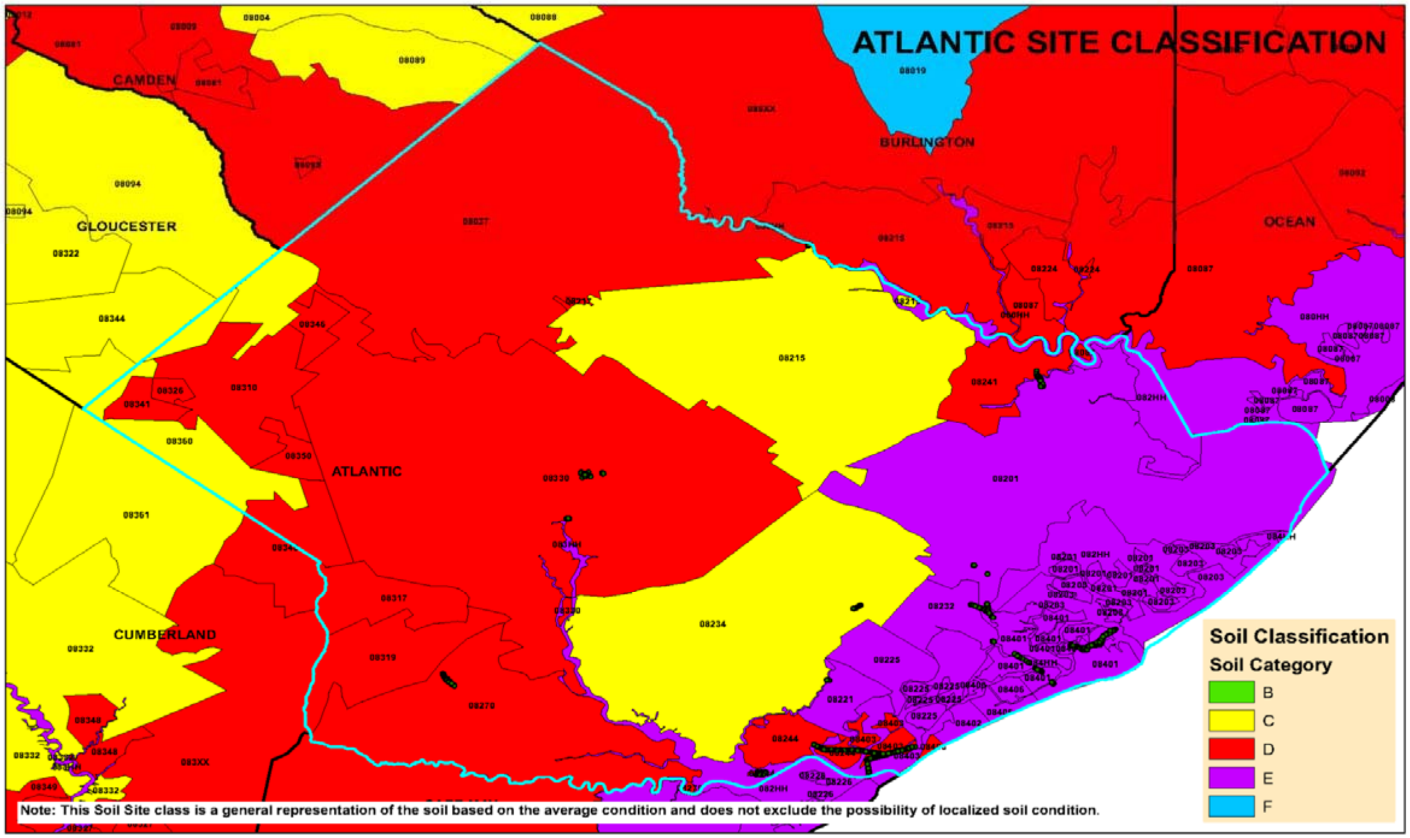


Figure II.6 Soil Site Class Map for Atlantic County of New Jersey

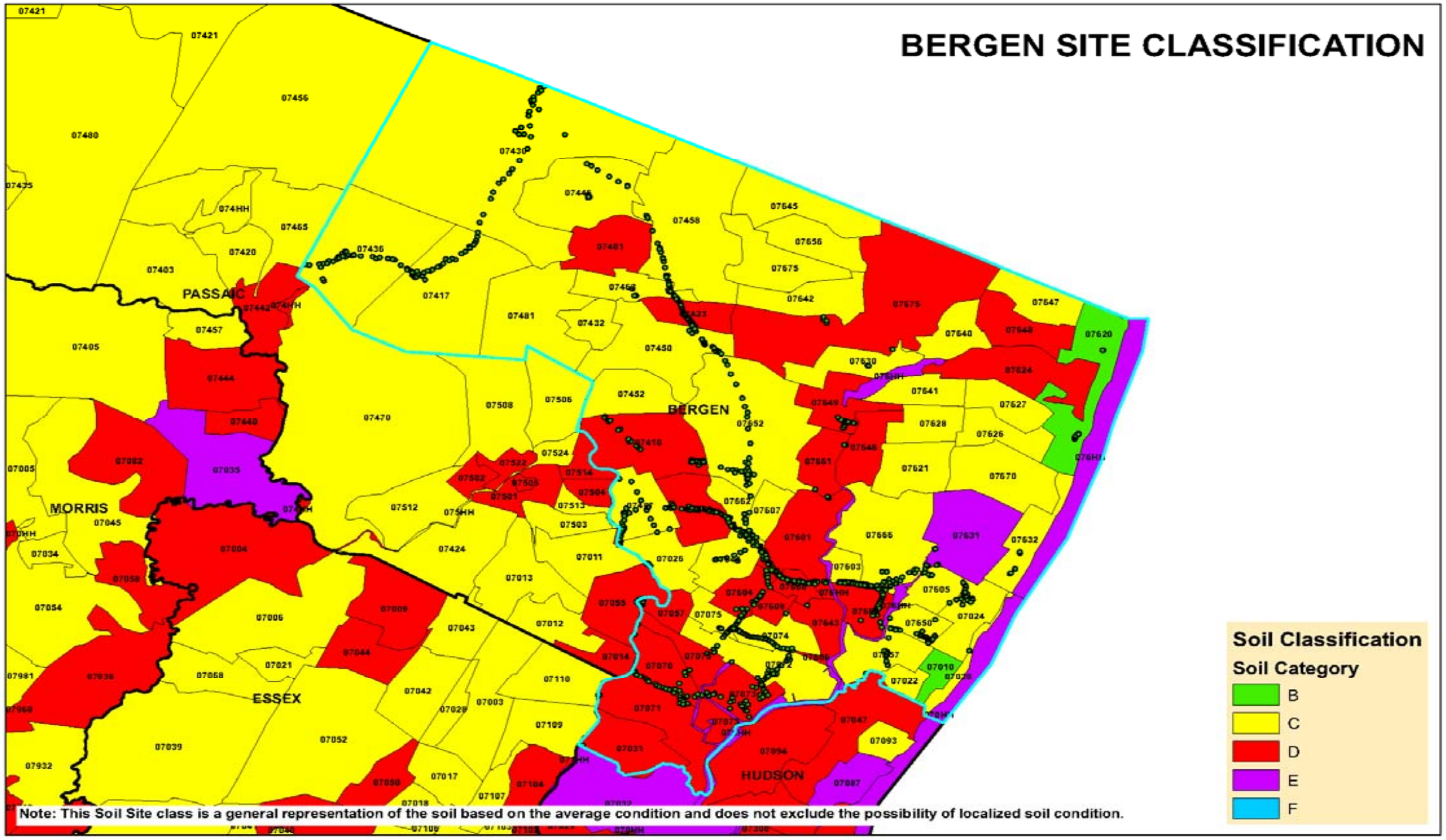


Figure II.7 Soil Site Class Map for Bergen County of New Jersey

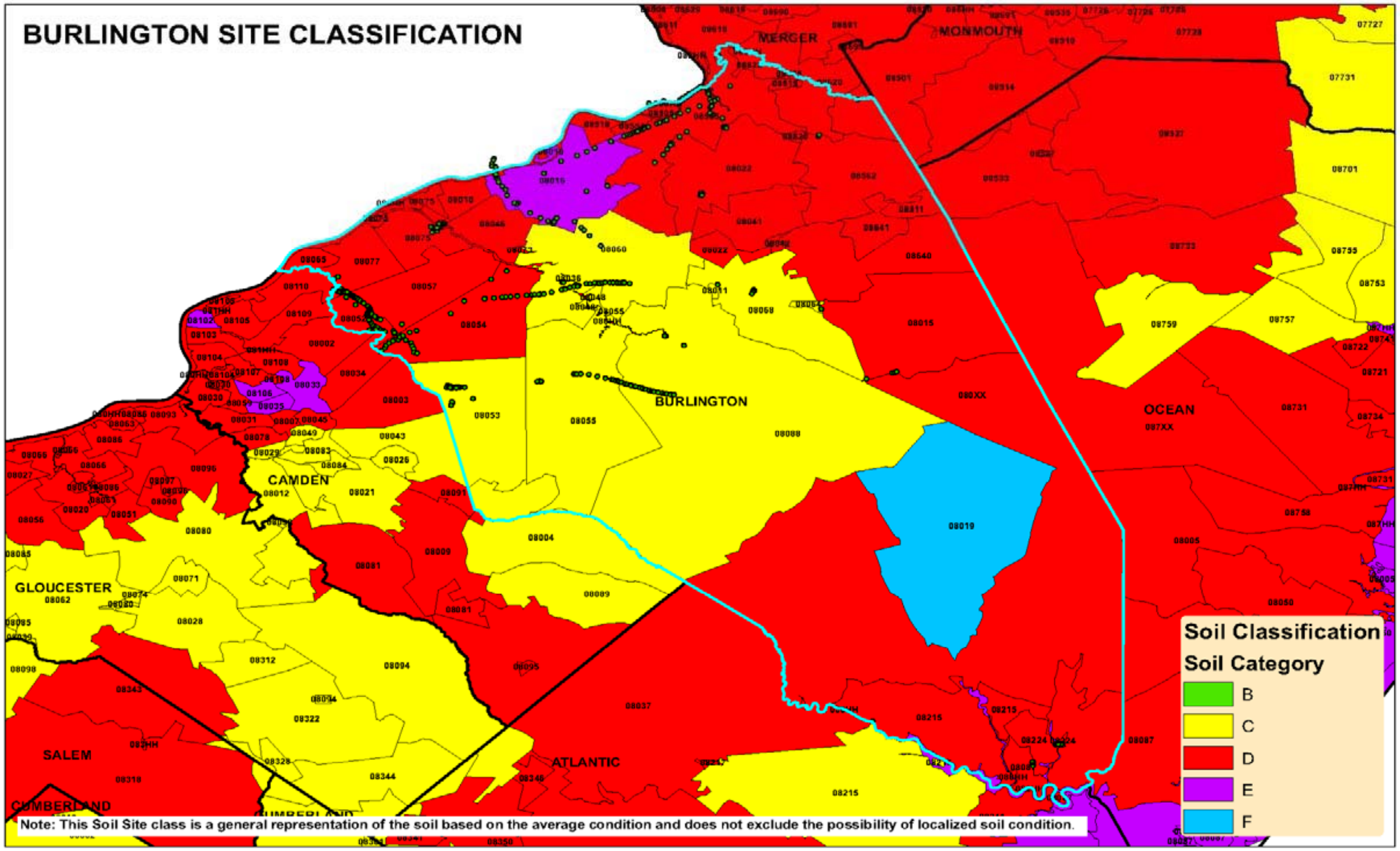


Figure II.8 Soil Site Class Map for Burlington County of New Jersey

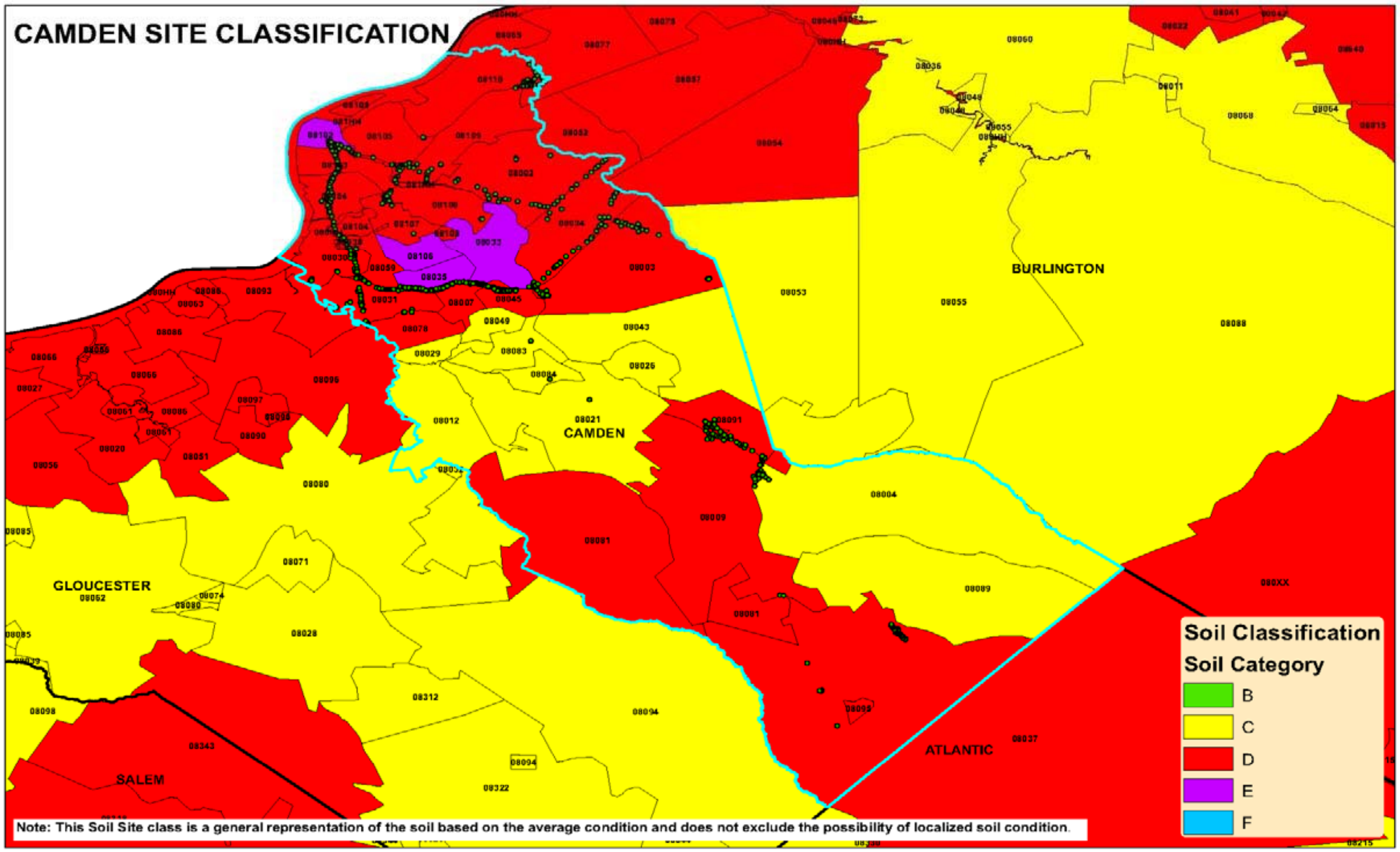


Figure II.9 Soil Site Class Map for Camden County of New Jersey

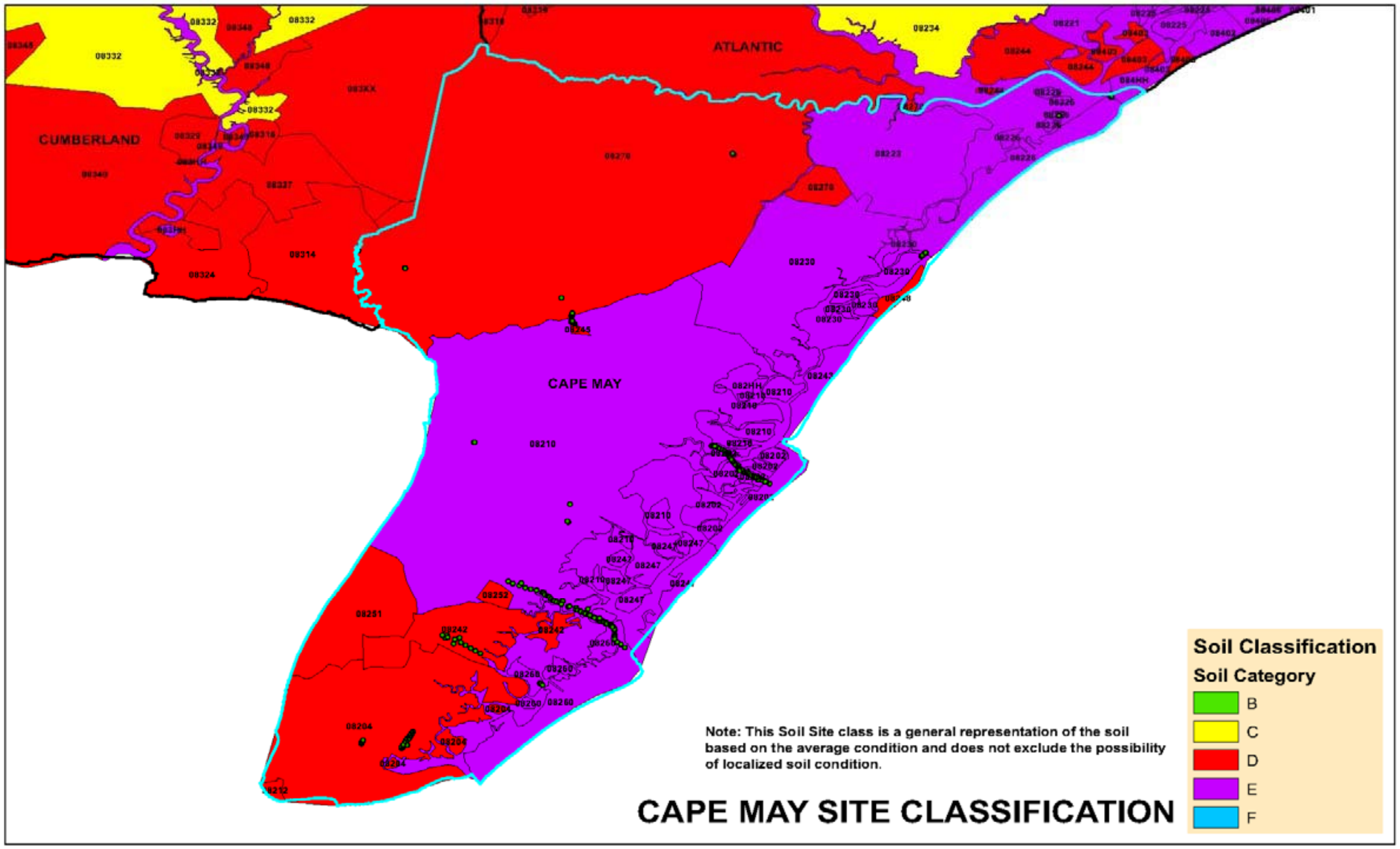


Figure II.10 Soil Site Class Map for Cape May County of New Jersey

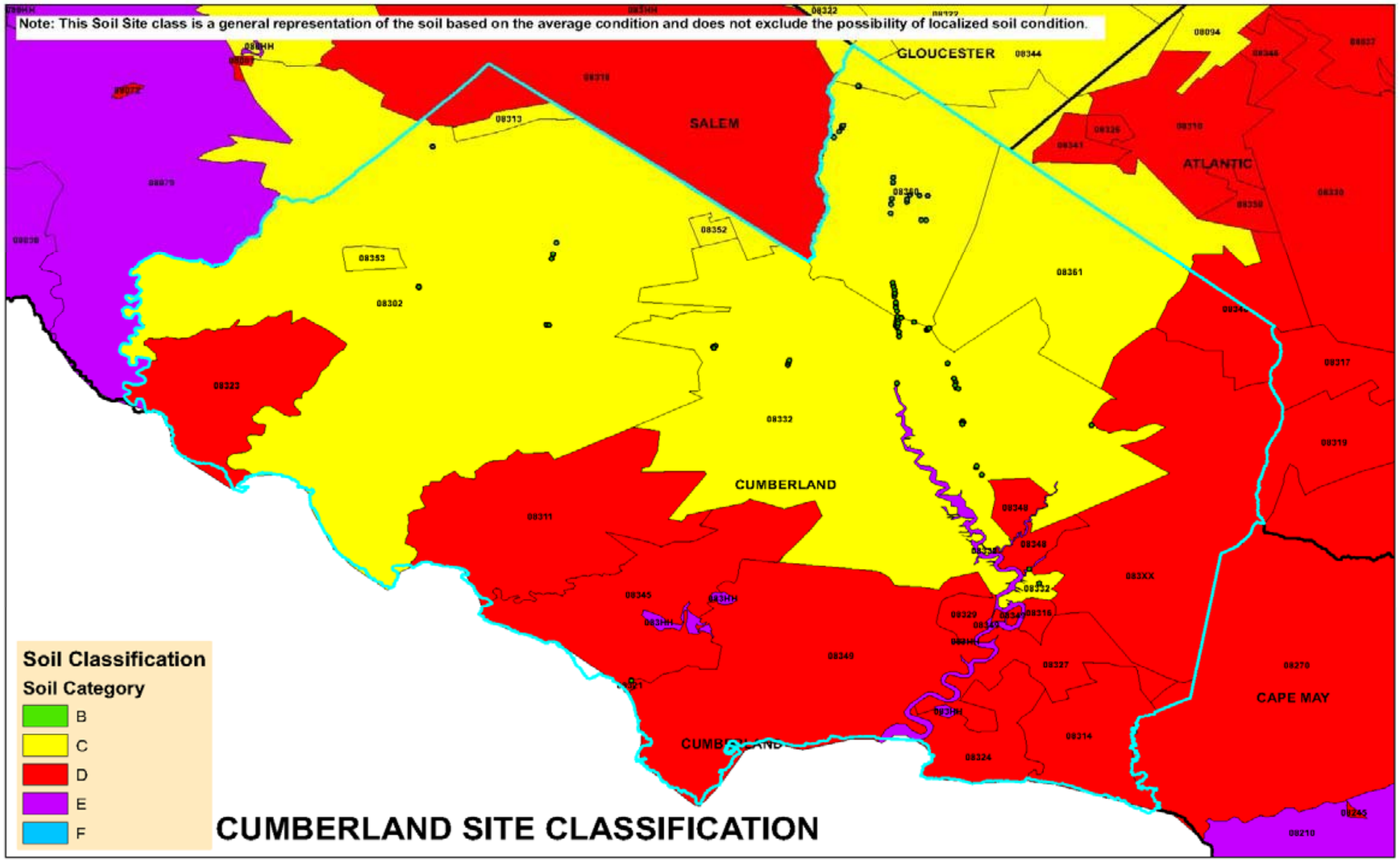


Figure II.11 Soil Site Class Map for Cumberland County of New Jersey

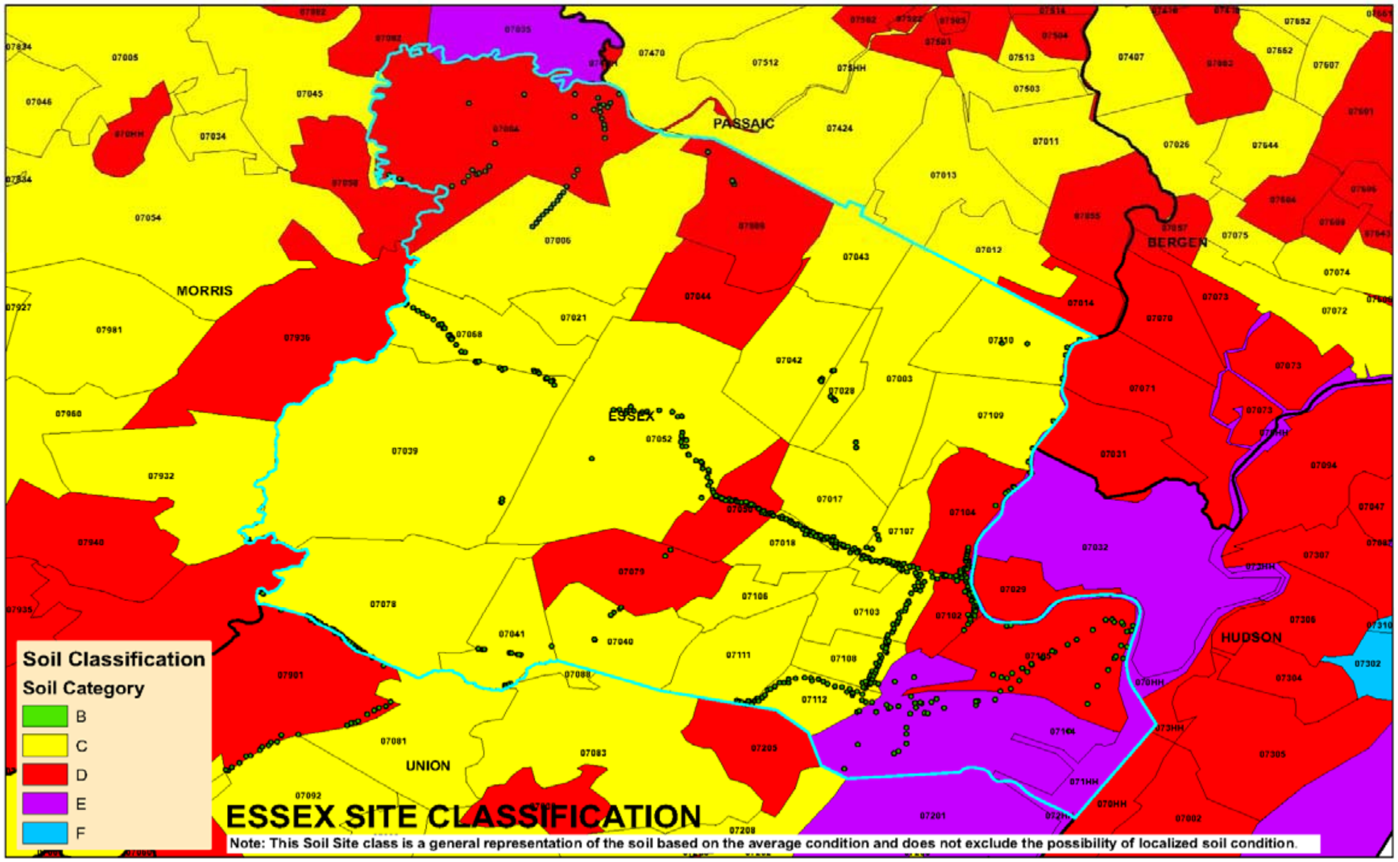


Figure II.12 Soil Site Class Map for Essex County of New Jersey

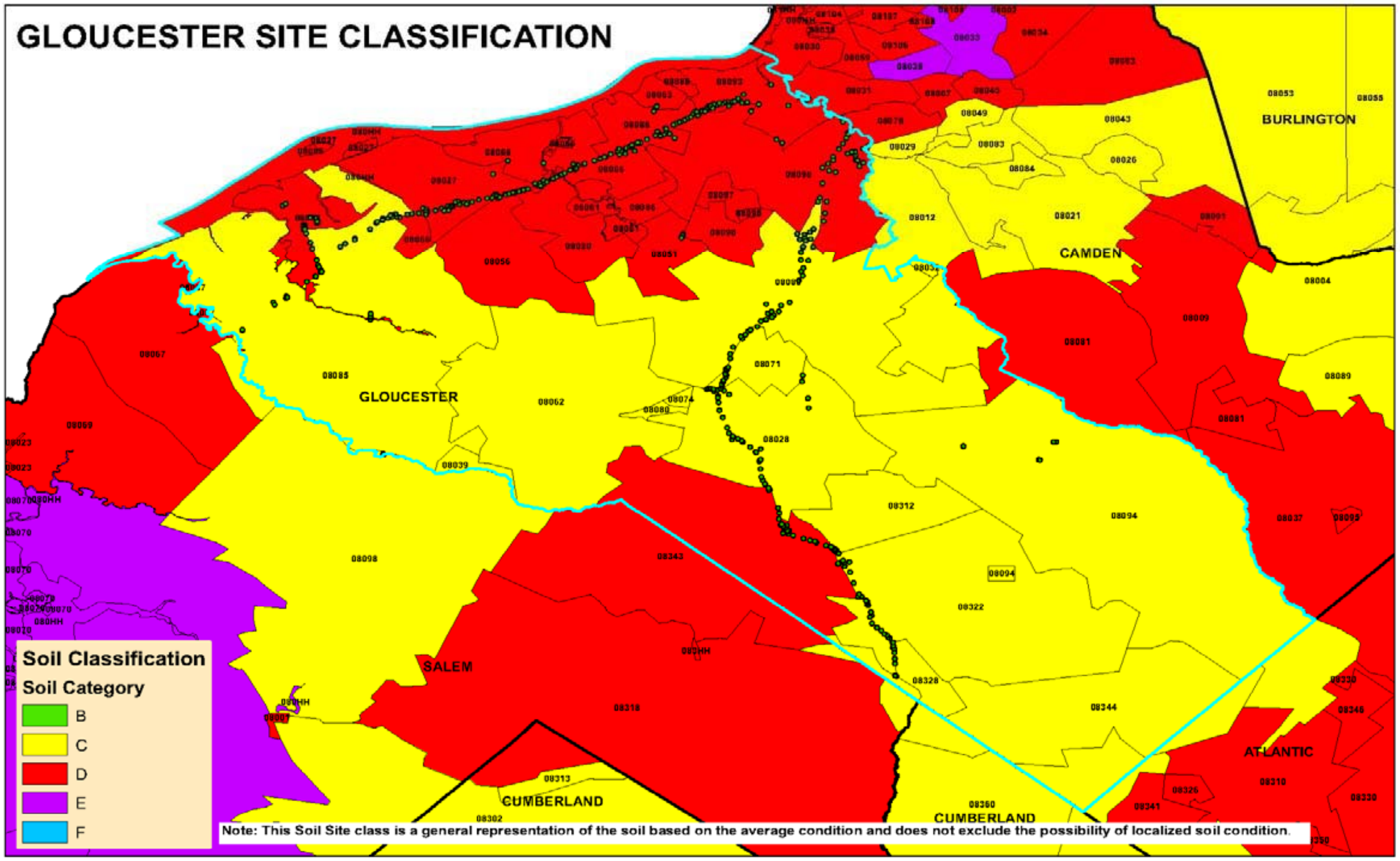


Figure II.13 Soil Site Class Map for Gloucester County of New Jersey

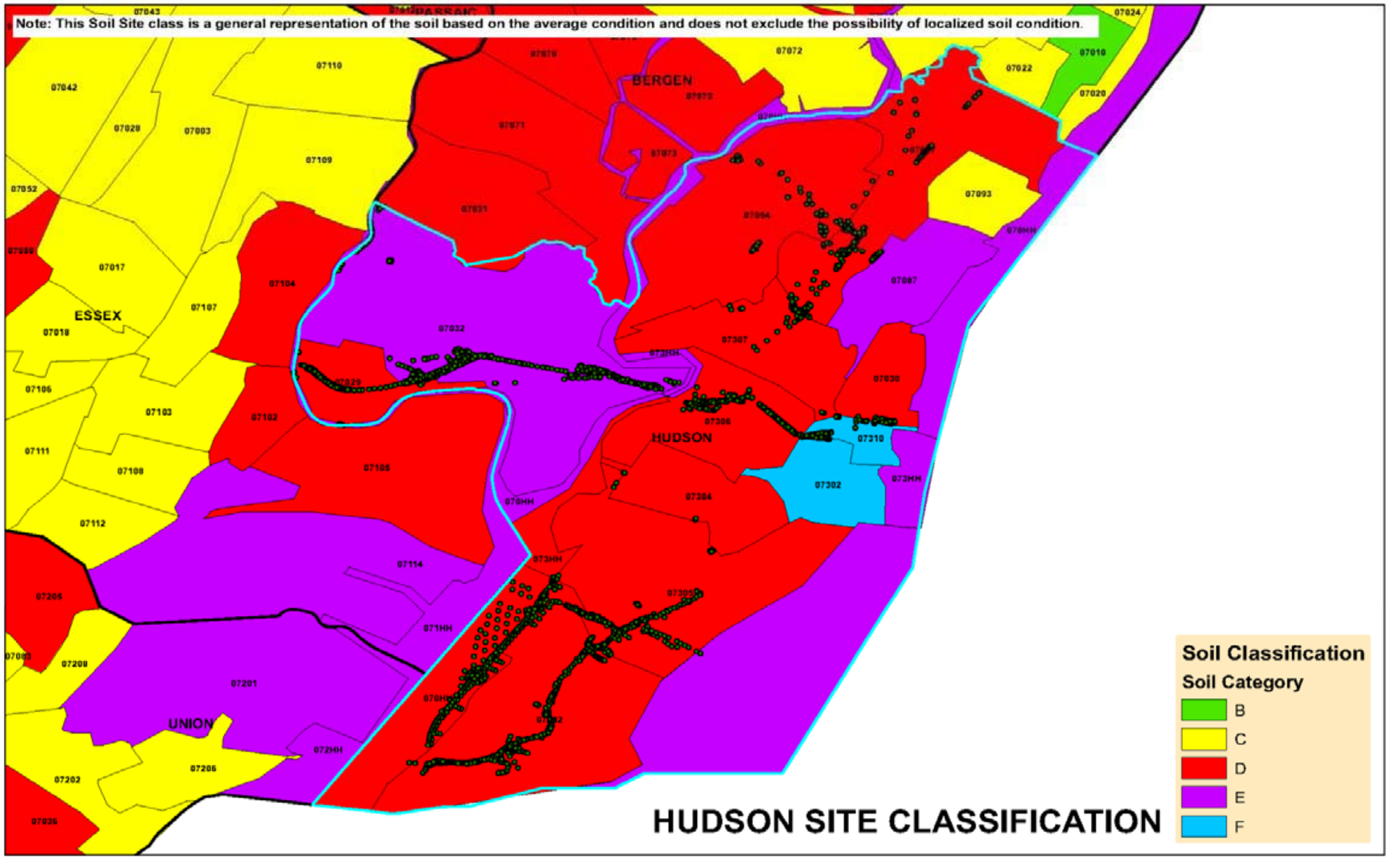


Figure II.14 Soil Site Class Map for Hudson County of New Jersey

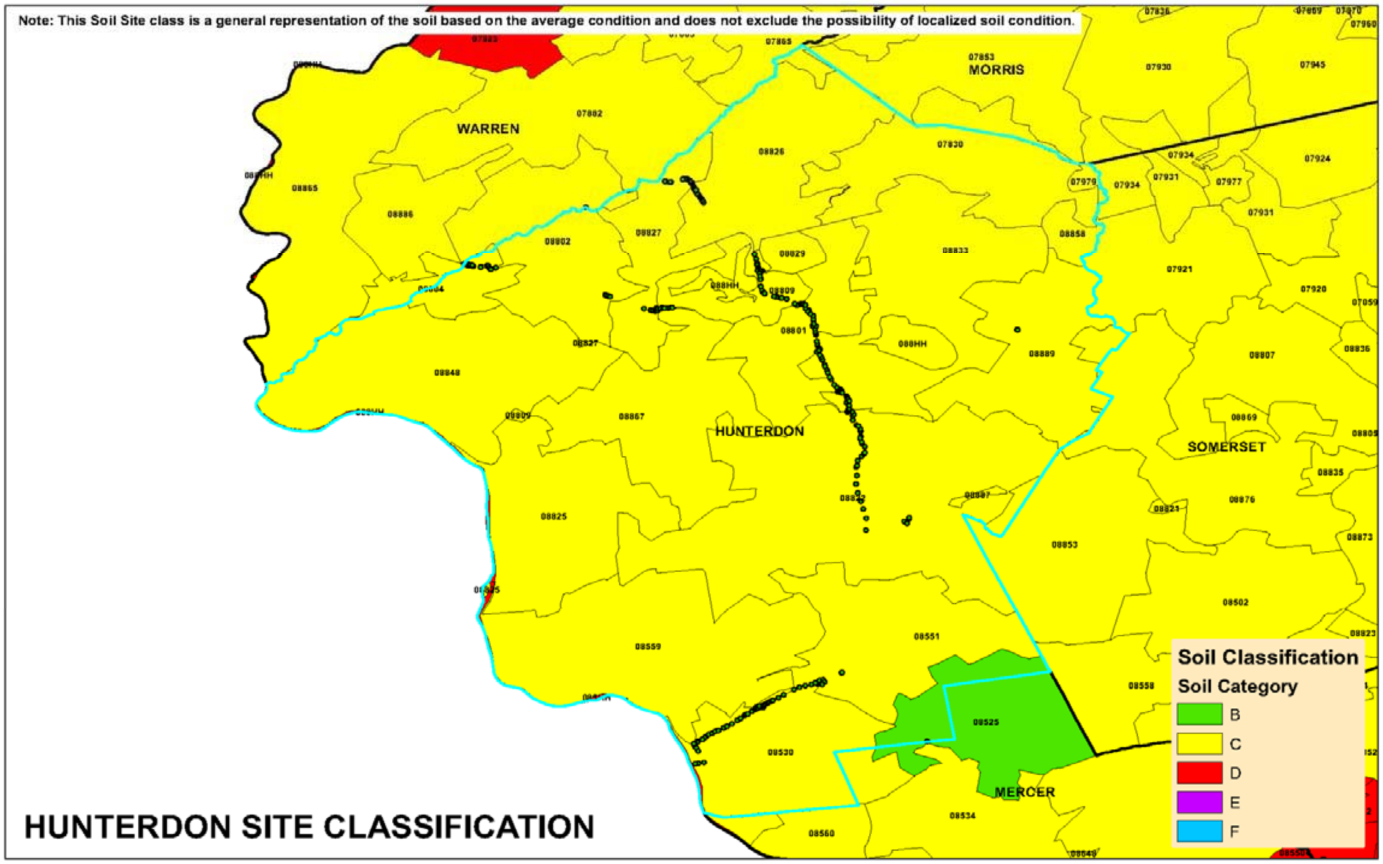


Figure II.15 Soil Site Class Map for Hunterdon County of New Jersey

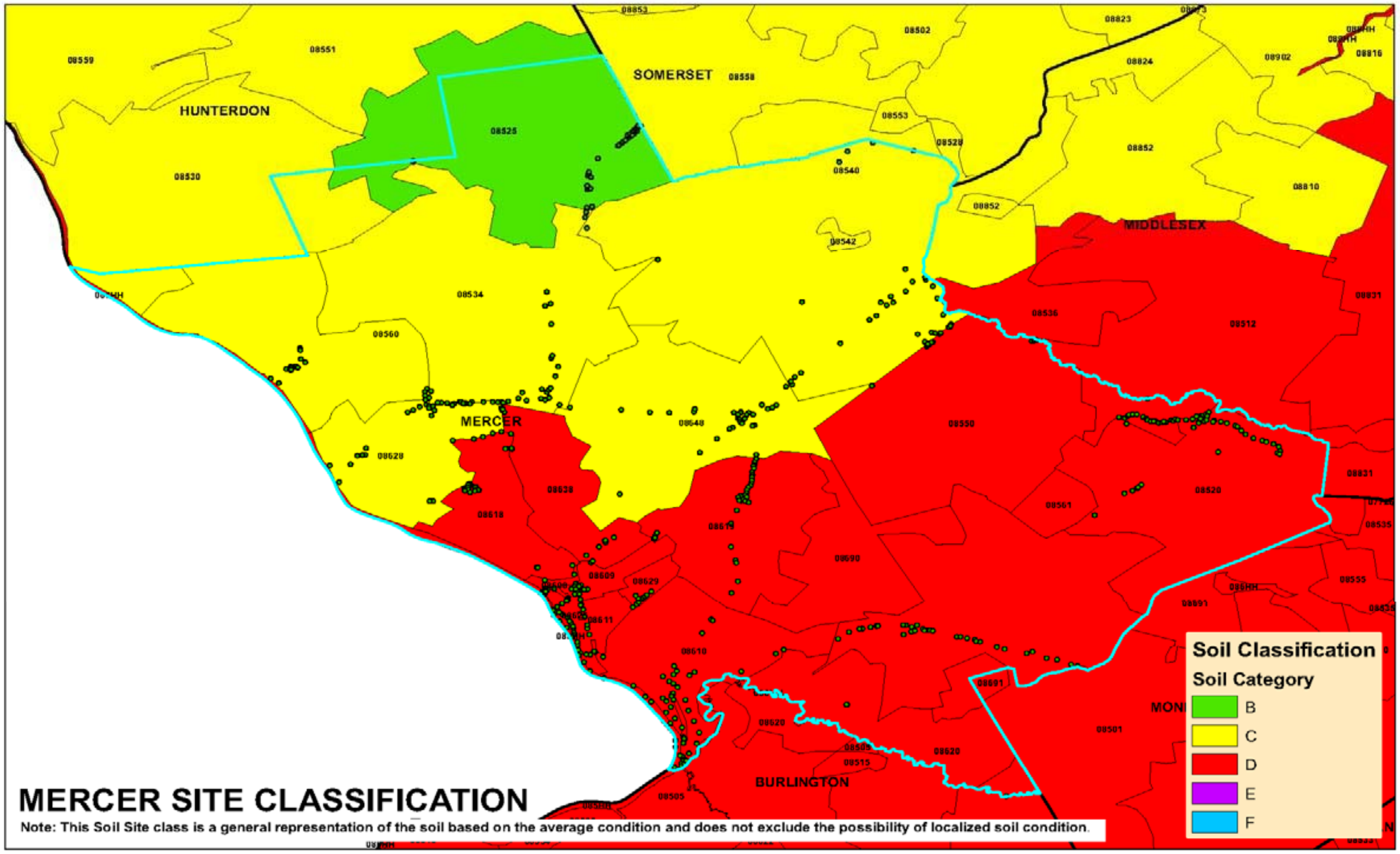


Figure II.16 Soil Site Class Map for Mercer County of New Jersey

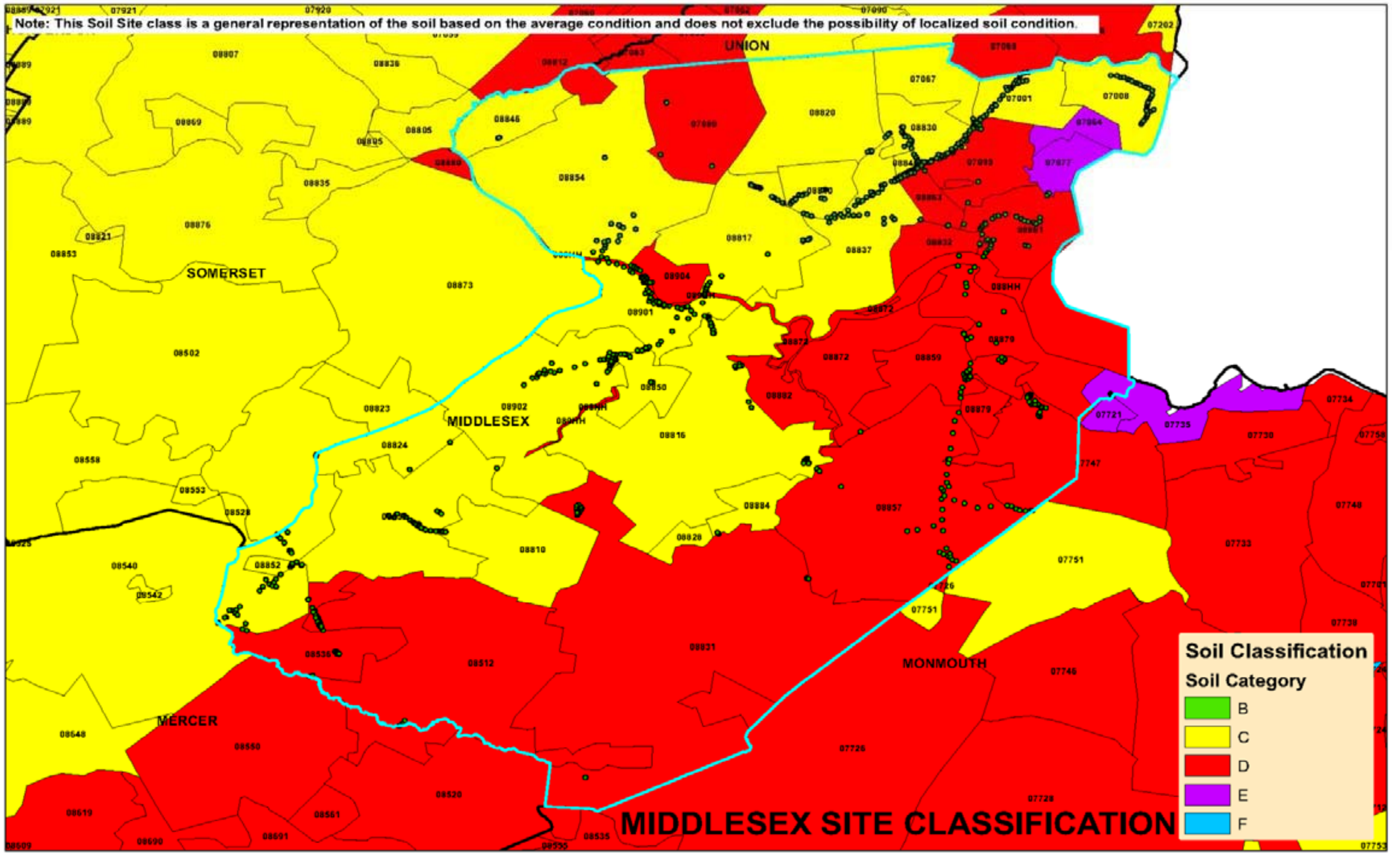


Figure II.17 Soil Site Class Map for Middlesex County of New Jersey

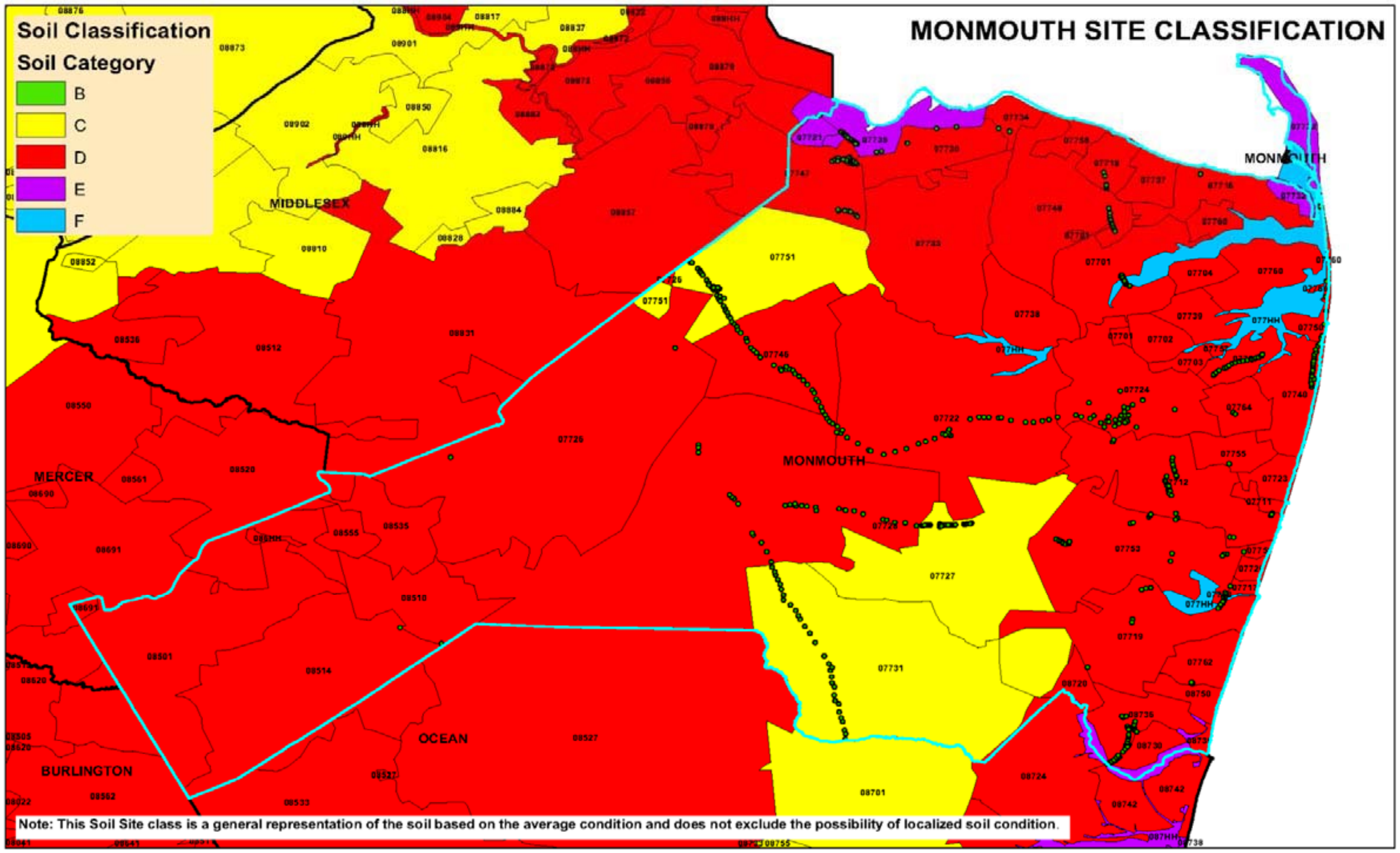


Figure II.18 Soil Site Class Map for Monmouth County of New Jersey

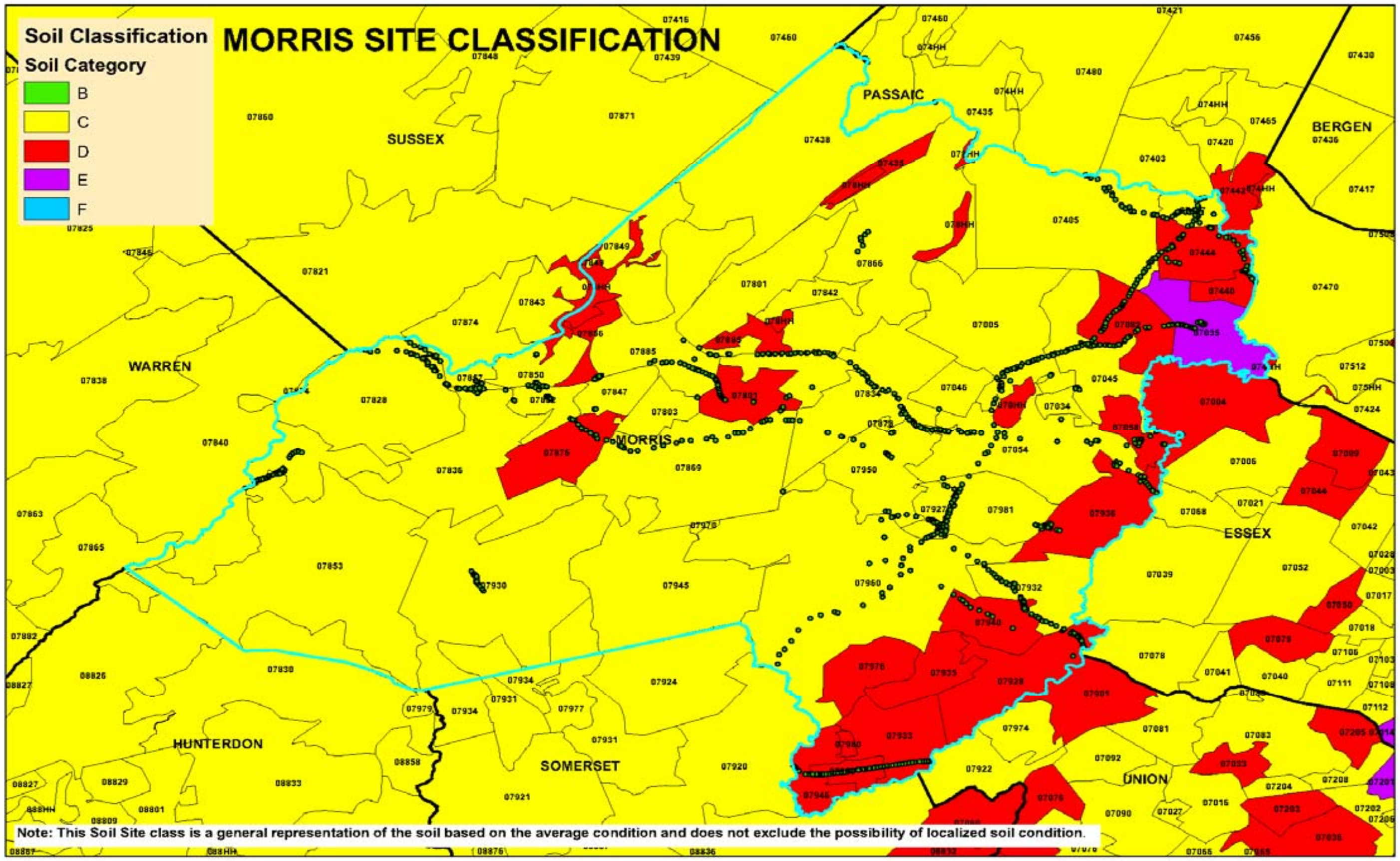


Figure II.19 Soil Site Class Map for Morris County of New Jersey

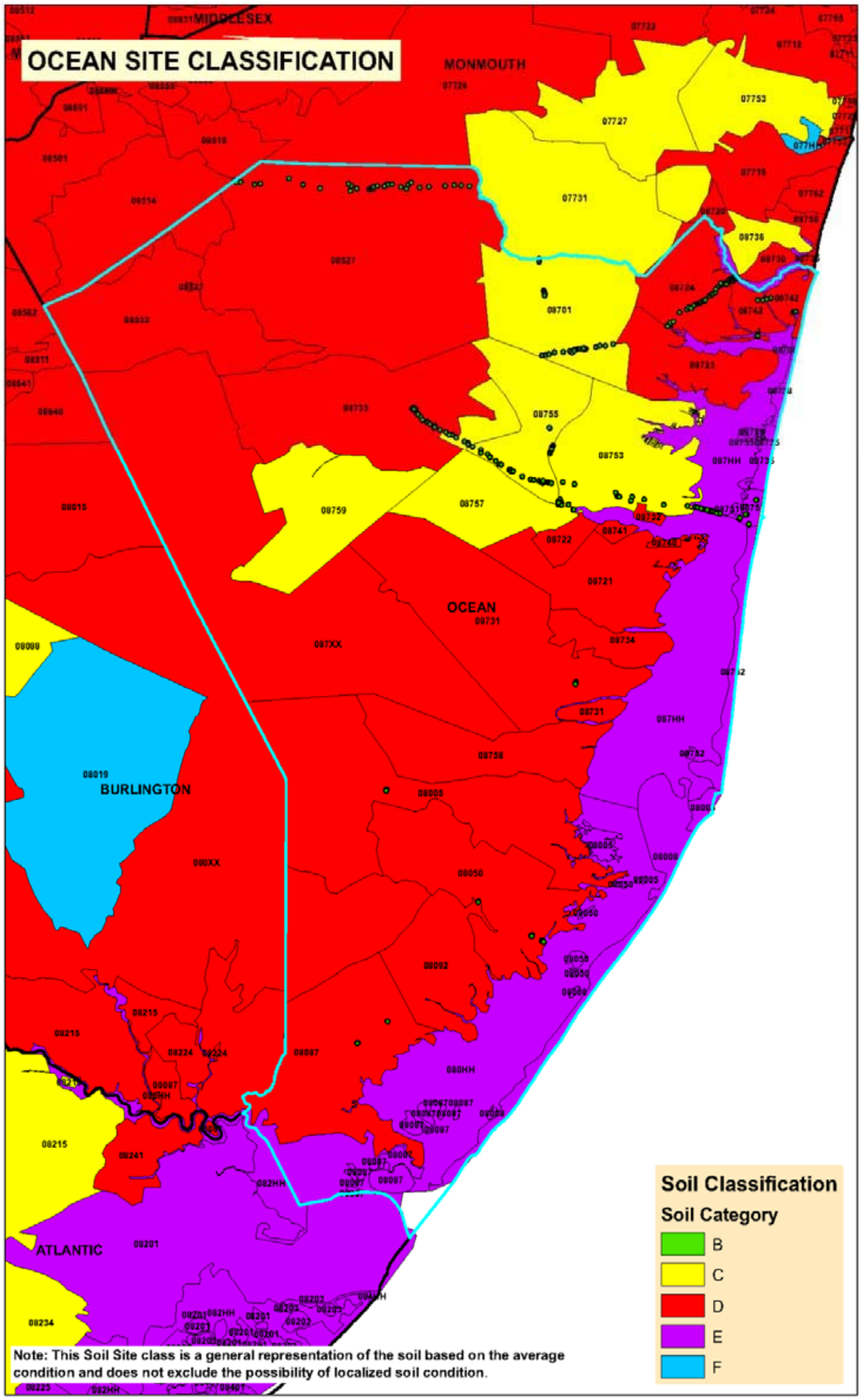


Figure II.20 Soil Site Class Map for Ocean County of New Jersey

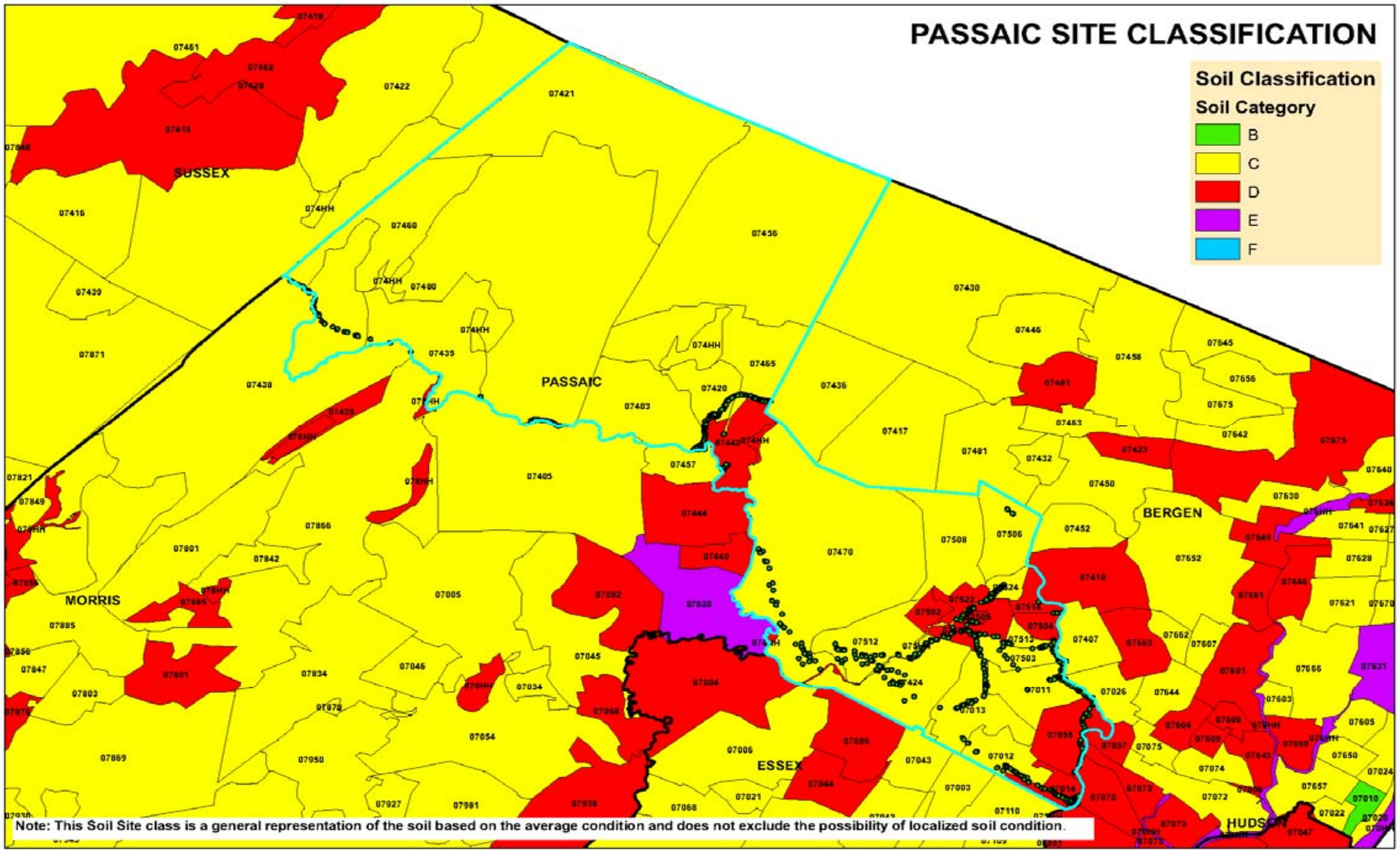


Figure II.21 Soil Site Class Map for Passaic County of New Jersey

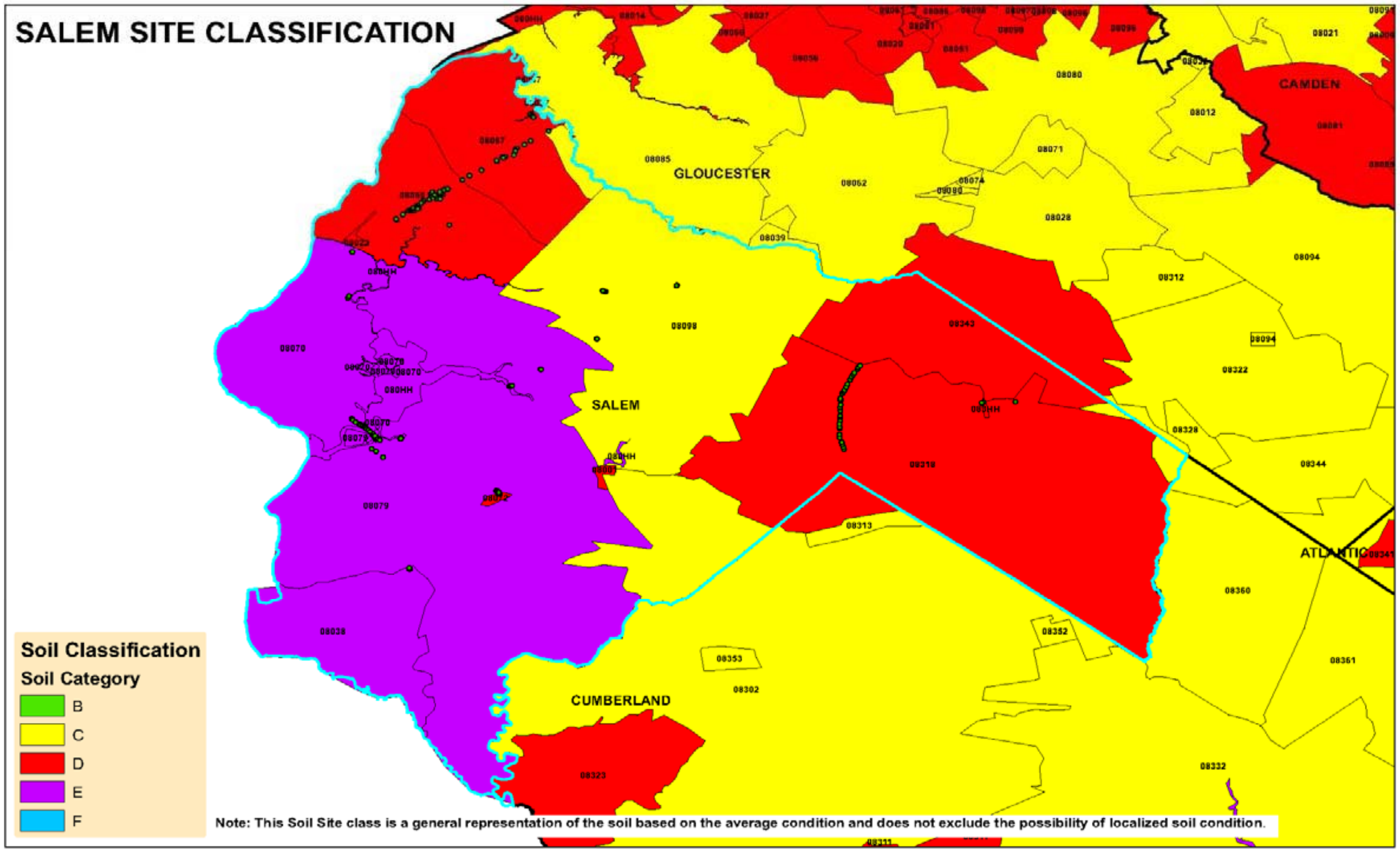


Figure II.22 Soil Site Class Map for Salem County of New Jersey

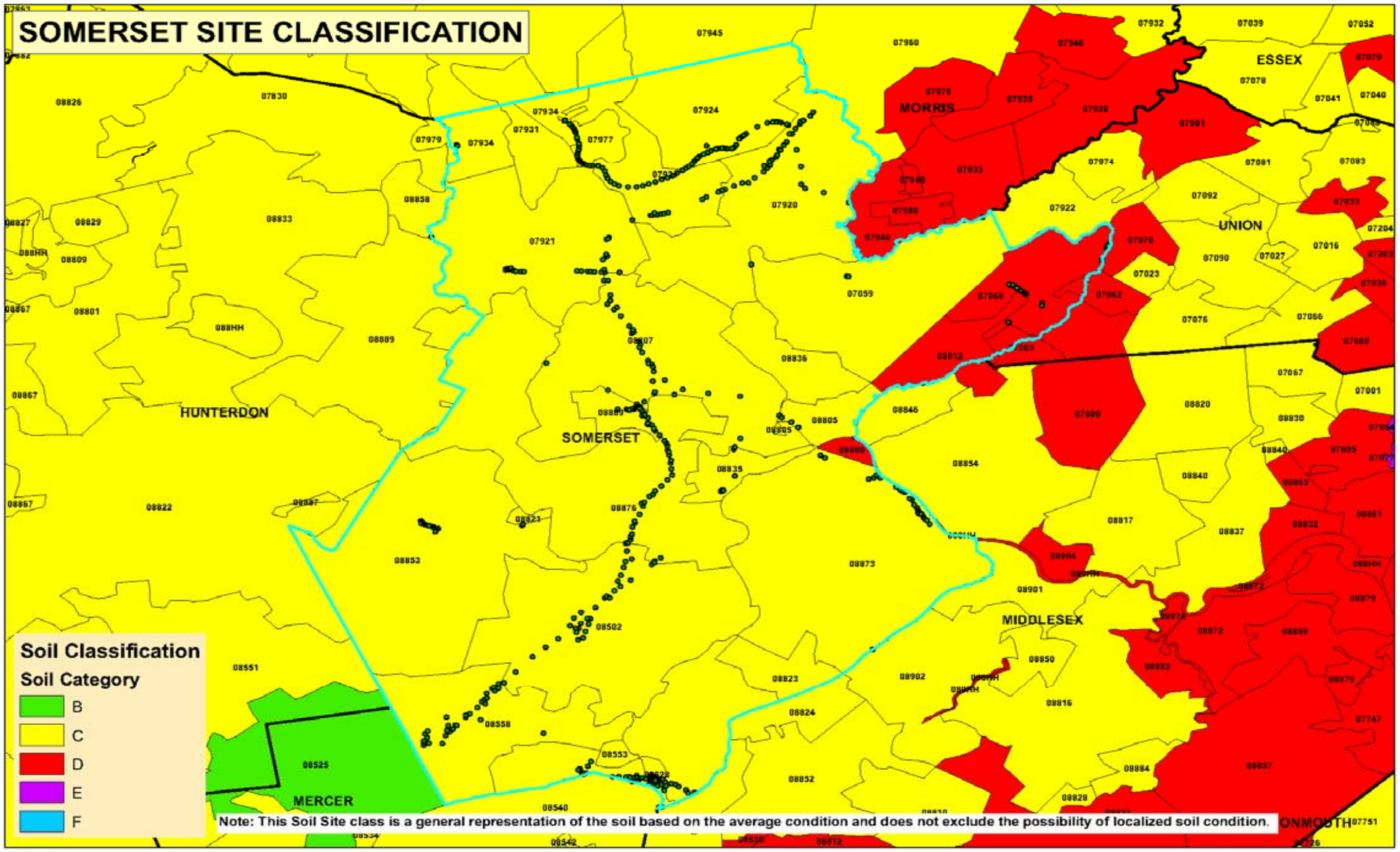


Figure II.23 Soil Site Class Map for Somerset County of New Jersey

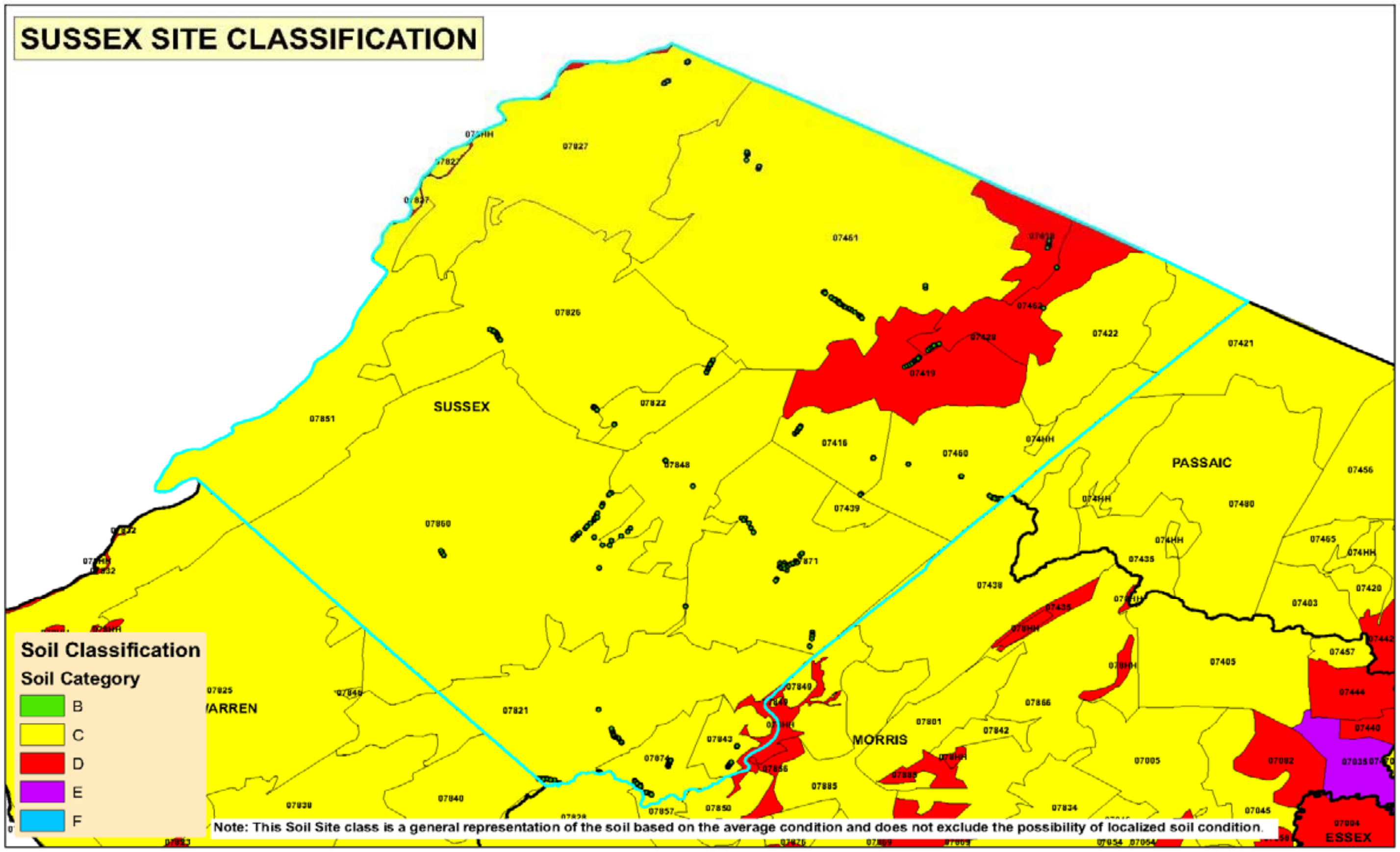


Figure II.24 Soil Site Class Map for Sussex County of New Jersey

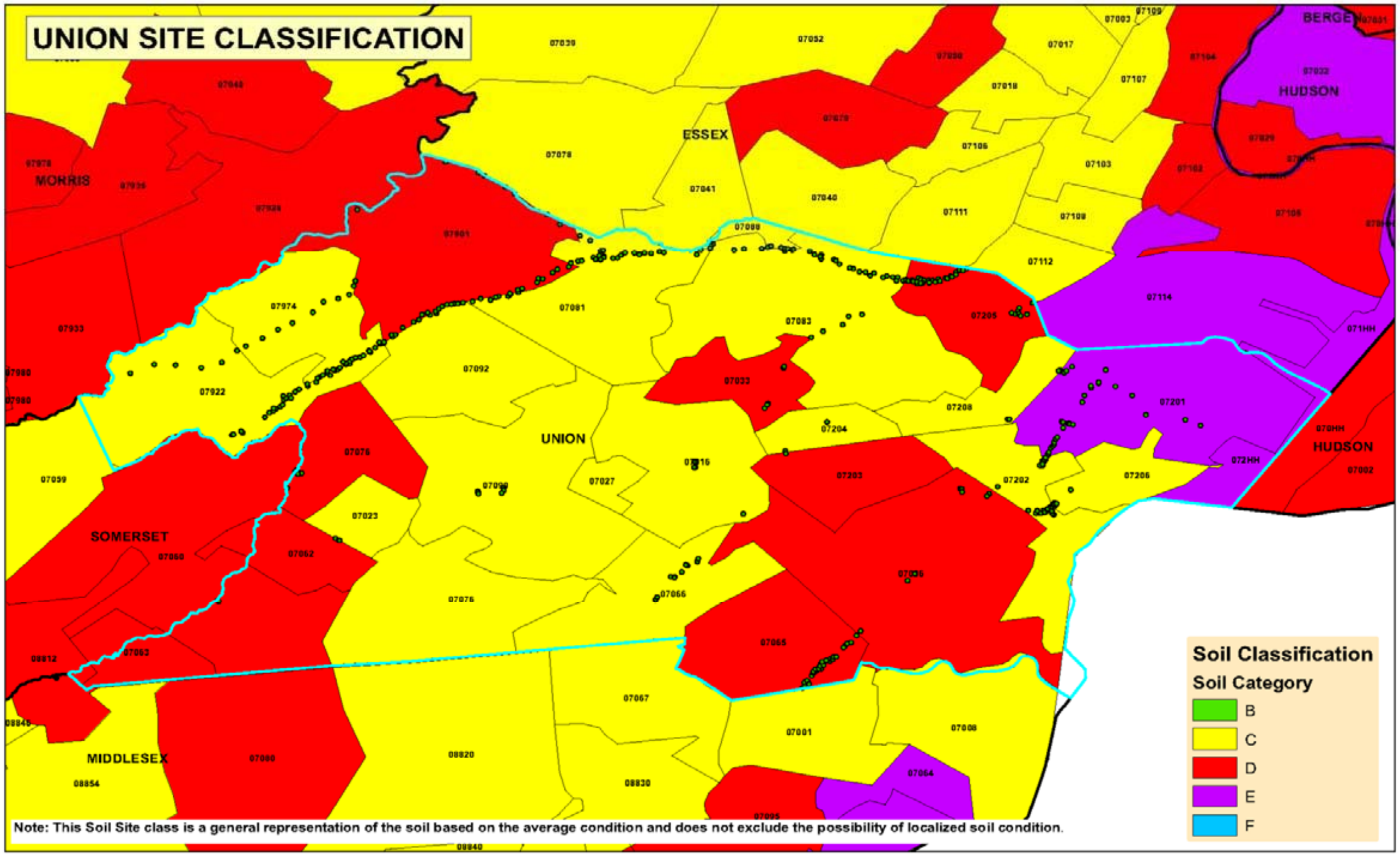


Figure II.25 Soil Site Class Map for Union County of New Jersey

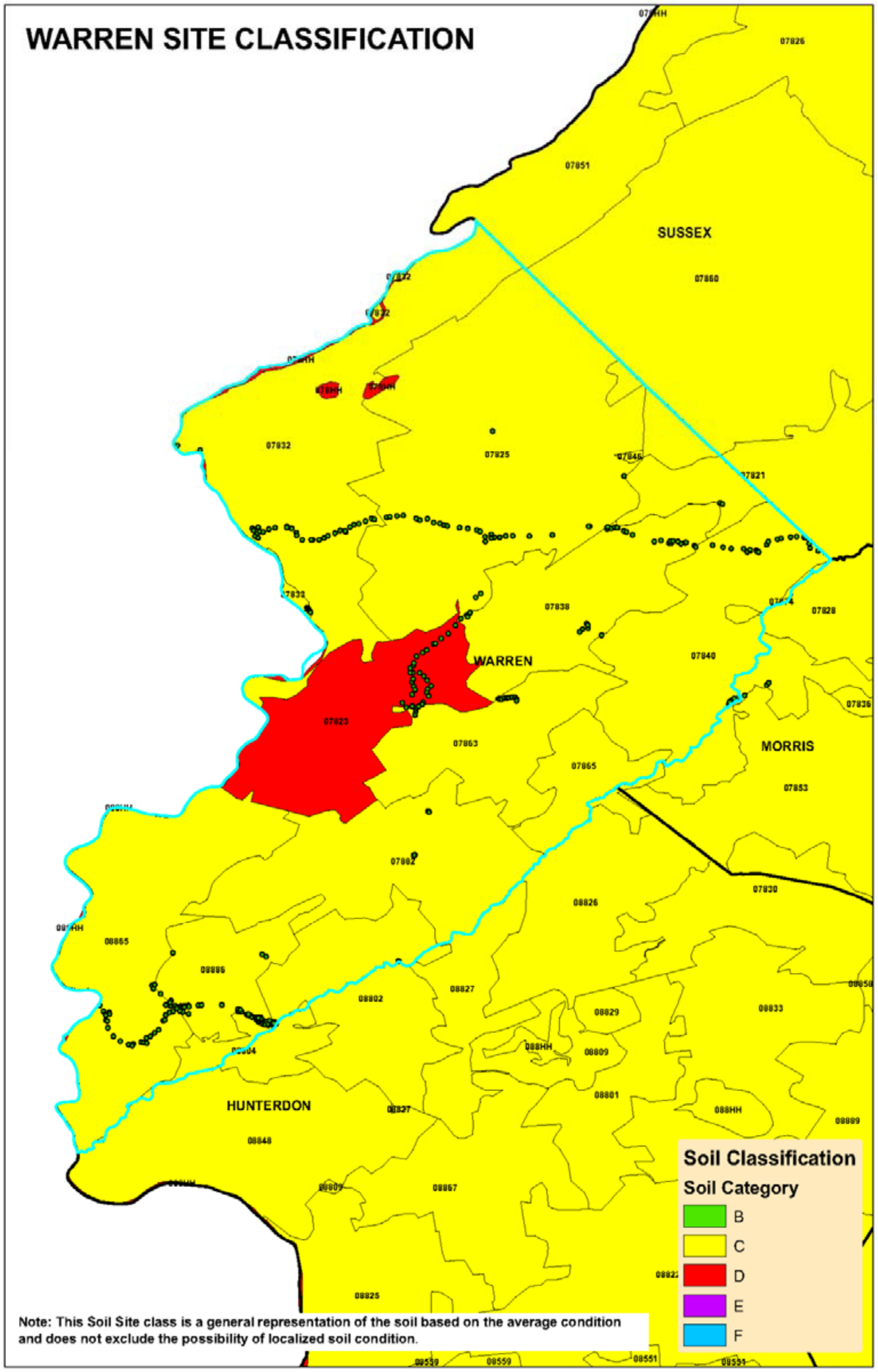


Figure II.26 Soil Site Class Map for Warren County of New Jersey

APPENDIX III: SEISMIC DESIGN CATEGORY MAPS, LIQUEFACTION ANALYSIS MAPS AND SITE-SPECIFIC ANALYSIS PROCEDURE FOR NEW JERSEY

Procedure for Liquefaction Potential Analysis

Sources of Data

The liquefaction analysis relied on two sources of data:

1) The main source is the NJDOT soil borings database, which consists of most of the boring logs from past transportation projects throughout the state of New Jersey (http://www.state.nj.us/transportation/refdata/geologic/). SPT blow counts and soil descriptions at different depths are available in the boring logs. However, the available boring logs are not evenly distributed in each zip code in New Jersey. There are zip codes that do not have boring logs in the database at present. For the generation of Soil Site Class map, boring logs were selected to conduct soil site classification analysis (detailed procedure outlined in the report for Task 9). These boring logs were again used to conduct liquefaction hazard analysis.

2) The second source of data is the Surficial Geological Map developed by New Jersey Geological Survey (NJGS) (http://www.state.nj.us/dep/njgs/geodata/dgs07-2.htm), which consists of the surficial soil descriptions and their depths in the State. This map was used in conjunction with the boring log database to yield a clear picture of soil type and the corresponding liquefaction hazard.

Liquefaction Potential Analysis Using SPT Blow Counts

The approach for screening liquefaction potential using SPT blow counts at a site is summarized in Youd et al. (2001). It is briefly introduced in the following.
At a specific site, if the peak horizontal acceleration a_{max} on ground surface as well as the earthquake magnitude is already determined, the cyclic stress ratio (CSR) is defined as:

$$CSR = 0.65(a_{\max}/g)\left(\sigma_{v0}/\sigma'_{v0}\right)r_d \tag{III.1}$$

where g is the gravitational coefficient, σ_{v0} and σ'_{v0} are the total and effective overburden stress, respectively, and r_d is the stress reduction factor (depth z is in the unit of meter):

$$\begin{aligned} r_d &= 1.0 - 0.00765z \quad \text{for } z \le 9.15m \\ r_d &= 1.174 - 0.0267z \quad \text{for } z > 9.15\text{m} \end{aligned} \tag{III.2}$$

The cyclic resistance ratio of granular soil at M_w (earthquake magnitude) = 7.5 is:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10\cdot(N_1)_{60}+45]^2} - \frac{1}{200} \tag{III.3}$$

in which $(N_1)_{60}$ is the corrected SPT blow counts, which takes into consideration the effects of overburden pressure, SPT device, bore hole parameters and fine contents. With these two ratios, the factor of safety against liquefaction is evaluated as:

$$FS = (CRR_{7.5} / CRS)MSF \tag{III.5}$$

where MSF is the scaling factor of earthquake magnitude:

$$MSF = 10^{2.24} / M_w^{2.56} \tag{III.6}$$

General Procedure for Liquefaction Potential Analysis of a Borehole

Assumptions

Overall the procedure tried to utilize as much as possible the available information and at the same time consider adequate conservativeness. The procedure is based on what is outlined above. However, since some information necessary for liquefaction potential analysis is not available, some conservative assumptions were considered in the analysis regarding the ground condition:

1) Unit weight of soil. The saturated or dry unit weight of soil is generally not provided in the boring logs analyzed. A conservative number of 121 pcf (or 19 kN/m^3) was assumed as the average unit weight in all boring logs to calculate the overburden total and effective stresses.

   Fine content of granular soils. The soil description in the boring log generally does not consist of the detailed information on grain-size-distribution. Instead, descriptive words such as “trace of”, “little” or “some” silt are used. In this study, the following percentages were assigned to these descriptive words:

Trace of silt	fine content of 5%
Little silt	fine content of 10%
Some silt	fine content of 20%.

   If a layer of soil is described as non-cohesive silt, a fine content of 35% was conservatively considered, according to the soil classification of AASHTO.

2) Water table. If water table is given in a boring log, it was used in the analysis; however, if the information is not provided, it is assumed that the water table is at the ground surface.

Input Data for Liquefaction Potential Analysis

For a specific borehole, the inputs for liquefaction potential analysis are obtained according to the following procedure:

1) ASSHTO GM 2.1 was used to obtain the peak ground acceleration (PGA) of each zip code on the outcrop surface of Site Class B rock.

2) The value was scaled to consider soil site effects. Considering the low seismicity in New Jersey, only Soil Site Class D and Class E were analyzed for liquefaction their potentials. Soil Site Class C was assumed to be non-liquefiable considering the large average SPT blow counts.

3) The a_{max}'s for liquefaction analysis of Site Classes D and E were obtained according to Table III.1. The amplification factors were decided based on Table III.2. According to AASHTO GM 2.1, PGA's of New Jersey for standard bridges are generally smaller than or equal to 0.1g, hence the amplification factors for Site Class D and Class E in column 1 of Table III.3 was used directly; on the other hand, for critical/essential bridges, a factor of 1.5 was applied to PGA. The amplification factors were then interpolated from what is directly specified in Table III.2.

Table III.1 a_{max} for liquefaction analysis

	Site Class D	Site Class E
Standard bridges (1000-year earthquake)	a_{max}=1.6*PGA. 1.6 is the amplification factor of Site Class D for PGA when PGA $\le$ 0.1g.	a_{max} =2.5*PGA. 2.5 is the amplification factor of Site Class E for PGA when PGA $\le$ 0.1g.
Critical bridges (1000 Yr times 1.5)	Two scenarios: 1) if 1.5*PGA < 0.1g, a_{max}= 1.6*1.5* PGA; 2) If 0.1g<1.5*PGA < 0.2g, a_{max} = 1.4+ [0.2-1.5*PGA] /0.1*0.2.	Two Scenarios: 1) if 1.5*PGA < 0.1g, a_{max}= 2.5*1.5* PGA; 2) If 0.1g<1.5*PGA < 0.2g, a_{max} = 1.7+ [0.2-1.5*PGA] /0.1*0.8.

4) Earthquake magnitude M_w is needed to calculate the factor of safety FS in Equation (4.7). It is proposed that M_w = 6.0 for both standard and critical/essential bridges. According to the deaggregations of PGA in New Jersey (http://eqint.cr.usgs.gov/deaggint/), the mean magnitude M_w of earthquake in New Jersey for a return period of 1000 year is between 5.5 and 5.9, depending on the specific location. The magnitude increases slightly for a return period of 2500 years. On the other hand, according to Youd et al. (2001), the accuracy of liquefaction potential based on the empirical procedure when M_w < 6.0 is smaller. Therefore, to

be conservative, it is suggested that M_w = 6.0 for both levels of earthquake hazard while screening the liquefaction potential for new bridges in New Jersey.

5) The SPT blow counts as found in available boring logs were corrected according to Youd et al. (2001) for SPT device and borehole parameters if they are available. Otherwise, the blow counts were assumed to be standard.

Table III.2 Factors of PGA according to site class (according to AASHTO LRFD)

Site Class	Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods				
	PGA≤ 0.10 S_s	$PGA = 0.20$ S_s	$PGA = 0.30$ S_s	$PGA = 0.40$ S_s	$PGA \geq 0.50$ S_s
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

Analysis procedure

An excel spread sheet has been developed to implement the procedure described above. The boring log was divided into layers according to the soil description and SPT blow counts, and the factor of safety FS against liquefaction was analyzed for each layer. A sample spread sheet is shown in Figure III.1.

SUBJECT: **Liquefaction Evaluation**

CALCULATED BY: HB DATE: 10/9/2008 CHECKED BY: DATE:

REFERENCES :
1. AASHTO LRFD Bridge Design Specifications, (2004, Third Edition with 2005, 2006 Interims)
2. Youd, T. L., et. Al, (2001), "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol, 127, No. 10, pp 817-833

Boring No.: B43683 Ground Water Depth: 1.00 ft = 0.305 m.
Earthquake Moment Magnitude, M_w: 6.0 Peak Ground Acceleration at Ground Surface, a_{max} (g): 0.16
C_E, Correction for Hammer Energy Ratio: 1.00 C_B, Correction Factor for Borhole Diameter: 1.00
Note: Hammer Type: Donut **Note:** Borehole Diameter (inch): 2.50
Length of Drilling Rod above Ground: 3.00 ft = 0.914 m
C_s, Correction For Sampler with or without Liners: 1.0 Standard Sampler? (Yes or Not) Yes
Average Unit Weight of Soil: 121.0 pcf 19.00014 kN/m^3
Earthquake Magnitude Scale Factor, MSF: $MSF = 10^{2.24} / M_w^{2.56}$ = **1.770**

Depth (ft)	Effective Overburden Pressure (kPa)	SPT Blow Counts (blows/ft)	Fine Contents (%)	Over Burden Correction, C_N	$(N_1)_{60}$	$(N_1)_{60cs}$	CSR	CRR	FS	Liquefaction?
1.0	5.79	18	10.00	1.700	23.0	24.3	0.104	0.494	4.75	**No**
6.0	19.80	5	10.00	1.577	5.9	6.9	0.180	0.154	0.85	**Yes**
11.0	33.80	7	10.00	1.435	8.5	9.6	0.192	0.194	1.01	**No**
16.0	47.81	3	10.00	1.316	3.4	4.3	0.195	0.119	0.61	**Yes**
21.0	61.81	15	10.00	1.215	17.3	18.6	0.195	0.351	1.80	**No**
26.0	75.82	0	10.00	1.129	0.0	0.9	0.194	0.087	0.45	**Yes**
27.0	78.62	9	10.00	1.113	9.5	10.6	0.193	0.209	1.08	**No**
28.0	81.42	5	10.00	1.098	5.2	6.2	0.193	0.144	0.75	**Yes**
30.0	87.03	49	10.00	1.069	52.4	54.4	0.191	Large N	Large N	**No, (Large SPT Value)**
36.0	103.83	85	10.00	0.989	84.1	86.8	0.185	Large N	Large N	**No, (Large SPT Value)**
41.0	117.84	71	20.00	0.931	66.1	75.0	0.177	Large N	Large N	**No, (Large SPT Value)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**
	0.00			1.700	0.0	0.0	#DIV/0!	Above Water	Above Water	**No (above water)**

Figure III.1 Sample spread sheet for liquefaction potential analysis.

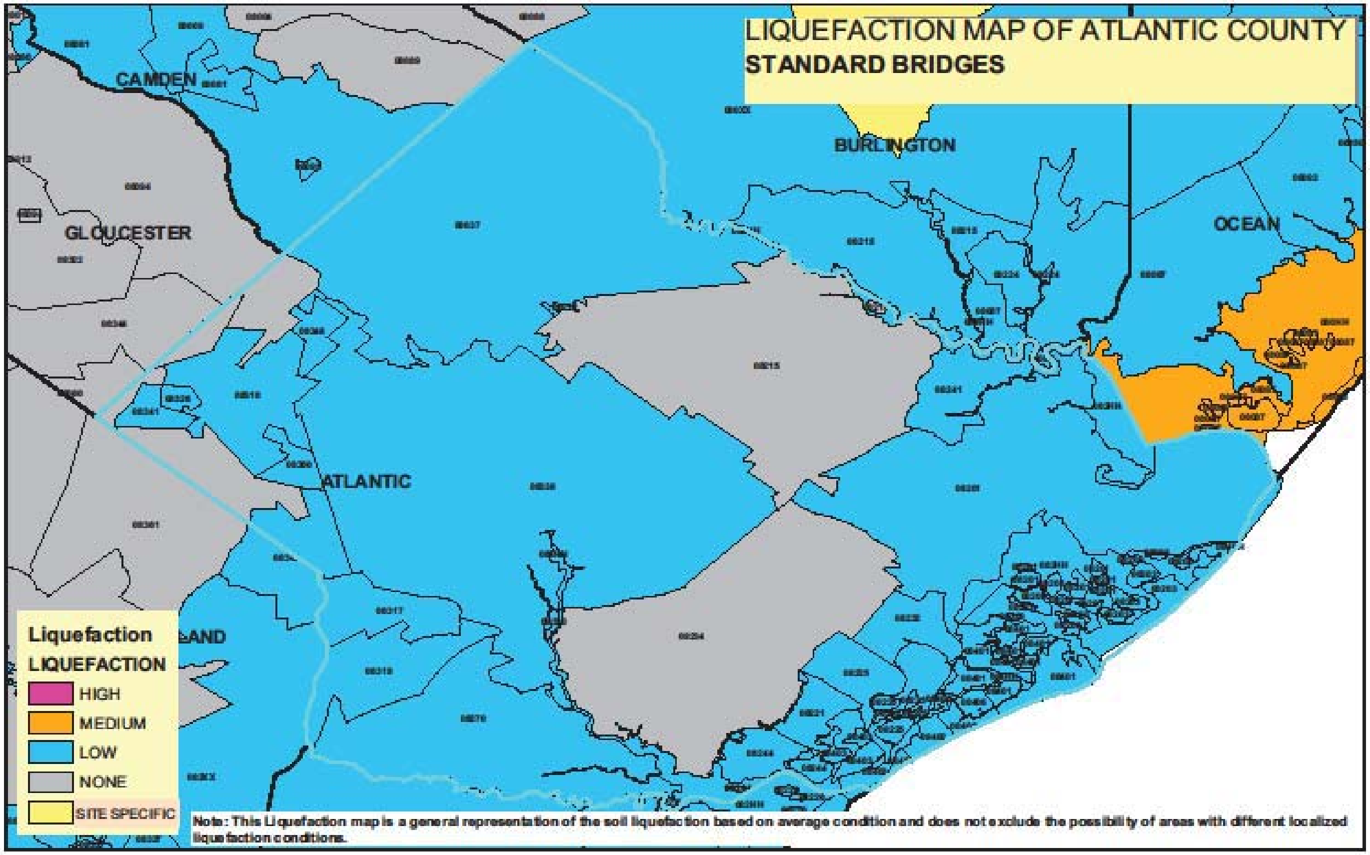


Figure III.2 Liquefaction Map for Standard Bridges in Atlantic County of New Jersey.

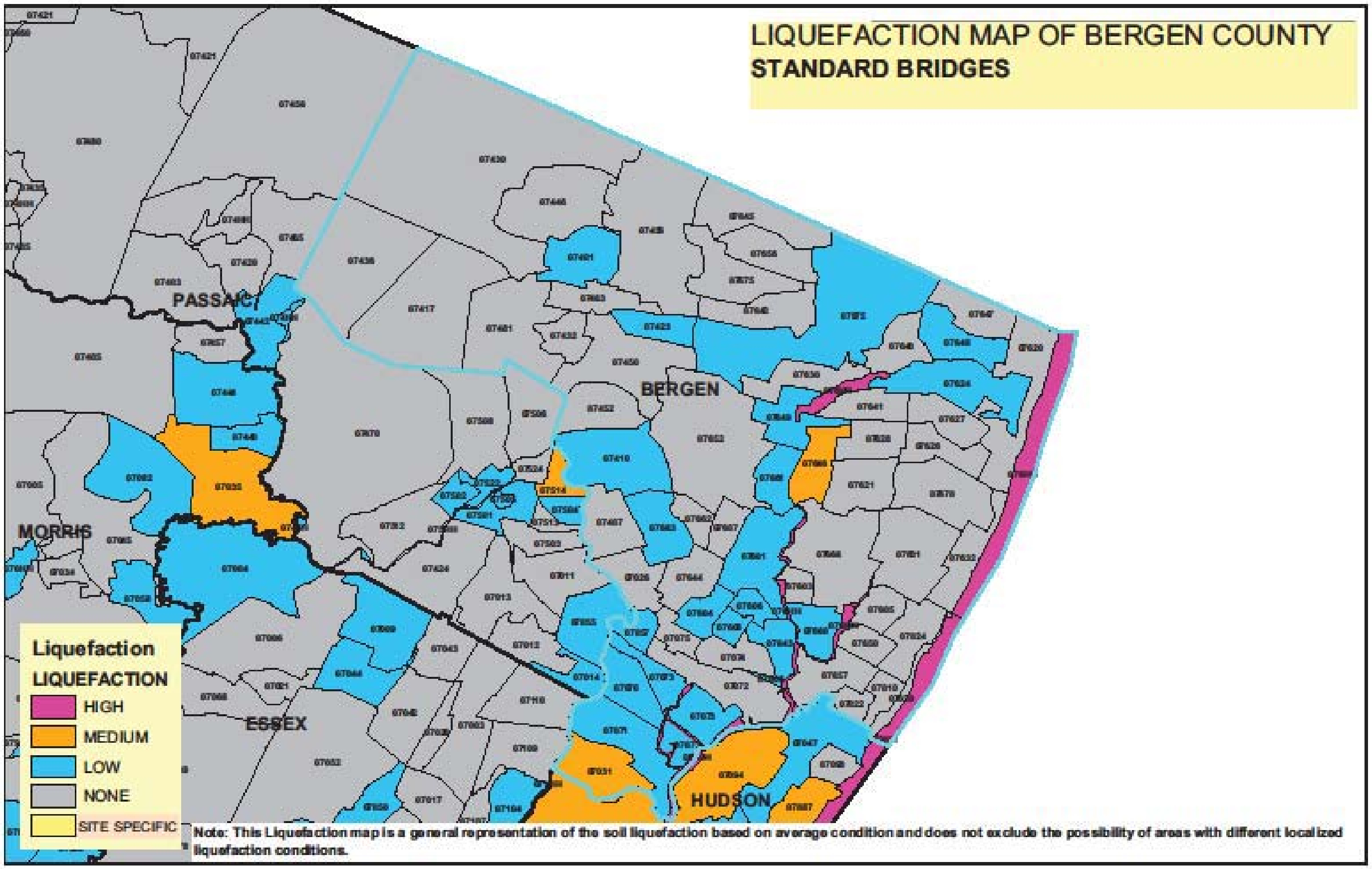


Figure III.3 Liquefaction Map for Standard Bridges in Bergen County of New Jersey

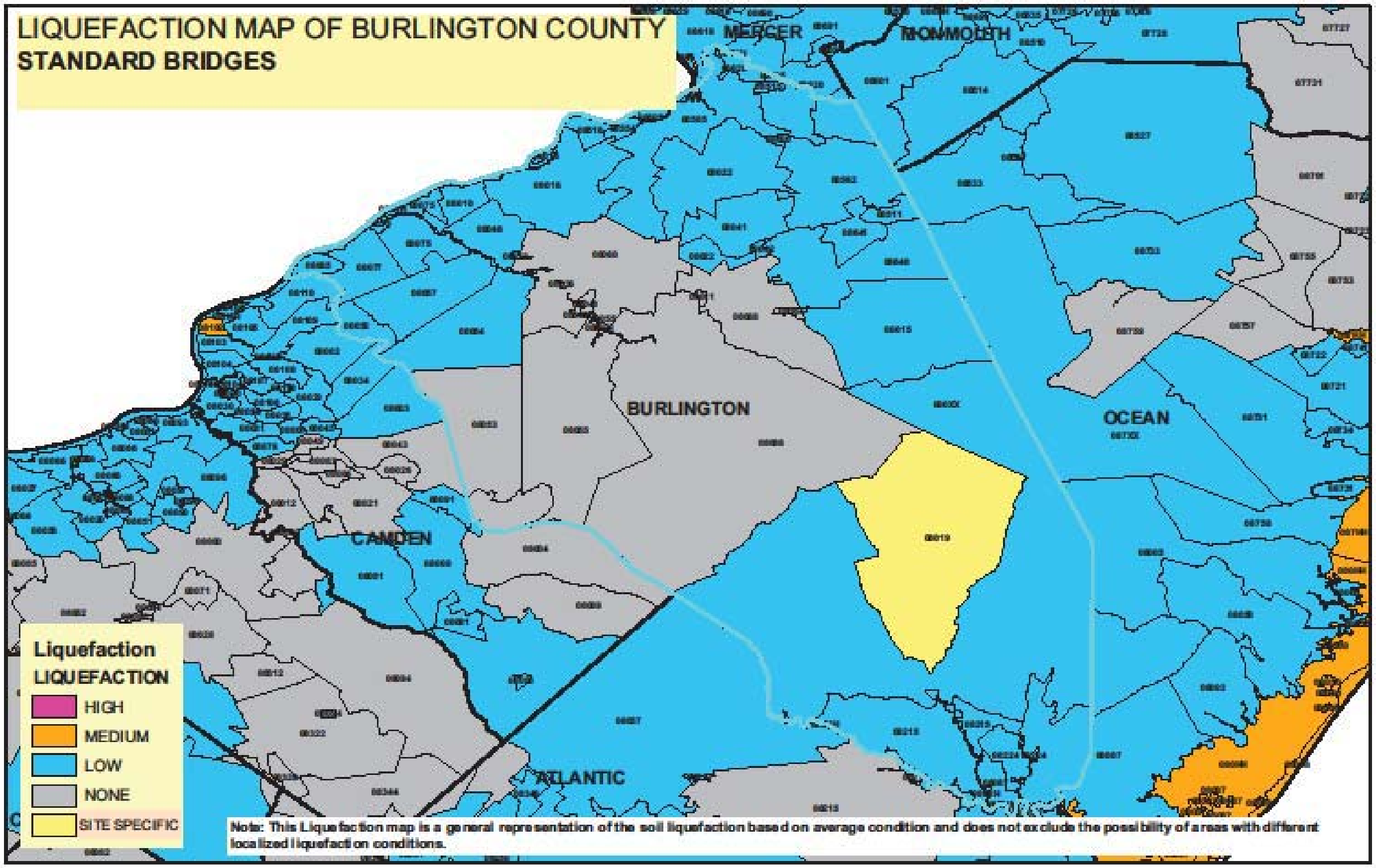


Figure III.4 Liquefaction Map for Standard Bridges in Burlington County of New Jersey

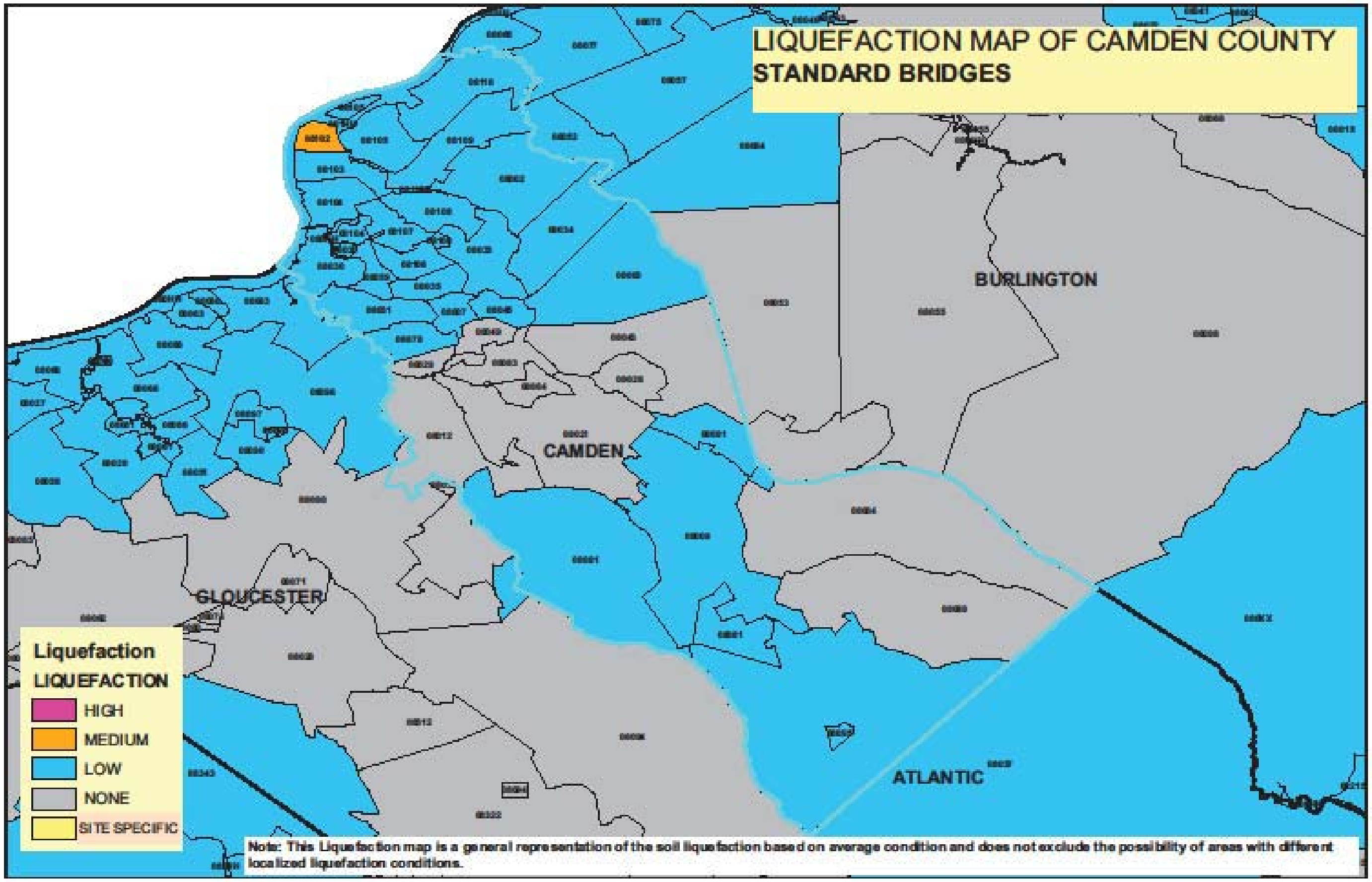


Figure III.5 Liquefaction Map for Standard Bridges in Camden County of New Jersey

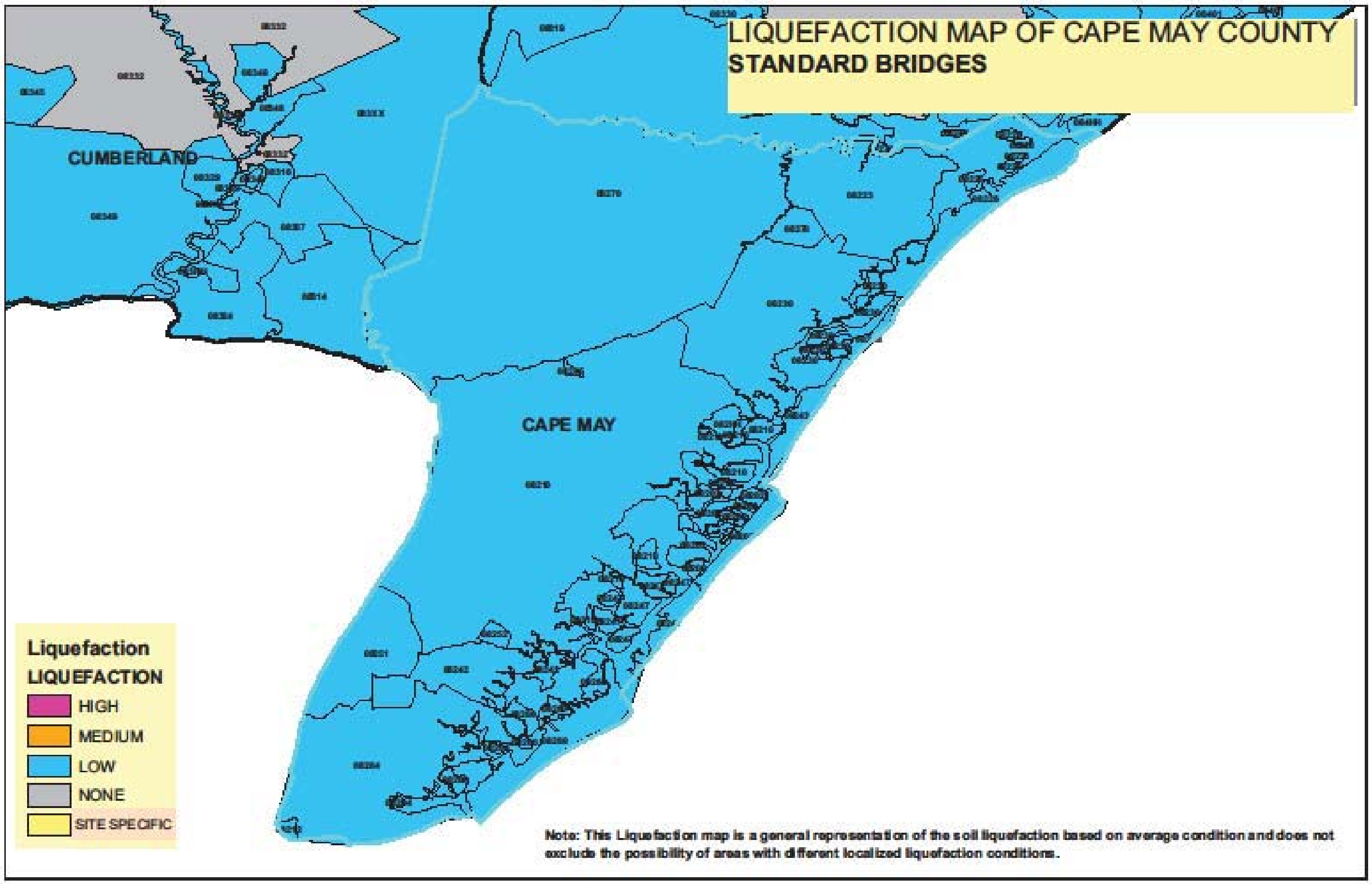


Figure III.6 Liquefaction Map for Standard Bridges in Cape May County of New Jersey

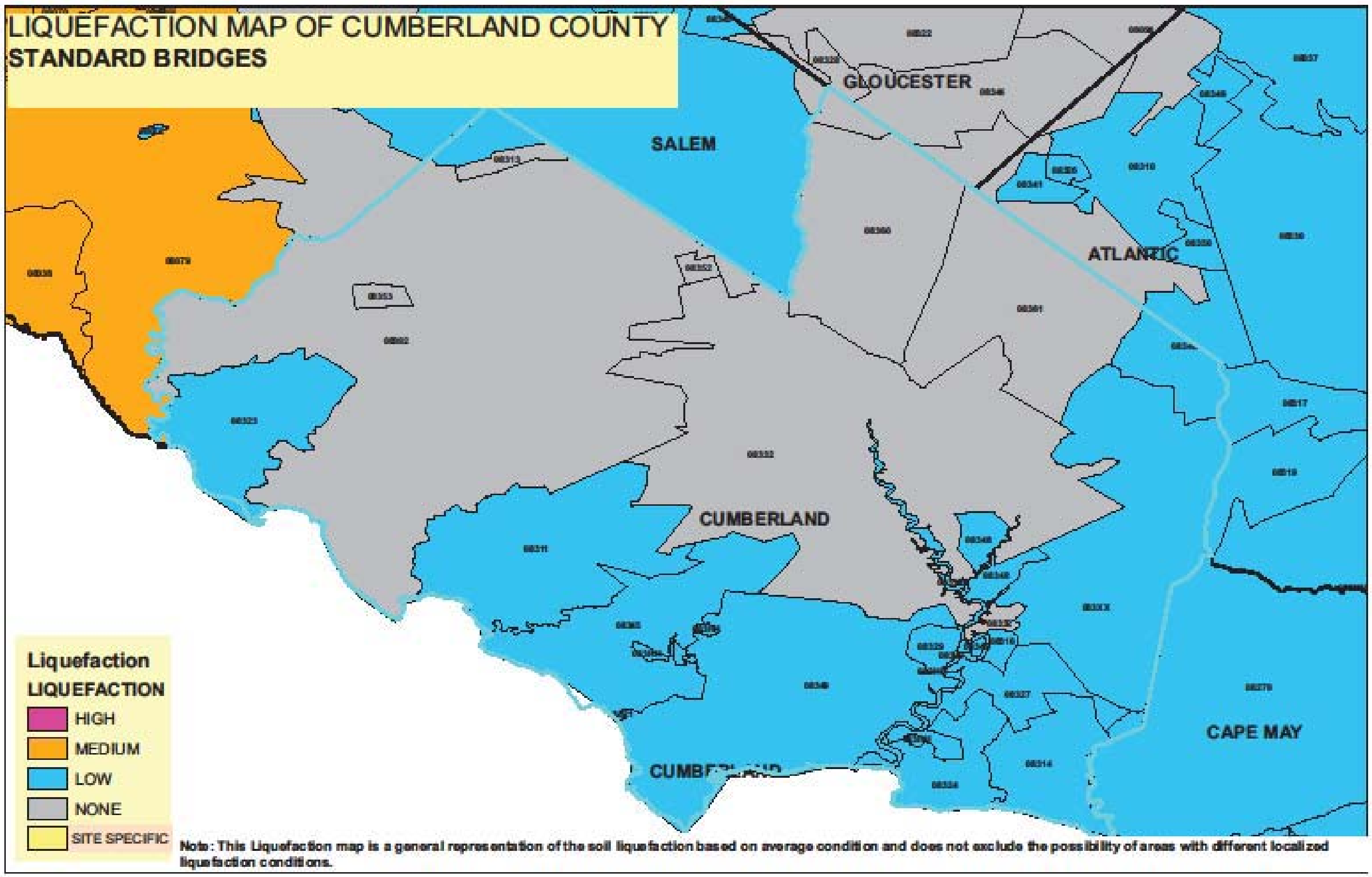


Figure III.7 Liquefaction Map for Standard Bridges in Cumberland County of New Jersey

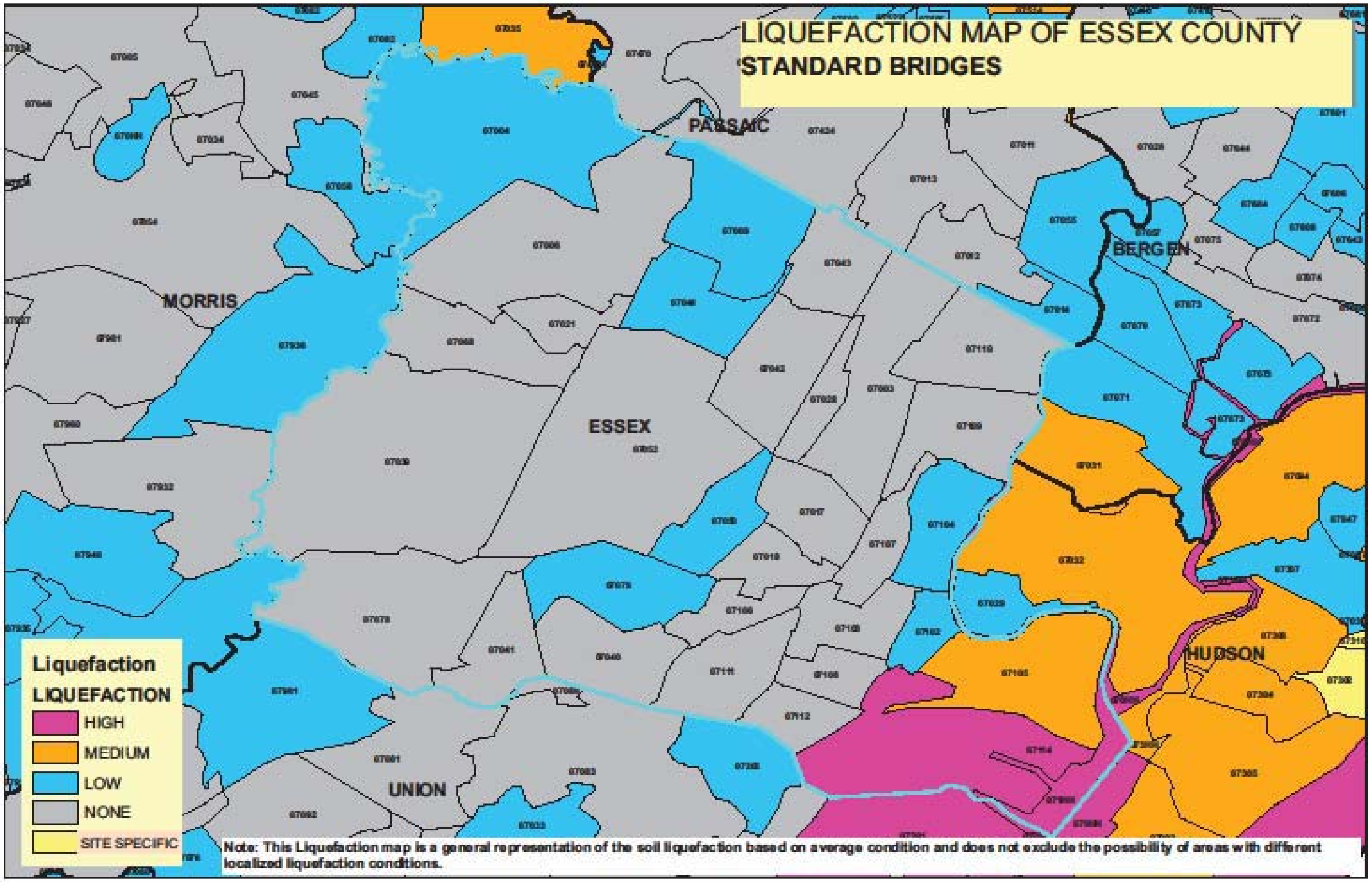


Figure III.8 Liquefaction Map for Standard Bridges in Essex County of New Jersey

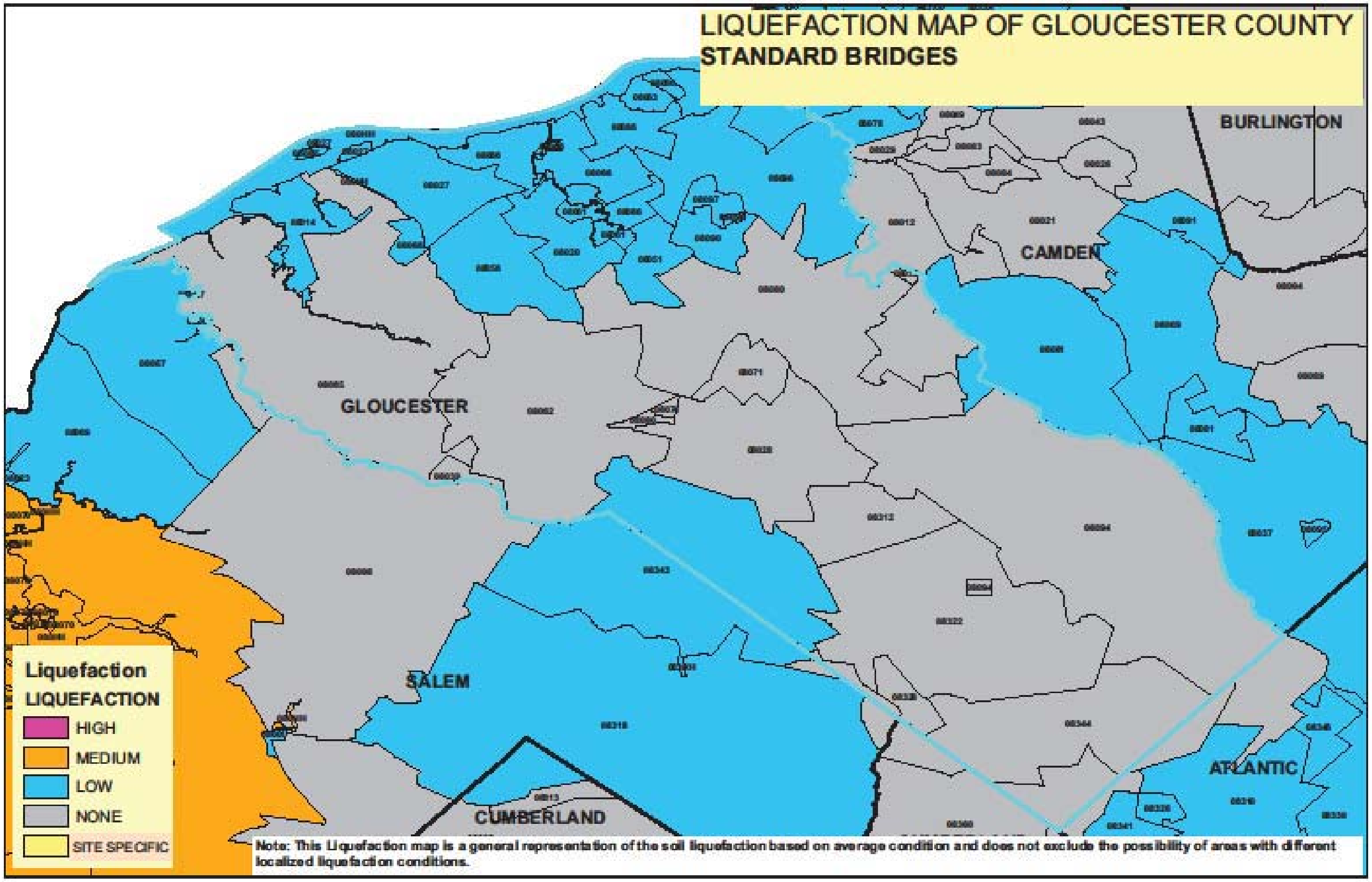


Figure III.9 Liquefaction Map for Standard Bridges in Gloucester County of New Jersey

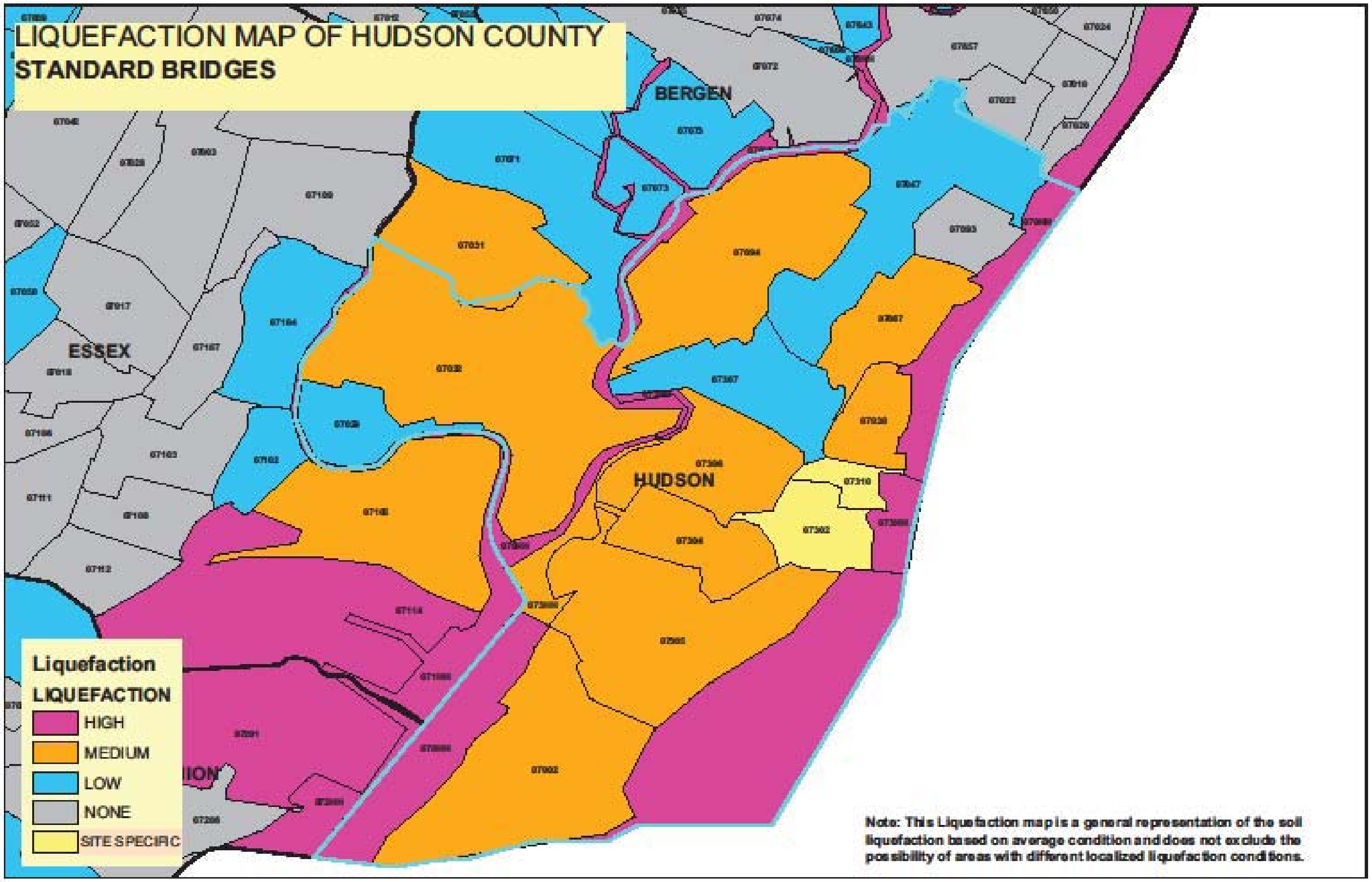


Figure III.10 Liquefaction Map for Standard Bridges in Hudson County of New Jersey

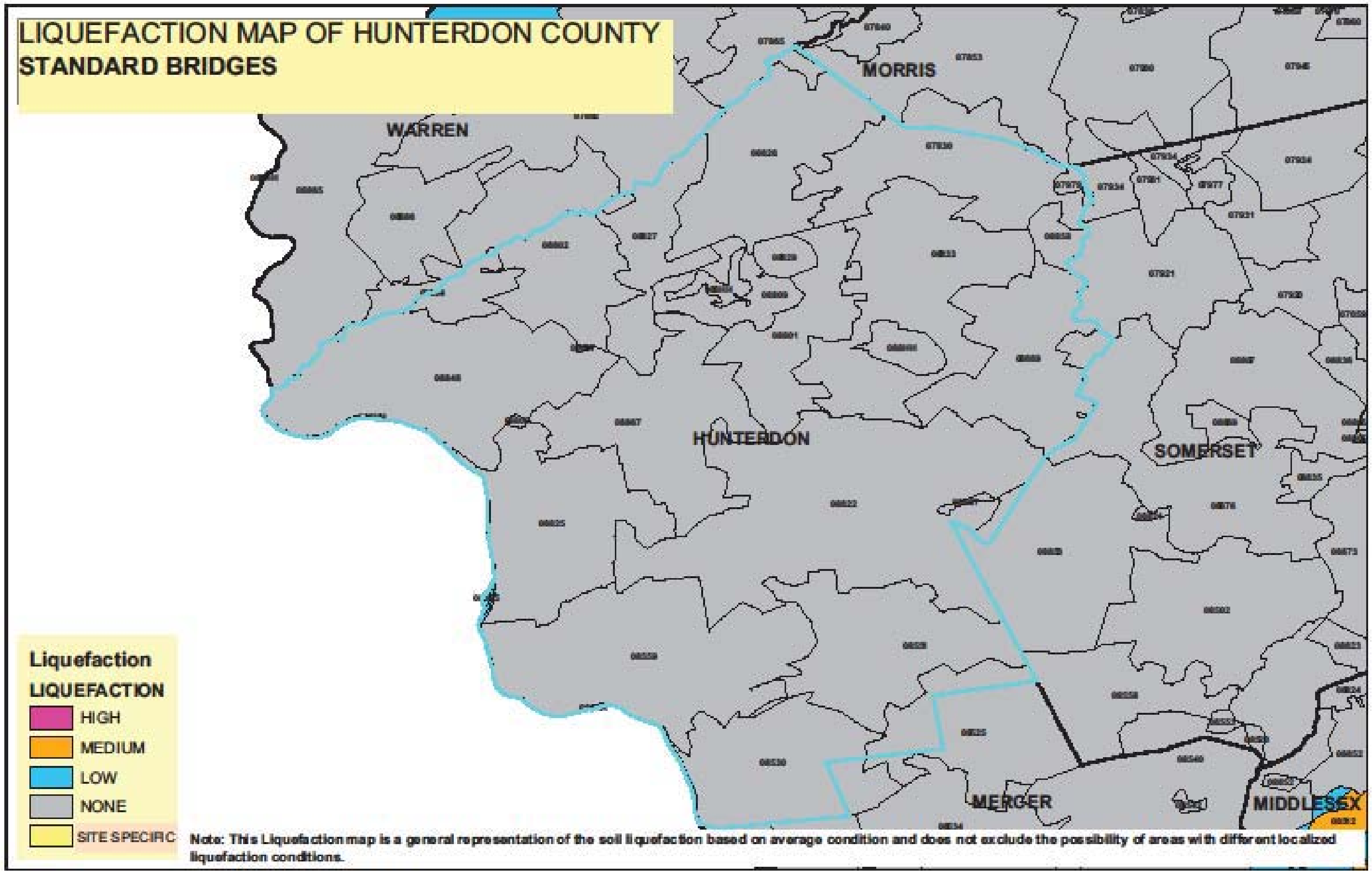


Figure III.11 Liquefaction Map for Standard Bridges in Hunterdon County of New Jersey

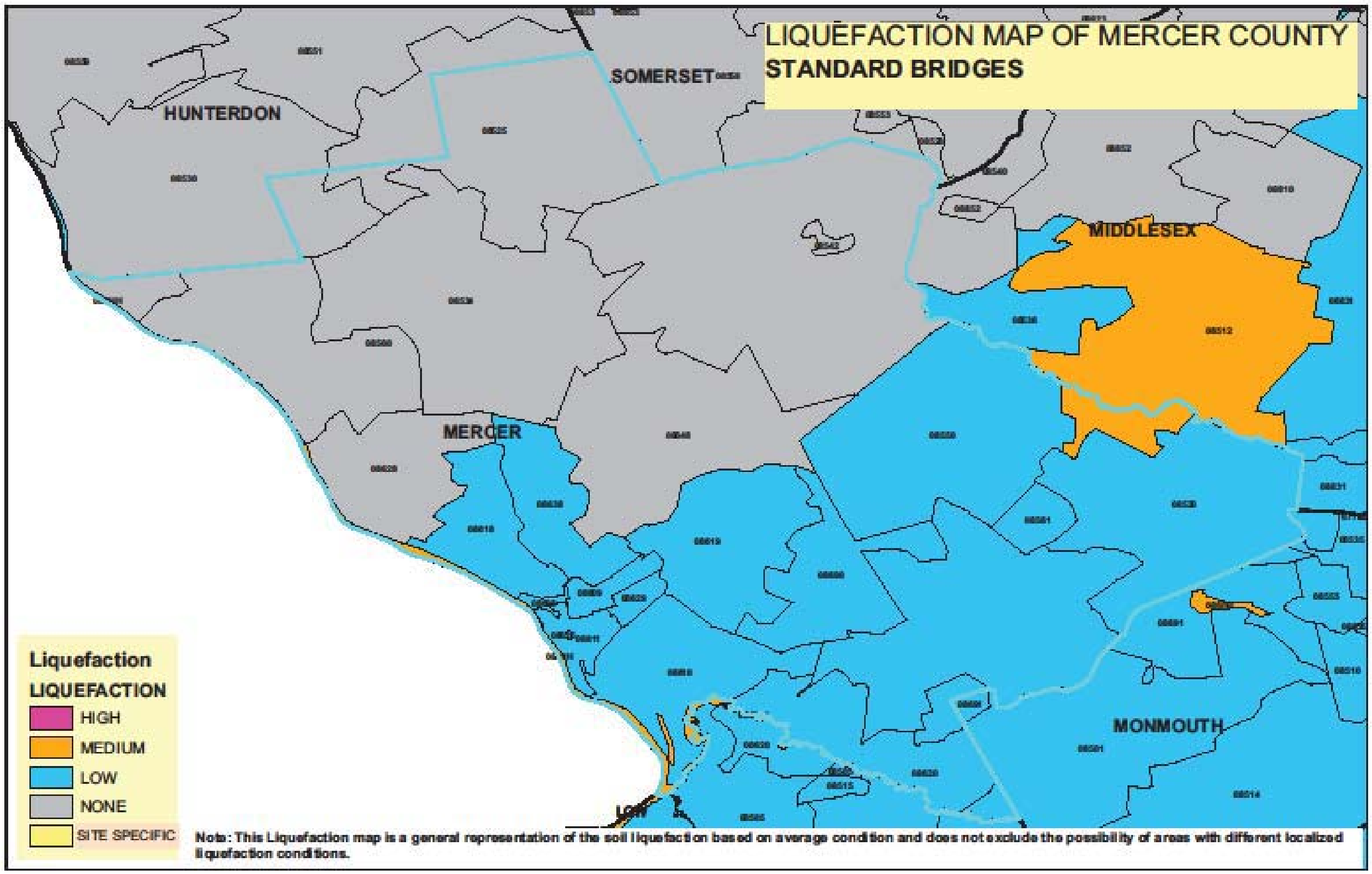


Figure III.12 Liquefaction Map for Standard Bridges in Mercer County of New Jersey

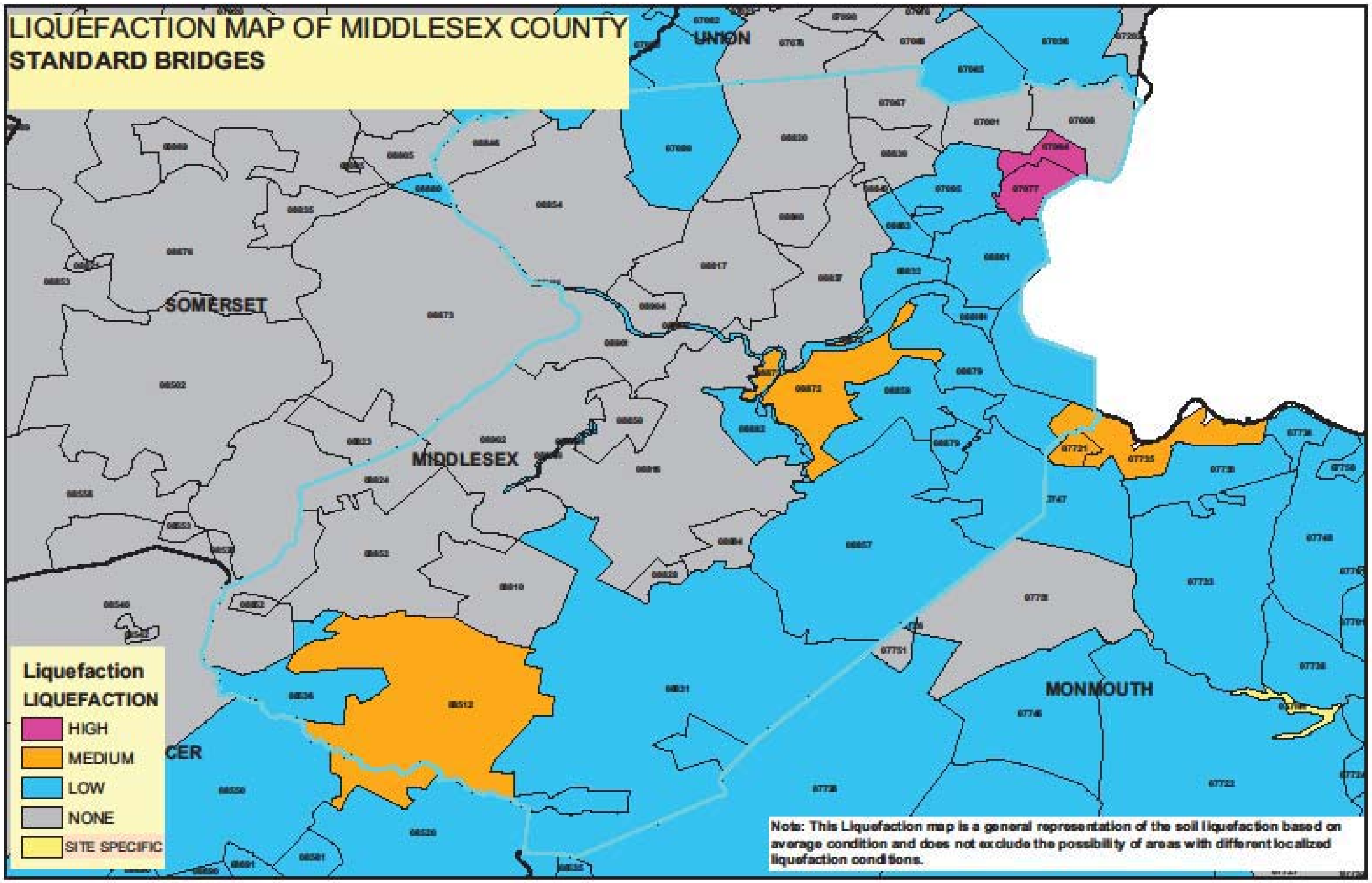


Figure III.13 Liquefaction Map for Standard Bridges in Middlesex County of New Jersey

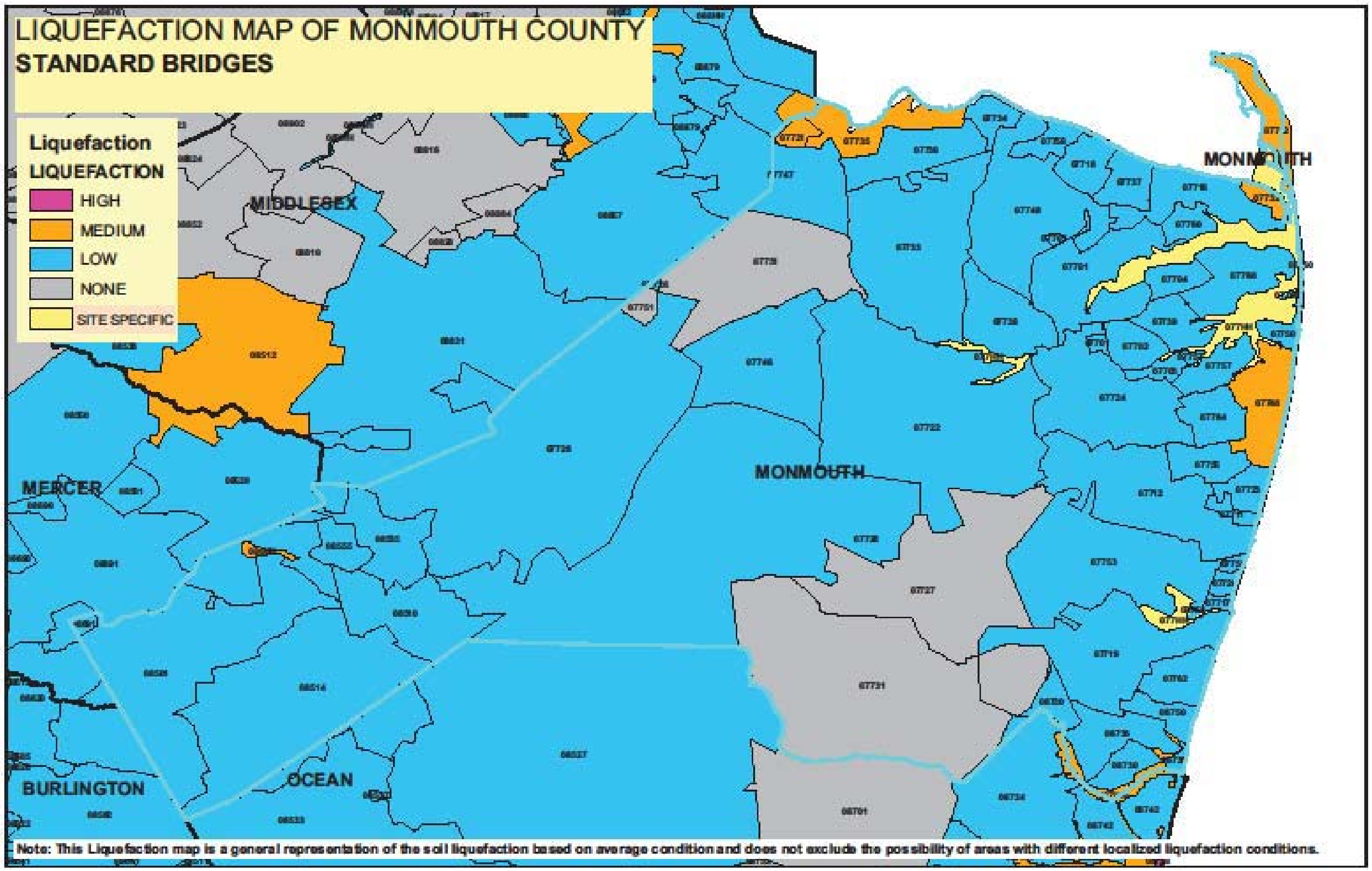


Figure III.14 Liquefaction Map for Standard Bridges in Monmouth County of New Jersey

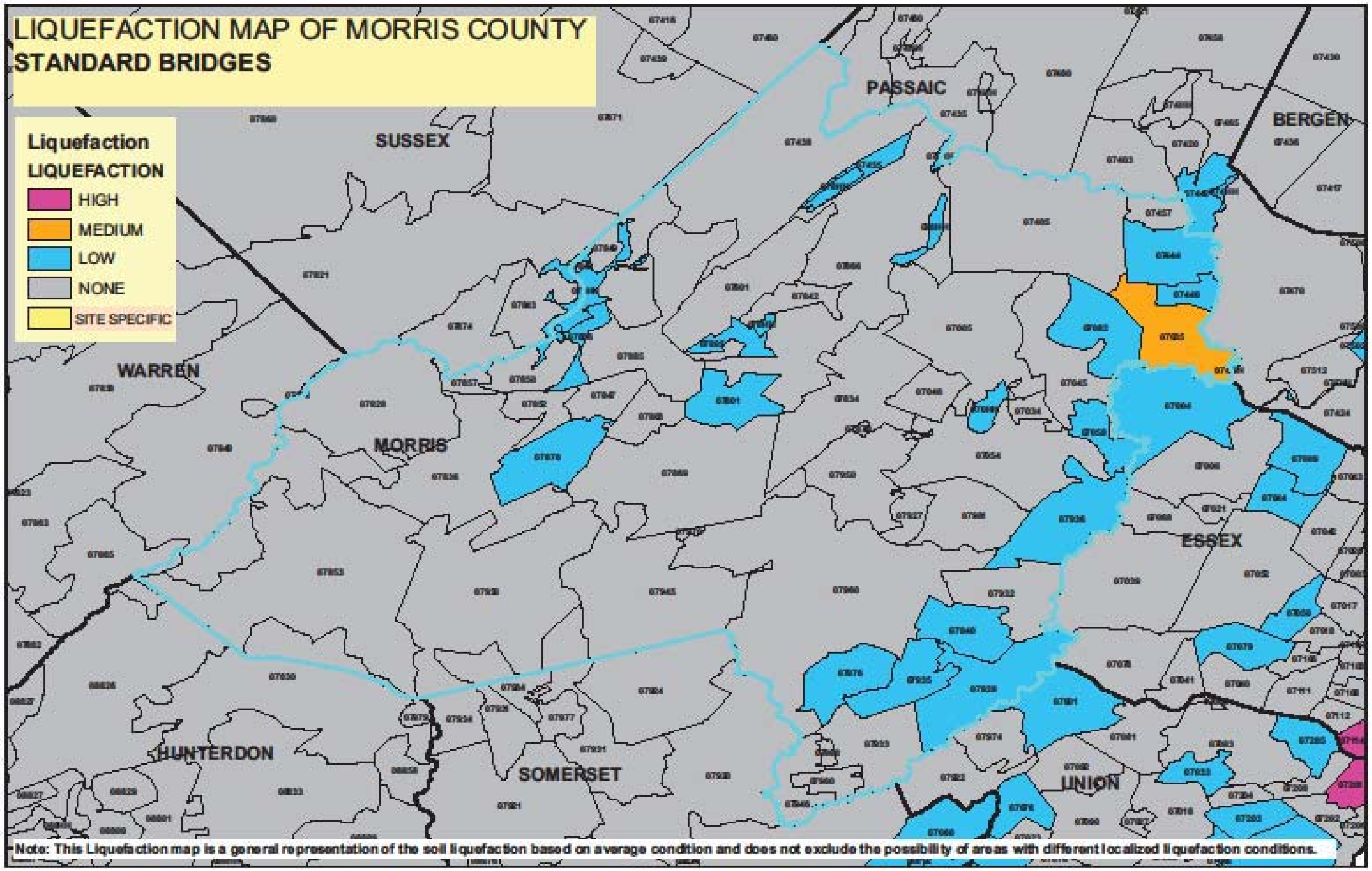


Figure III.15 Liquefaction Map for Standard Bridges in Morris County of New Jersey

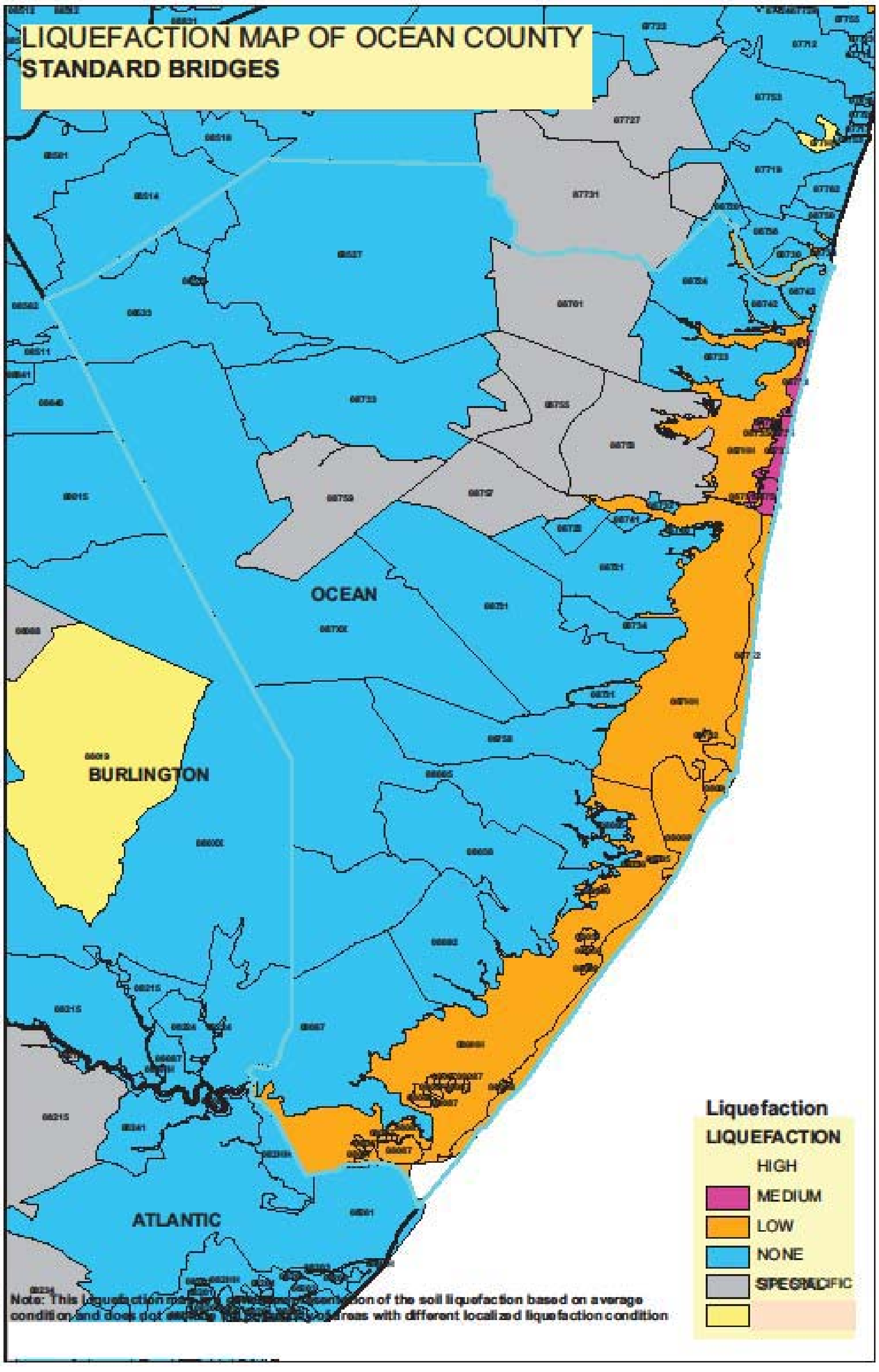


Figure III.16 Liquefaction Map for Standard Bridges in Ocean County of New Jersey

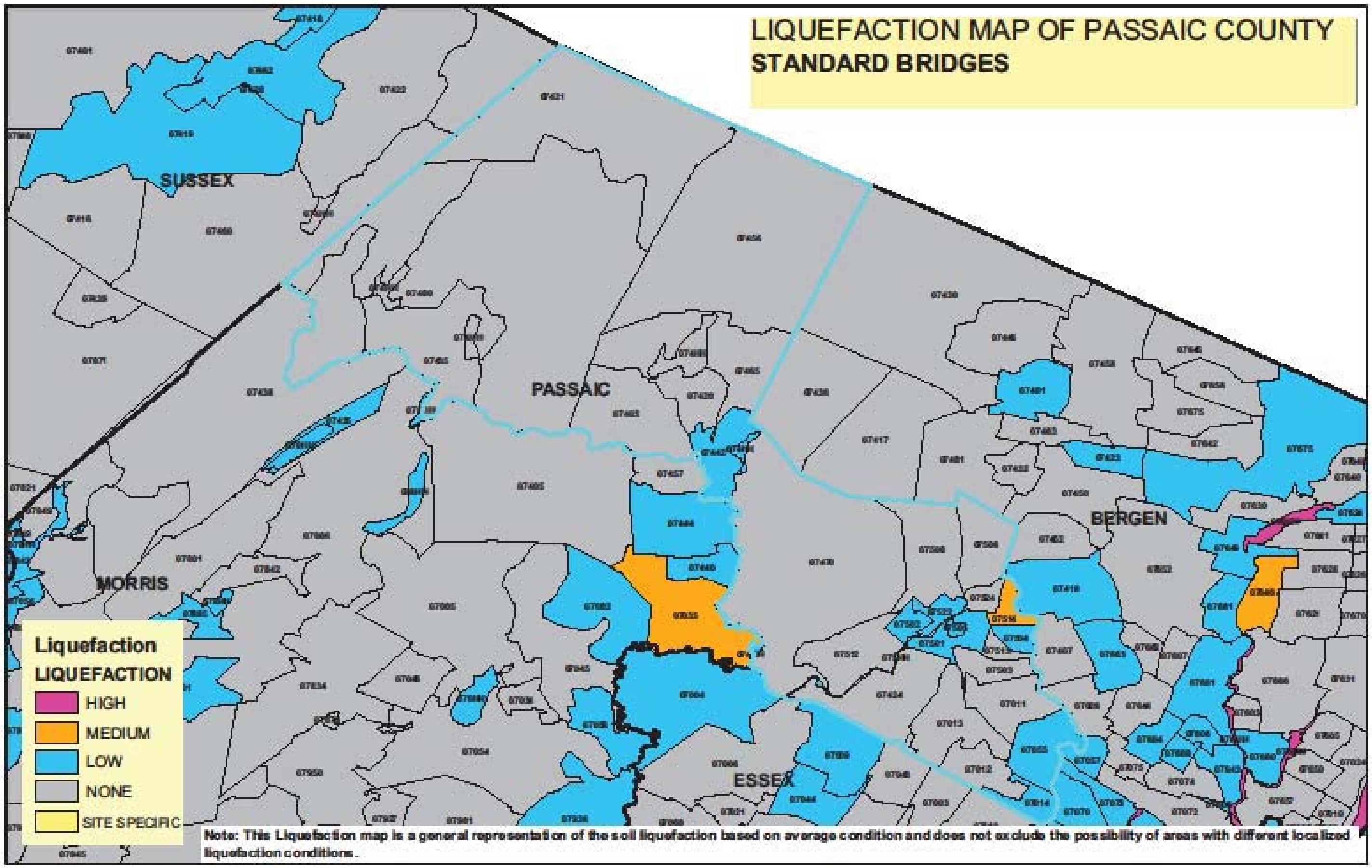


Figure III.17 Liquefaction Map for Standard Bridges in Passaic County of New Jersey

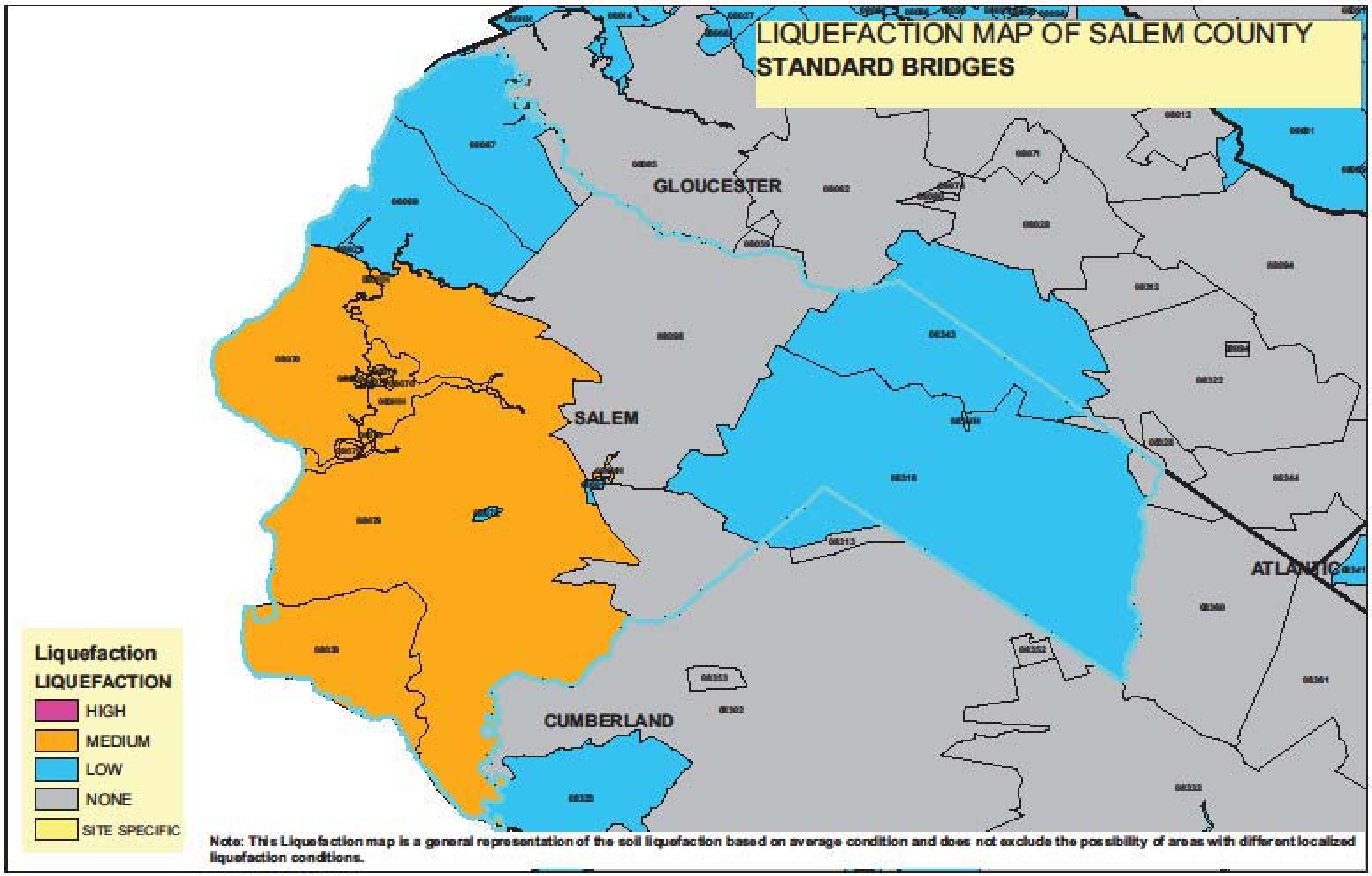


Figure III.18 Liquefaction Map for Standard Bridges in Salem County of New Jersey

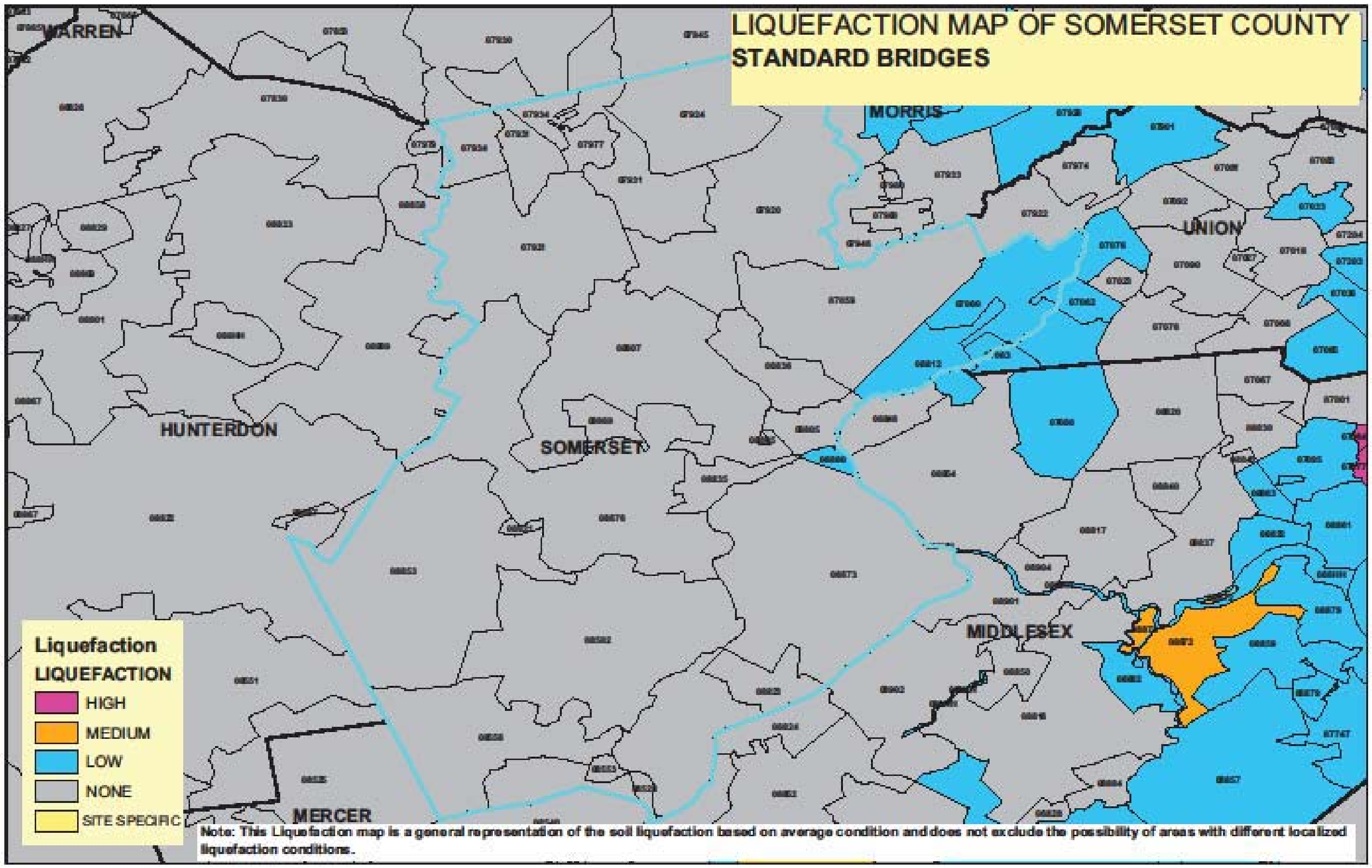


Figure III.19 Liquefaction Map for Standard Bridges in Somerset County of New Jersey

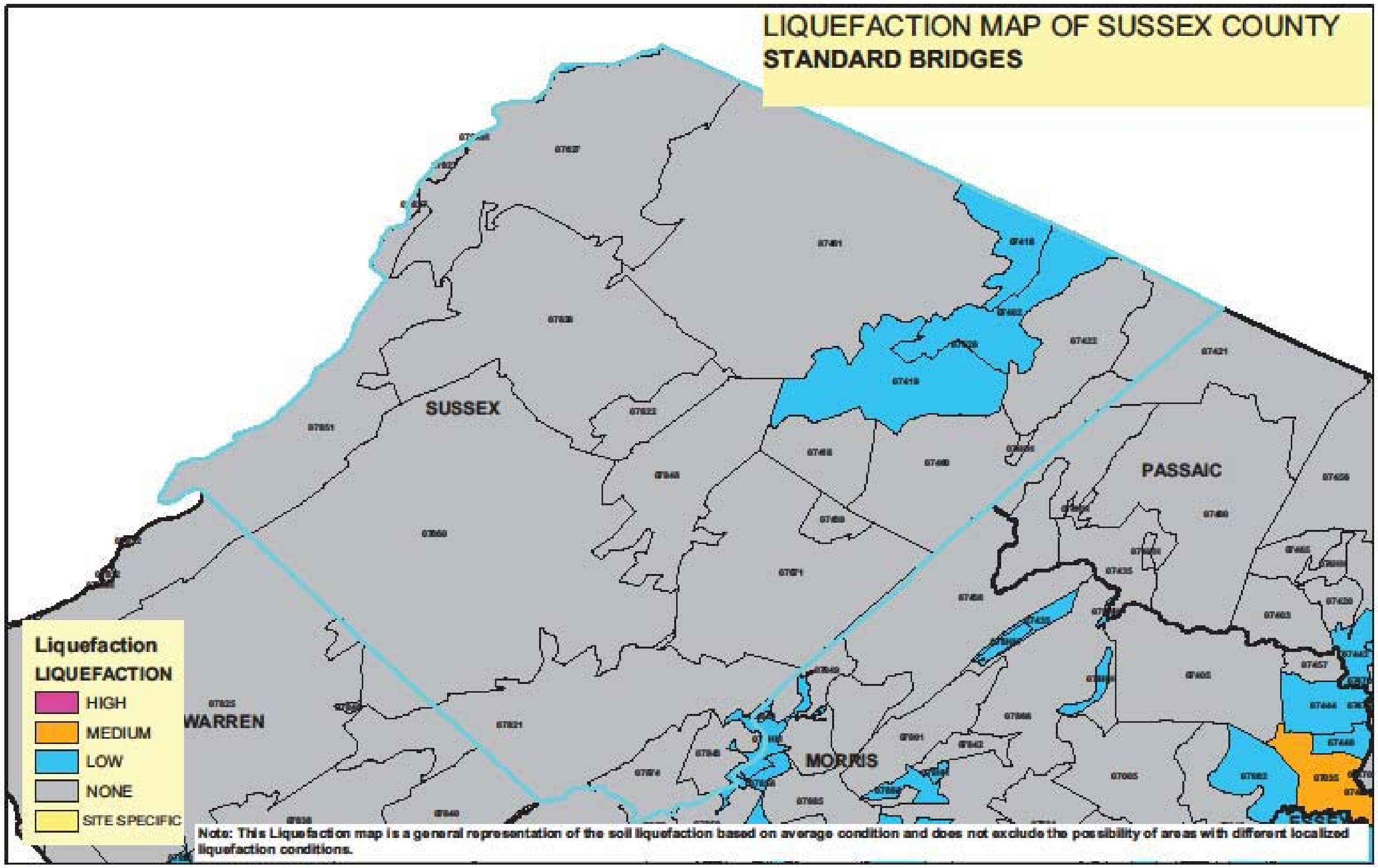


Figure III.20 Liquefaction Map for Standard Bridges in Sussex County of New Jersey

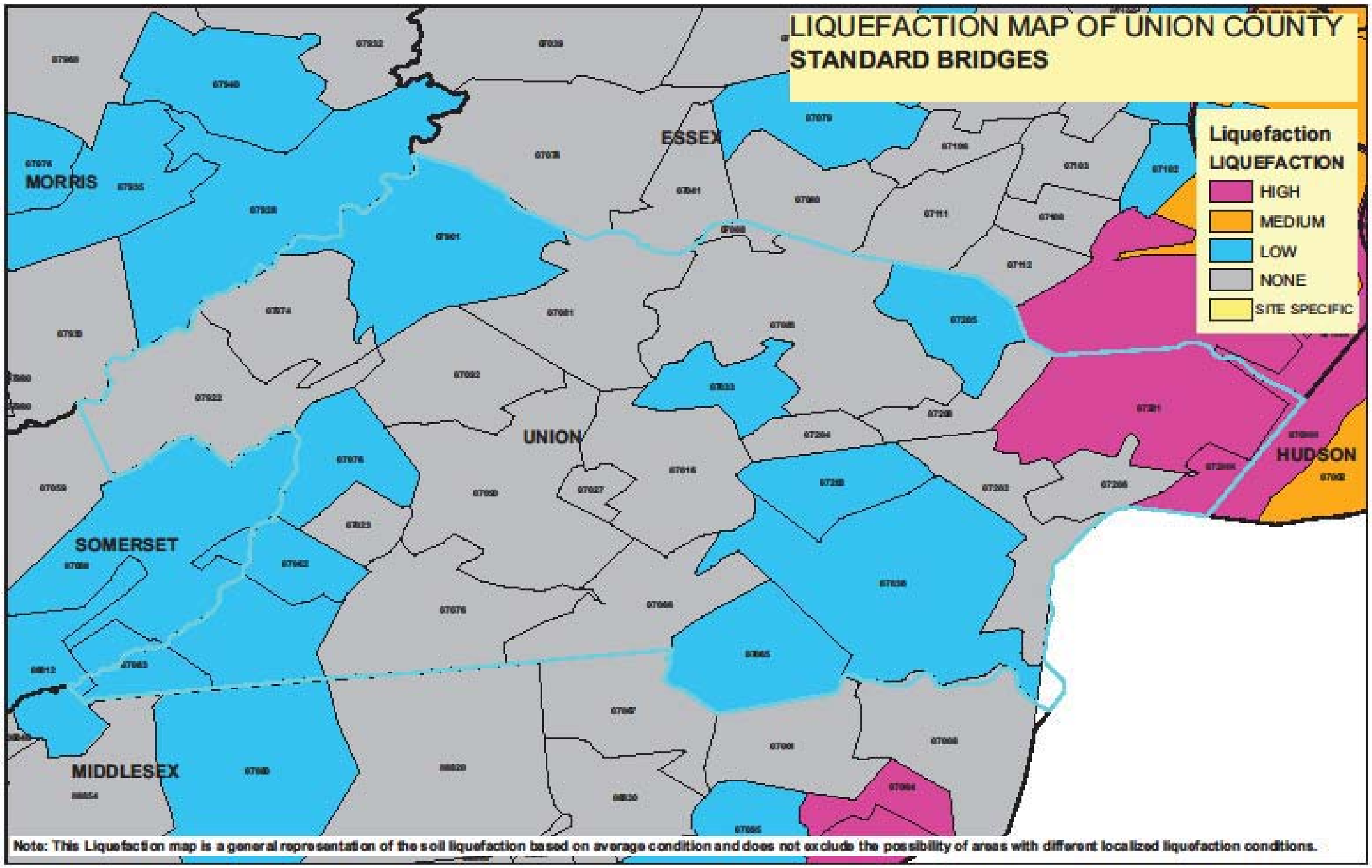


Figure III.21 Liquefaction Map for Standard Bridges in Union County of New Jersey

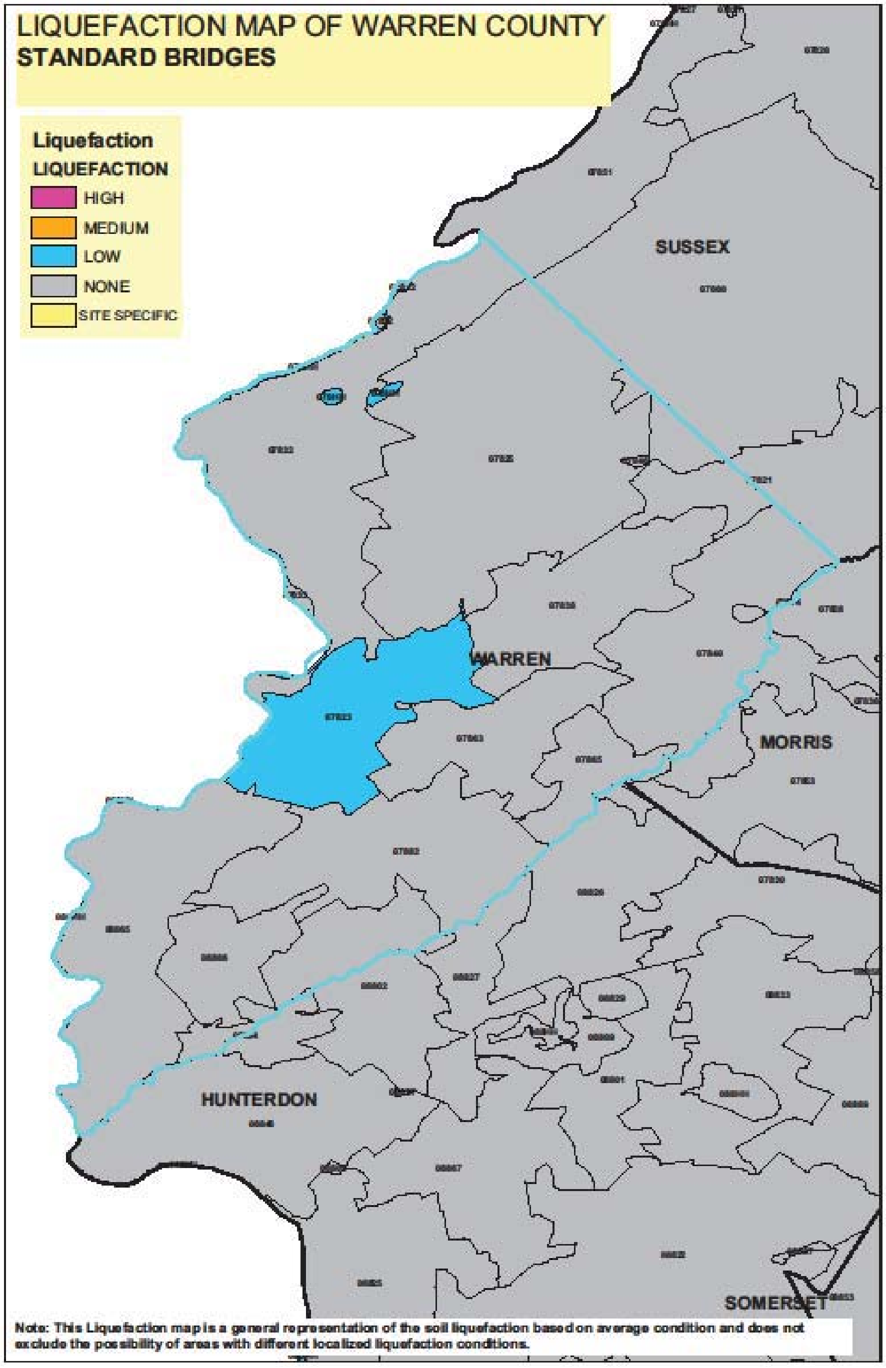


Figure III.22 Liquefaction Map for Standard Bridges in Warren County of New Jersey

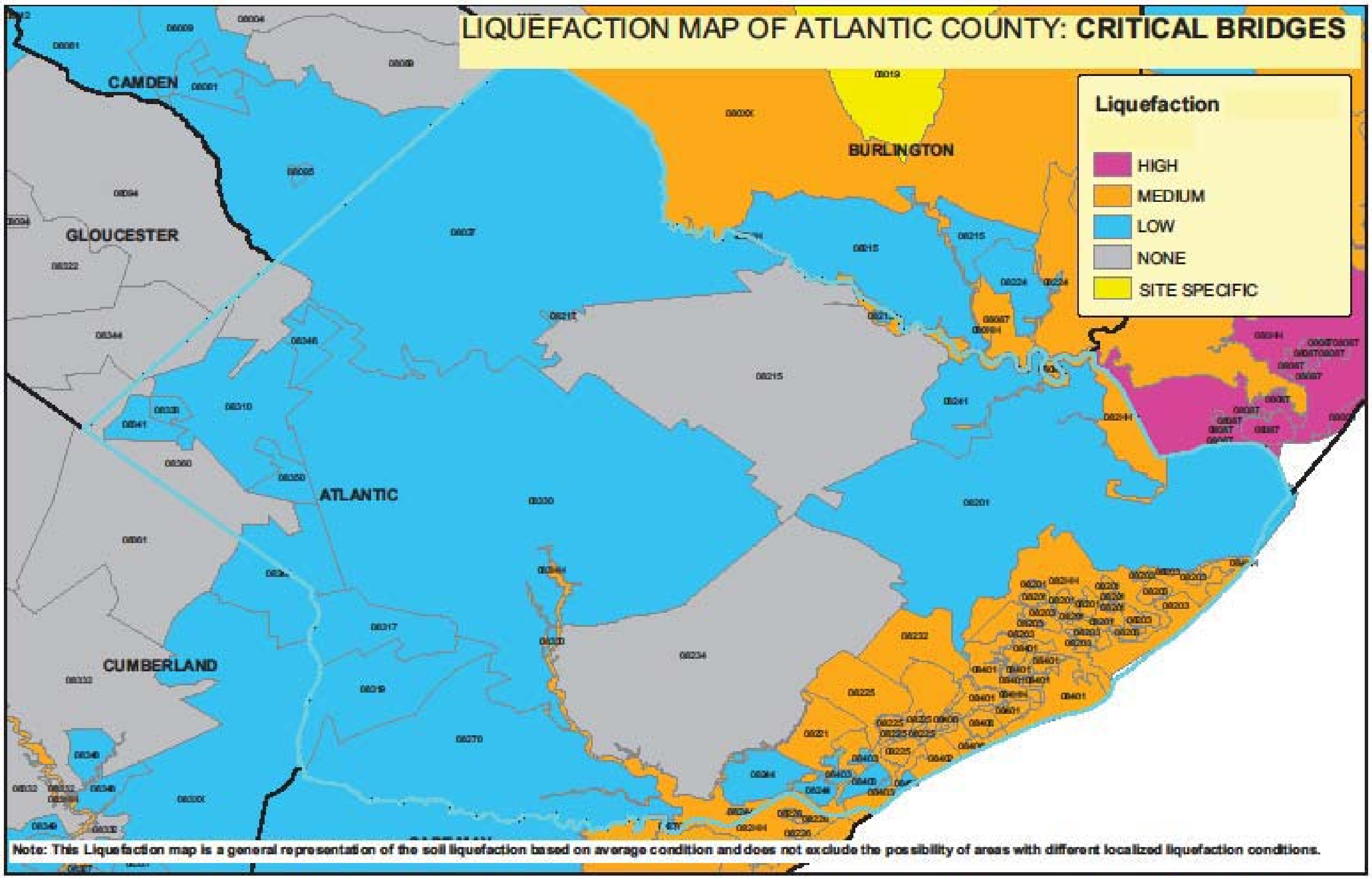


Figure III.23 Liquefaction Map for Critical Bridges in Atlantic County of New Jersey

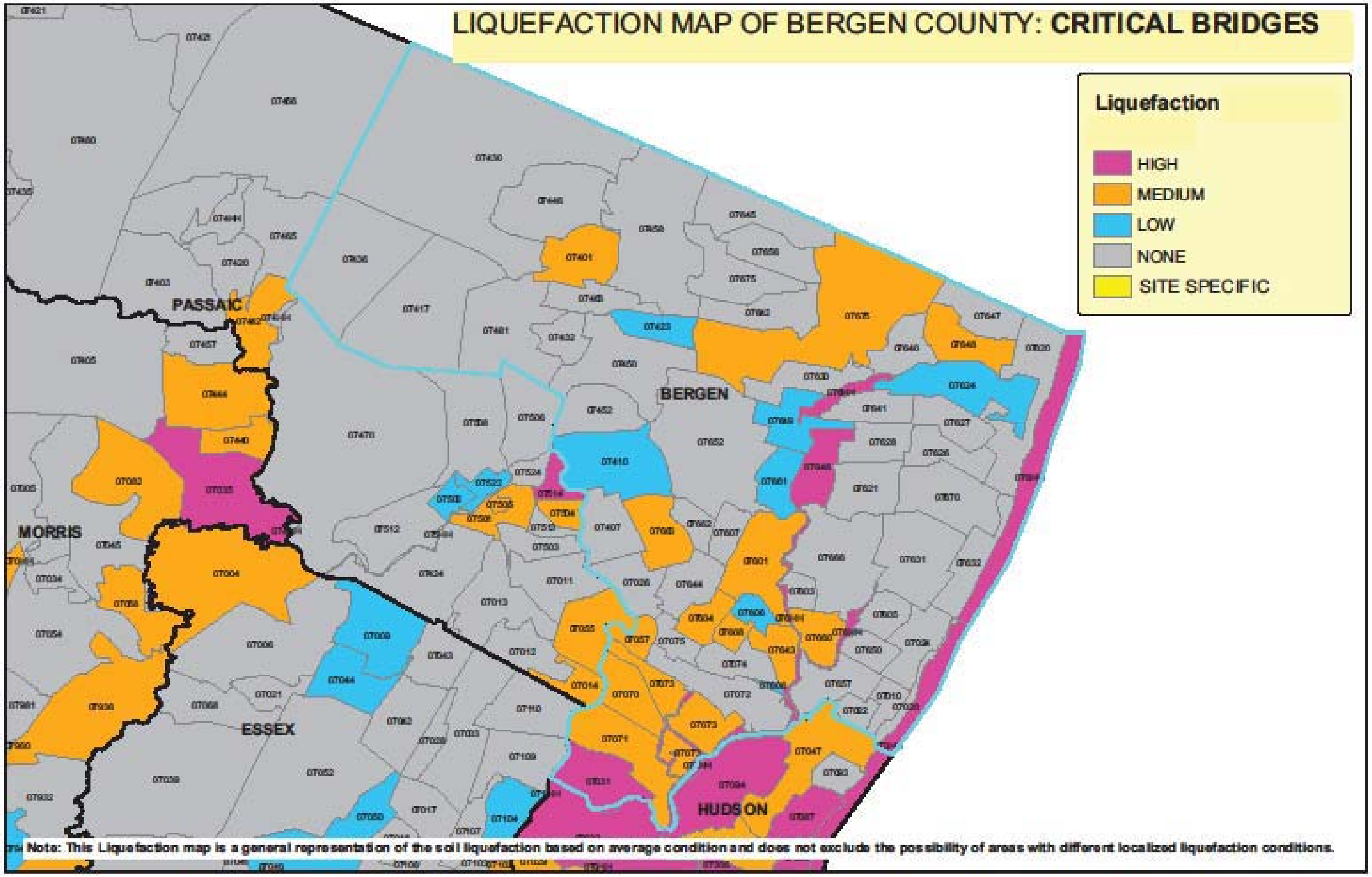


Figure III.24 Liquefaction Map for Critical Bridges in Bergen County of New Jersey

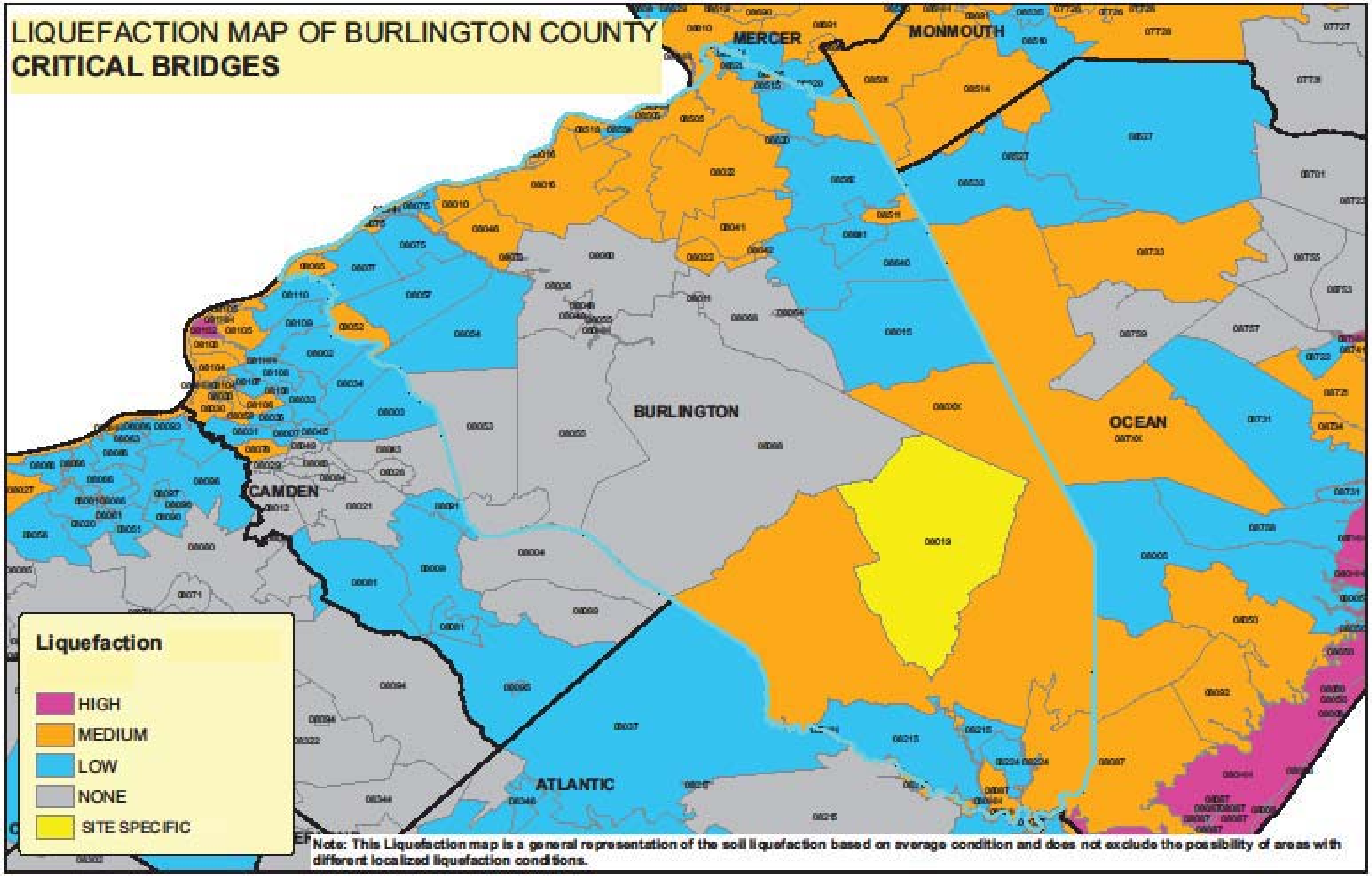


Figure III.25 Liquefaction Map for Critical Bridges in Burlington County of New Jersey

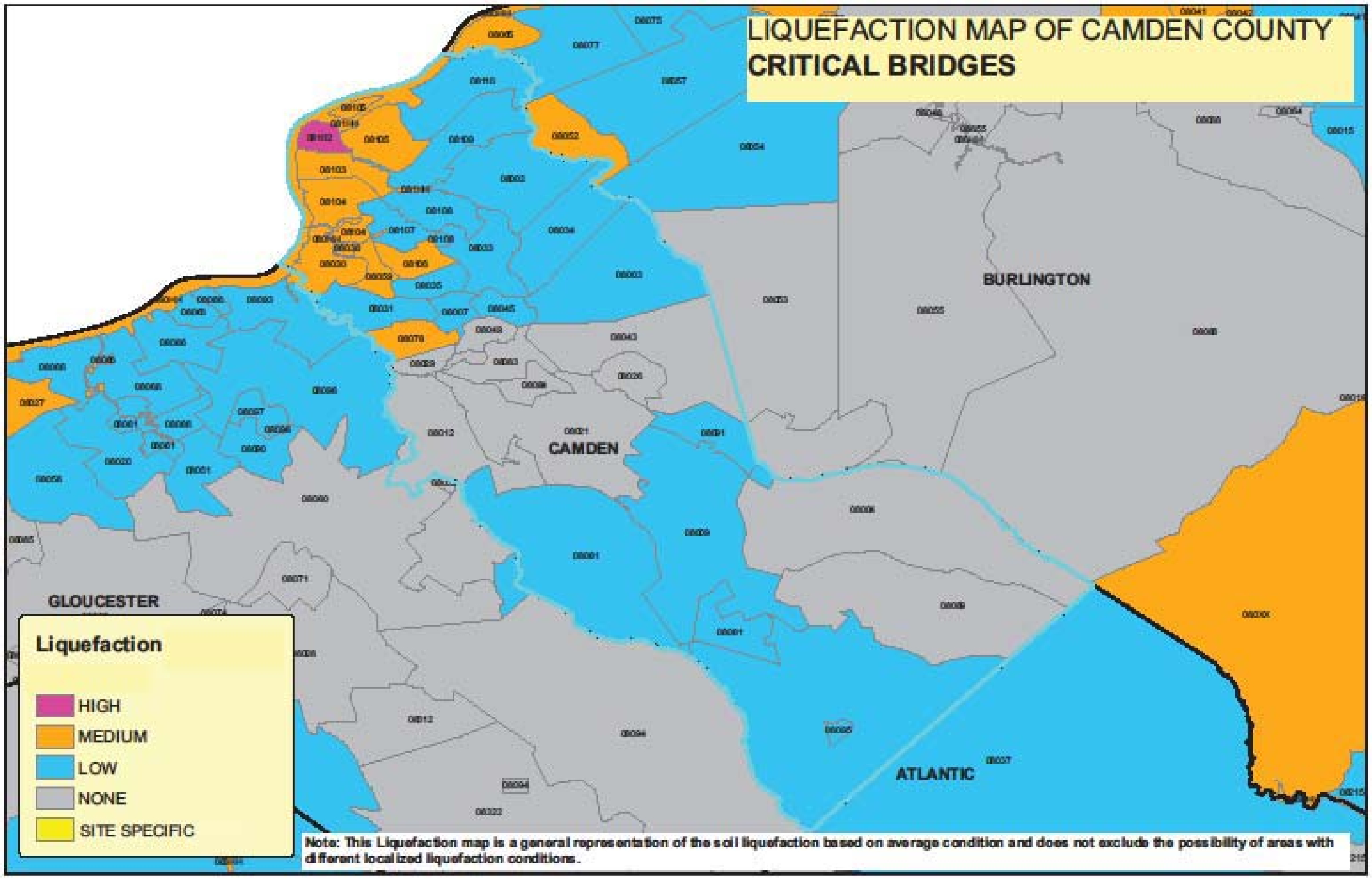


Figure III.26 Liquefaction Map for Critical Bridges in Camden County of New Jersey

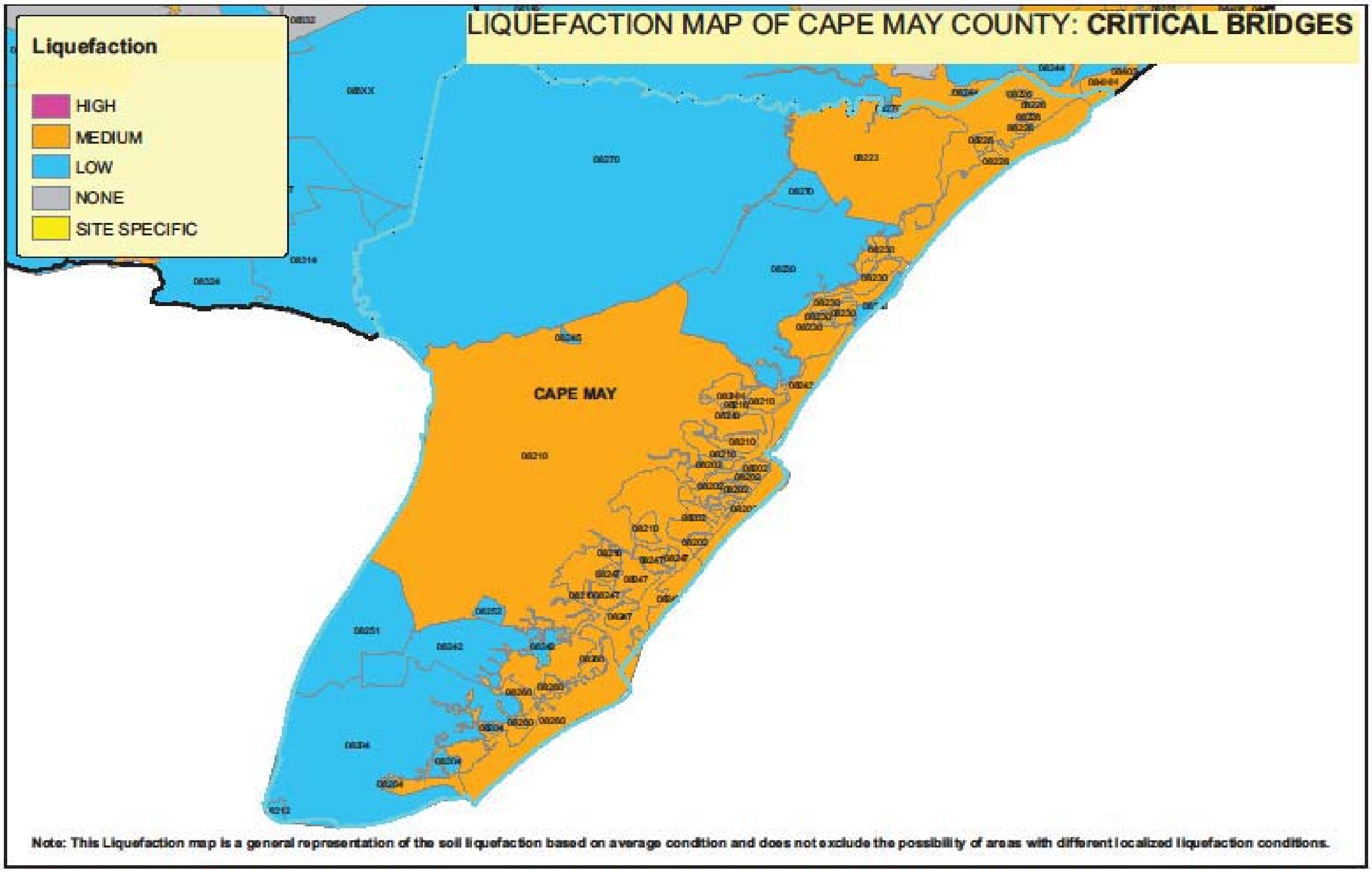


Figure III.27 Liquefaction Map for Critical Bridges in Cape May County of New Jersey

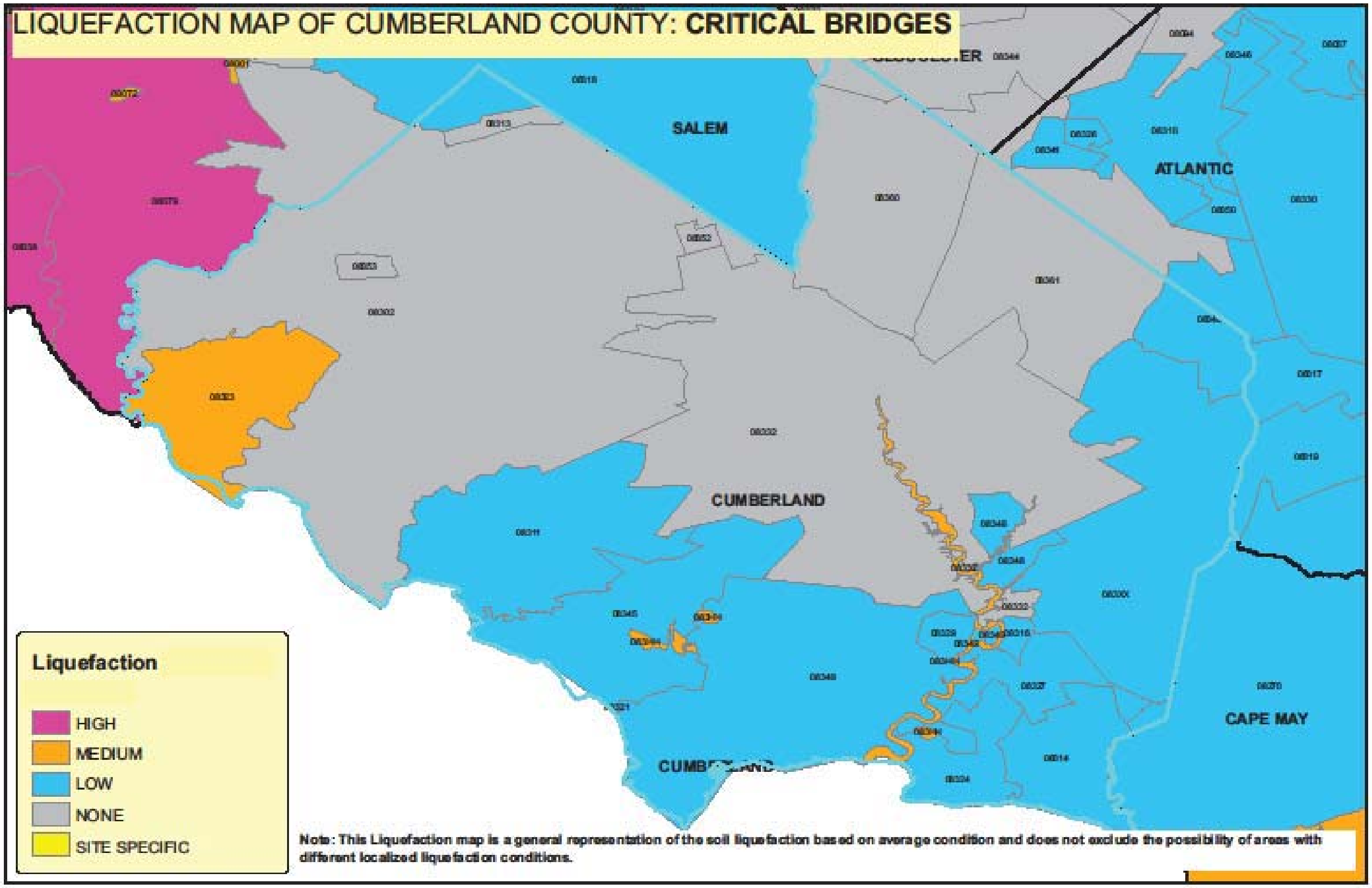


Figure III.28 Liquefaction Map for Critical Bridges in Cumberland County of New Jersey

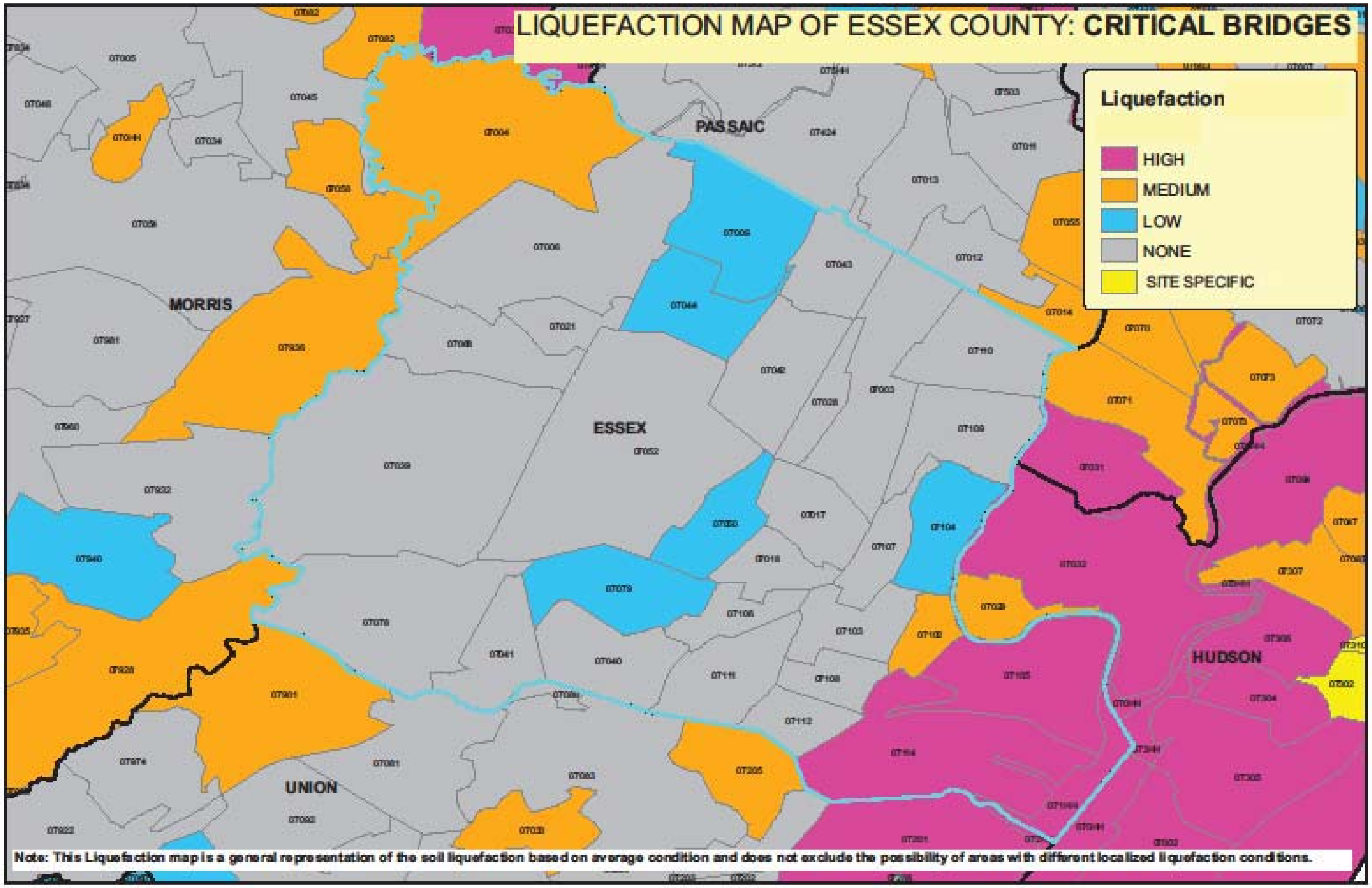


Figure III.29 Liquefaction Map for Critical Bridges in Essex County of New Jersey

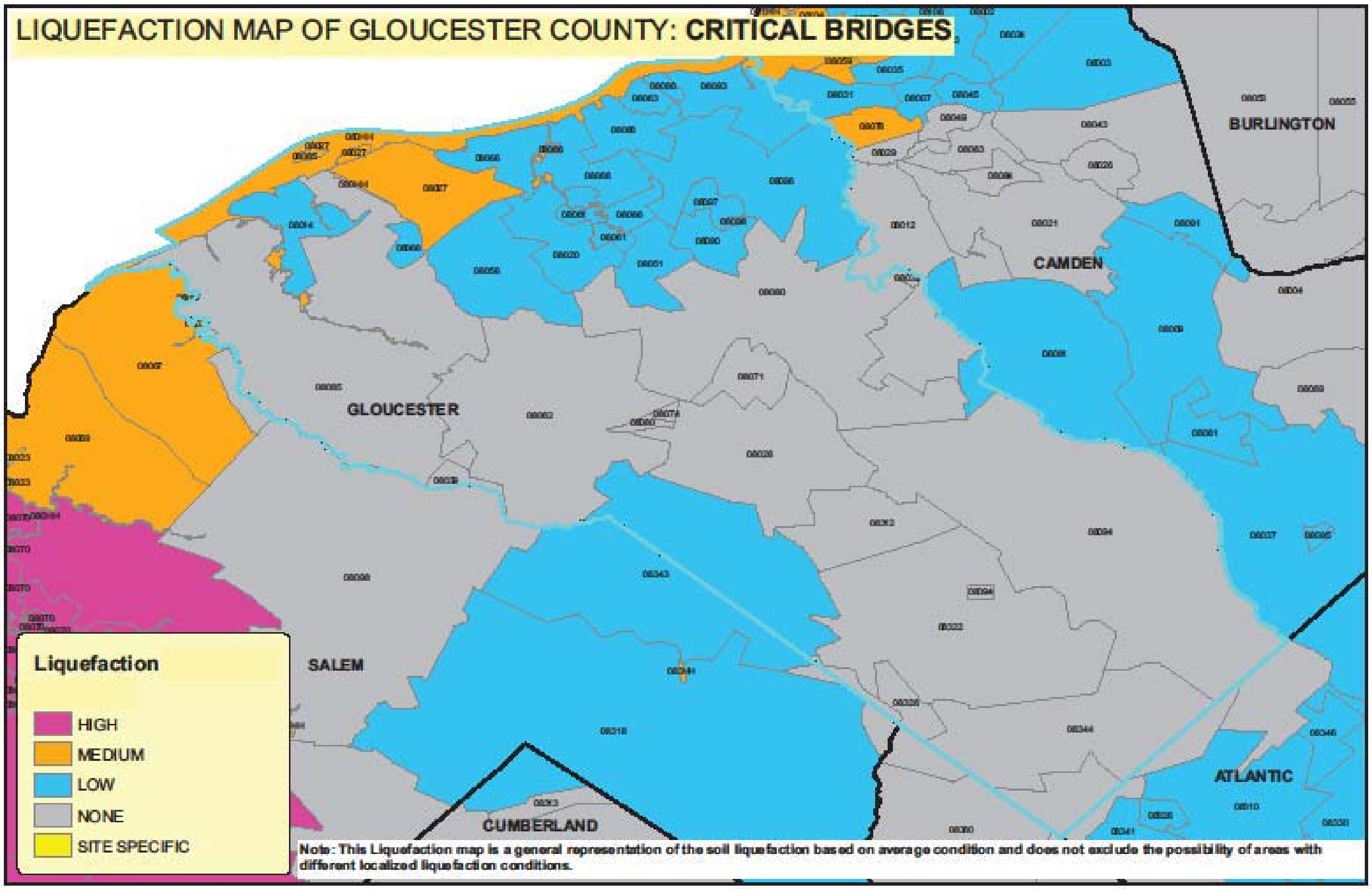


Figure III.30 Liquefaction Map for Critical Bridges in Gloucester County of New Jersey

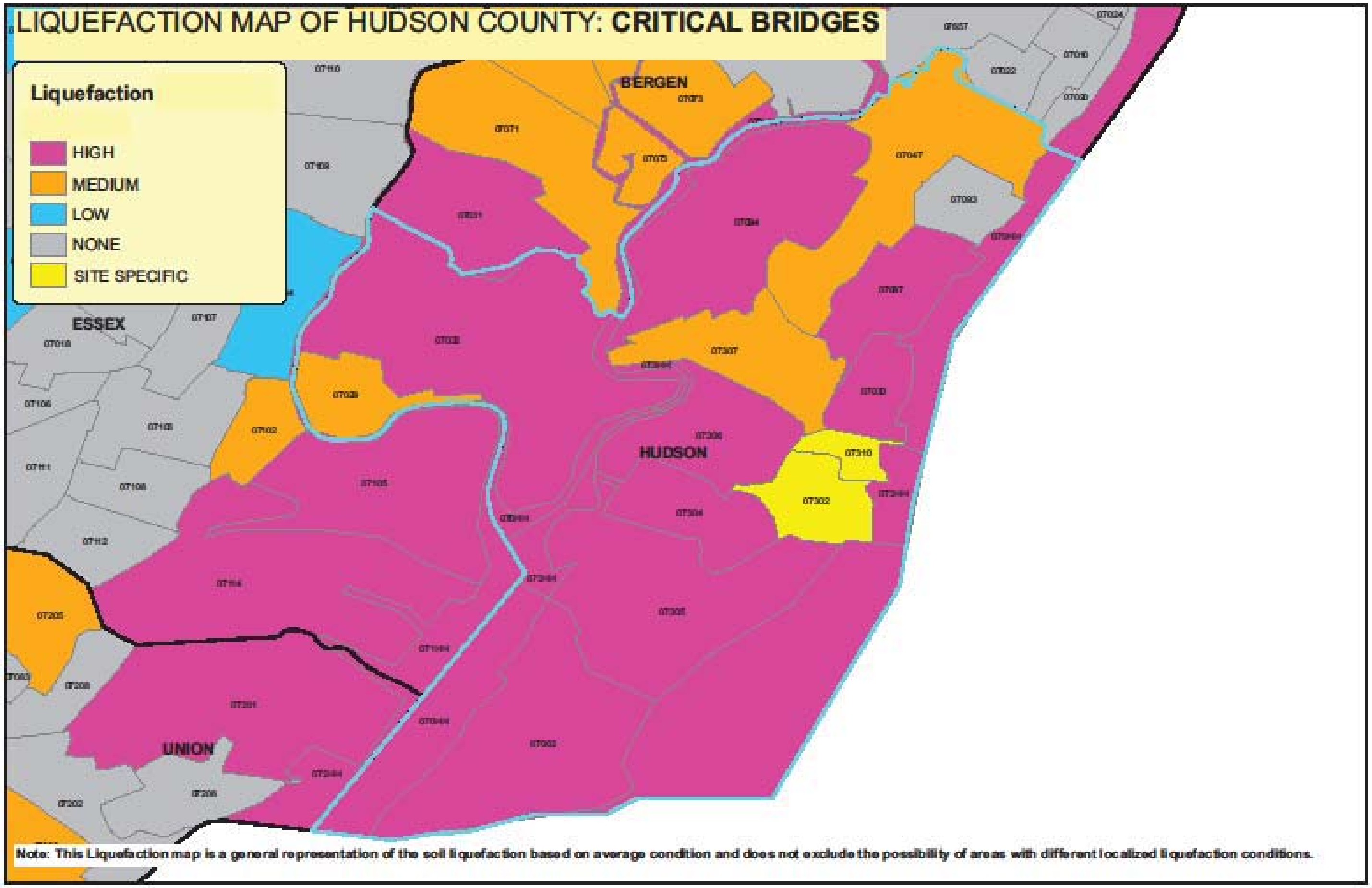


Figure III.31 Liquefaction Map for Critical Bridges in Hudson County of New Jersey

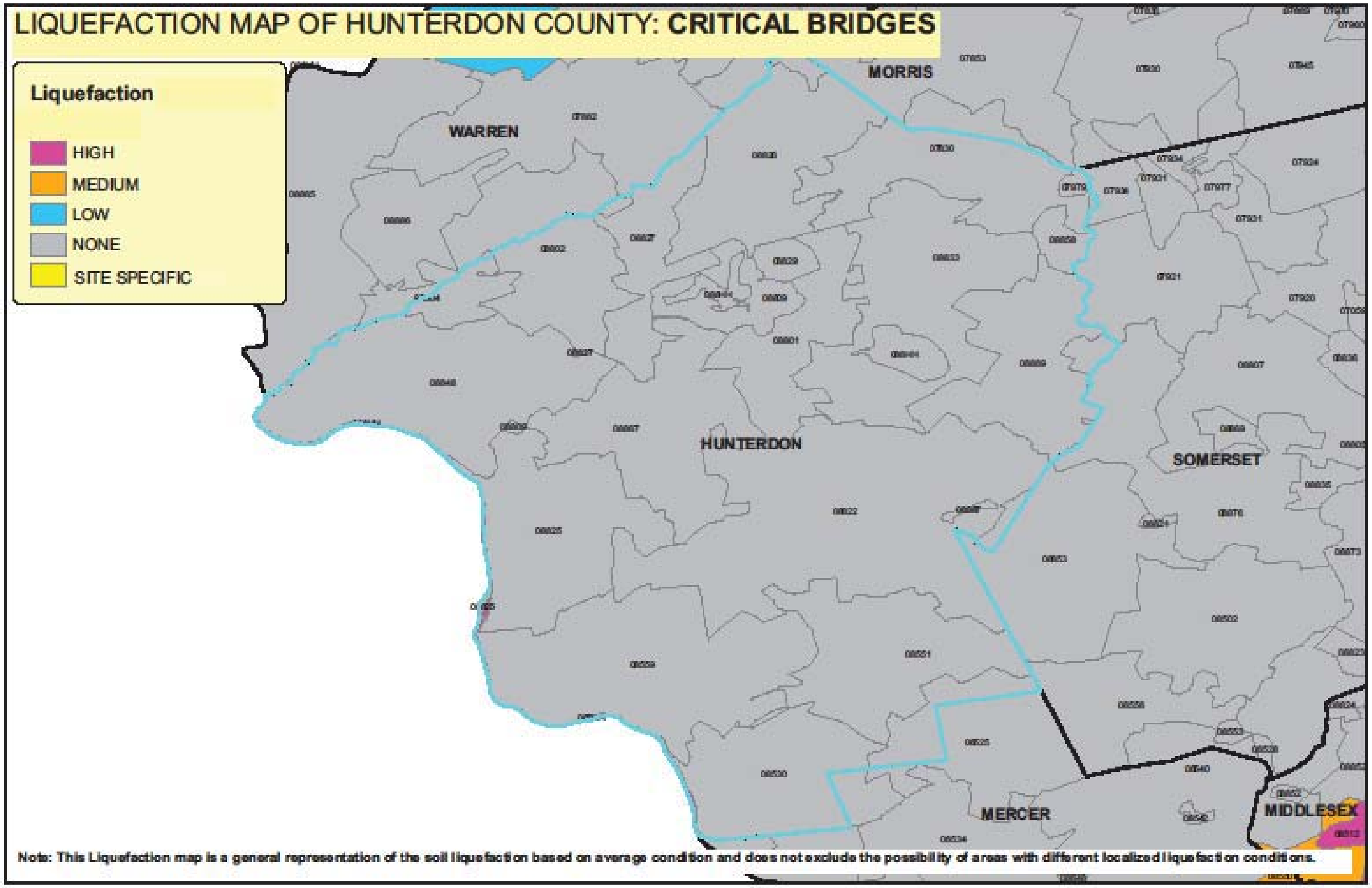


Figure III.32 Liquefaction Map for Critical Bridges in Hunterdon County of New Jersey

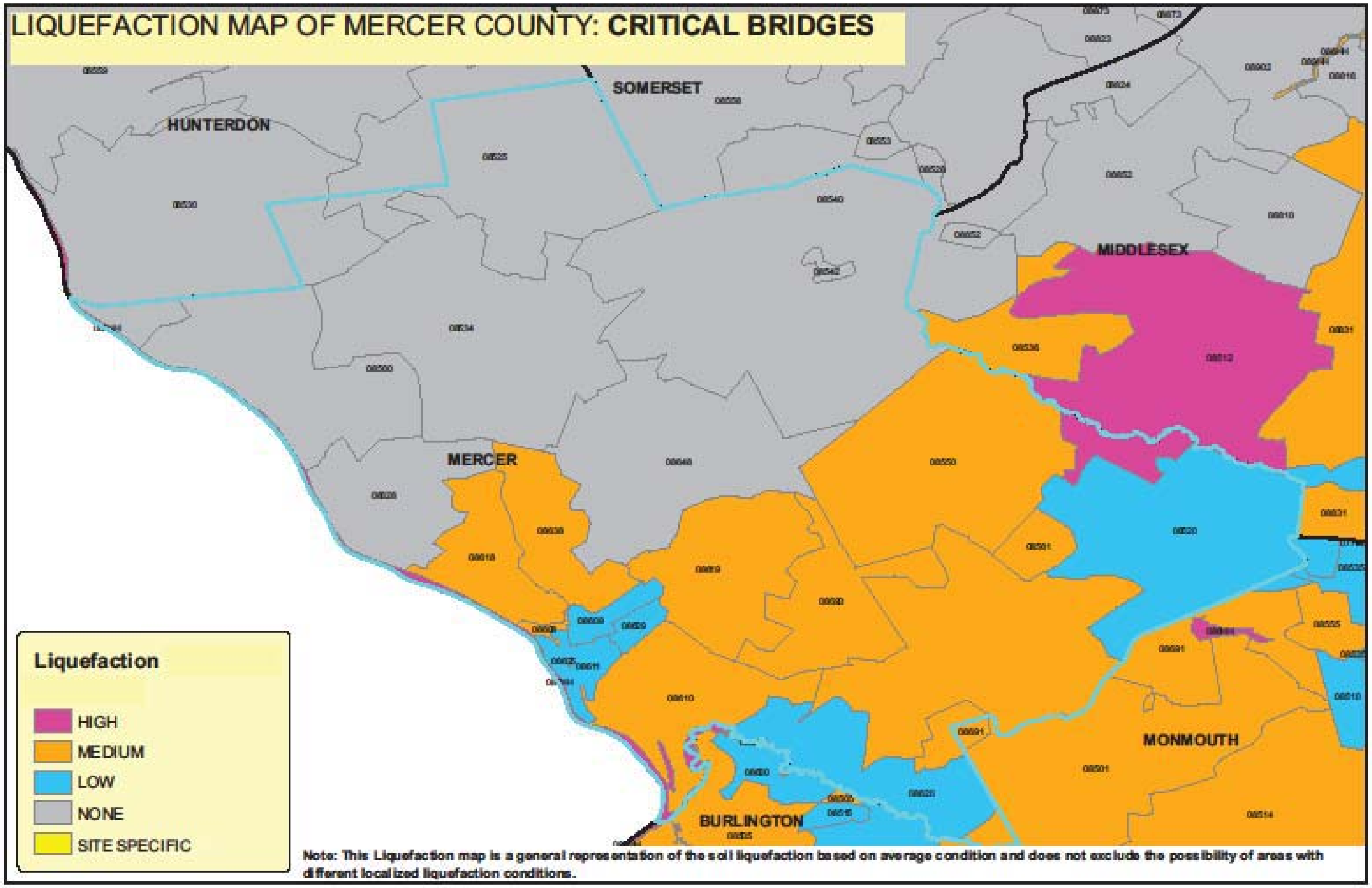


Figure III.33 Liquefaction Map for Critical Bridges in Mercer County of New Jersey

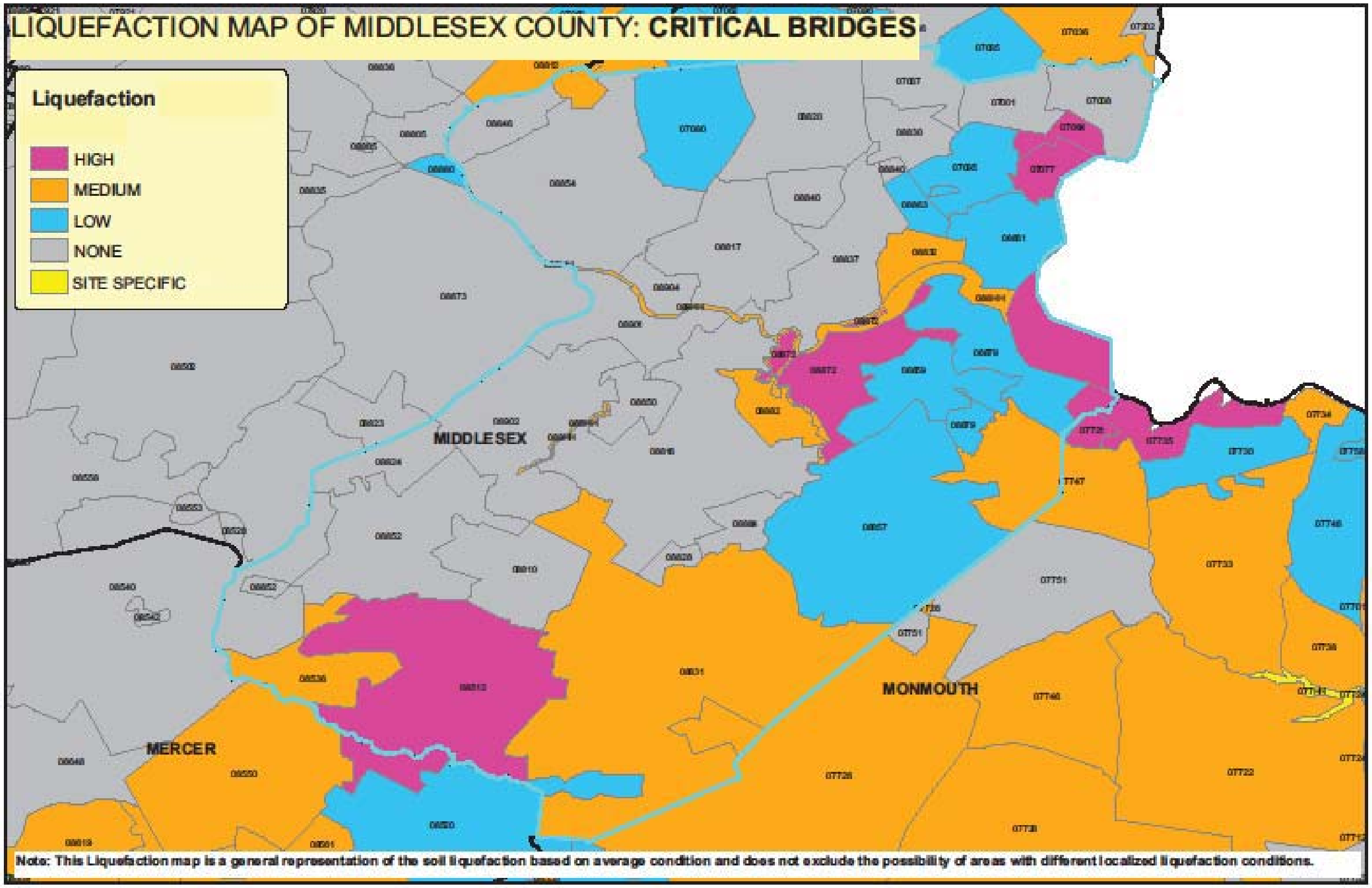
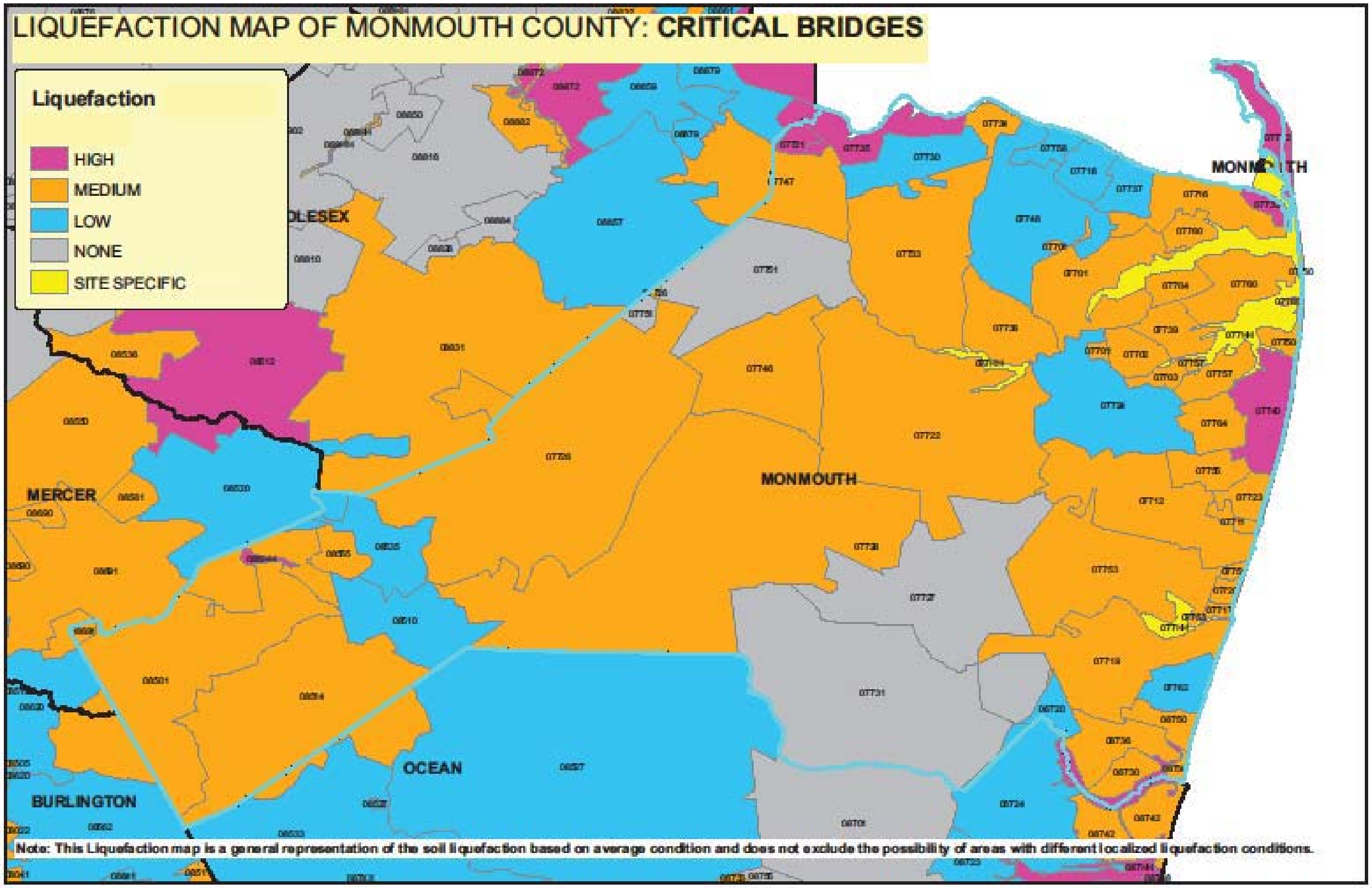


Figure III.34 Liquefaction Map for Critical Bridges in Middlesex County of New Jersey



.Figure III.35 Liquefaction Map for Critical Bridges in Monmouth County of New Jersey

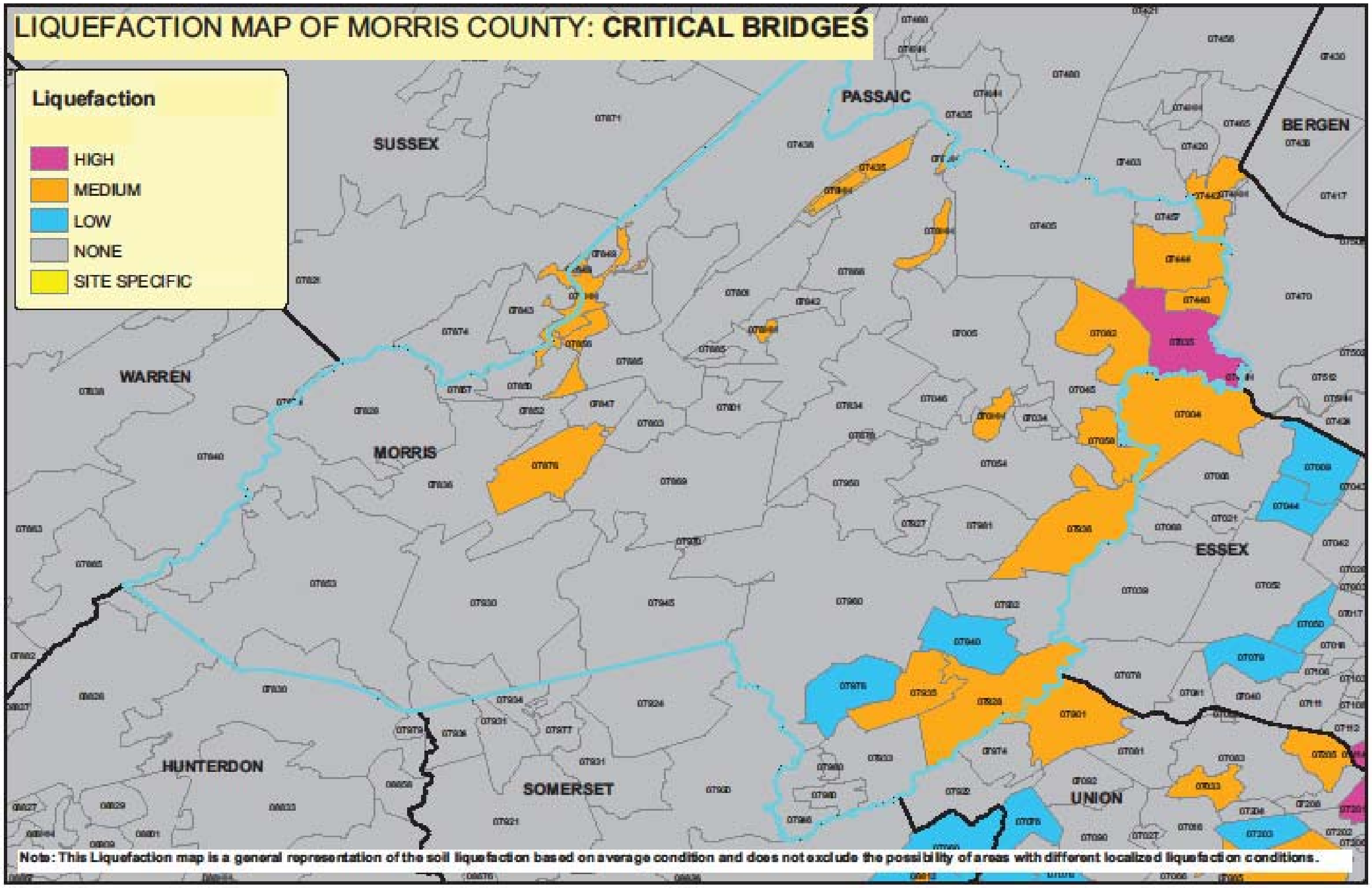


Figure III.36 Liquefaction Map for Critical Bridges in Morris County of New Jersey

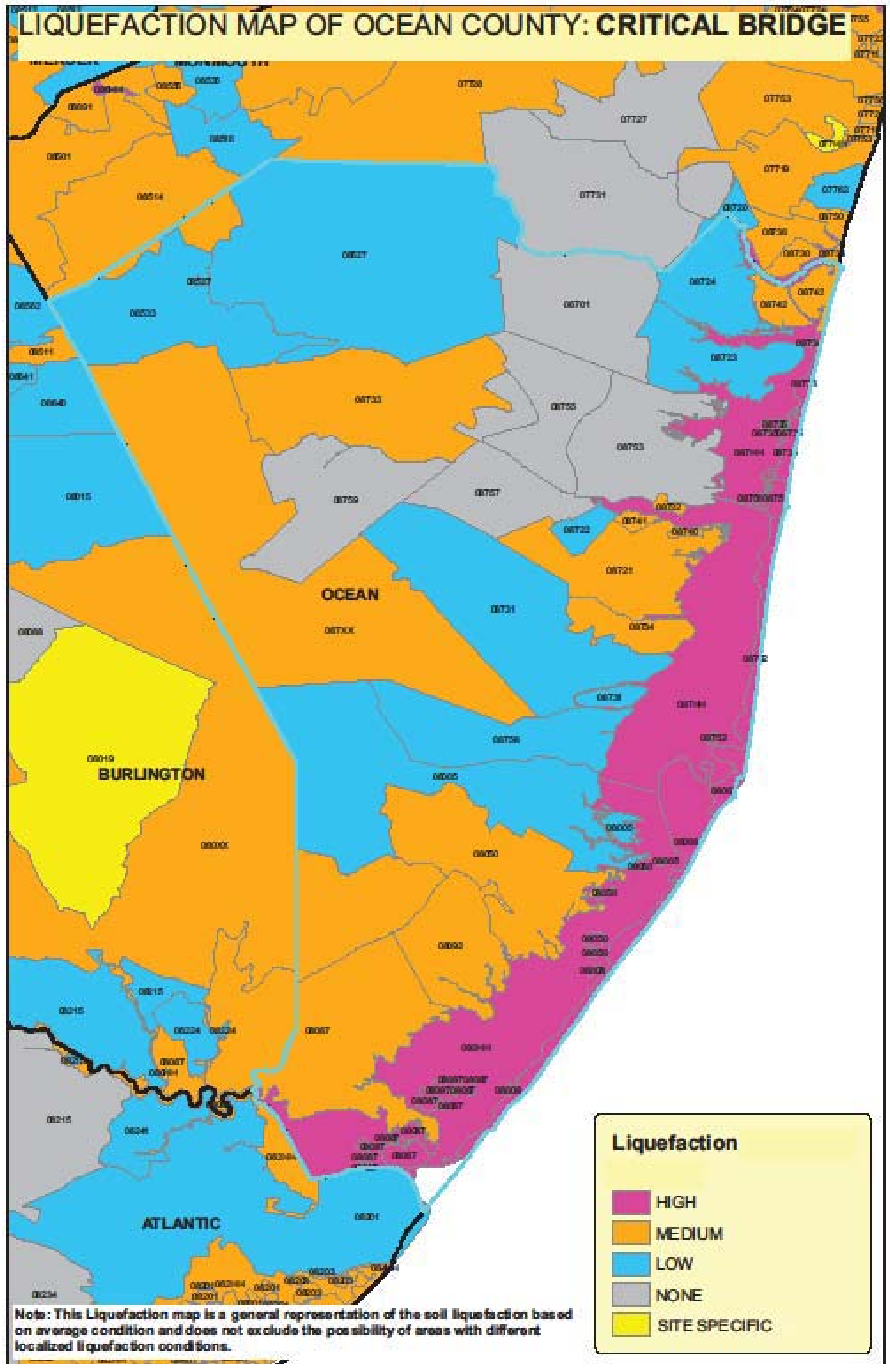


Figure III.37 Liquefaction Map for Critical Bridges in Ocean County of New Jersey

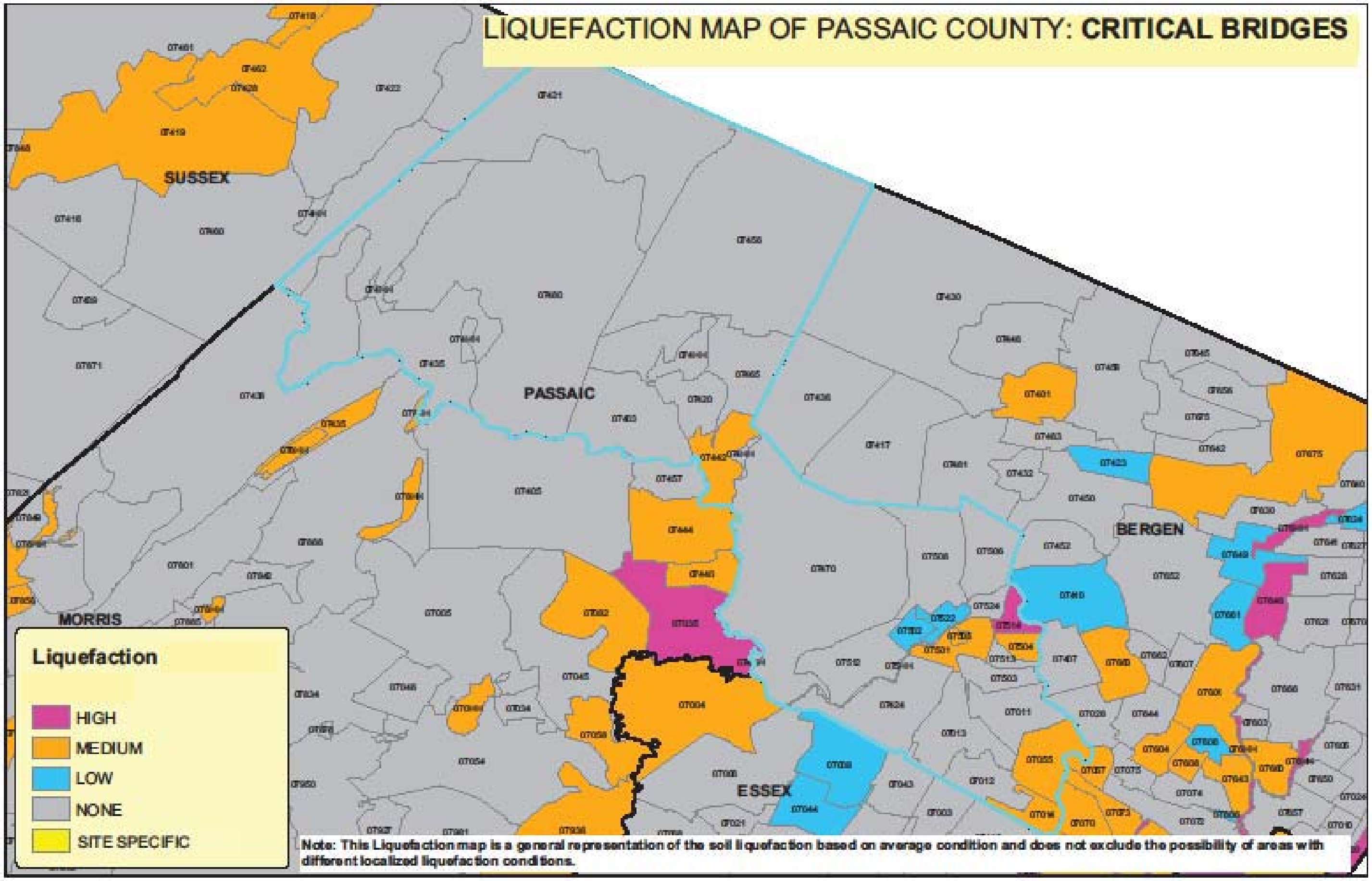


Figure III.38 Liquefaction Map for Critical Bridges in Passaic County of New Jersey

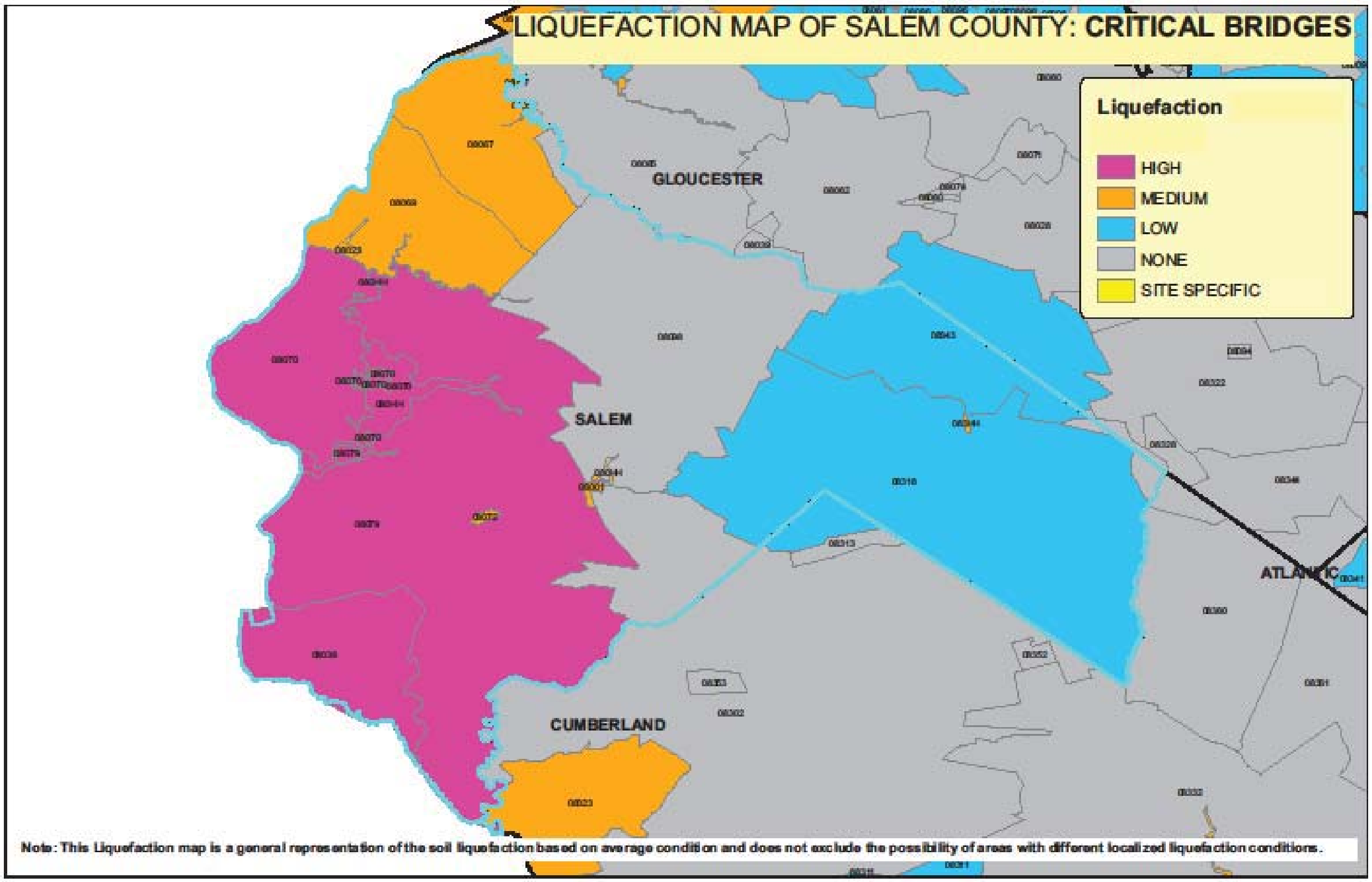


Figure III.39 Liquefaction Map for Critical Bridges in Salem County of New Jersey

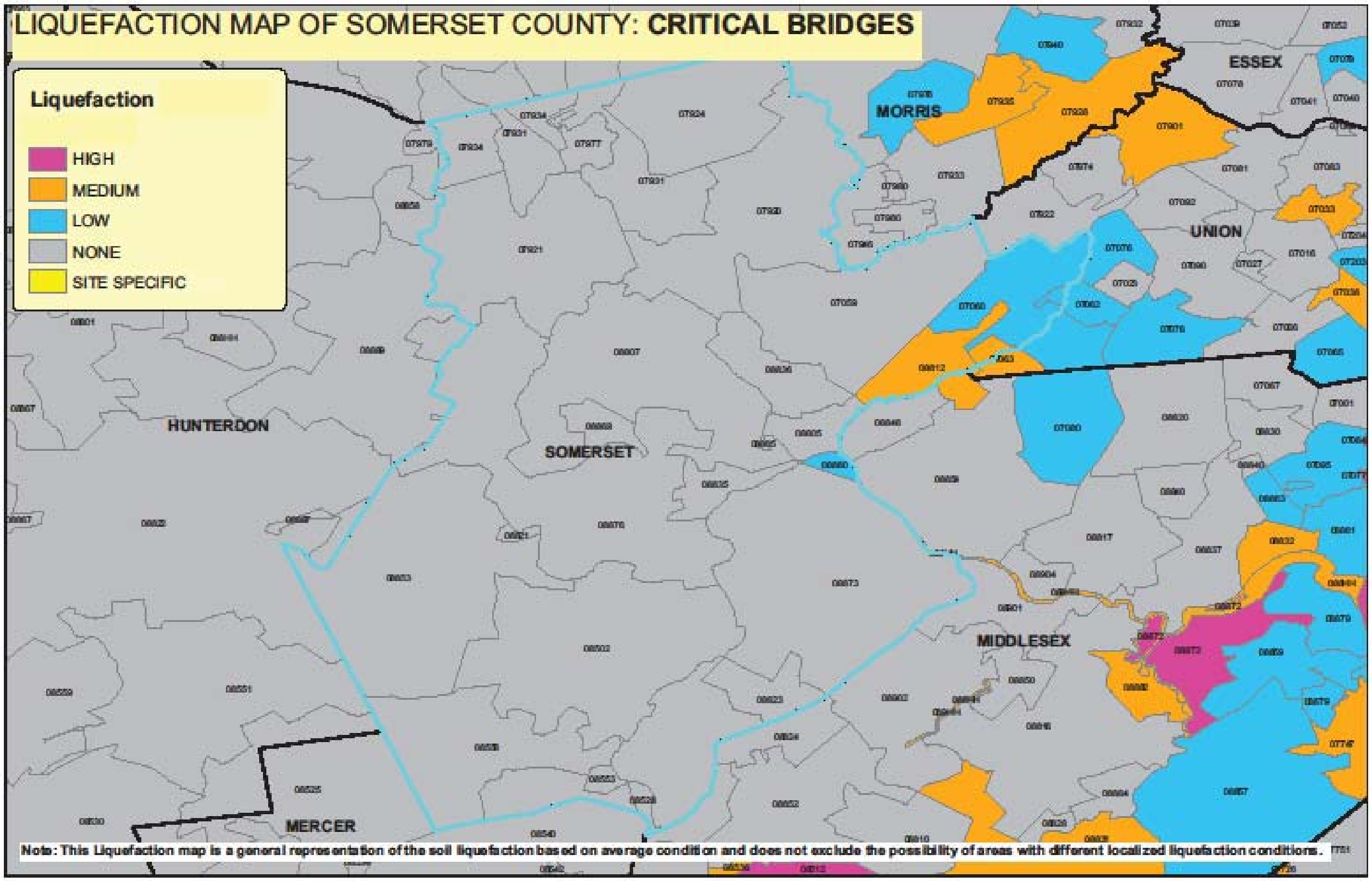


Figure III.40 Liquefaction Map for Critical Bridges in Somerset County of New Jersey

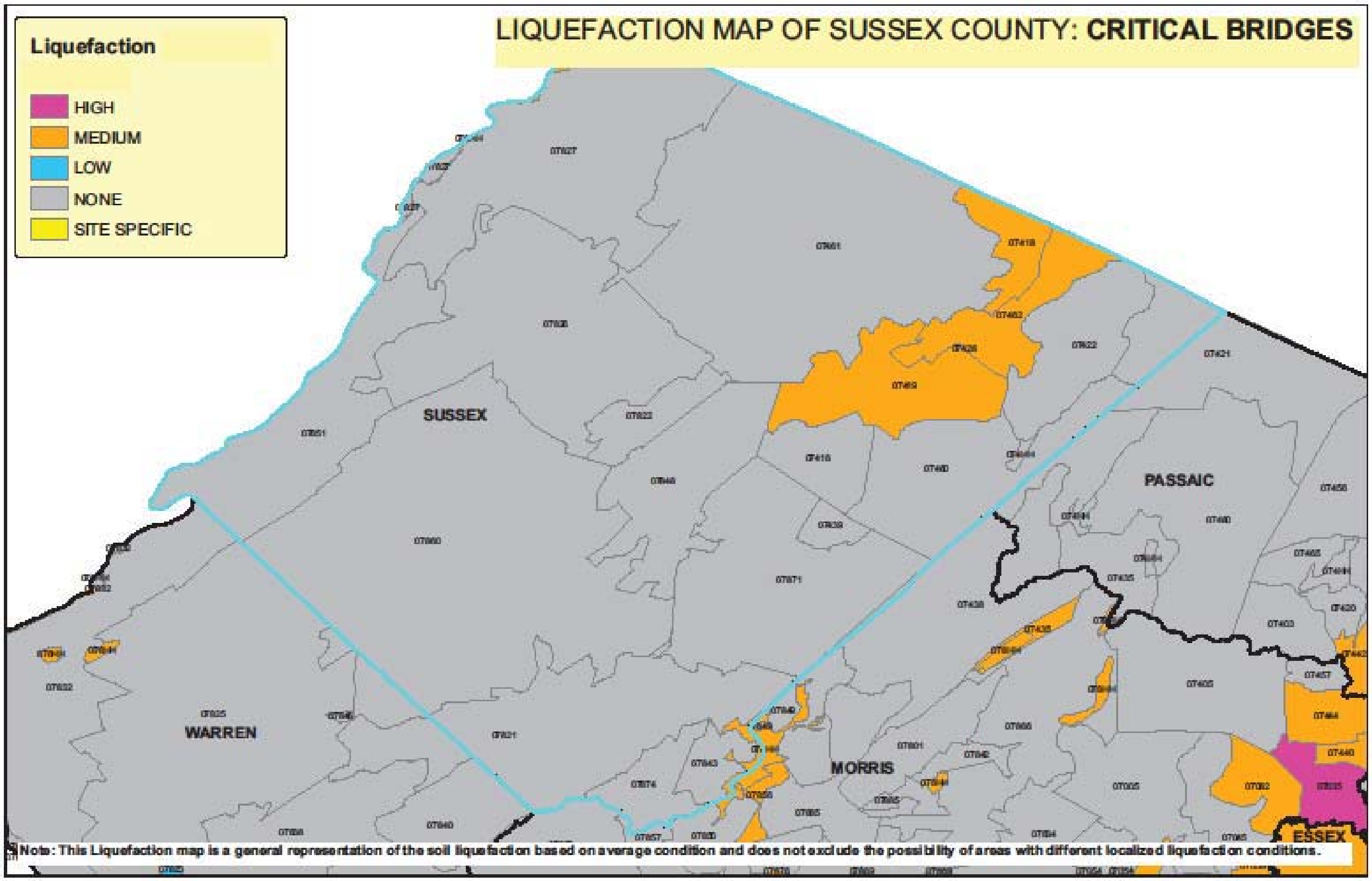


Figure III.41 Liquefaction Map for Critical Bridges in Sussex County of New Jersey

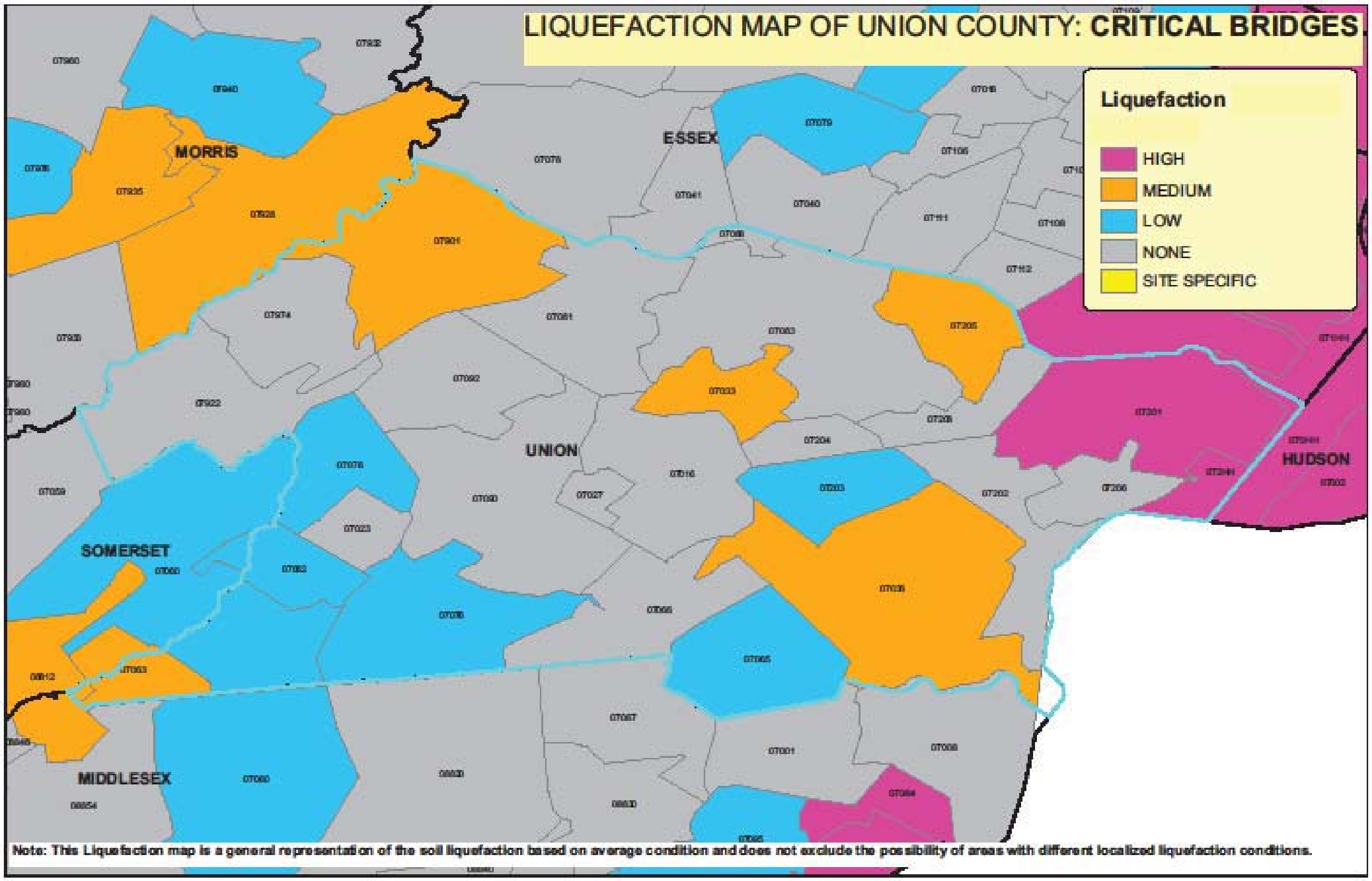


Figure III.42 Liquefaction Map for Critical Bridges in Union County of New Jersey

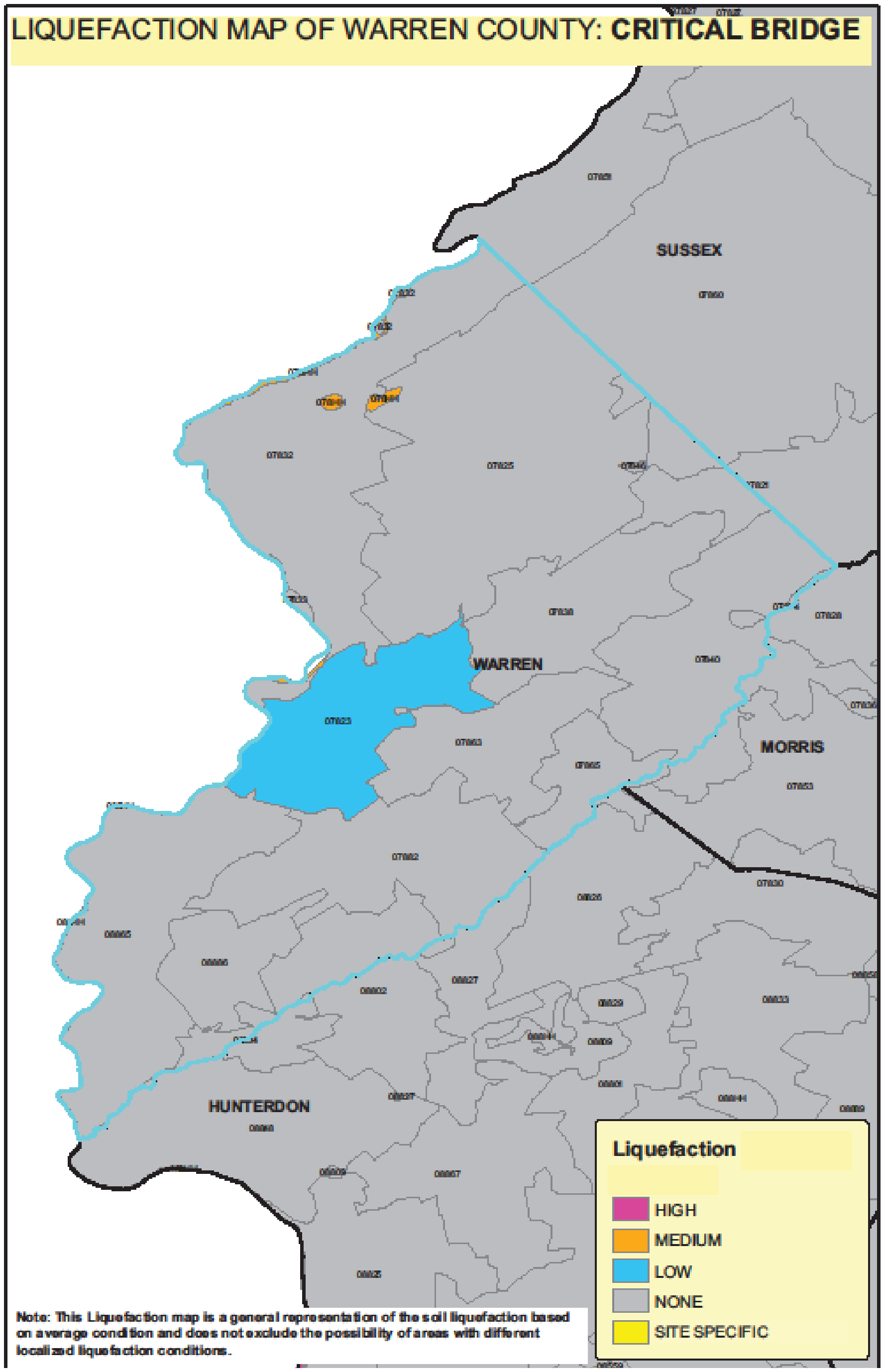


Figure III.43 Liquefaction Map for Critical Bridges in Warren County of New Jersey

User Manual for Site Specific Analysis Tools

Introduction

The entire process of generating site-specific spectra is completed using two separate computer programs. Using these programs, users can generate synthetic surface ground motions by inputting design response spectrum, for example generic spectra based on AASHTO (2008). In the first stage of the process, time-histories of bedrock motions based on the input response spectrum is generated by a 'SIMQKE embedded MATLAB GUI Tool' developed at the City College of New York. These time histories of bedrock motion are used by the computer program "DEEPSOIL", which is available freely, to generate surface ground motion and surface response spectra.

SIMQKE embedded MATLAB GUI Tool

This program provides a graphic user interface, which enables users to generate synthetic bedrock motion time-histories using a design response spectrum.

System requirements and files description

MATLAB must be installed (It can be used with MATLAB version 2007b. However, the current version of this tool was developed under MATLAB version 2009b). The executable file of SIMQKE must be located in the same folder where the MATLAB GUI tool is saved.

User Manual

- Launching the program

  This tool is not a MATLAB independent program. Hence, it must be executed in the MATLAB environment. The user should launch MATLAB first to run the program. As shown in Figure III.44.

  Change the current folder to the work folder, for example, in this case, the folder name is Alpha. After changing the folder, all the files in the folder can be shown in the 'Current Folder' window.

  Type 'Surface_GM_Generating_Alpha1' in the 'Command Window' of MATLAB, the GUI of this tool is prompted (See Figure III.45).

- Inputting the design response spectrum

  Users can input the 6 parameters, PGA, S_s, S_1, F_{pga}, F_a and F_v, which define the design response spectrum. For example, according to AASHTO map, these parameters for Hudson, NJ (zip code: 07029, site class B) are PGA=0.1g, S_s=0.183g, S_1=0.038, F_{pga}=1, F_a=1 and F_v=1.

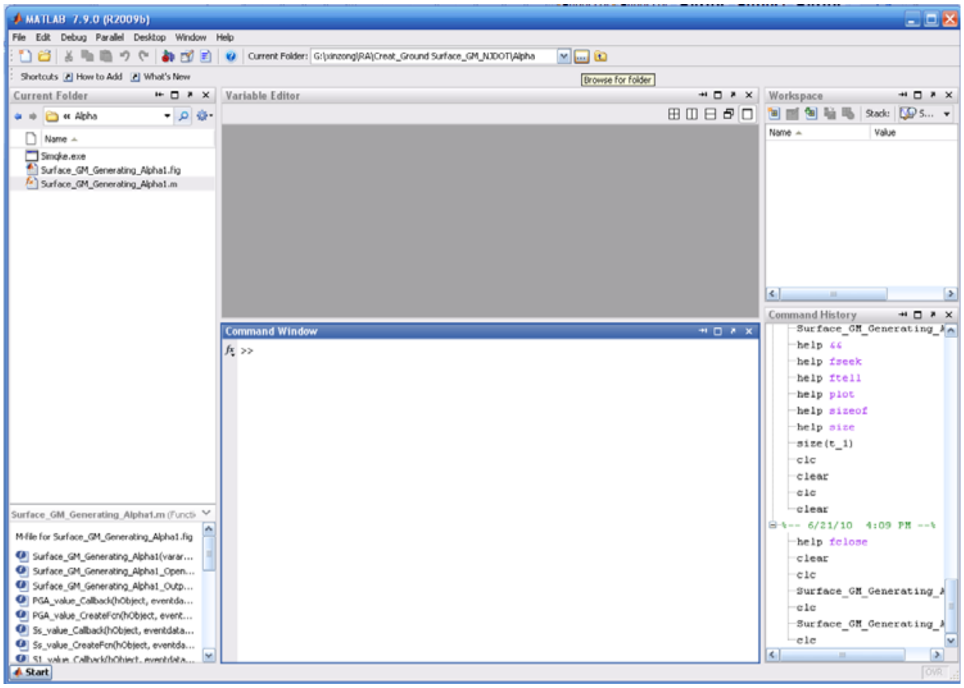


Figure III.44 Launching MATLAB

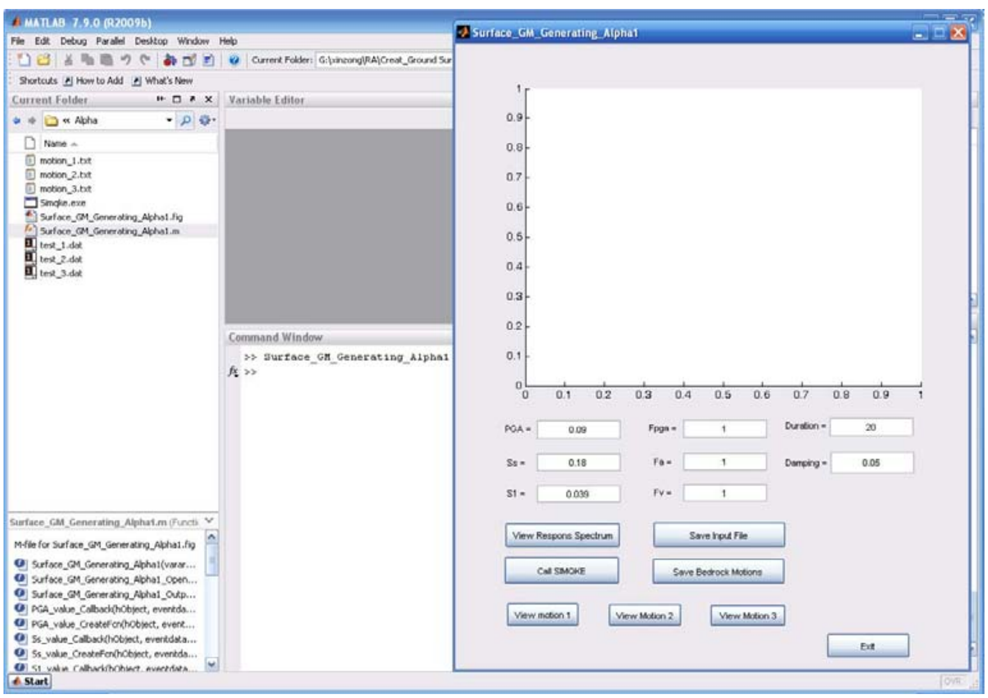


Figure III.45 Launching GUI Tool

In Figure III.45, parameter 'Duration' is the duration of target bedrock motions, which can be changed by the user. However, based on ground motion time histories generated for New York City, a conservative duration of 20 seconds recommended. The damping ratio should be maintained at 0.05, which is the one corresponding to the input response spectrum as per ASSHTO (2008).

After inputting the values of all parameters, click 'View Response Spectrum' button to view the spectrum (See Figure III.46). Please make appropriate changes if the response spectrum hasn't been developed correctly.

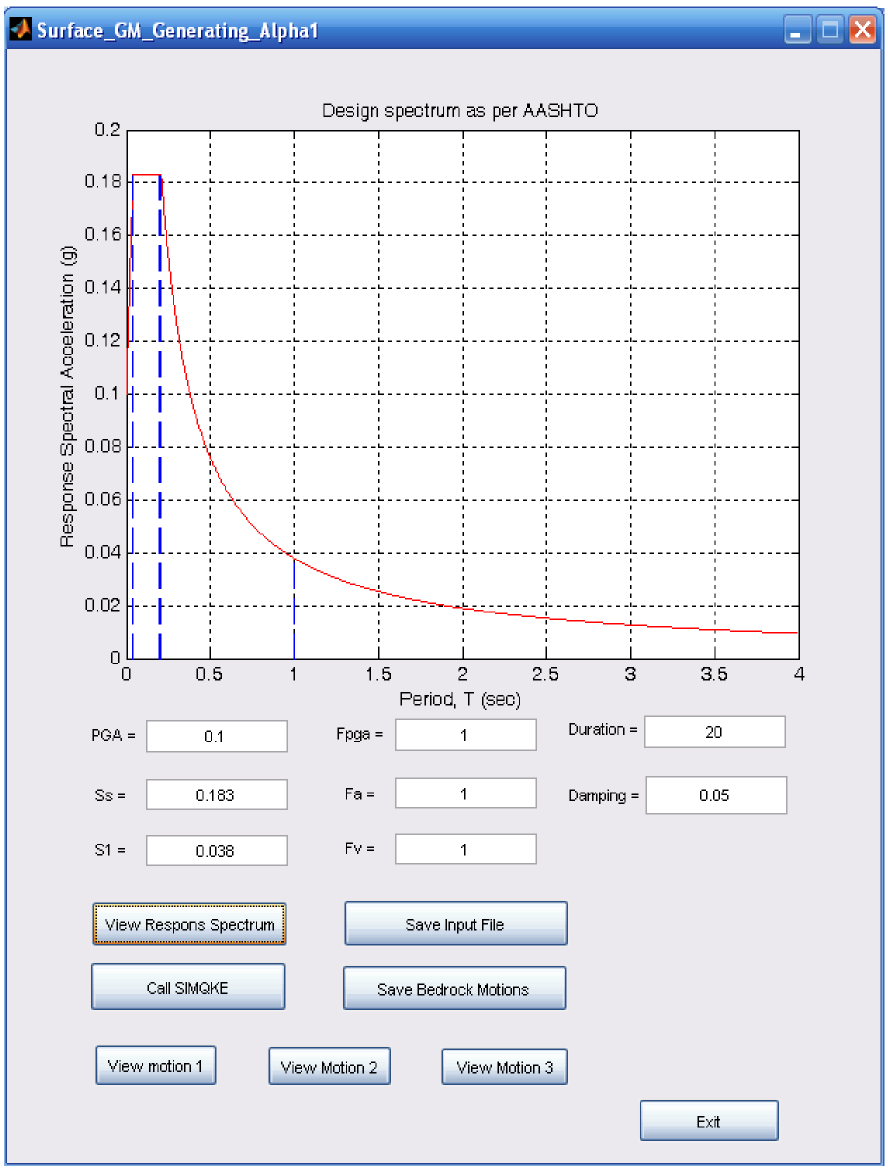


Figure III.46 Inputting parameters for response spectrum

Saving input file for SIMQKE

Once the design response spectrum has been generated, click ‘Save Input File’ button to create and save an input file for SIMQKE analysis. The file name is set to be ‘input.dat’, and **can’t be changed**.

Running SIMQKE

Once the input file has been saved, launch SIMQKE by clicking ‘Call SIMQKE’ button to prompt SIMQKE in MATLAB main window (see Figure III.47).

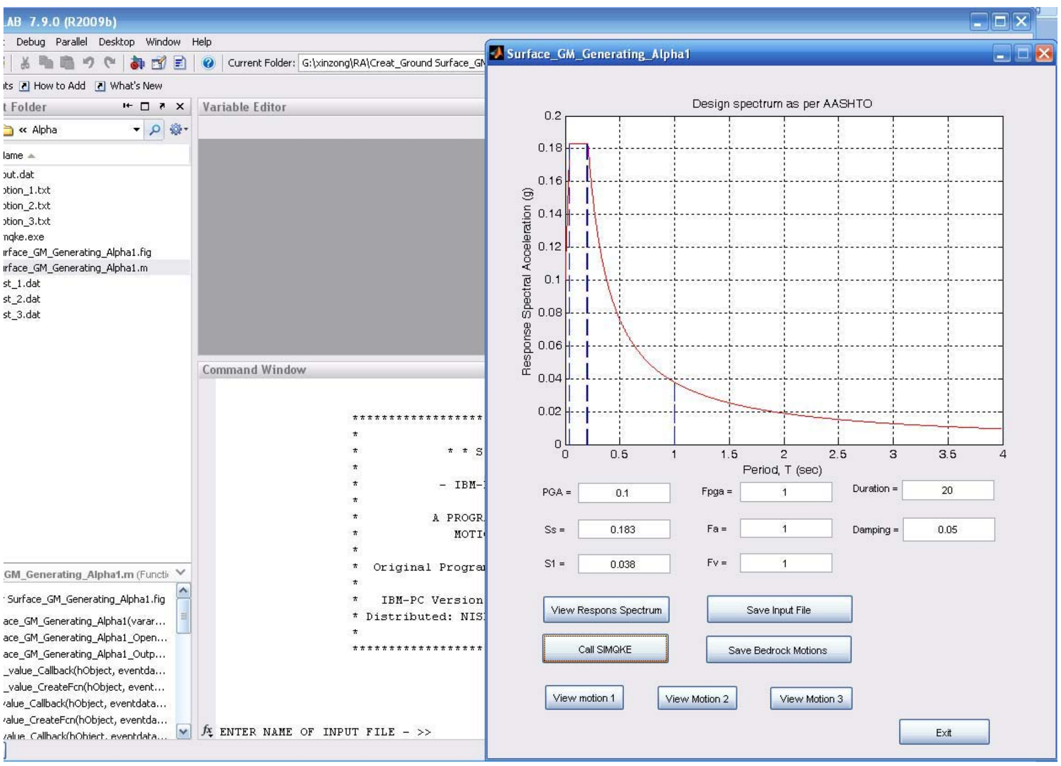


Figure III.47 Prompting SIMQKE

The user will need to enter the name of the input file in the main window of MATLAB. Input the input file name in the ‘Command Window’ as it says, (See Figure III.48, NOTE AGAIN that the file name has to be ‘input.dat’), and press ‘Enter’ button on your keyboard.

You will be asked to enter the name of the output file in the main window of MATLAB (e.g., ‘ENTER NAME OF OUTPUT FILE’). Type a file name for output file after the short dash, and press ‘Enter’. NOTE: The name of the output file MUST be ‘output.dat’ in this version, otherwise there will be errors and the motions cannot be generated.

When this process is completed, user can see a command line saying ‘Stop – Program terminated’ (See Figure III.48). At this point, the user can switch back to the GUI tool.

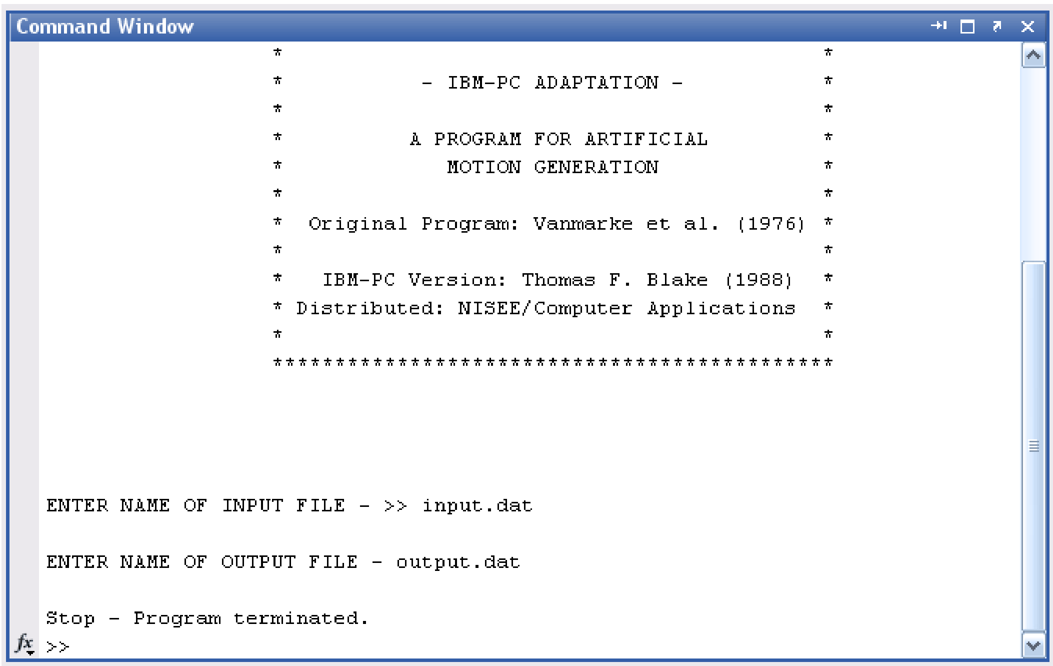


Figure III.48 Running SIMQKE

<u>Saving and viewing bedrock motions</u>

Once switched back to the GUI tool, user can click ‘Save Bedrock motions’ button to save the motion time histories to files named ‘motion_1.txt’, ‘motion_2.txt’ and ‘motion_3.txt’, which are located in the work folder set in the beginning. The three time-histories that are compatible to the input response spectrum are generated according to AASHTO (2008) (See Figure III.49). Users can view them by clicking ‘View Motion 1’, ‘View Motion 2’ and ‘View Motion 3’, respectively.

These motions will be the input motions for DEEPSOIL to use to conduct the site response analysis. See later subsection on “Motion & Output Control” or “STEP 4/6” for reference.

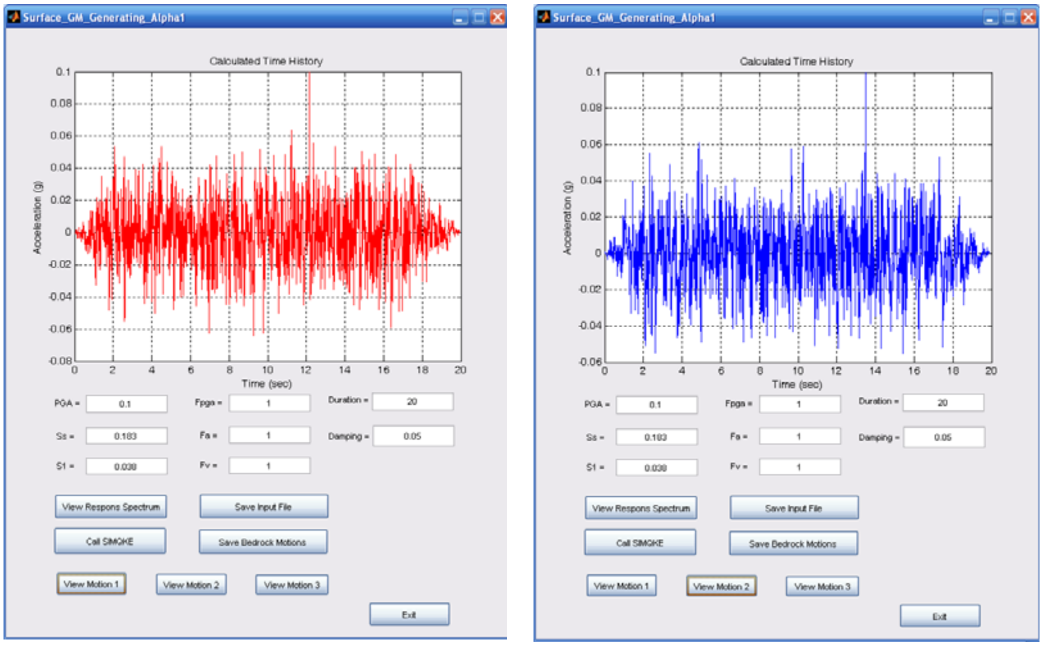


Figure III.49 Viewing bedrock motions

DEEPSOIL User Manual

DEEPSOIL Structure

The DEEPSOIL graphic user interface consists of 5 (for equivalent linear analysis) / 6 (for nonlinear analysis) stages and intuitively guides the user from the beginning to the end of the 2nd stage (from bedrock motion to surface motion) of site-specific ground response analysis. In this application, only the equivalent-linear analysis is necessary.

Initialization

After starting the DEEPSOIL program, the user is presented with the initialization screen shown in Figure III.50.

At this stage, the user must select whether Standard or Batch Mode analysis will be performed. In the Standard analysis, the user defines a profile and corresponding properties and propagates a single input motion through the profile. The Standard Mode is always used in our applications.

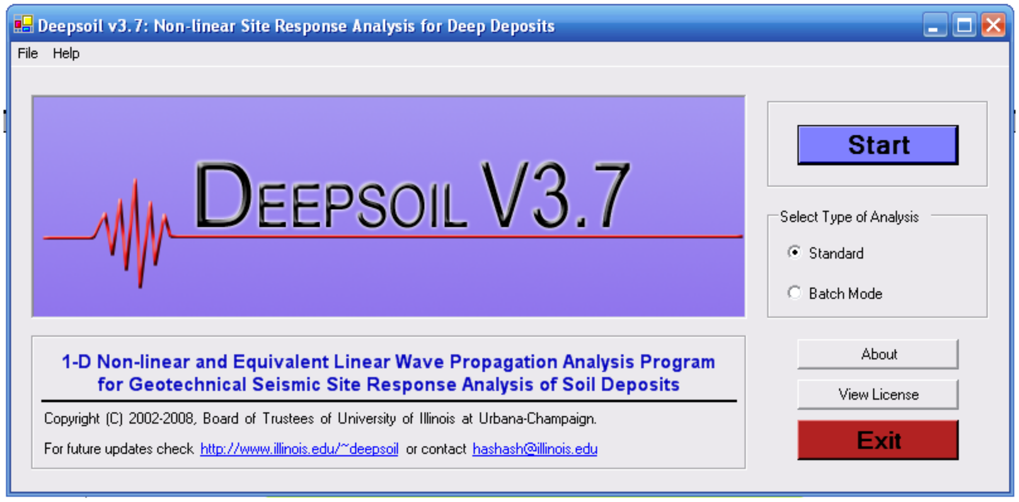


Figure III.50 DEEPSOIL Start/Initialization screen

The “About” button provides summary data regarding the DEEPSOIL program.

“View License” will allow the user to review the DEEPSOIL license agreement.

The “Exit” button closes the DEEPSOIL program.

Once the user has selected either the Standard or Batch Mode analysis, the “Start” key will initialize the corresponding program. If the user is using the program for the first time, the license agreement will appear and will need to be accepted before the program will function.

<u>Selection of Analysis Type: Step 1 of 6</u>

The first step in the analysis requires the selection of analysis type. Figure III.51 illustrates the form of Step 1. At this stage, the user may either: a) open a previously saved profile by clicking the “Open Existing Profile” button, or b) create a new analysis. The user may also specify a workspace or “working directory” to use during this session.

Before creating a new profile or opening an existing profile, it is recommended to verify the “Current Workspace Directory” at the bottom of the page. The DEEPSOIL “Working” directory is chosen by default. If a different directory is preferred, press the “Change Work Space” button to bring up a folder browser and select the new directory. The specified directory should be automatically updated in Step 1/6.

To use a previously saved profile, click the “Open Existing Profile” button located at the top-right corner of the form. A browser window will appear which allows the user to navigate folders to find an existing profile. Note that the default directory will be either:

a) the user-defined working directory, or b) the DEEPSOIL program directory (if the user-defined working directory does not exist).

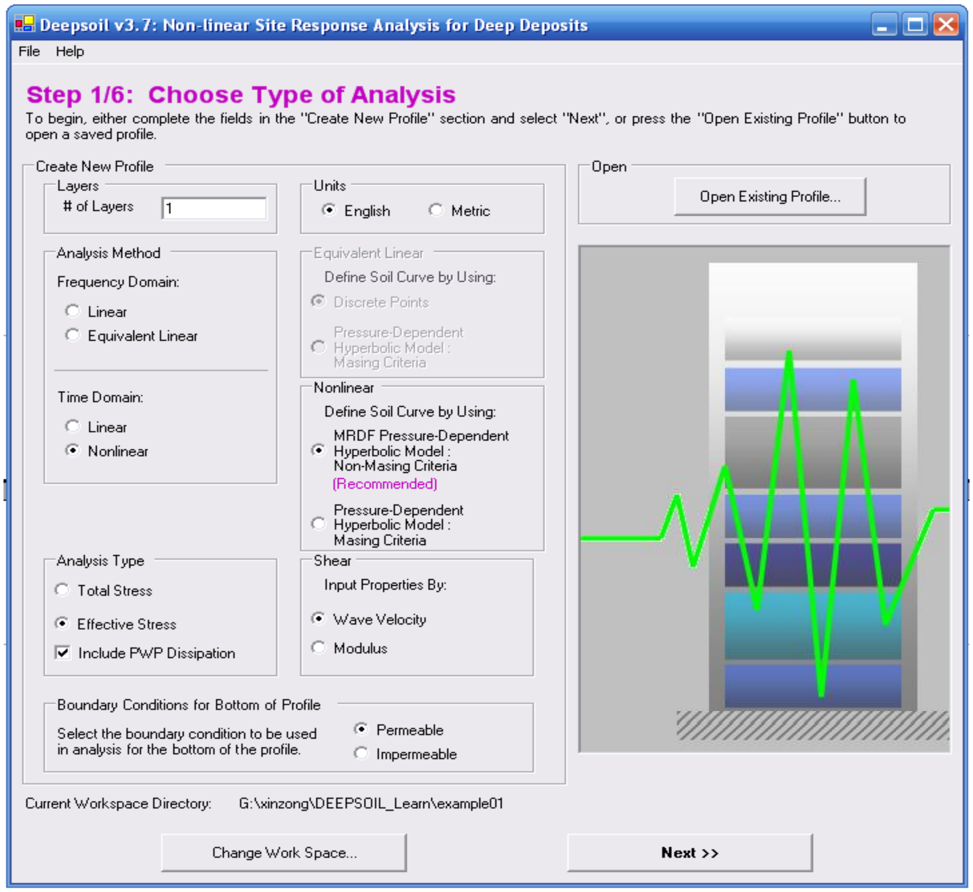


Figure III.51 Step 1/6: Choosing type of analysis

To create a new analysis, the user must specify the type of analysis before proceeding to the next stage of analysis. The user must specify:

1. The number of layers to be used in the profile.
2. The analysis method:
   - Frequency Domain
     - Linear
     - Equivalent Linear
   - Time Domain
     - Linear

    - Nonlinear

3. The type of input for shear properties:
   - Shear Modulus
   - Shear Wave Velocity
4. The units to be used in analysis:
   - English
   - Metric
5. The analysis type:
   - Total Stress Analysis
   - Effective Stress Analysis (Pore Water Pressure generation only)
     - Include PWP Dissipation (PWP generation and dissipation)
6. The method to define the soil curve:
   - For Equivalent Linear
     - Discrete Points
     - Pressure-Dependent Hyperbolic Model
   - For Nonlinear
     - MRDF Pressure-Dependent Hyperbolic Model
     - Pressure-Dependent Hyperbolic Model
7. The boundary conditions (for Effective Stress Analysis Incl. PWP Dissipation)
   - Permeable
   - Impermeable

The Effective Stress Analysis option is only available for a Nonlinear (Time Domain) analysis. Note that (1), (3) and (4) can be changed in the next stage.

For this application, always choose Equivalent Linear for (2), Total Stress Analysis for (5), and Discrete Points for (6).

<u>Defining Soil Profile & Model Properties: Step 2a of 6</u>

This stage is divided into two partitions. The partition to be considered requires the user to define the soil profile and specify the soil properties of each layer (Figure III.52). The type of input required depends on the analysis parameters selected in Step 1.

The entire form is broken up into three sections. The section located at the left is a visual display of the soil profile. The section at the right is the table where the values for required input parameters must be entered, but the location of the water table must also be specified. The section in the middle contains layer property information, conversion functions, and soil modifier commands.

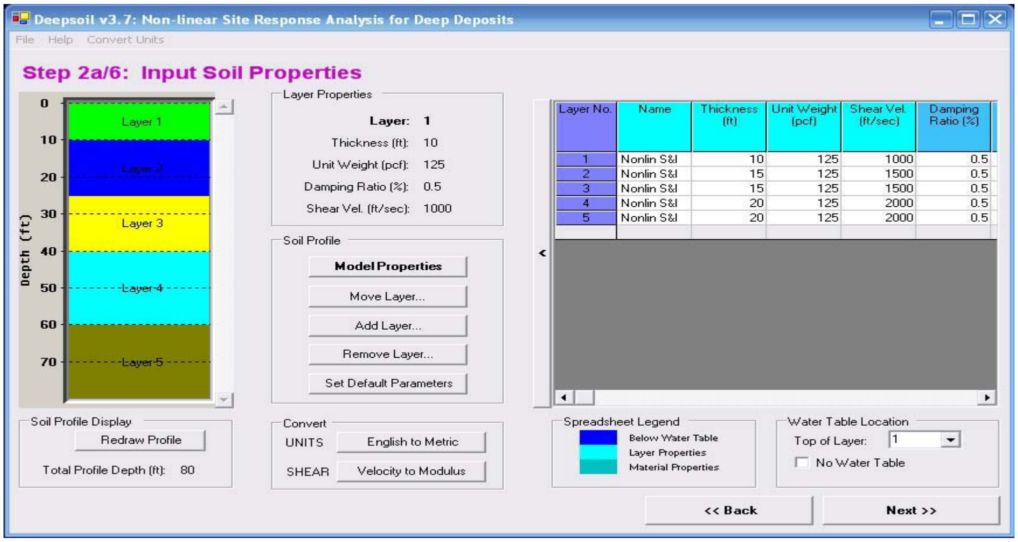


Figure III.52 Step 2a/6: Inputting Soil Properties

If a total stress analysis is selected, the user must specify the typical soil properties of each layer based on the type of analysis that was selected.

Checking Maximum Frequency (for Time Domain Analysis only)

This step is not necessary in our applications.

<u>Defining Rock Properties: Step 2b of 6</u>

After defining the soil and model properties, the user must now define the rock/half-space properties of the bottom of the profile (Figure III.53).

The user has the option of selecting either an Elastic Half-Space or a Rigid Half-Space. An information display makes the user aware that a rigid half-space should be chosen if a within motion will be used, and an elastic half-space should be selected if an outcrop motion is being used. If a rigid half-space is being used, no input parameters are required. If an elastic half-space is being used, the user must supply the shear velocity (or modulus), unit weight, and damping ratio of the half-space.

Bedrock properties can be saved by giving the bedrock an appropriate name and pressing **Save Bedrock**. The new bedrock will appear in the list of saved bedrocks below. To use saved bedrock, select the appropriate file from the list box and press the **Show Bedrock** button.

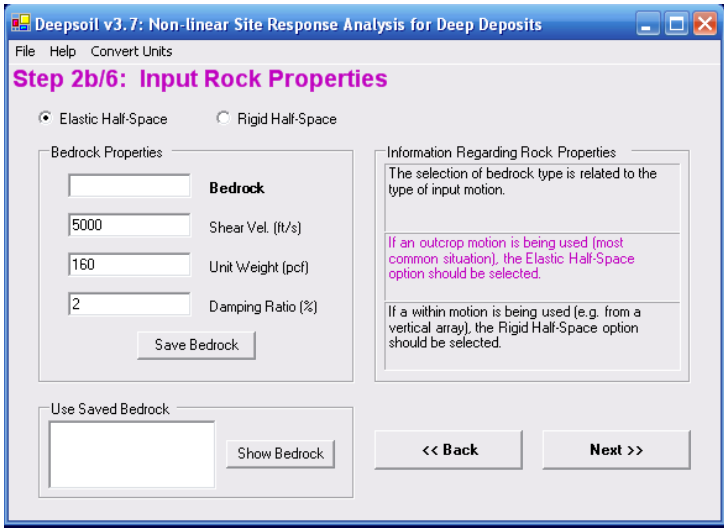


Figure III.53 Step 2b/6: Input Rock Properties

<u>Analysis Control: Step 3 of 6</u>

In this stage of analysis, the user may specify options to be used either the frequency domain or time domain analysis (Figure III.54). For this application, select “Frequency Domain Analysis”.

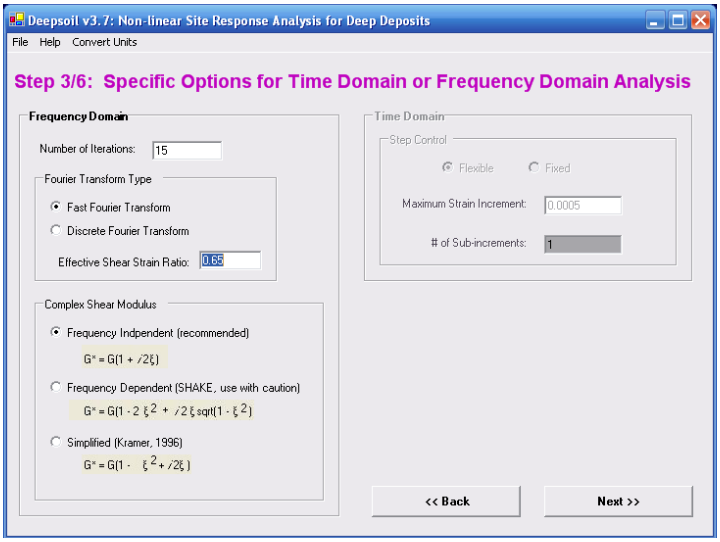


Figure III.54 Step 3/6: Specific Options for Time Domain or Frequency Domain Analysis

Motion & Output Control

Based on the user's selected option during the Initialization stage, the options available in this stage of analysis will vary depending on if a Standard (Figure III.55) or Batch Mode analysis is being performed. In both cases, the input motion(s) and layer(s) for output display will be selected. The user should also choose the damping ratio for the calculated response spectra. The default damping ratio is set to 5%. If the Standard option was chosen during the Initialization stage, deconvolution can also be performed. Generally deconvolution is not needed.

The motion control stage allows the user to specify the input motion to be used in analysis and to select of the layers to be analyzed. In this application, the users should use the three generated time-histories as input motions. The layers to be analyzed may be selected by checking the appropriate checkbox at the left of the form.

The number of calculation points is relevant in the frequency domain analysis and should be specified when using the Fast Fourier Transform. Note that DEEPSOIL will provide an estimate of the number of points to be used for any input motion.

Further options include: a) Convert Input Motion, b) Baseline Correction, and c) Deconvolution (for Standard analysis mode). The Damping Ratio of the output response spectra should also be specified.

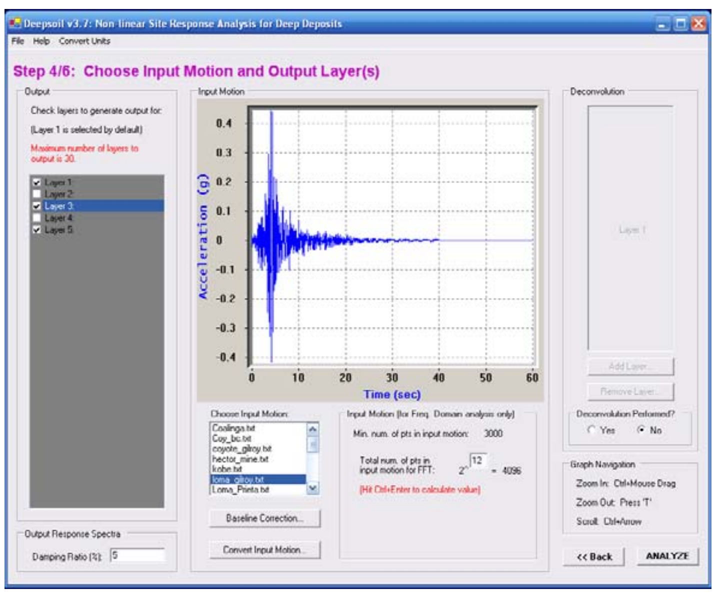


Figure III.55 Step4/6: Choose Input Motion and Output Layer(s) [Standard Analysis].

If the analysis type is a Frequency Domain Analysis, the user may click the "ANALYZE" button to perform the analysis.

If the analysis type is a Time Domain Analysis, the user must click the "Next" button to proceed to Step 5 of the analysis. This option is not used in this application.

Viscous Damping Formulation / Optimum Modes Selection: Step 5 of 6

This stage will only appear for time domain analysis, and is not necessary in this application.

Output: Step 6 of 6

Upon completion of analysis, the following output for each selected layer will be directly exported to a file the user specified (By pushing "OK" button in the reminder window, see Figure III.56, and specifying a filename and folder, see Figure III.57):

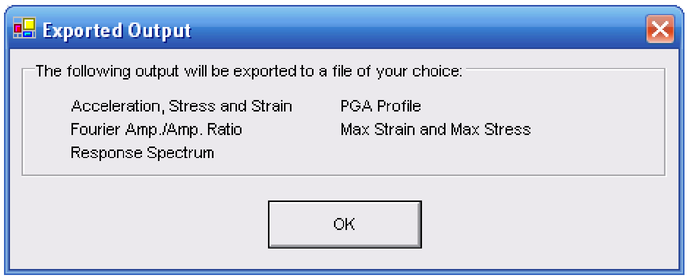


Figure III.56 Reminder Window for Exported Output

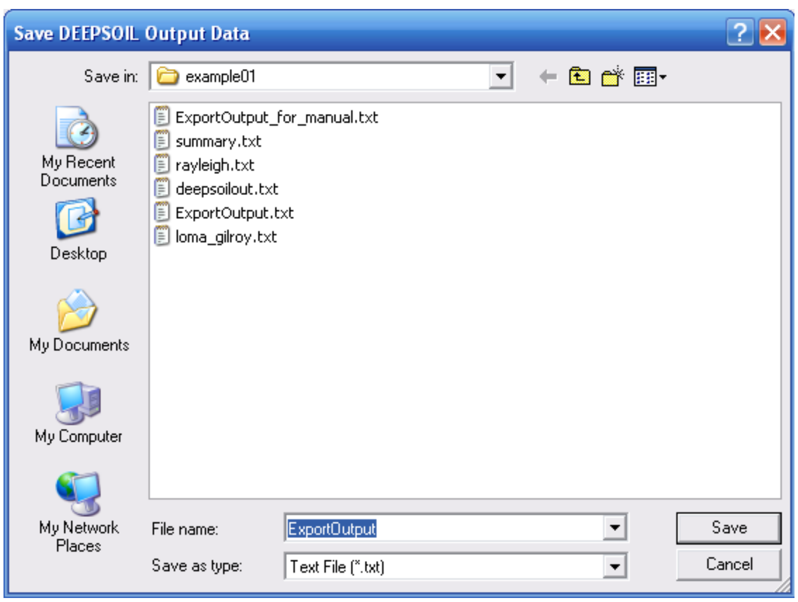


Figure III.57 Export Output data to file

Output from the "Total Stress Analysis"

- Acceleration (g) vs. Time (sec)

- Strain (%) vs. Time (sec)
- Stress (shear/effective vertical) vs. Time (sec)
- Response Spectra: PSA (g) vs. Period (sec)
- Fourier Amplitude (g-sec) vs. Frequency (Hz)
- Fourier Amplitude Ratio (surface/input) vs. Frequency (Hz)
- PGA Profile: Max PGA vs. Depth
- Strain Profile: Max Strain vs. Depth

For “Effective Stress Analysis”, in addition to above output, we get the following output,

- Pore Water Pressure (pwp/effective vertical) vs. Time (sec)
- PWP Profile: Max PWP vs. Depth

If a Batch Mode analysis was selected, the user will be notified to move the exported data to a safe directory and will then close out the session. For a Standard analysis, the user may immediately view the following output visually (Figure III.58) by selecting the appropriate tab for the selected layer:

- Acceleration (g) vs. Time (sec)
- Strain (%) vs. Time (sec)
- Stress (shear/effective vertical) vs. Time (sec)
- Stress (shear/effective vertical) vs. Strain (%)
- Fourier Amplitude (g-sec) vs. Frequency (Hz)
- Fourier Amplitude Ratio (surface/input) vs. Frequency (Hz)
- Response Spectra: PSA (g) vs. Period (sec)

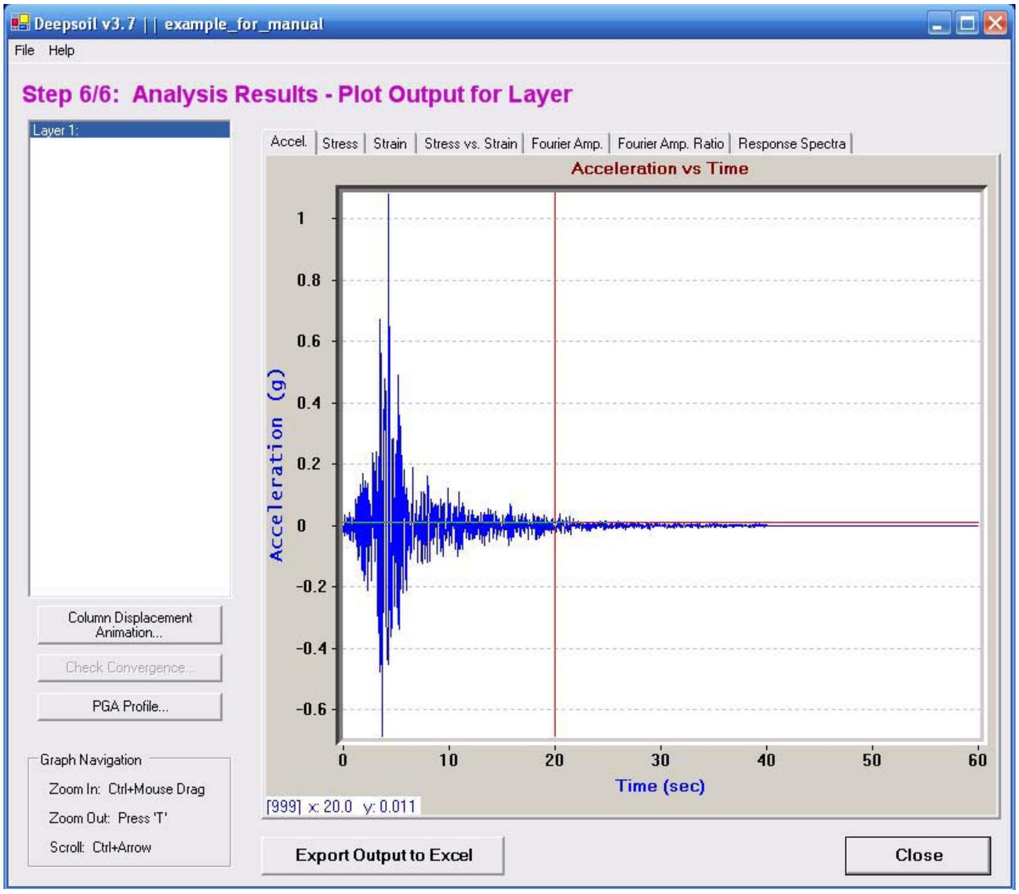


Figure III.58 Step 6/6: Analysis Results – Plot Output for Layer

<u>Example: Equivalent Linear Frequency Domain Analysis / Multi-Layer, Elastic Rock</u>

Three soil profiles are shown on Figure III.59. For every available soil profile (e.g., the left most one in Figure III.59), two additional soil profiles should be generated by increasing or decreasing the shear wave velocity by 20% with respect to the shear wave velocity of the given soil profile. For every soil profile, 3 motions from the same design spectrum (see previous subsection) should be analyzed. Hence, we need to analyze $3\times3=9$ cases.

This example is prepared to guide the users through various features of DEEPSOIL for one case, and user can follow the same procedure to complete the rest of the 8 cases.

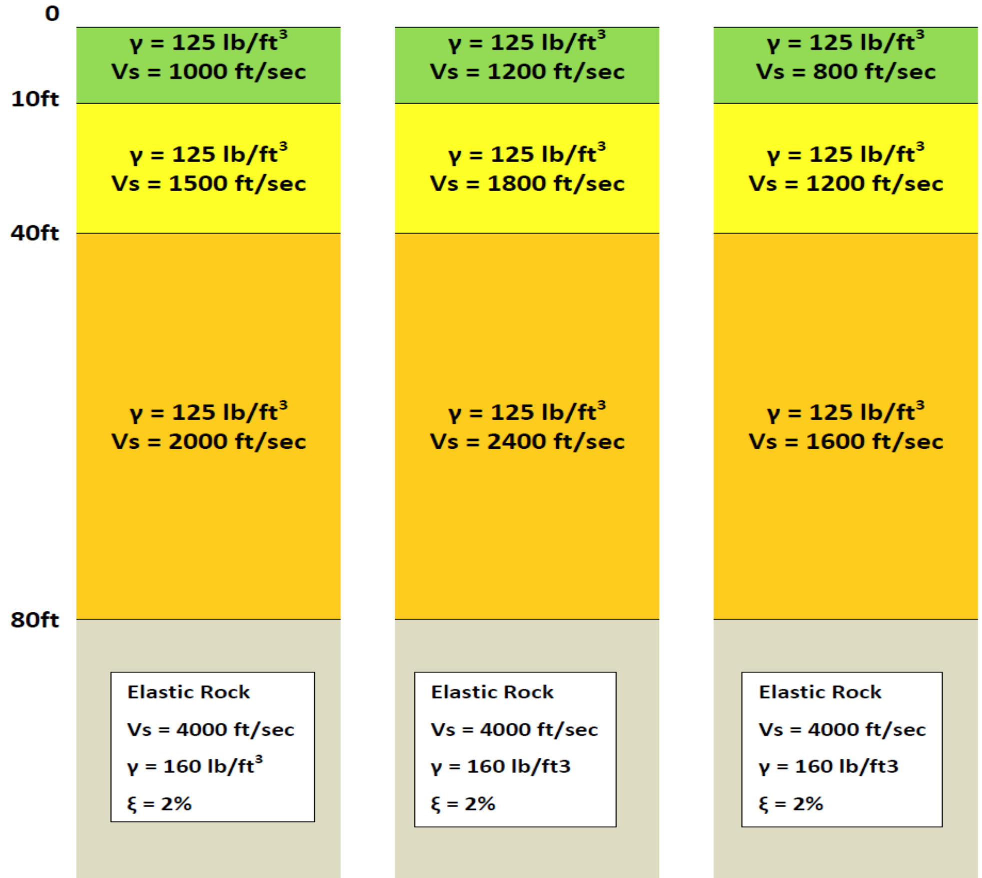


Figure III.59 Soil profiles for the example

Choose “Standard” analysis type at the initialization stage.

STEP 1/6

For Step 1/6, as shown in Figure III.60, first input 3 for the “# of Layers” (default value is 1) according to the soil profile and choose “English” for “Units” system.

For “Analysis Type”, select the Frequency Domain – “Equivalent Linear” analysis. This will enable the “Equivalent Linear” options.

For an equivalent linear analysis, the G/G_{MAX} and damping ratio curves can be defined using either a) Discrete Points or b) the Modified Hyperbolic Model.

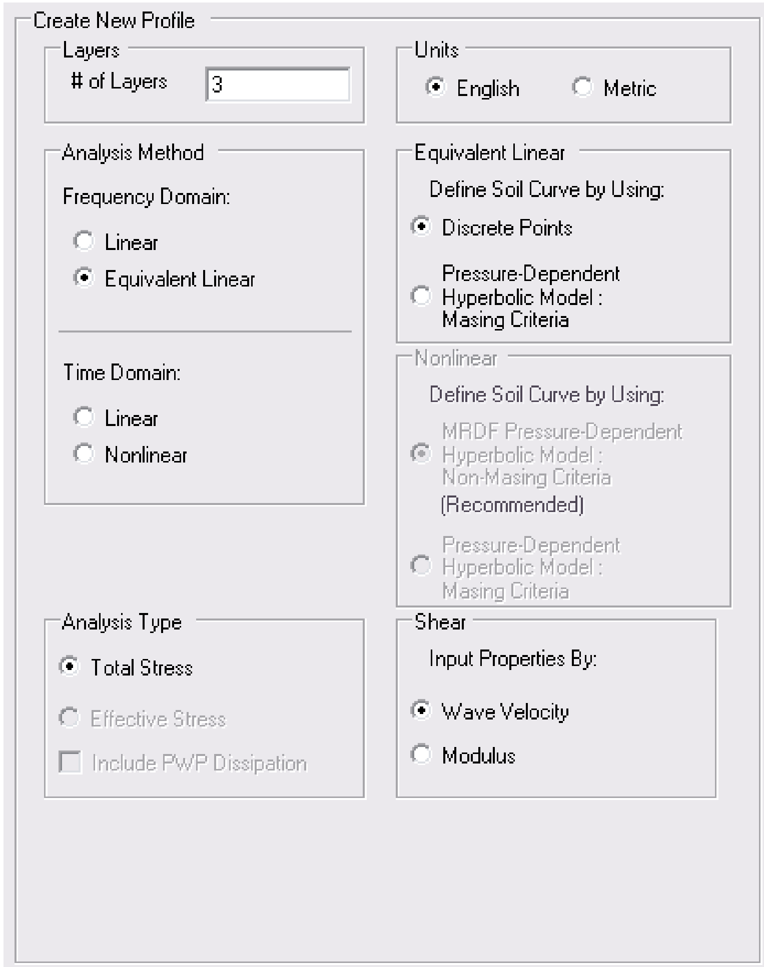


Figure III.60 Step 1 of 6

If Discrete Points are selected, the G/G_{MAX} and damping ratio will be defined in discrete points at various strain levels. It is also possible to define the G/G_{MAX} and damping curve using the modified hyperbolic model. In that case, the user needs to define the nonlinear parameters for the soil model. DEEPSOIL will automatically develop corresponding G/G_{MAX} and damping ratio curves. For this example, select "Discrete Points".

Now you must choose whether to define the stiffness of the layer in shear wave velocity or shear modulus. Select "Wave Velocity".

Finally, the stress type analysis will be "Total Stress Analysis". Check that "Total Stress" analysis is selected and press the "Next" button.

STEP 2a/6

In Step 2a/6, the user must define the soil column and soil properties. Figure III.61 shows the window that displays the soil properties.

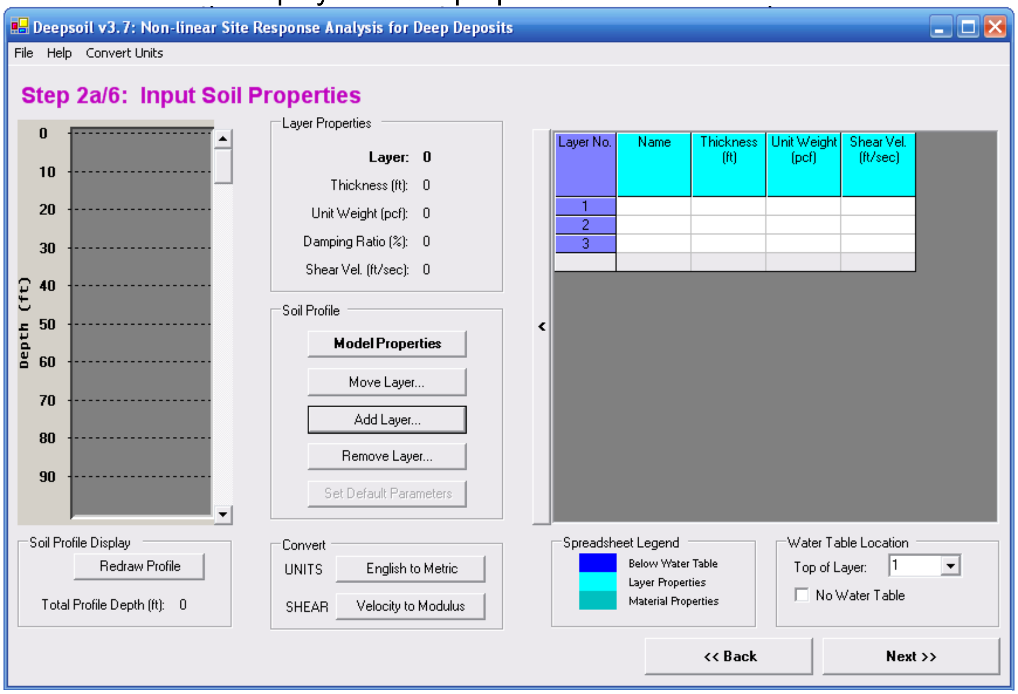


Figure III.61 Step 2a of 6

The soil properties can be selected by expanding the soil column worksheet. Do this by clicking on the vertical bar with the expanding arrow. Specify the material properties of the layer as follows:

Layer No.	Name	Thickness (ft)	Unit Weight (pcf)	Shear Vel. (ft/sec)
1	E Lin S&I	10	125	1000
2	E Lin S&I	30	125	1500
3	E Lin S&I	40	125	2000

Then click the same vertical bar with contracting arrow to go back.

The user can go directly to the spreadsheet, the graphical soil column, or use the “Model Properties” button to define the soil curves.

From the spreadsheet, left-click any cell of the layer for which you want to define the soil curves, and then right-click the same cell to bring up the spreadsheet pop-up menu. Select “Save/Calc Curves”, as shown in Figure III.62.

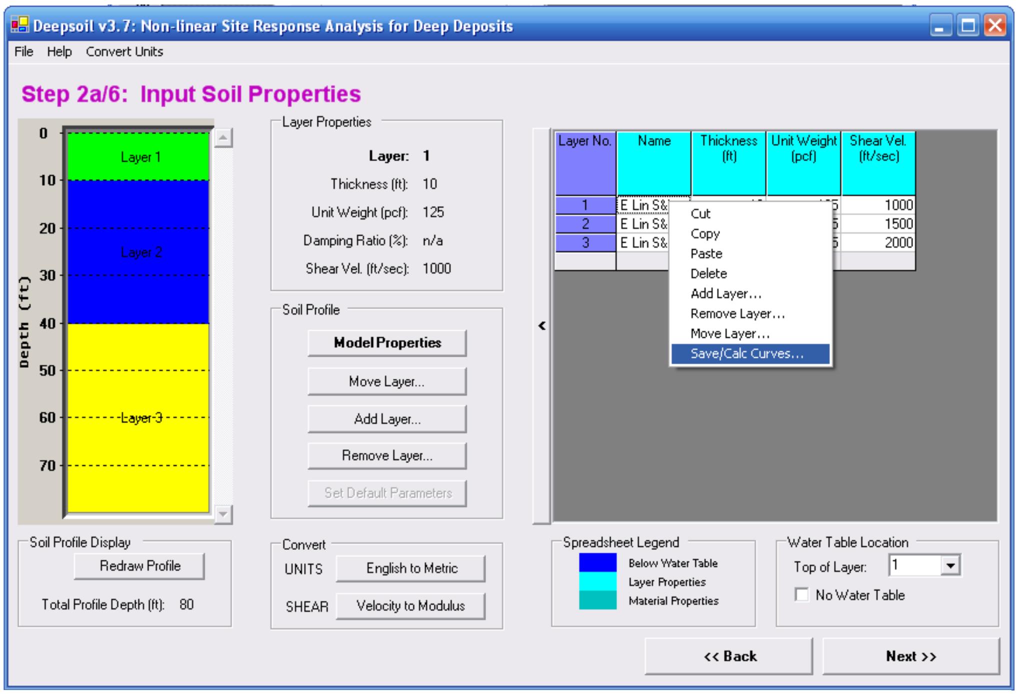


Figure III.62 Select "Save/Calc Curves" from spreadsheet

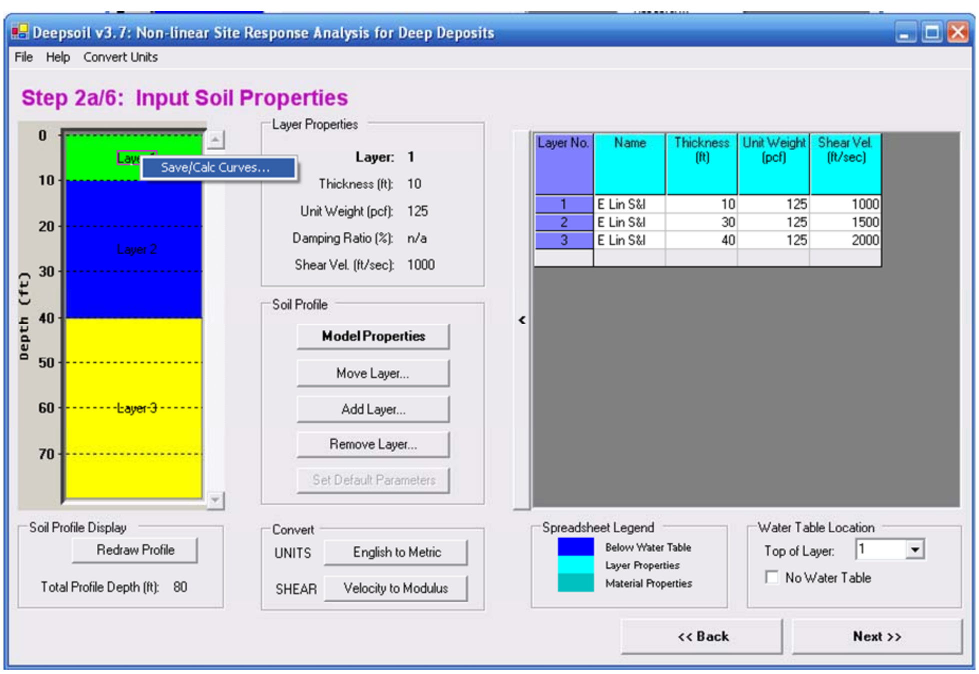


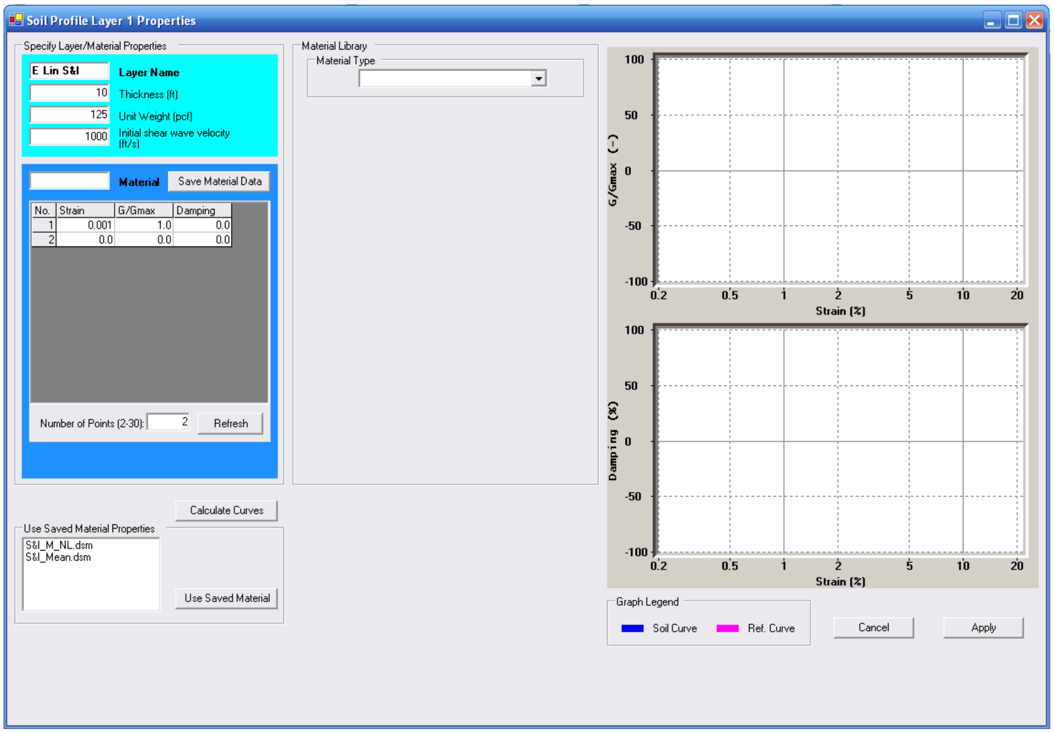
Figure III.63 Select "Save/Calc Curves" from graphical soil column

From the graphical soil column, left-click the Layer name to bring up the soil column pop-up menu. Select “Save/Calc Curves” (Figure III.63).

The “Model Properties” button will allow you to define the

soil curves for whichever layer is currently selected as shown under Layer Properties.

Pressing “Save/Calc Curves” or clicking the “Model Properties” button will open a new window displays below:



The user can define the G/G_{MAX} and damping properties by first defining the number of data points. Note that the number of data points should be identical for G/G_{MAX} and damping. The strain and damping values should be entered as a percent [%].

To save the data points, type a name to identify the properties and press “Save Material Date”. Once saved, the newly saved file will appear in the “User Saved Material Properties” list box.

The user can also use pre-saved material properties by selecting the appropriate file from the list box and pressing the “Use Saved Material” button.

A Material Library is also available to the user to define the soil curves. You will use this method in this example. To use this method, the user must define a) the Material Type, and b) the Target Curve.

Click on the Material Type drop-down menu and select “Sand”. Two new items will appear: Basic Parameters and Target Curve. The Basic Parameters for this case simply displays the vertical stress at the midpoint of the layer. Now you must define the Target Curve.

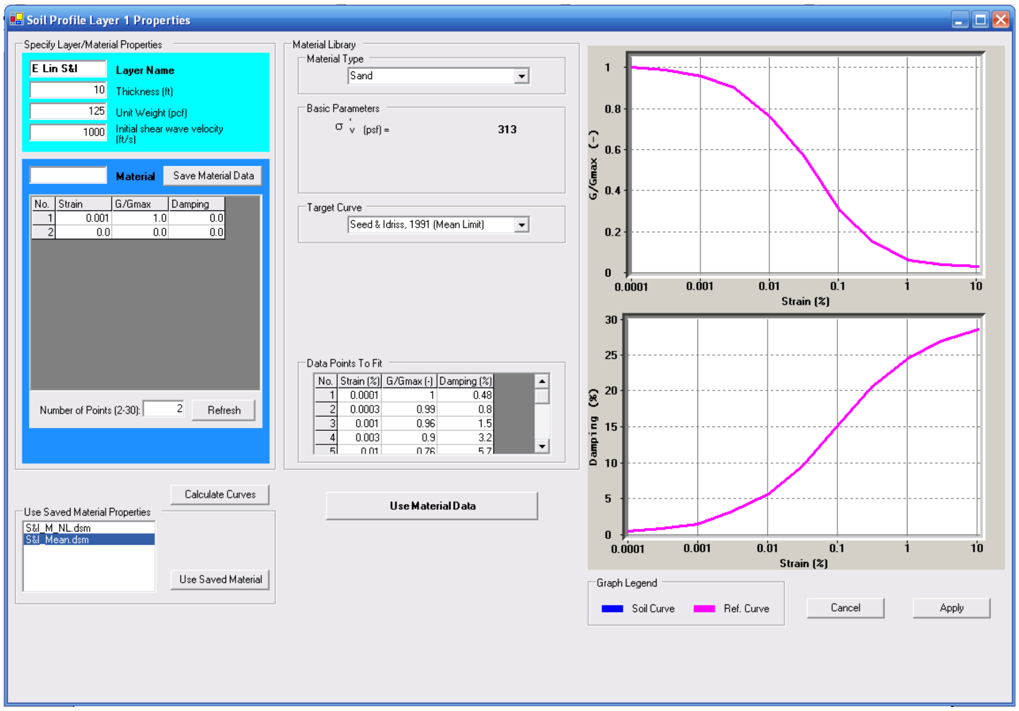


Figure III.64 Soil curves

Click on the Target Curve drop-down menu. A list of various models for sand will appear. Select the “Seed & Idriss, 1991 (Mean Limit)” item. The model soil curves will be plotted in red for your reference (Figure III.64). In addition, a new item appears labeled: “Data Points to Fit”. These are the points which define the model curves. To use this model data, click the “Use Material Data” button. The discrete points of your soil model will be updated to match these points. Click “Calculate Curves” to verify that the models are the same (Figure III.65).

Once you are satisfied with your soil curves, press the “Apply” button to apply the properties. Do the same things for other layers.

When you have finished checking the data, press the “Next” button to proceed.

STEP 2b/6

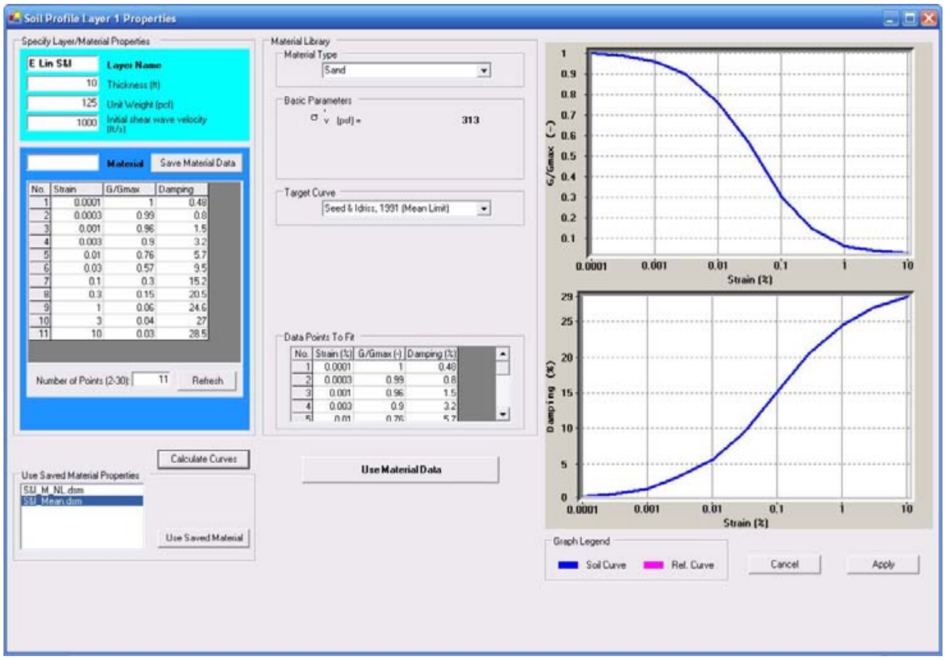


Figure III.65 Check soil curves

In this step, you will define the elastic properties of the bedrock. Select the “Elastic Half-Space” option to define the elastic bedrock properties. Enter the input for Shear Velocity, Unit Weight, and Damping Ratio as 5000 ft/sec, 160 pcf, and 2% respectively. You can also save the bedrock properties by giving the bedrock a name and then clicking the “Save Bedrock” button. Press the “Next” button to proceed to Step 3/6.

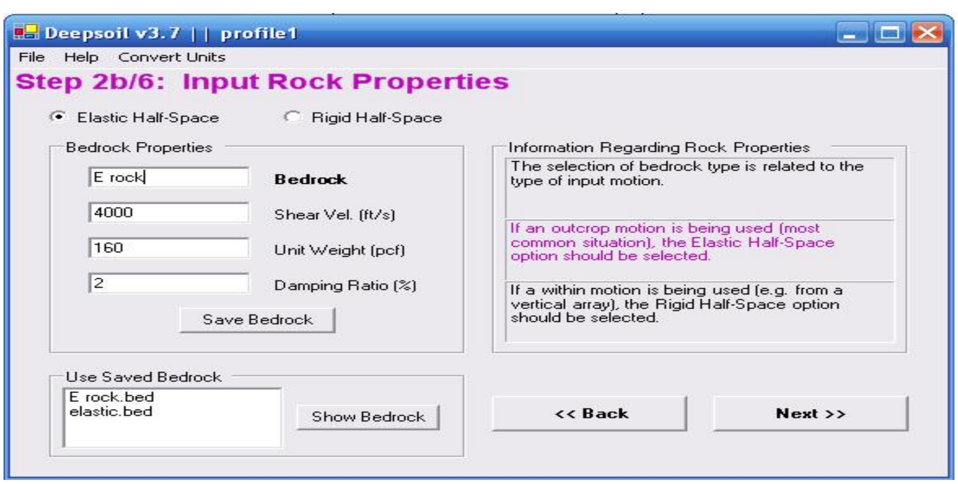


Figure III.66 Bedrock properties

STEP 3/6

The third stage of analysis is the analysis control stage.

Equivalent linear analysis requires a number of iterations to obtain more accurate results. The recommended number of iteration is 15. For the sake of accuracy, you should not choose less than 10 iterations. For this example, choose (at least) 10 iterations.

Select the Fast Fourier Transform (FFT).

The next step is selecting the effective shear strain ratio. The equivalent linear analysis selects shear modulus and damping ratio at a representative shear strain at an effective strain as a ratio of maximum shear strain. Enter an effective shear strain ratio of 0.65.

Select the Frequency Independent Complex Shear Modulus for use in this analysis. See Figure III.67.

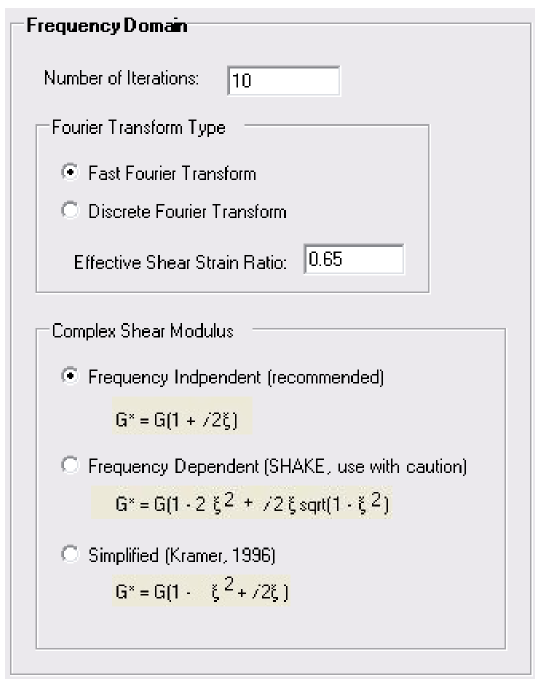


Figure III.67 Step 3/6: Analysis control

Finally, press the “Next” button to proceed to the input motion and output layer(s) selection window (Step 4/6).

STEP 4/6

Step 4/6 involves the selection of a) input motion and b) layers for output.

A motion library is provided which will automatically plot the selected motion for the user's inspection. Users should add their own motions produced by the tools discussed in Section 2 to this motion library whose default direction is C:/program files/ UIUC/Deepsoil v3.7/Working/Input Motions.

Select the input motion "motion_1.txt" from the motion library. This is one of the three response-spectrum compatible acceleration histories generated in previous subsection for the example location.

In the frequency domain analysis, the number of points for the FFT must be defined. The number of points is a power of 2. DEEPSOIL will calculate the minimum number of points needed for the input motion and automatically sets the number of points to be used in the FFT to this minimum value. Note that the number of points for FFT should not be smaller than the minimum value recommended by DEEPSOIL.

After selecting the input motion and associated parameters, select the layer(s) for output (shown in the left column). Layer 1 is selected by default. In this example, you may select to analyze Layers 2 and 3, if you like, by checking each layer's corresponding checkbox.

Finally, enter the damping ratio for the output response spectrum (shown in the lower left corner). The recommended value is 5%. See Figure III.68.

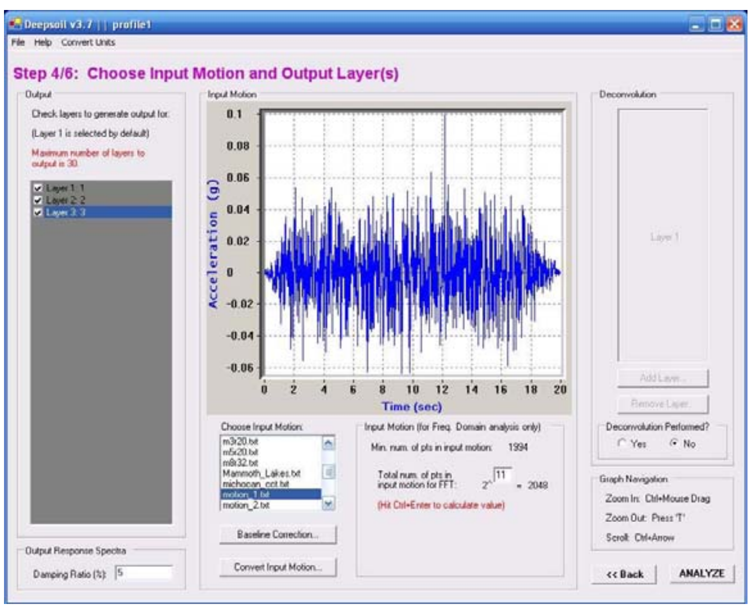
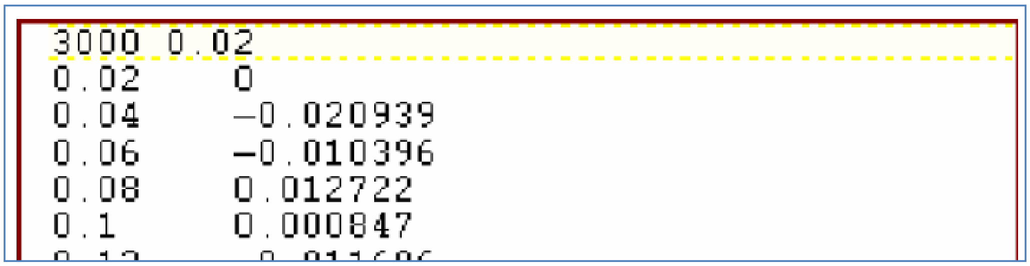


Figure III.68 Step 4 of 6

Press the "ANALYZE" button to begin the analysis. Before the analysis begins, you will be prompted to save the profile. You will be prompted to save the profile before proceeding with the analysis regardless if any changes have been made. If you wish to save the profile, click "Yes". To continue with the analysis, click "No" when prompted.

In case the user wishes to define new motions, the format of the ground motion file should be as shown in Figure III.69 below.



```
3000 0.02
0.02     0
0.04     -0.020939
0.06     -0.010396
0.08     0.012722
0.1      0.000847
```

Figure III.69 User Defined Format of Ground Motions

The first number is the total number of data points; the second number is the time interval. The actual time history should be written in two columns, the first column is the time step and the second column is the acceleration. The acceleration should be in units of g. Users don't have to worry about this format since the format of the motions have been written into the corresponding format by the tool mentioned in previous subsetion. Users just need to make sure to save the motions they wanted into the right folder of DEEPSOIL.

STEP 5/6
This step is only for time domain analysis, and always skipped in this application.

STEP 6/6
Following the introduction above, you can save the analysis results.

The Figure III.70 below shows the computed surface acceleration and response spectra. Check that your results match with those shown.

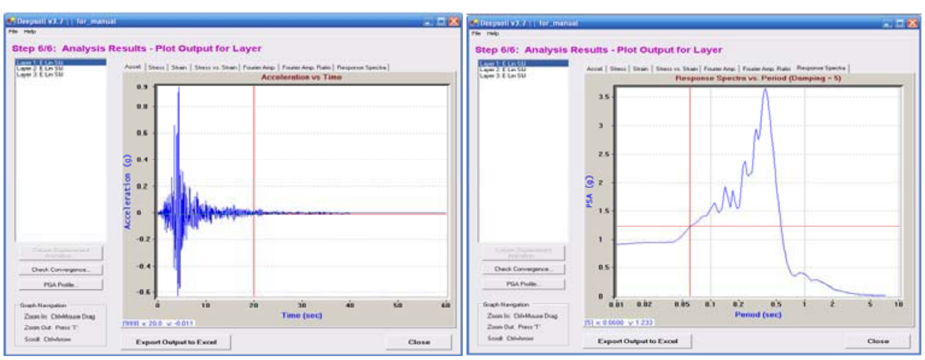


Figure III.70 Computed Surface Acceleration and Response Spectra

Once all 9 cases have been analyzed, user must obtain the final response spectrum which is the envelope of the response spectra of the 9 cases, as shown in Figure III.71. The program cannot accomplish this task automatically. Rather, the user is required to extract data of the 9 response spectra from the 9 cases, and obtain the envelope response spectrum using appropriate tool such as Matlab or Excel Spreadsheet.

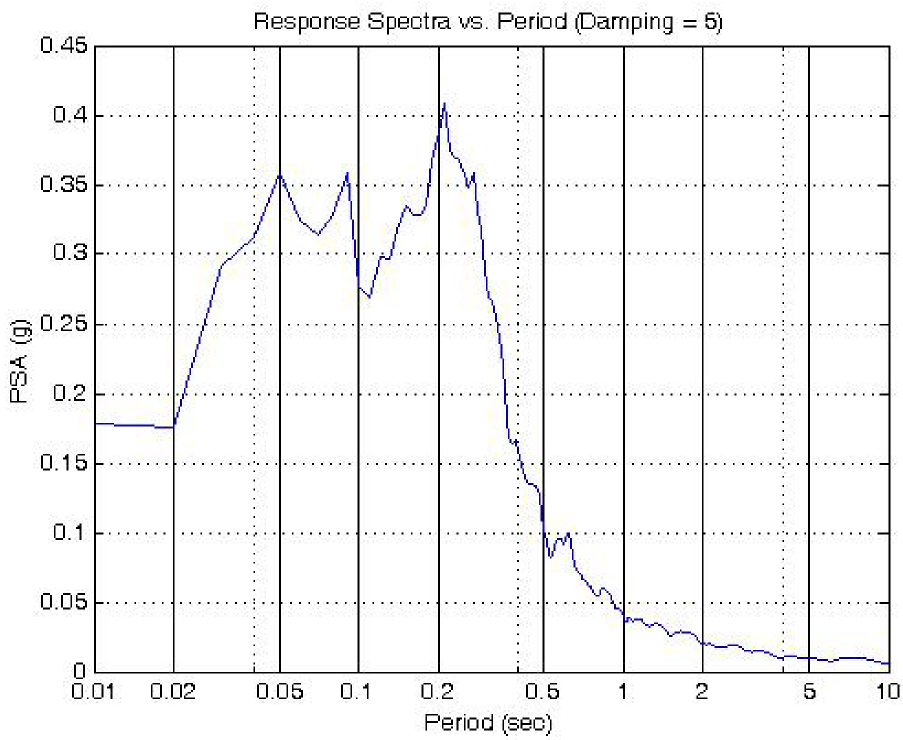
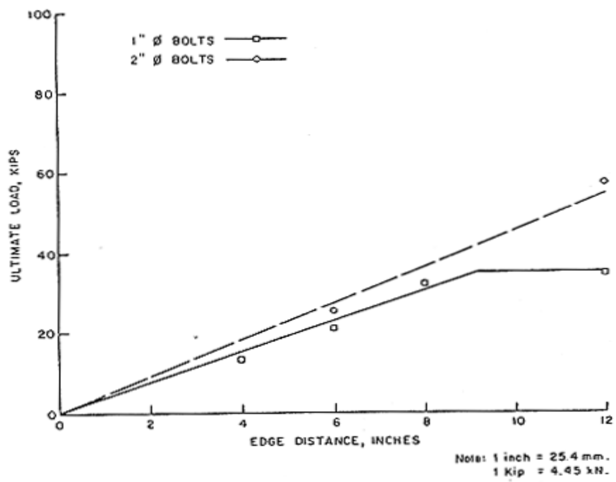


Figure III.71 Final Envelope Response Spectra Based on 9 Cases

APPENDIX IV: EXAMPLE ON DESIGN OF SINGLE SPAN STEEL BRIDGE

IV.A: The experimental findings based on report FLTWA-CA-ST4167-77-12 prepared by Caltrans in cooperation with the U.S Department of Transportation Federal Highway Administration are used to confirm values for the ultimate shear strength of various bolts. The figure below shows ultimate load for 1" and 2" bolts in relation to the edge distance. The table below shows the strength capacity in relation to crack width, for 1" and 2" bolts (A307 &A449)



Comparison between average ultimate loads versus edge distances of cast-in-place one-inch and two-inch diameter anchor bolts in nominally reinforced concrete.

Bolt Diameter (inches)	Average Loads (kips) and Corresponding Average Deflections (inches)											
	Mild Steel (ASTM A307)						High-Strength Steel (ASTM A449)					
	Test Nos.	P @ Δ = 0.05"	P Crack	Δ Crack	P Ult.	Δ Ult.	Test Nos.	P @ Δ = 0.05"	P Crack	Δ Crack	P Ult.	Δ Ult.
1	73,74	2.8	13.7	0.96	38	>1.0	81,82	4.7	13.8	0.29	38.5	>1.0
2	85,86	18.4	16.8	0.04	61.7	>1.0	89,90	16.5	15.8	0.04	61.8	=1.0

-63-

NOTES: 1. Refer to Appendix A for complete information about above test numbers and abbreviations.

2. 1 inch = 25.4 mm.
1 kip = 4.45 kN.

Figure IV.A.1 Comparison Between Average Loads and Deflections of Mild Steel and High-Strength Steel Achor Bolts

IV.B Full Scale Cycle Testing of Foundation Support Systems for Highway Bridges

Part II: Abutment Backwalls.
UCLA-SGEL Report 2007/02

The specimen, located at the UCLA-Caltrans test site in Hawthorne, California, consists of a full-scale (in height) model of a backwall with dimensions of 8.5 ft (height) by 15 ft (width) by 3 ft (thick) with a total weight of 58,000 lbs. The wall is located at a clear distance of 11 feet from a reaction block with dimensions of 24 ft in length, 12 ft in width and 6 ft in height. As shown in Figure IV.B.1 and IV.B.2, five hydraulic actuators were installed between the test specimen and the reaction block to control the horizontal and vertical displacement of the wall.

The natural clayey soils at the site were excavated as shown in Figure IV.B.3 so that the failure plane would be entirely within backfill. Side panels of plywood were erected to simulate wingwalls. These panels are located approximately 0.3 m from the backwall. To reduce the friction between the sidewalls and the backfill, the plywood wing walls were furnished with two layers of 0.006 in PVC foil. Testing was conducted under displacement control, in which horizontal displacements (normal to the wall) were prescribed and all other displacements and all rotations were held to zero.

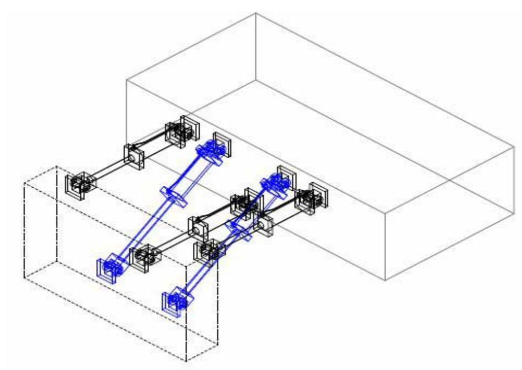


Figure IV.B.1 Actuator Configuration

Figure IV.B.2 Photograph of actuator configuration

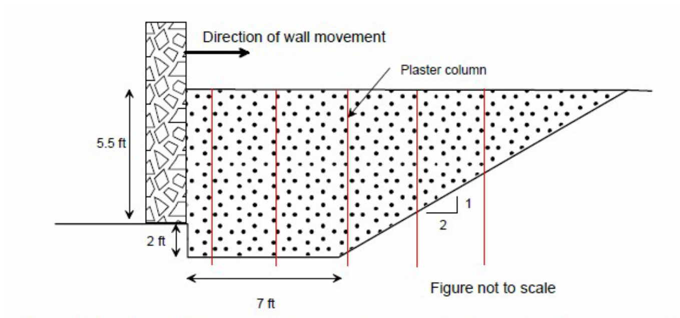


Figure IV.B.3 Schematic cross section through test specimen illustrating the shape of the excavated surface and backfill

The first test sequence involved displacing the wall with no backfill to establish the load-deflection relationship associated with base friction. The second sequence involved testing with backfill. Figure B.4 presents the load-deflection data collected without backfill. Five "cycles" of testing were performed in the positive direction only (towards the backfill), with the largest amplitude being 1.0 in (25 mm). The sudden jumps in the graph indicate that the internal displacement limits were reached. Those limits are set in the control system for safety reasons. The limits were extended and the test resumed, after it was assured that the specimen and the surrounding soil were in a stable condition. This process has no influence on the measured test data.

Testing with the backfill in place was again performed with only positive cycles (displacements into the backfill). Unloading was carefully controlled so as to always maintain positive contact between the backfill and wall (no gapping allowed). Table IV.B.1 shows the loading and unloading sequence for the load displacement curve with backfill soil. The complete load-displacement response is plotted in Figure IV.B.4. As shown in Figure IV.B.4, the maximum lateral load for the test with backfill was about 520 kips, which was reached at a displacement of about 2.0 in.

Table IV.B.1 Loading and Unloading Sequence

initial ave reading at controller PC	Δ displacement	start point [inch]	end point [inch]	total time of movement [sec]
0.81				
0.91	0.1	0	0.1	60
0.88	-0.03	0.1	0.07	60
1.01	0.13	0.07	0.2	60
0.96	-0.05	0.2	0.15	60
1.21	0.25	0.15	0.4	60
1.11	-0.1	0.4	0.3	60
1.56	0.45	0.3	0.75	120
1.41	-0.15	0.75	0.6	60
1.81	0.4	0.6	1	120
2.31	0.5	1	1.5	120
2.16	-0.15	1.5	1.35	60
2.81	0.65	1.35	2	180
2.66	-0.15	2	1.85	60
3.81	1.15	1.85	3	360
3.66	-0.15	3	2.85	60
5.81	2.15	2.85	5	720
5.66	-0.15	5	4.85	60
7.81	2.15	4.85	7	720
6.81	-1	7	6	240

The contribution of the wall base friction to the overall measured horizontal loads can be estimated by comparing the wall displacements with and without backfill soil. Figure IV.B.6 presents the hysteretic test data for both cases up to a peak displacement of 1.0 inch. Backbone curves were created using the peak load-displacement data associated with each initial cycle from both data sequences, with the results shown in Figure IV.B.7 Also shown in Figure IV.B.7 is the percent contribution of the base friction on the measured peak loads from the tests with backfill. The lateral resistance for the test without backfill reached a peak load of 40 kips at a lateral displacement level of 0.4 inches. The lateral resistance for a displacement of 1.0 inches dropped to about 30 kips. Based on the shear strengths of the natural clayey soils underlying the wall, the expected base resistance would be approximately 34 and 39 kips, which is consistent with the test data. For larger displacement levels (> 1.0 in.) the friction along the bottom of the wall was assumed to remain at 30 kips amounting to approximately 50% of the dead load. This level of friction force may be used in confirming the lateral load path into the abutment foundations as required in SDC B illustrated in this example bridge.

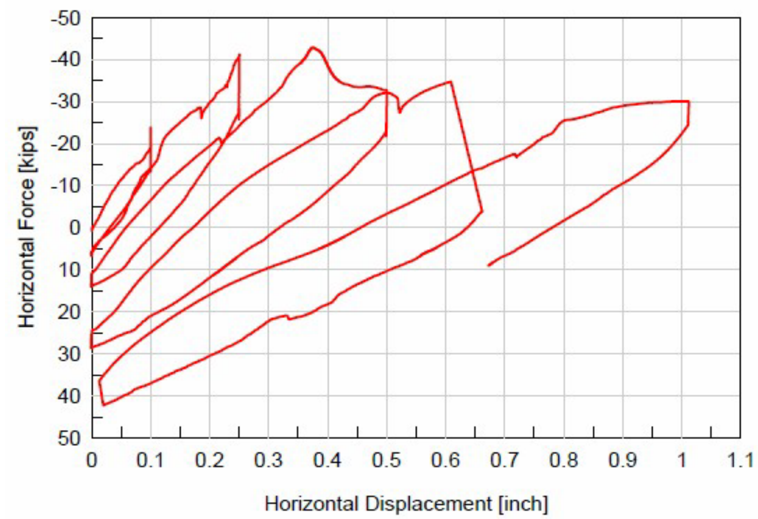


Figure IV.B.4 Load-displacement curve without backfill soil

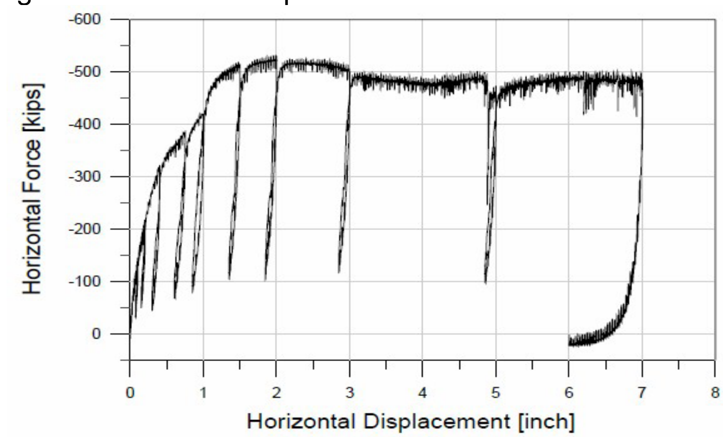


Figure 5.2. Load-displacement curve with backfill soil

Figure IV.B.5 Load-displacement curve with backfill soil

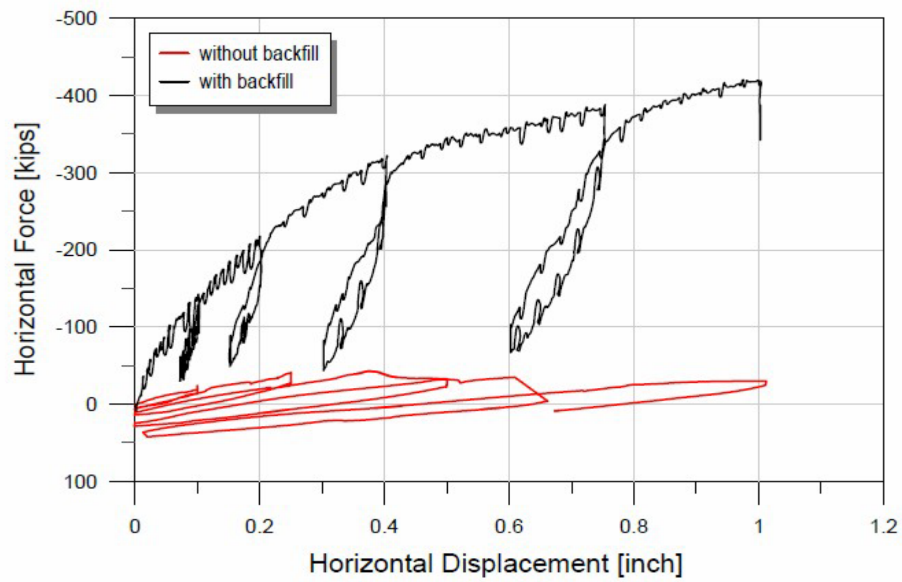


Figure IV.B.6 Load-deflection curves up to max displacement of 1.0 in with and without backfill

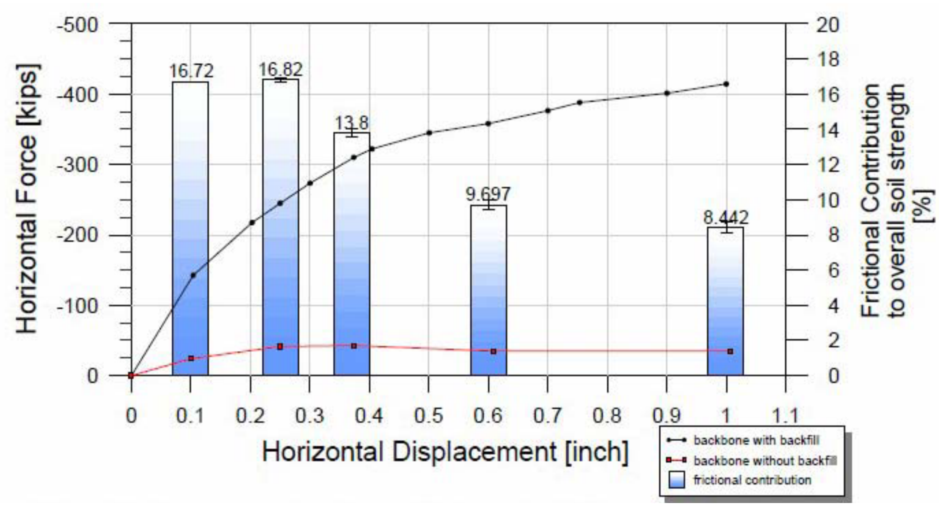


Figure IV.B. 7 Backbone curves and contribution of base friction up to 1.0 inch displacement

APPENDIX V: EXAMPLE ON DESIGN OF TWO-SPAN STEEL BRIDGE

CSI_SAP Version 12 is used to model the moment-curvature relationship for the as-built column section shown in Figure 5.26 and modeled in Figure V.1. For SDCs B and C, the expected material properties shall be used to determine the section stiffness and overstrength capacities. The stress-strain properties are used as shown in Table V.1. The column reinforcement consists of ASTM A615 Grade 60 #9 longitudinal reinforcement and #4 transverse reinforcement with material properties as follows:

#9	$f_y = 60Ksi$	$\varepsilon_{sh} = 0.0125$
	$f_{ye} = 68Ksi$	$\varepsilon_{su}^R = 0.06$
	$f_{ue} = 95Ksi$	$\varepsilon_{su} = 0.09$
#4	$f_y = 60Ksi$	$\varepsilon_{sh} = 0.015$
	$f_{ye} = 68Ksi$	$\varepsilon_{su}^R = 0.06$
	$f_{ue} = 95Ksi$	$\varepsilon_{su} = 0.09$

The spiral reinforcement splice detail shown in Figure V.2 is not considered an ultimate splice detail; therefore the allowable strain of the #4 bar reinforcement is constrained to a value of 2% well below the ultimate strain value of the rebar mentioned above as 9% strain. The modification described above is depicted in Figure V.3.

Figure V.4 shows the modeling of the #9 longitudinal reinforcement based on LRFD_GS 8.4.2 with the ultimate strain value constrain to a reduced value of 6%. The section shown in Figure 2A-1 consist of unconfined concrete cover and confined concrete core modeled as shown in Figure V.5 according to LRFD_GS 8.4.4.

Figure V.6 shows the results of the SAP Moment-Curvature Analysis corresponding to an axial compressive force of 633 Kips equal to the column dead load.

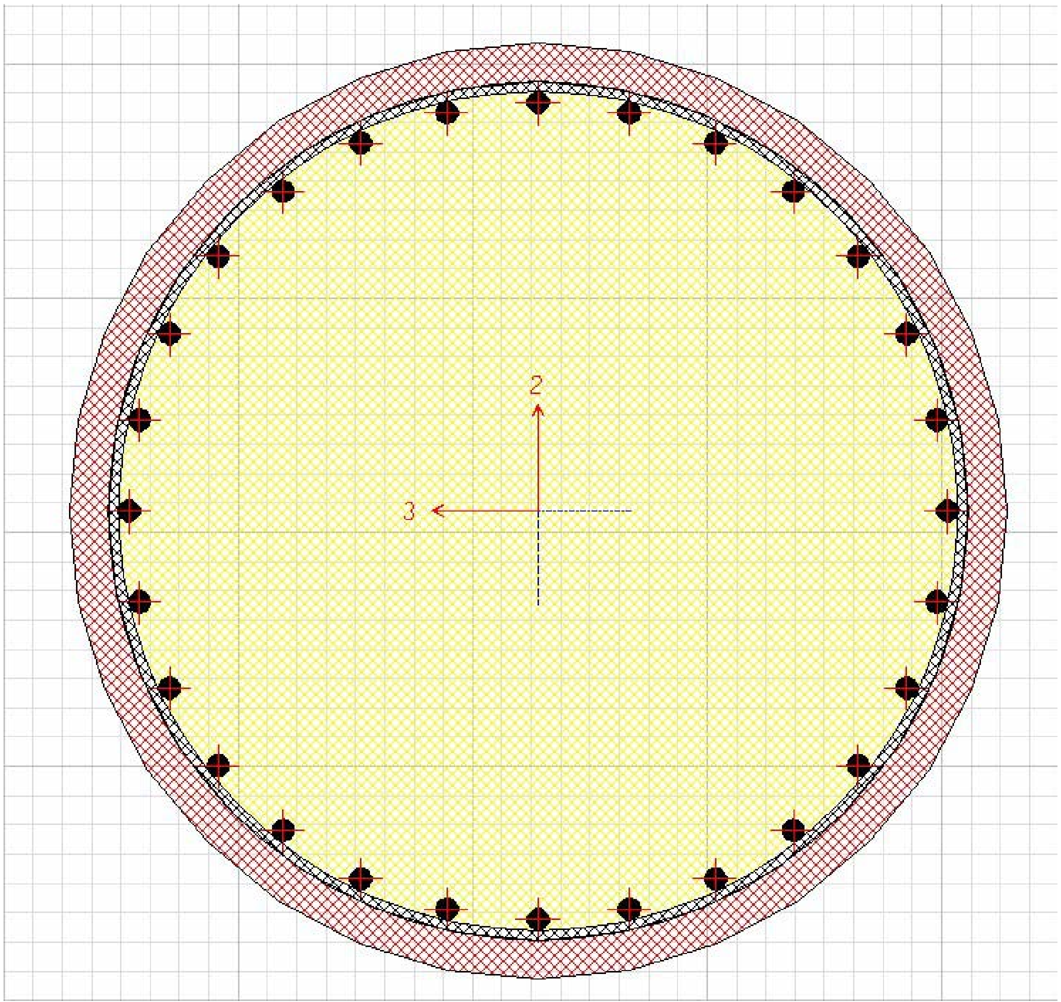
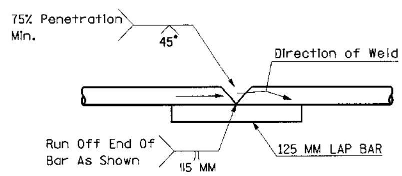


Figure V.1 Colum Section Model

Table V.1Stress Properties of Reinforcement Steel Bars.

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3 - #18	60	60
Expected yield stress (ksi)	f_{ye}	#3 - #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3 - #18	95	95
Expected yield strain	ε_{ye}	#3 - #18	0.0023	0.0023
Onset of strain hardening		#3 - #8	0.0150	0.0150
		#9	0.0125	0.0125
	ε_{sh}	#10 - #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^{R}	#4 - #10	0.090	0.060
		#11 - #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4 - #10	0.120	0.090
		#11 - #18	0.090	0.060

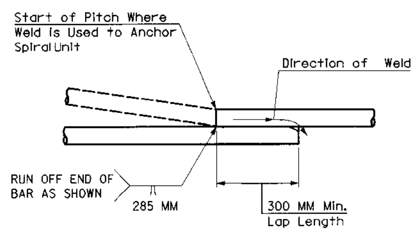


NOTES:

1. Butt weld to be made first.
2. Butt weld to be made in flat or horizontal position.
3. Lap bar to be centered on splice.
4. Flare weld to be made in direction shown.
5. Lap bar to be equal in size to spiral bar.

VEE GROOVE WELDED SPLICE OPTION

N.T.S.



NOTES:

Flare weld to be made in direction shown.

WELDED LAP SPLICE AND ANCHOR

N.T.S.

SPIRAL REINFORCEMENT SPLICE AND ANCHOR DETAIL

FIGURE V.2 SPIRAL REINFORCEMENT DETAIL

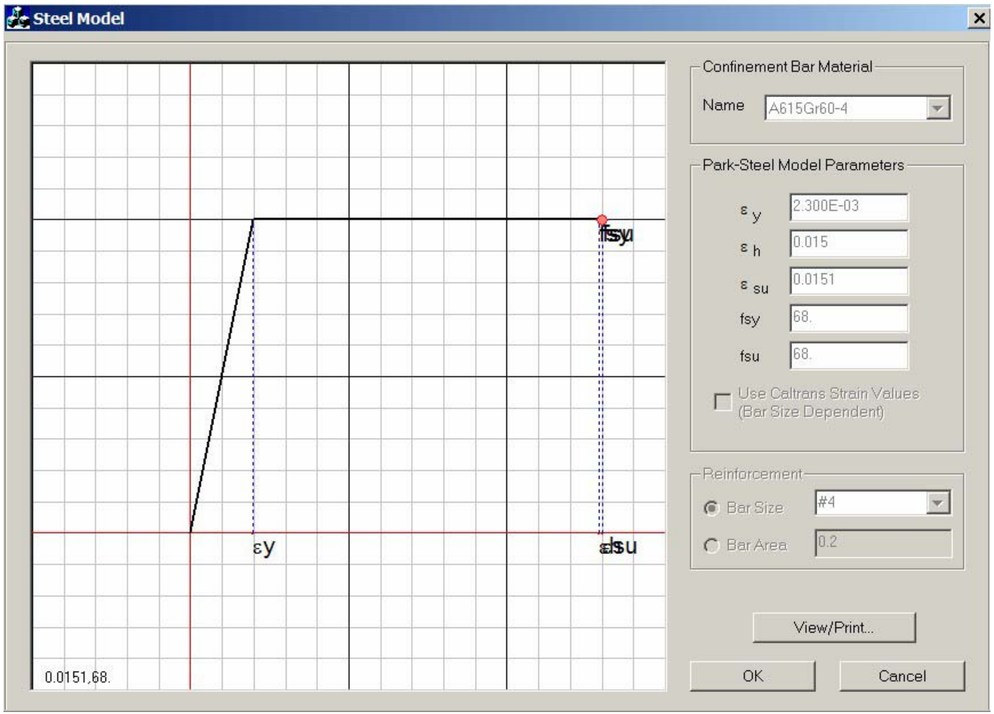


FIGURE V.3 TRANSVERSE REINFORCEMENT STRESS-STRAIN MODEL.

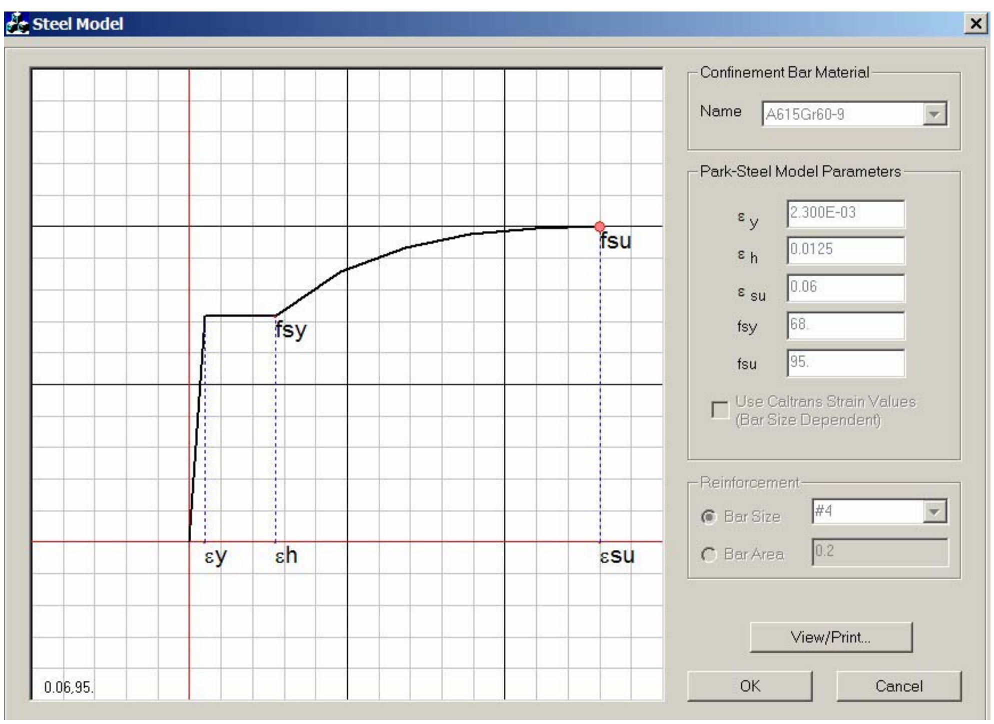


FIGURE V.4 LONGITUDINAL REINFORCEMENT STRESS-STRAIN MODEL.

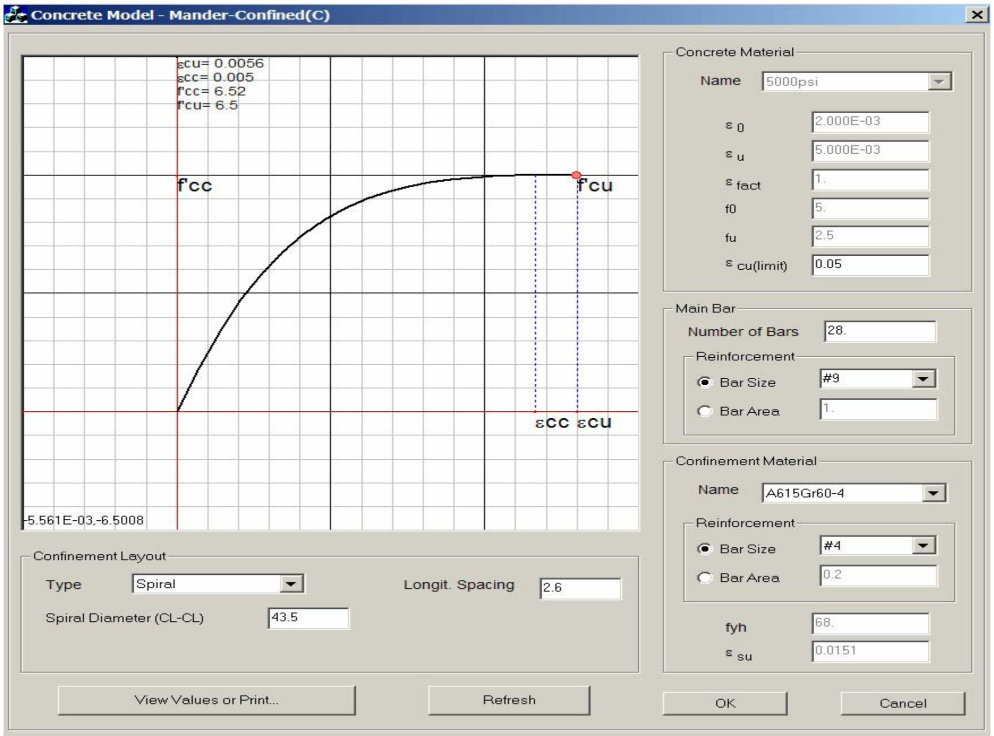


FIGURE V.5 CONFINED CONCRETE CORE.

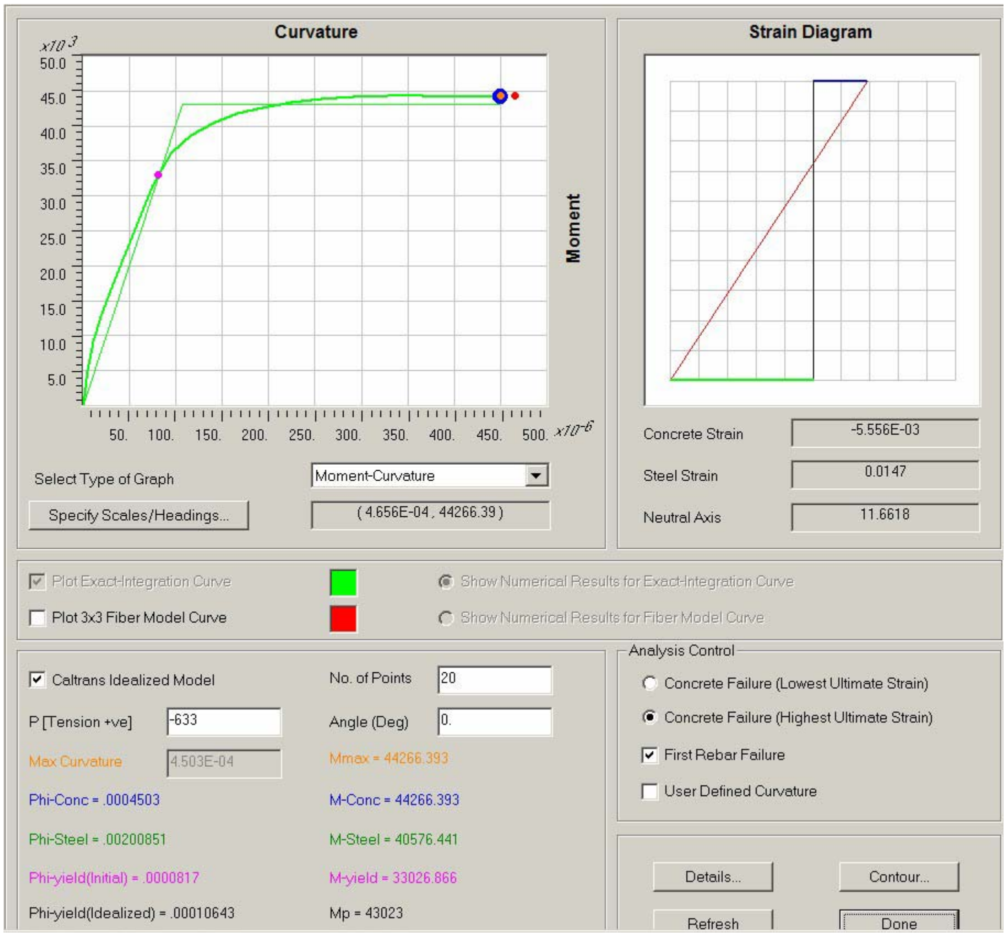


FIGURE V.6 SAP MOMENT-CURVATURE ANALYSIS

APPENDIX VI: EXAMPLE ON DESIGN OF THREE-SPAN STEEL BRIDGE

VI.A: Pier Analysis

CSI-SAP12 is used to model the bent ar piers 3, 4, 5, and 6 shown in Figure VI.C.4 through VI.C.11 of Appendix VI.C. Three cross sections are considered to obtain the gross and effective properties. The moment curvature relationship is also derived and plotted. The three cross secion refer to :

1.) Column Section (CS)
2.) Column Section Embedded in the Shaft (CSE)
3.) Shaft Casing (SC)

Pier 4 is the longitudinal seismic collector bent for Frame 2 that include spans 4, 5, and 6 . Pier 4 having fixed bearings is a collector in the longitudinal direction for a total bridge length of 167.5m or 558 ft. In the transverse direction, the seismic load is shared by all piers. Several Models are generated to capture the response of Piers 3, 4, 5, and 6. These models are described below.

Model 1

Model 1 has the following characteristics:

a.) A crash wall is included as it impacts the transverse direction response.
b.) The point of fixity of the shaft is considered at a depth of 3 times the equivelent shaft diameter below the fill material. The fill material considered equal to 25 ft deep is shown in the Boring Log of Figures 5.52 and 5.53.
c.) The cross sections material modeling based on LRFD-GS 8.4 include:
   - An unconfined concrete model for the column concrete cover shown in Figure VI.A.1.
   - A confined concrete model fo the core concete surrounded by the transverse spiral reinforcement shown in Figure VI.A.2.
   - Reinforcment steel stress strain model shown in Figure VI.A.3.
   - Shaft casing material stress-strain considered at minimum to be ASTM-A53 shown in Figure VI.A.4.

For SDCs B and C, the expected material properties shall be used to determine the column section stiffness and overstrength capacities. The stress-strain properties are used as shown in Table VI.A.1. The column reinforcement consists of ASTM A615 Grade 60 #9 longitudinal reinforcement and #5 transverse reinforcement with material properties as follows:

#9 $f_y = 60Ksi$ $\varepsilon_{sh} = 0.0125$

$f_{ye} = 68Ksi$ $\varepsilon_{su}^R = 0.06$

$f_{ue} = 95Ksi$ $\varepsilon_{su} = 0.09$

#5 $f_y = 60Ksi$ $\varepsilon_{sh} = 0.015$

$f_{ye} = 68Ksi$ $\varepsilon_{su}^R = 0.06$

$f_{ue} = 95Ksi$ $\varepsilon_{su} = 0.09$

Table VI.A.1 Stress Properties of Reinforcement Steel Bars.

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3 - #18	60	60
Expected yield stress (ksi)	f_{ye}	#3 - #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3 - #18	95	95
Expected yield strain	ε_{ye}	#3 - #18	0.0023	0.0023
Onset of strain hardening		#3 - #8	0.0150	0.0150
		#9	0.0125	0.0125
	ε_{sh}	#10 - #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^R	#4 - #10	0.090	0.060
		#11 - #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4 - #10	0.120	0.090
		#11 - #18	0.090	0.060

For Model 1, the gross and effective properties are generated for the following:

- Column section (CS1) having a 3.5 ft diameter (See Figure VI.A.5, VI.A.6)
- Column Section Embedded in the Shaft (CSE1) having a diameter of 4.0 ft and casing thickness of ½ in. (See Figures VI.A.7, VI.A.8)
- The Shaft Casing (SC1) having a diameter of 4.0ft and a casing thickness of ½ in. (See Figure VI.A.9, VI.A.10)
- The Bent Cap (See Figures VI.A.11, VI.A.12)

The Transverse and Longitudinal Loadings of Model 1 are shown in Figures VI.A.13 and VI.A.14. Figure VI.A.15 shows the transverse response for the bent subject to 1000 kips of loading at the cap elevation.

Figure VI.A.16 shows the longitudinal response for the bent subject to a total of 1000 kips of loading distributed equally to the three columns and imposed at the cap elevation.

Figure VI..A17 shows the moment distribution for Model 1 subject to the 1000 Kips transverse loading described above. Figure VI.A.18 shows the shear distribution for Model 1 subject to 1000 Kips of transverse loading.

The Following is a list of figures associated with Model 1:

Figure VI.A.1: Unconfined Concrete Stress strain Model

Figure VI.A.2: Confined Concrete Stress Strain Model

Figure VI.A.3: A615 Grade Reinforcement Stress-Strain Model

Figure VI.A.4: ASTM A53 Grade Steel Casing Stress Strain Model

Figure VI.A.5: Column Gross Properties (CS1)

Figure VI.A.6: Moment Curvature Analysis (CS1)

Figure VI.A.7: Column Casing Gross Properties (CSE1)

Figure VI.A.8: Moment Curvature Analysis (CSE1)

Figure VI.A.9: Shaft Casing Gross Properties (SC1)

Figure VI.A.10: Moment Curvature Analysis (SC1)

Figure VI.A.11: Bent Cap Gross Properties

Figure VI.A.12: Bent Cap Moment Curvature Analysis

Figure VI.A.13: CSI-SAP Model 1 subjected to 1000 Kips Transverse Loading

Figure VI.A.14: CSI-SAP Model 1 Subjected to 1000 Kips Longitudinal Loading

Figure VI.A.15: Response of Model 1 Subjected to 1000 Kips of Transverse Loading

Figure VI.A.16: Response of Model 1 Subjected to 1000 Kips of Longitudinal Loading

Figure VI.A.17: Model 1 Moment Distribution for Bent Subject to 1000 Kips of Transverse Loading

Figure VI.A.18: Model Shear Distribution for Bent Subject to 1000 Kips of Transverse Loading

<u>Model 2:</u>

Model 2 is similar to Model 1 except for the casing having a thickness equal to 0.75in instead of 0.5 in. for Model 1.

For Model 2, the gross and effective properties are generated for the following:

- Column section (CS1) having a 3.5 ft diameter (See Figure VI.A.5, VI.A.6)
- Column Section Embedded in the Shaft (CSE2) having a diameter of 4.0 ft and casing thickness of ¾ in. (See Figures VI.A.19, VI.A.20)
- The Shaft Casing (SC2) having a diameter of 4.0ft and a casing thickness of ¾ in. (See Figure VI.A.21, VI.A.22)
- The Bent Cap (See Figures VI.A.11, VI.A.12)

The transverse and Longitudinal Loadings of Model 2 are shown in Figures VI.A.13 and VI.A.14. Figure VI.A.23 shows the transverse response for the bent subject to 1000 kips of loading at the cap elevation.

Figure VI.A.24 shows the longitudinal response for the bent subject to a total of 1000 kips of loading distributed equally to the three columns and imposed at the cap elevation.

The following is a list of Figures associated with Model 2:

Figure VI.A.19: Column Casing Gross Properties (CSE2)

Figure VI.A.20: Moment Curvature Analysis (CSE2)

Figure VI.A.21: Shaft Casing Gross Properties (SC2)

Figure VI.A.22: Moment Curvature Analysis (SC2)

Figure VI.A.23: Response of Model 2 Subjected to 1000 Kips of Transverse Loading

Figure VI.A.24: Response of Model 2 subjected to 1000 Kips of Longitudinal Loading

Model 3:

Model 3 is also similar to Model 1 except that the increase in strength of the Pier include enlarging the column diameter to 4.5 ft and the shaft casing diameter to 5.0 ft.

For Model 3, the gross and effective properties are generated for the following:

- Column section (CS3) having a 4.5 ft diameter (See Figure VI.A.25, VI.A.26)
- Column Section Embedded in the Shaft (CSE3) having a diameter of 5.0 ft and casing thickness of 1 in. (See Figures VI.A.27, VI.A.28)
- The Shaft Casing (SC3) having a diameter of 5.0ft and a casing thickness of 1 in. (See Figure VI.A.29, VI.A.30)

The transverse and Longitudinal Loadings of Model 3 are similar to Model 1shown in Figures VI.A.13 and VI.A.14. Figure VI.A.31 shows the transverse response for the bent subject to 1000 kips of loading at the cap elevation.

Figure VI.A.32 shows the longitudinal response for the bent subject to a total of 1000 kips of loading distributed equally to the three columns and imposed at the cap elevation.

Figure VI.A.33 shows the moment distribution for Model 3 subject to the 1000 Kips Transverse loading described above. Figure VI.A.34 shows the shear distribution for Model 3 subject to 1000 Kips of transverse loading.

The following is a list of Figures associated with Model 3:

Model 4:

Model 4 is similar to Model 1 except that the crash wall is removed as it is the case for pier 6. This change mainly impacts the transverse response and the moment distribution.

The Longitudinal and Transverse Loadings of Model 4 are shown in Figures VI.A.35 and VI.A.36. Figure VI.A.37 shows the transverse response for the bent subject to 1000 kips of loading at the cap elevation.

Figure VI.A.38 shows the moment distribution for Model 4 subject to the 1000 Kips Transverse loading described above.

The following is a list of figures associated with Model 4:

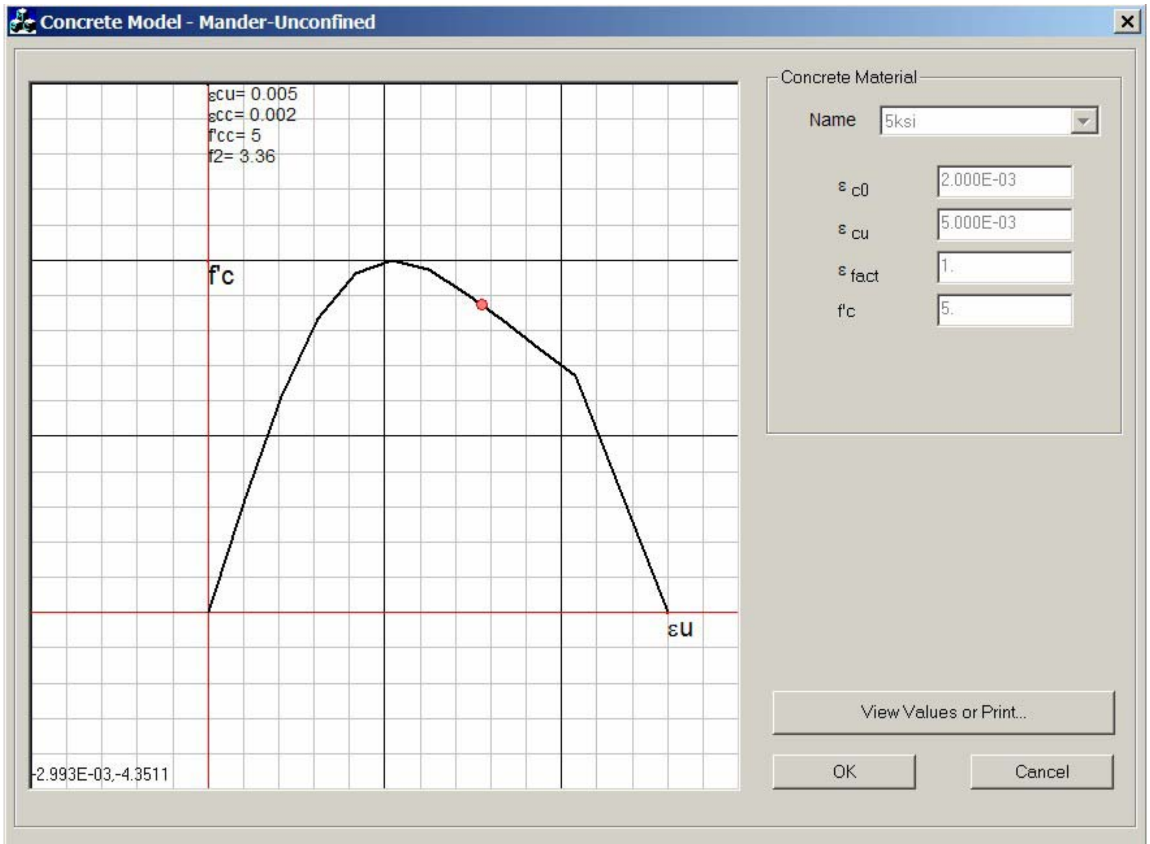


Figure VI.A.1 Unconfined Concrete Stress Strain

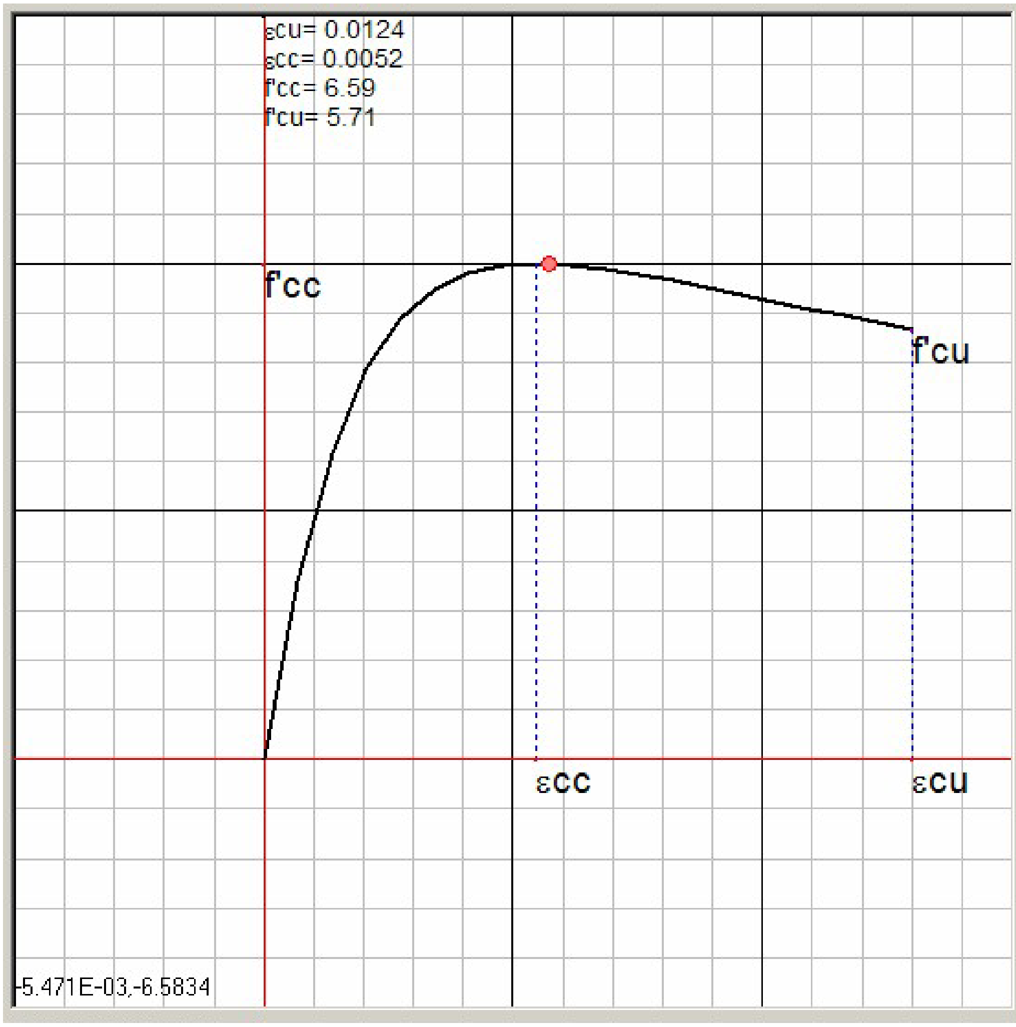


Figure VI.A.2 Confined Concrete Stress Strain Model

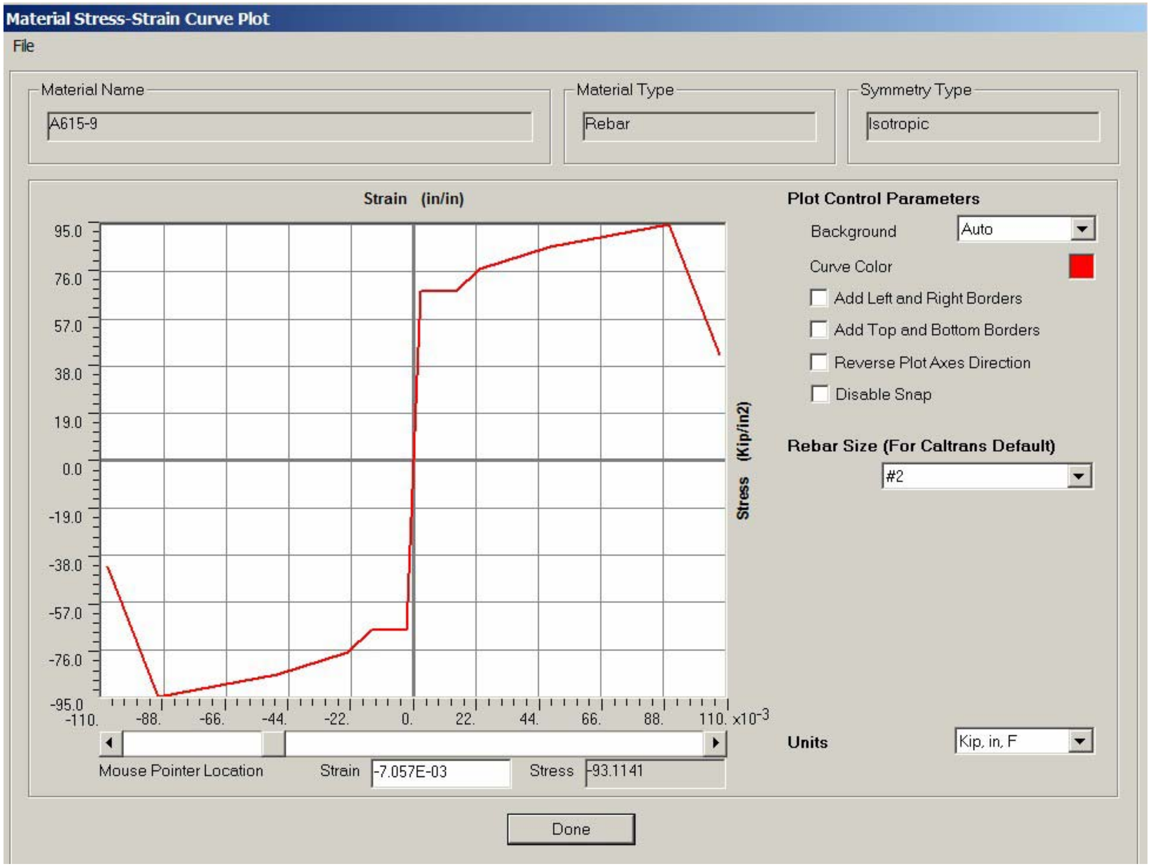


Figure VI.A.3 A615 Grade Reinforcement Stress Strain Model

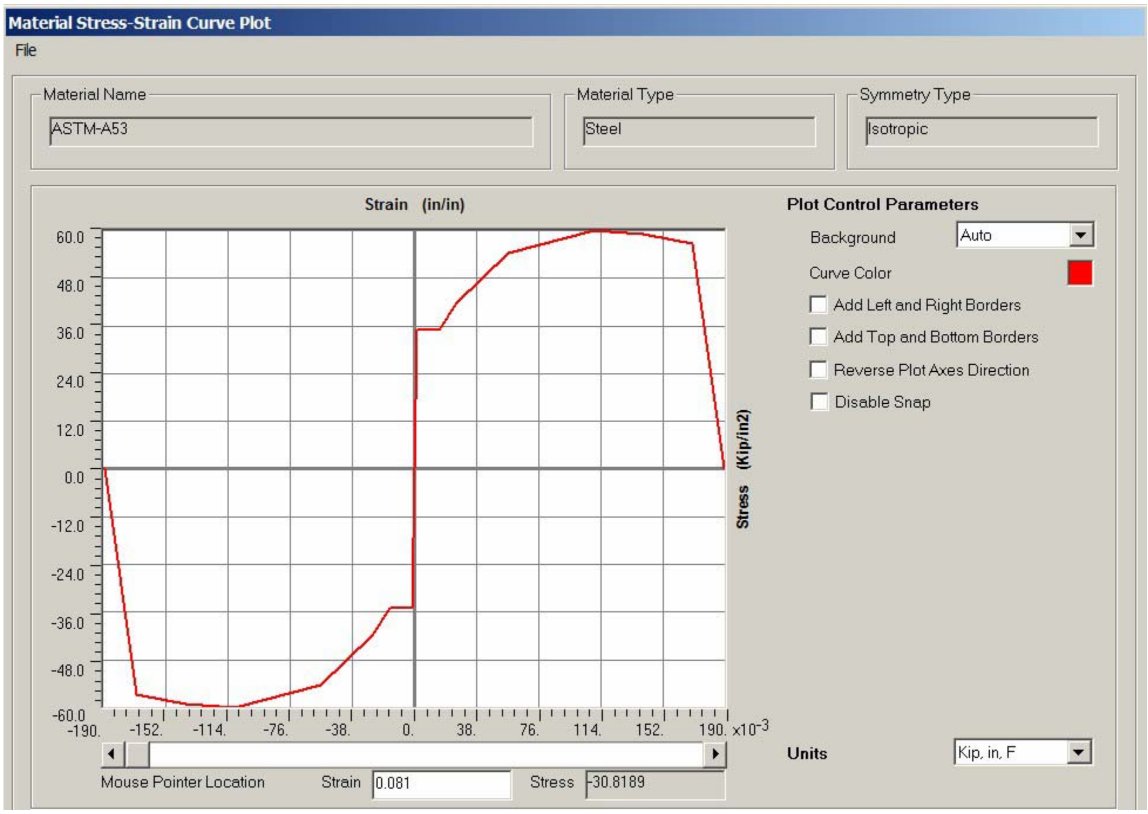


Figure VI.A.4 ASTM-A53 Grade Steel Casing Stress-Strain Model

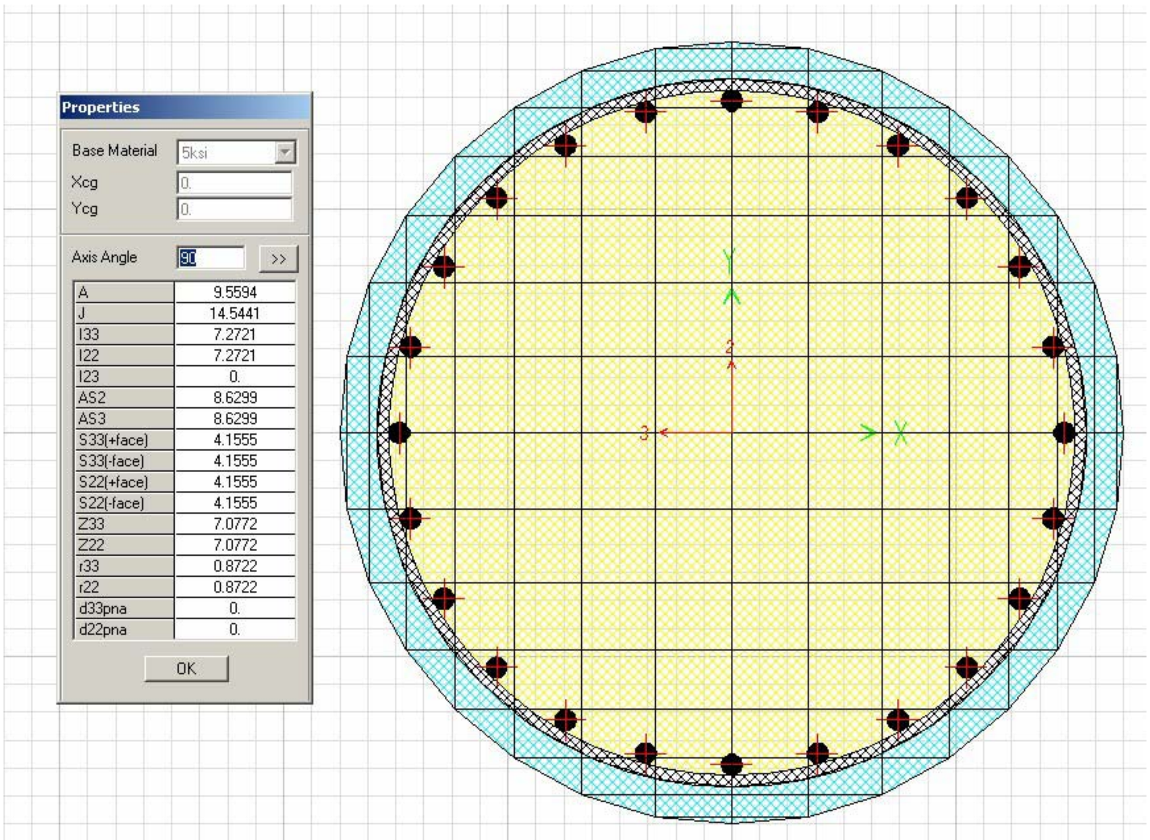


Figure VI.A.5 Column Gross Properties (CS1)

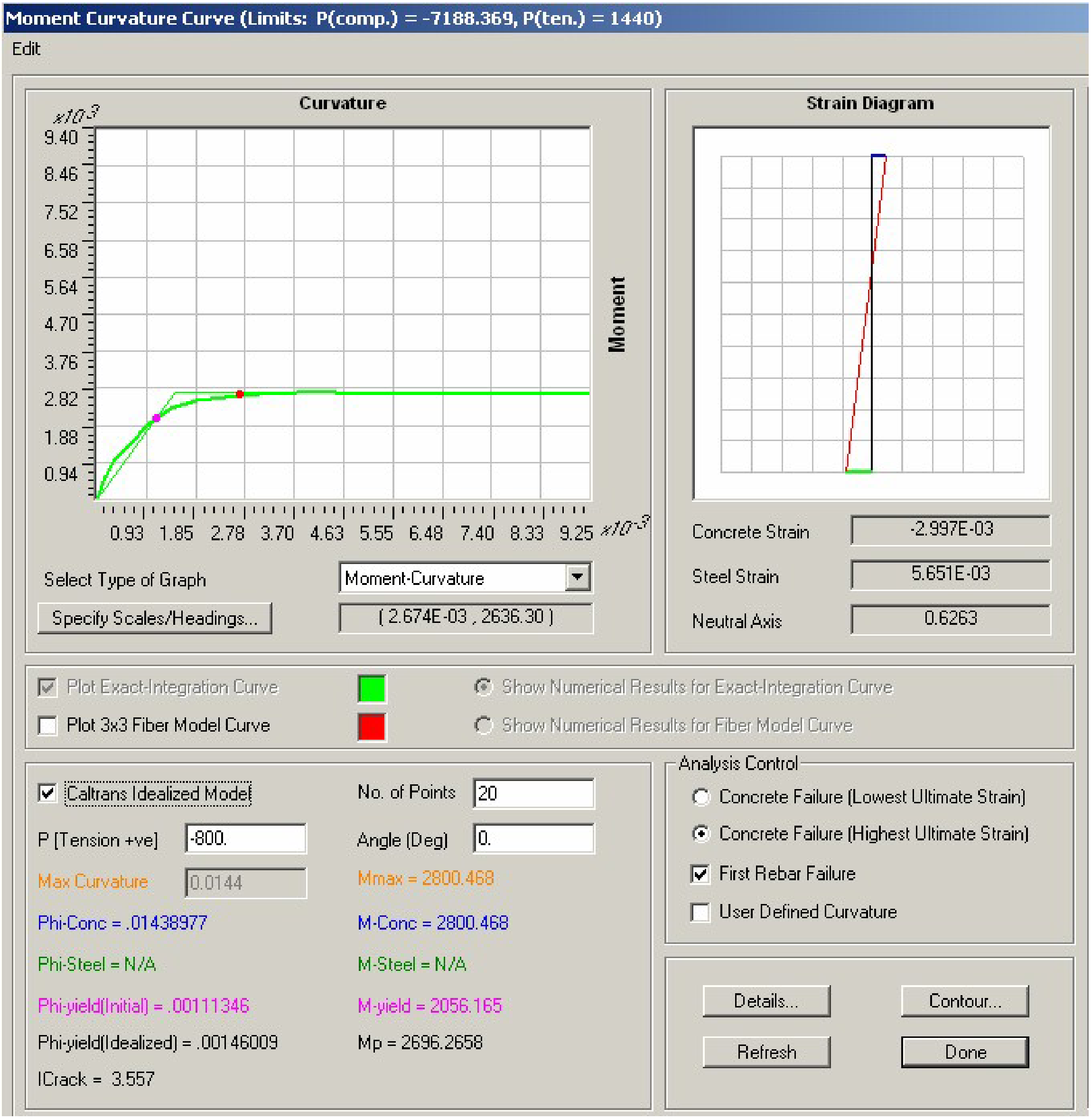


Figure VI.A.6 Moment Curvature Analysis (CS1)

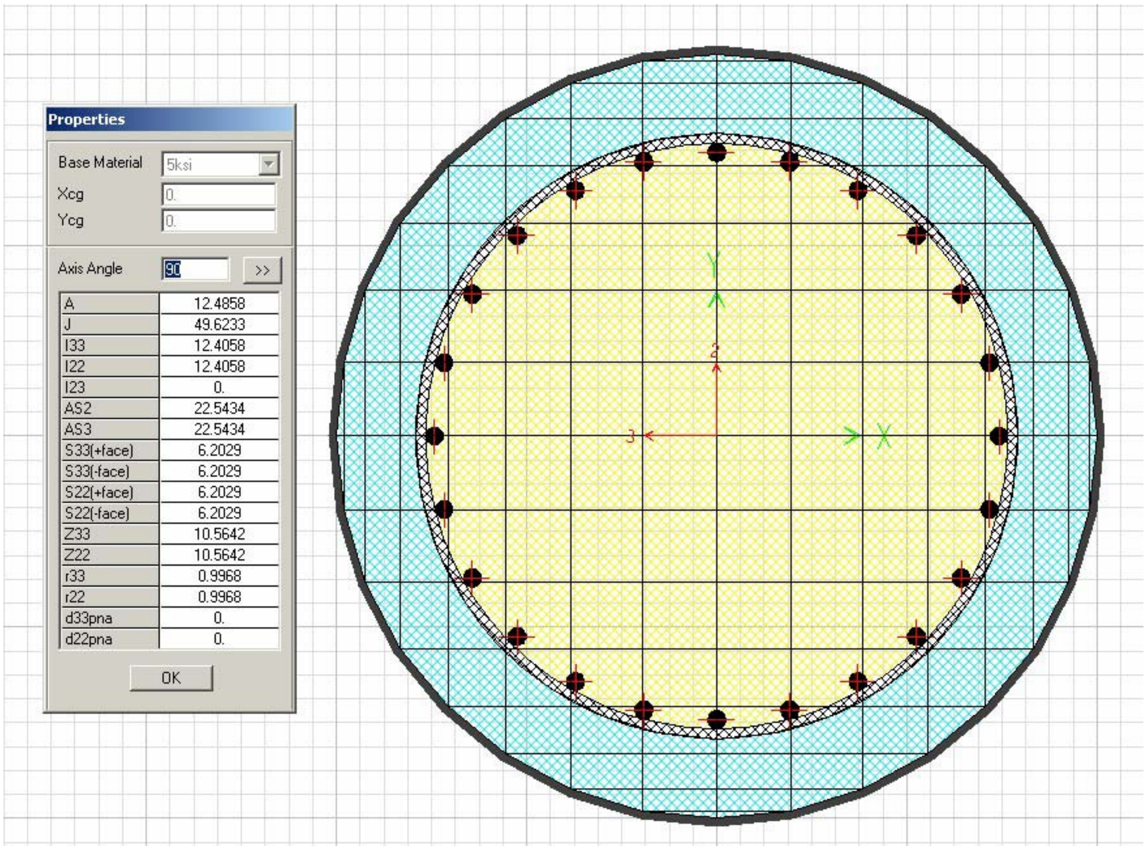


Figure VI.A.7 Column Casing Gross Properties (CSE1)

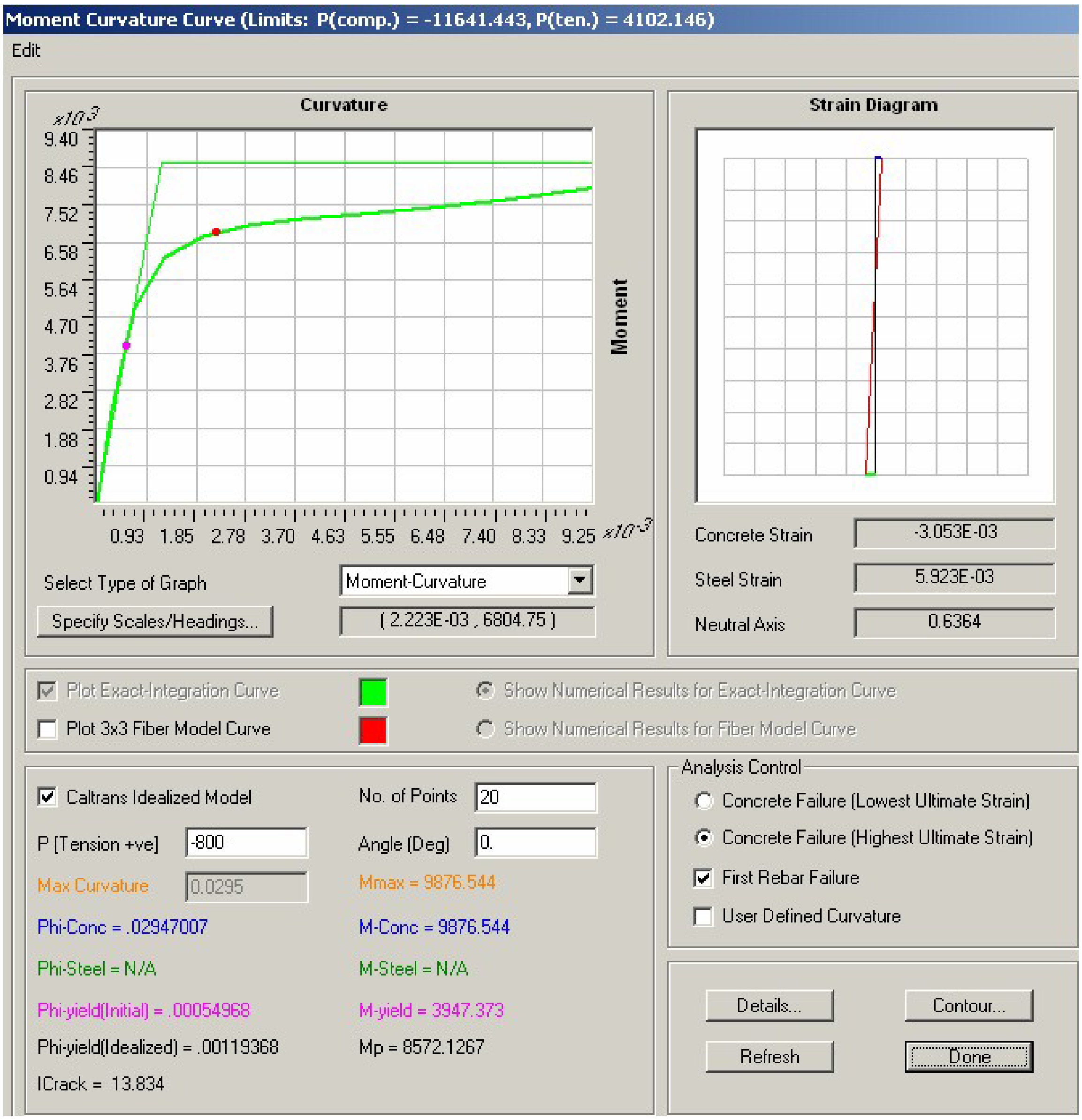


Figure VI.A.8 Moment Curvature Analysis (CSE1)

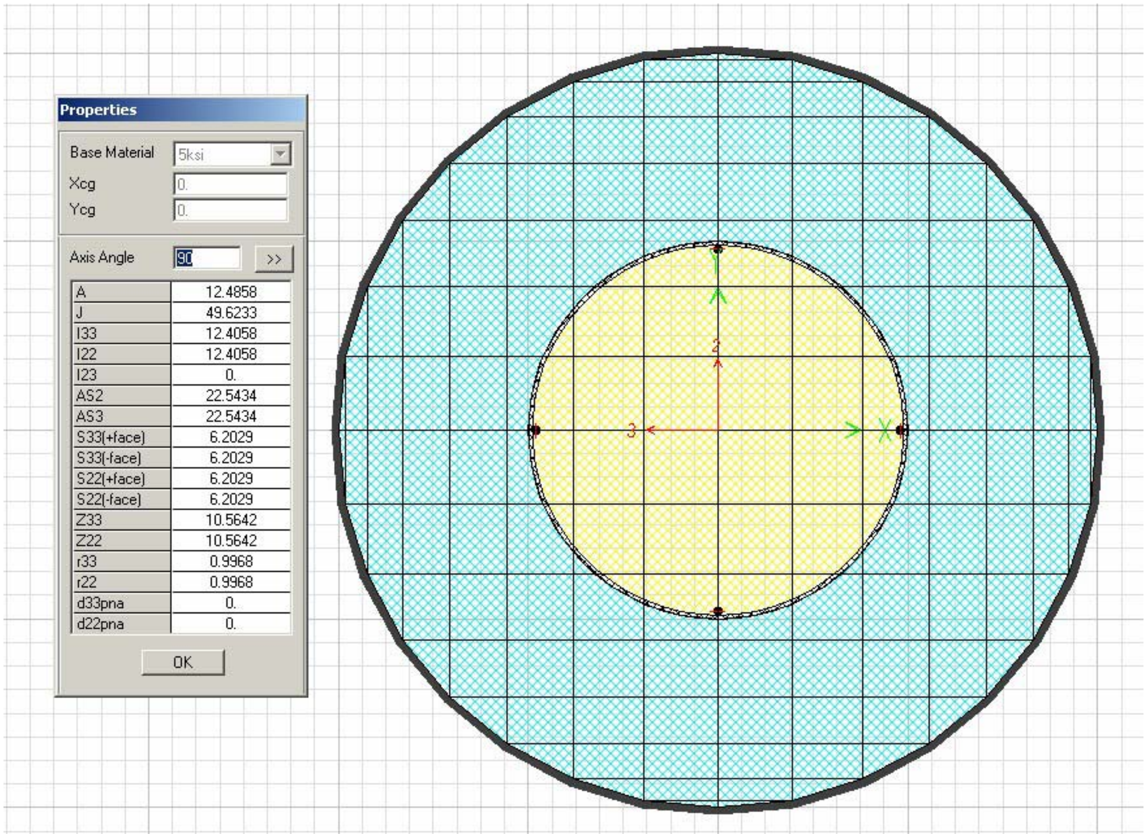


Figure VI.A.9 Shaft Casing Gross Properties (SC1)

Figure VI.A.10 Movement Curvature Analysis (SC1)

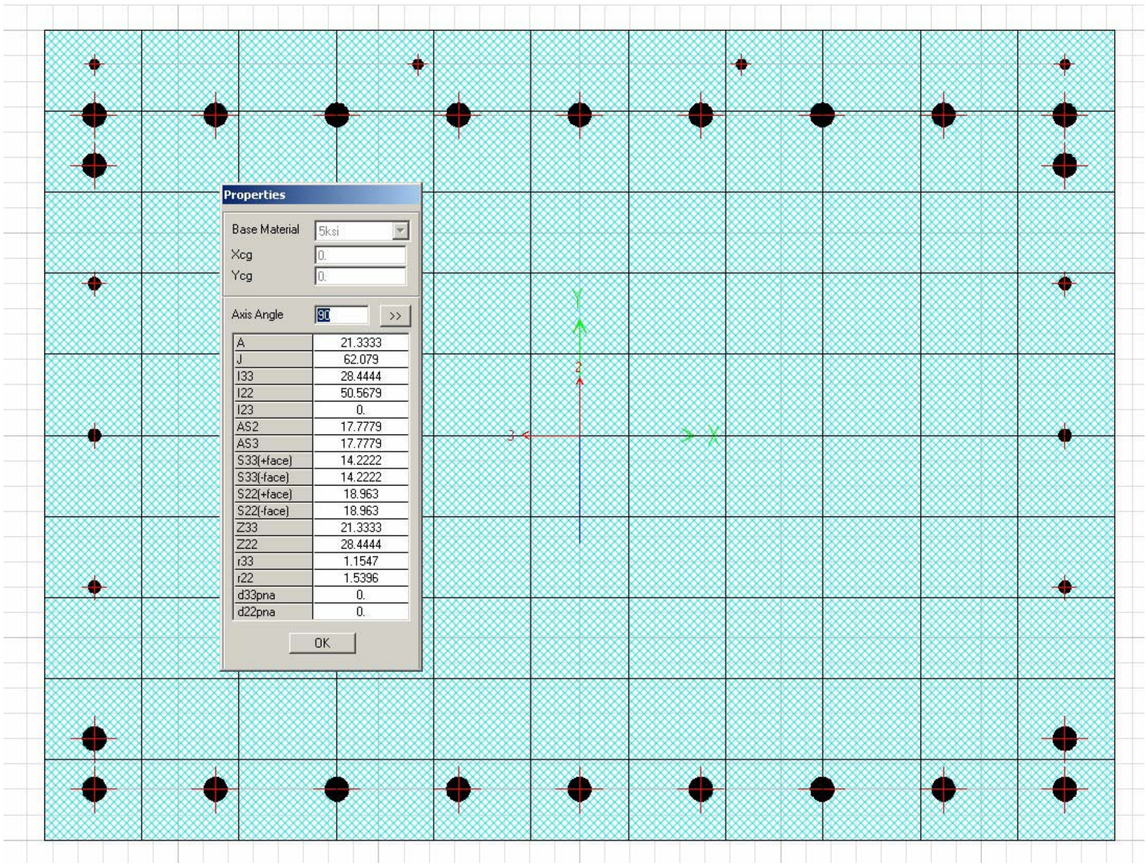


Figure VI.A.11 Bent Cap Gross Properties

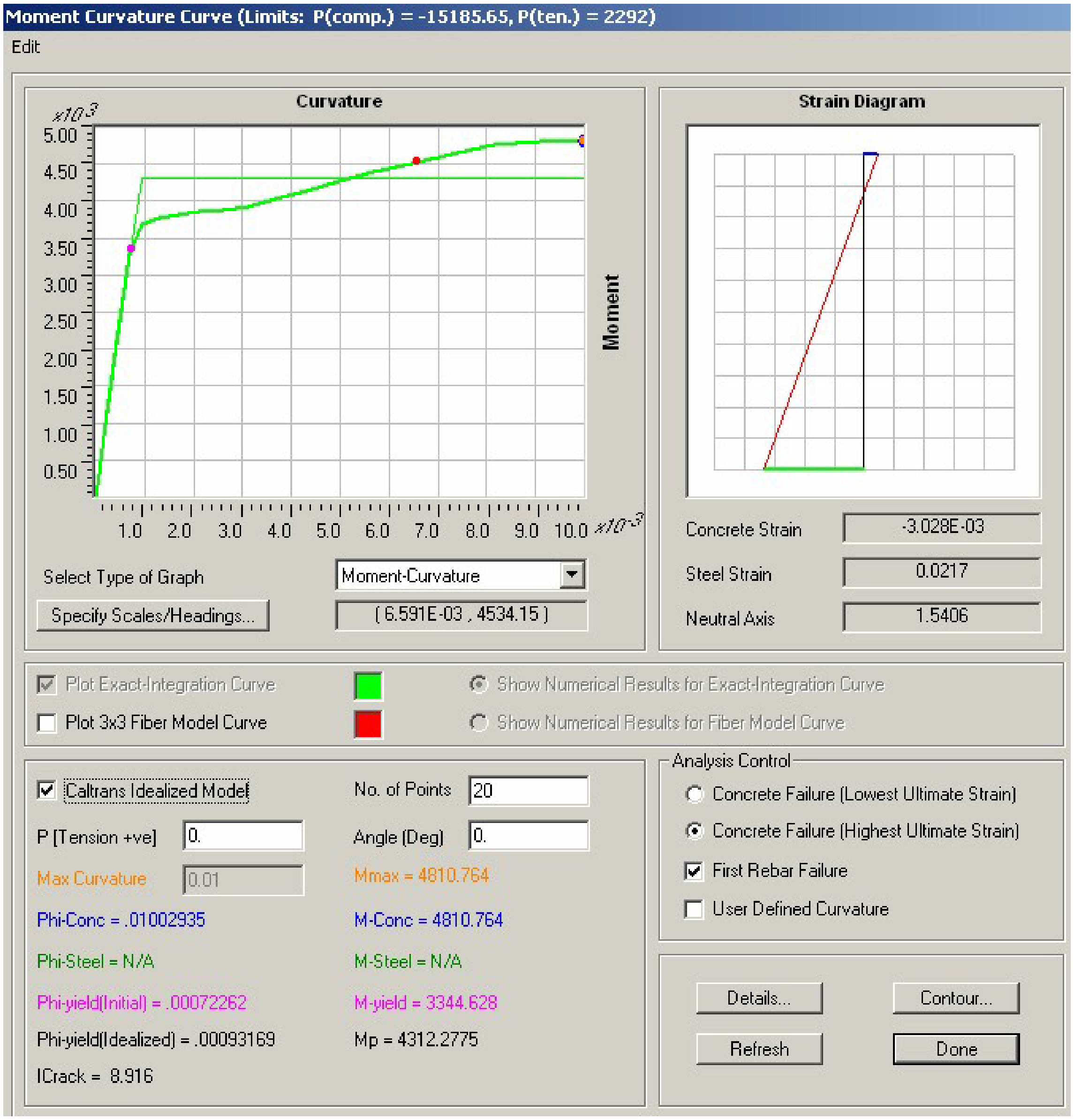


Figure VI.A.12 Bent Cap Movement Curvature Analysis

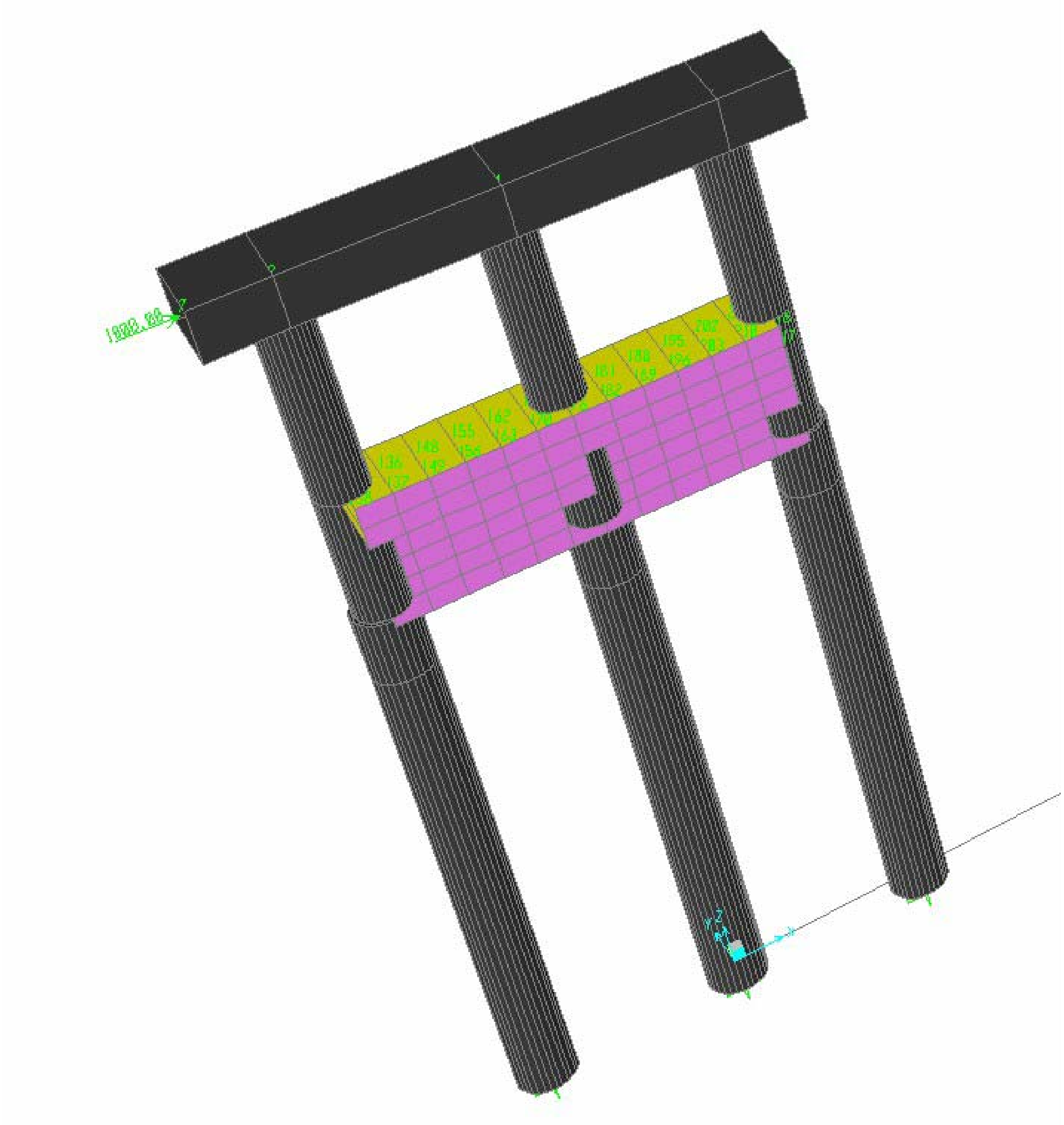


Figure VI.A.13 Model 1 Subjected to 1000 Kips Transverse Loading

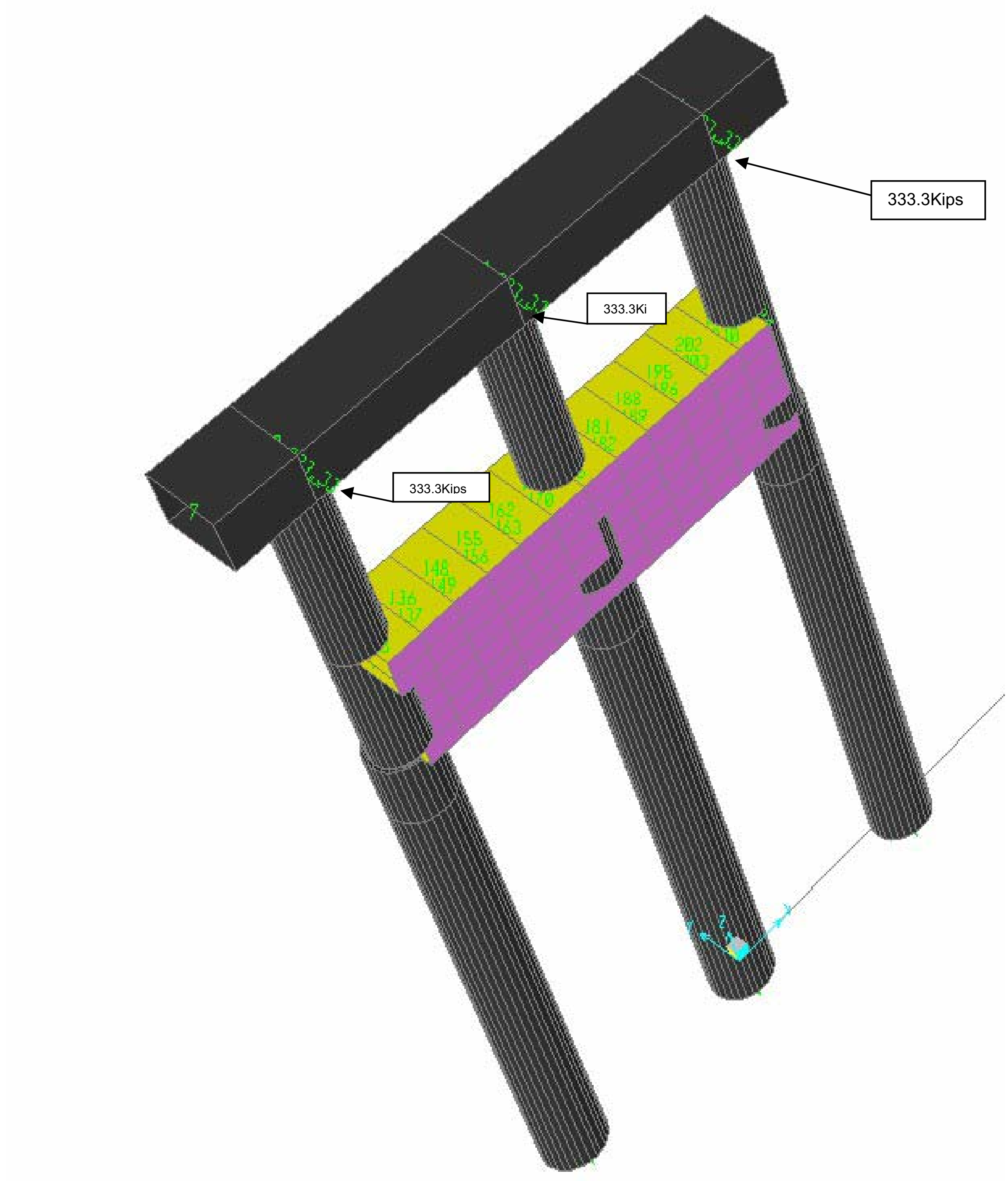


Figure VI.A.14 CSI_SAP Model 1 Subjected to 1000 Kips Longitudinal Loading

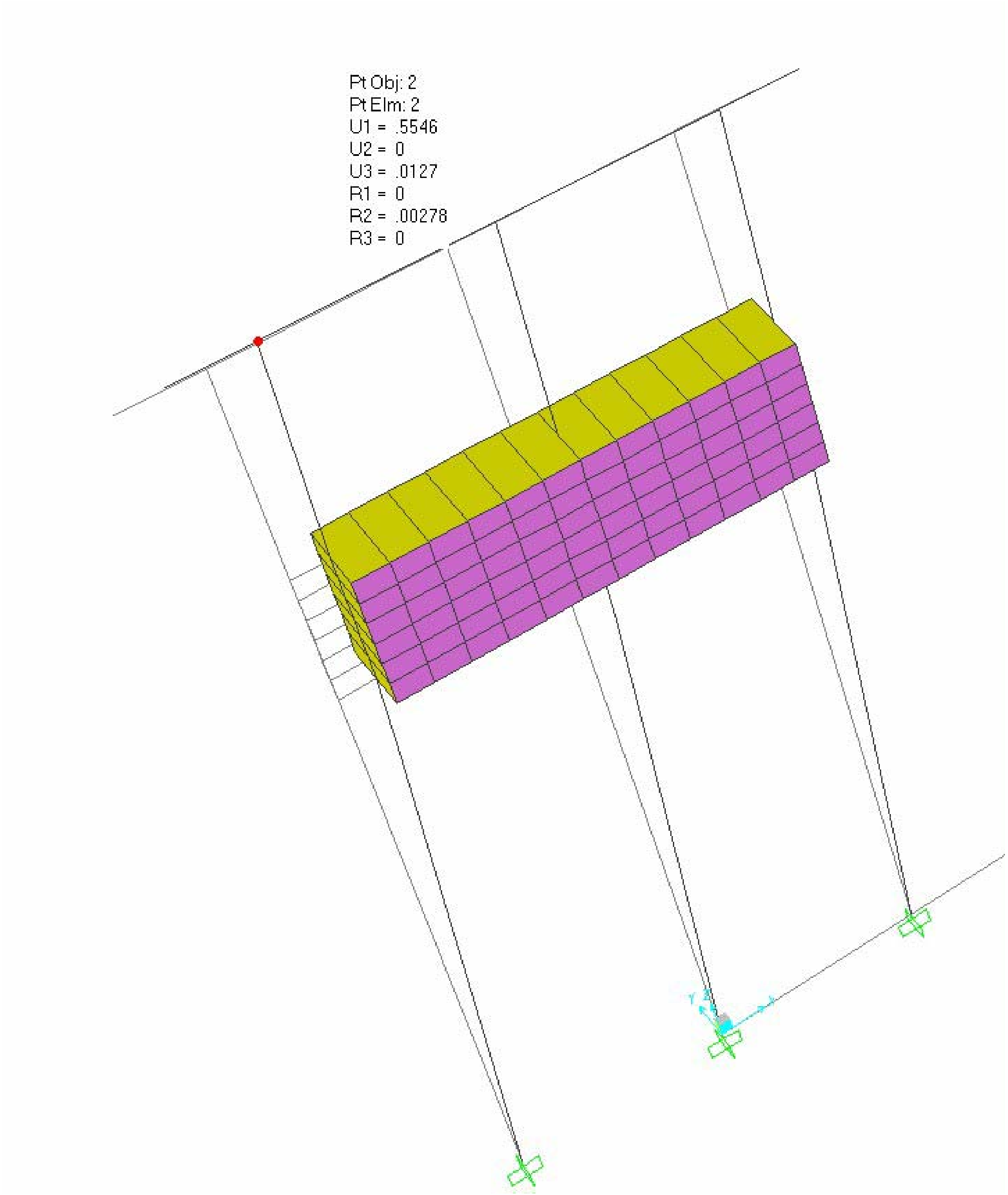


Figure VI.A.15 Response of Model 1 Subjected to 1000 Kips of Transverse Loading

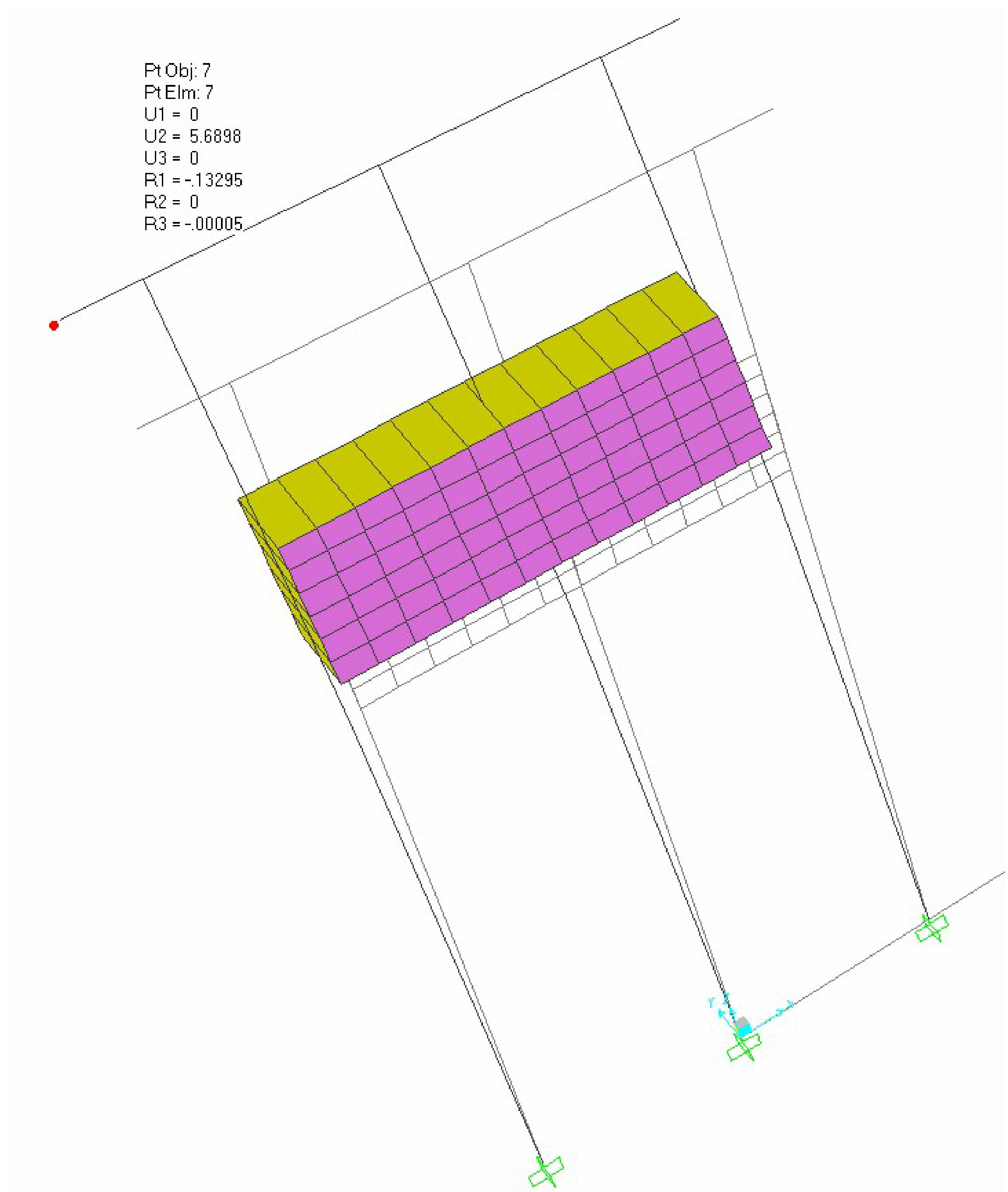


Figure VI.A.16 Response of Model 1 Subjected to 1000 Kips of Longitudinal Loading

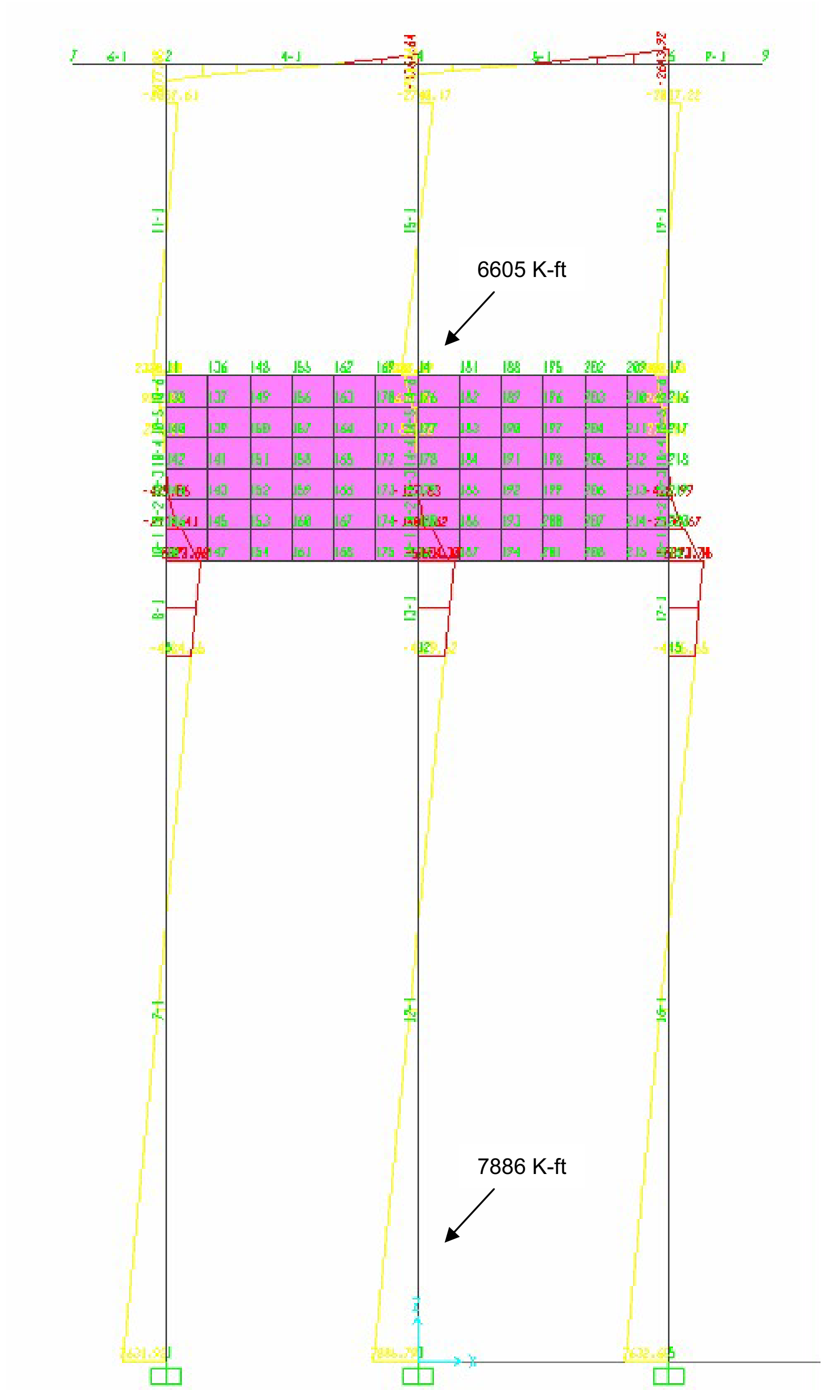


Figure VI.A.17 Model 1 Moment distribution for Bent Subject to 1000 Kips of Transverse Loading

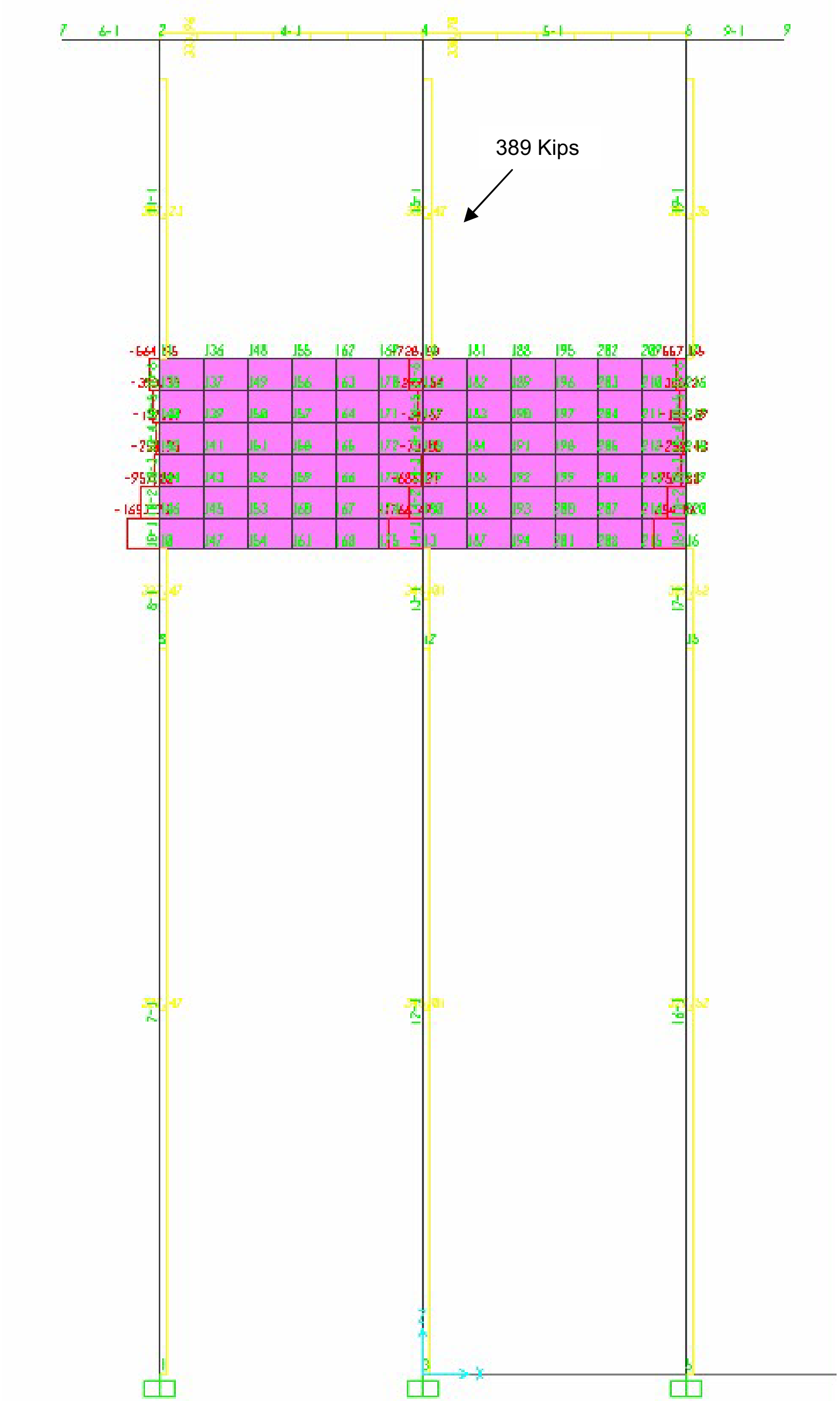


Figure VI.A.18 Model Shear Distribution for Bend Subject to 1000 Kips of Transverse Loading

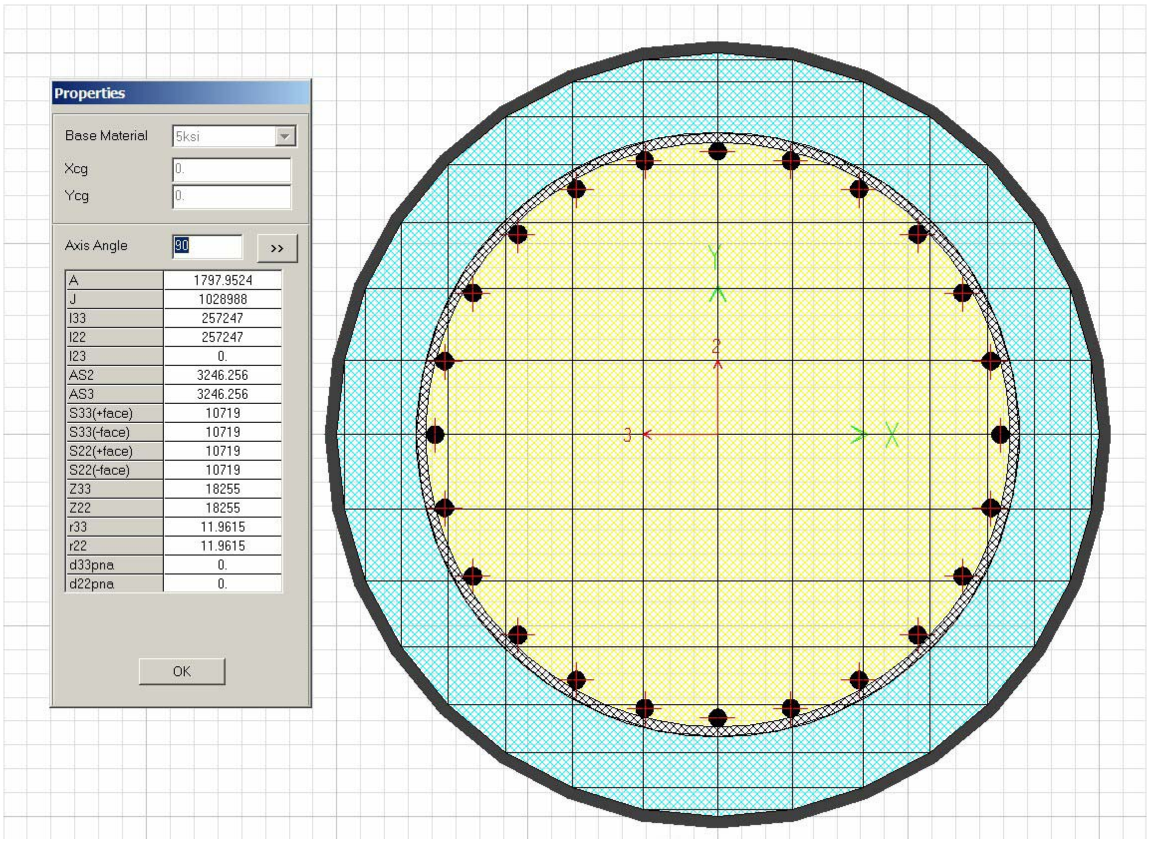


Figure VI.A.19 Column Casing Gross Properties CSE2

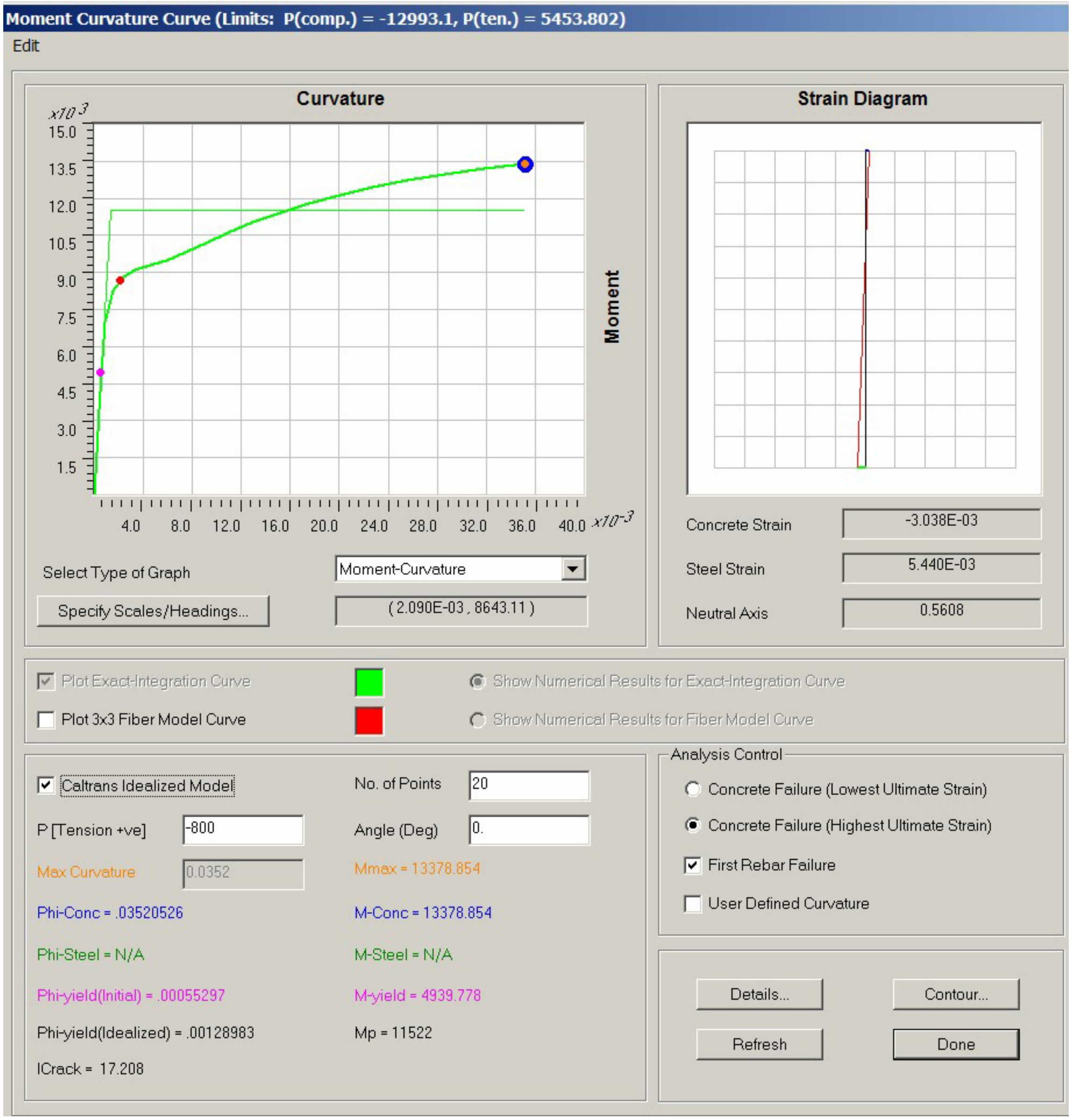


Figure VI.A.20 Moment Curvature Analysis (CSE2)

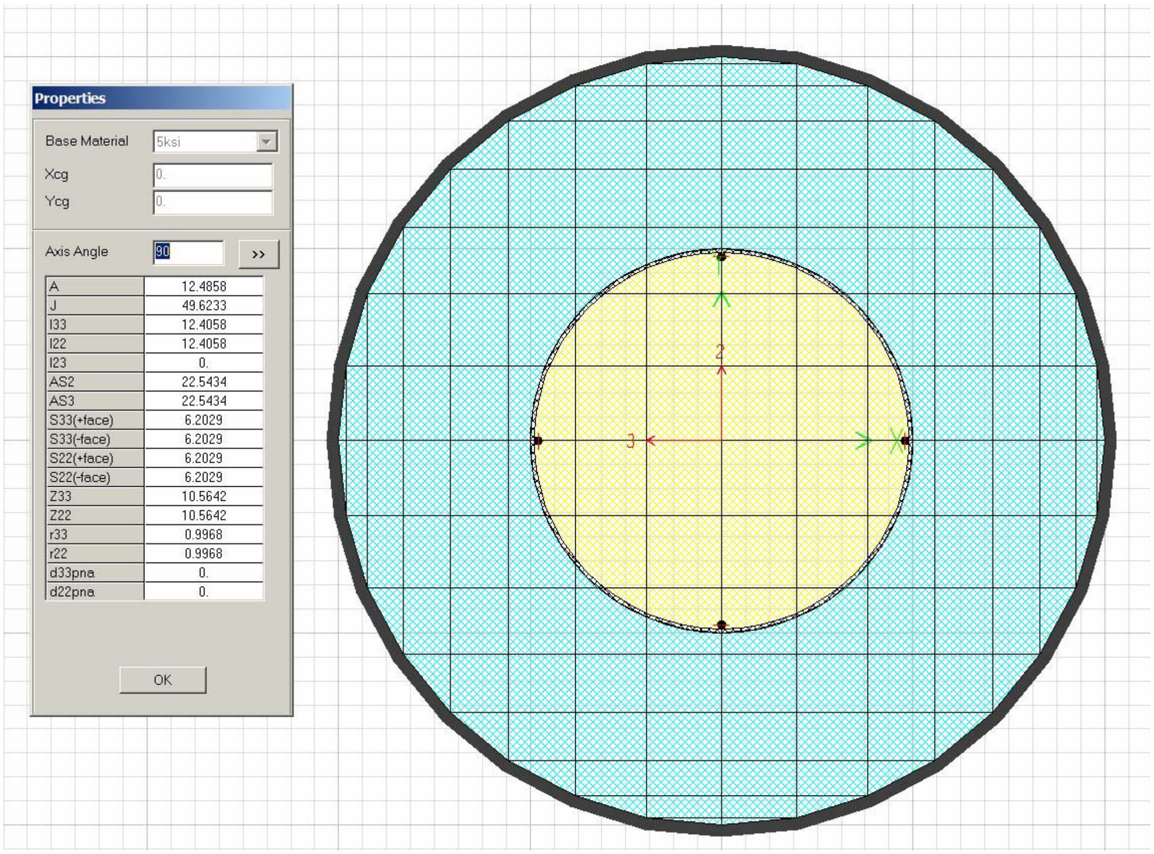


Figure VI.A.21 Shaft Casing Gross Properties (SC2)

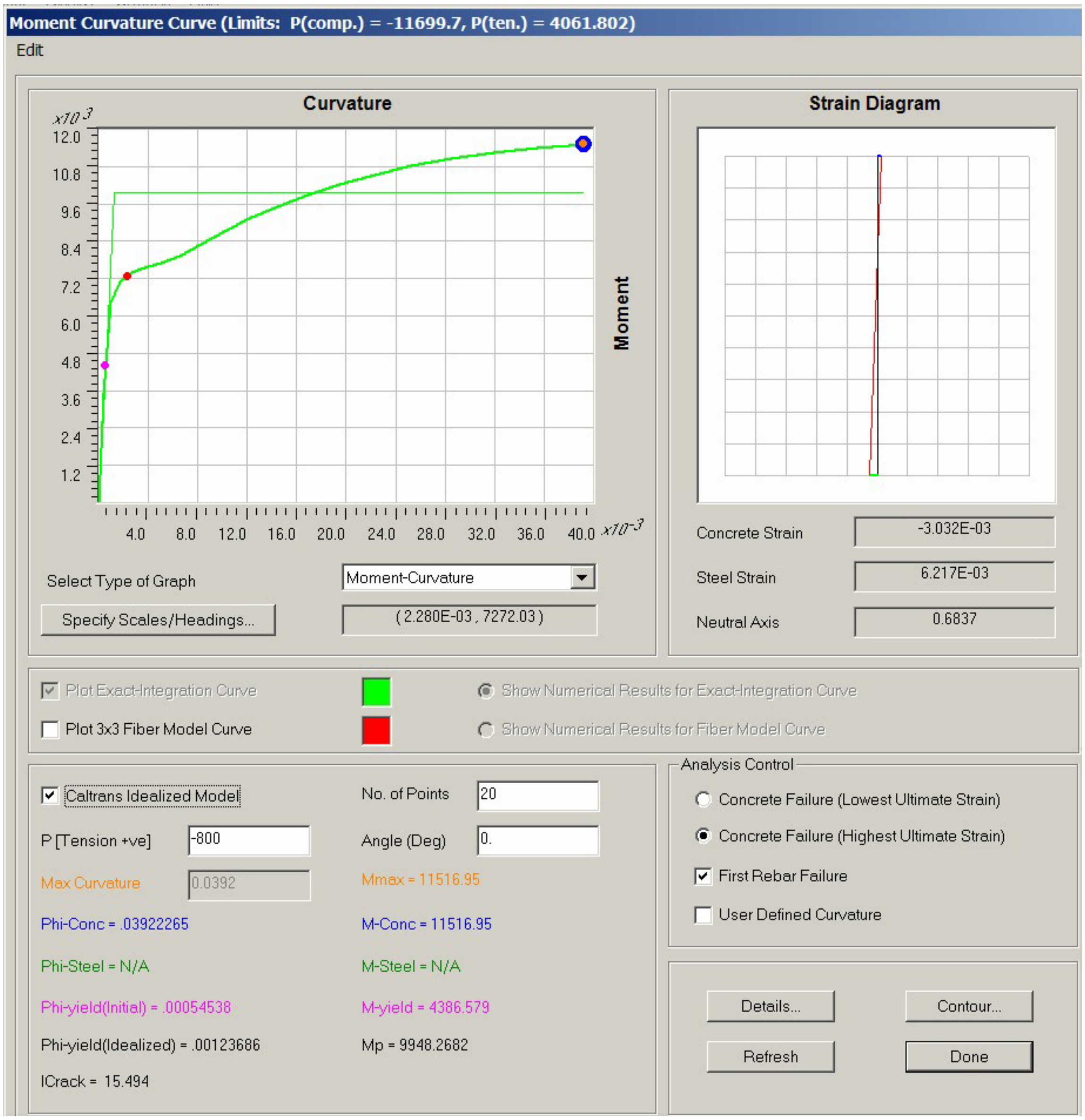


Figure VI.A.22 Moment Curvature Analysis (SC2)

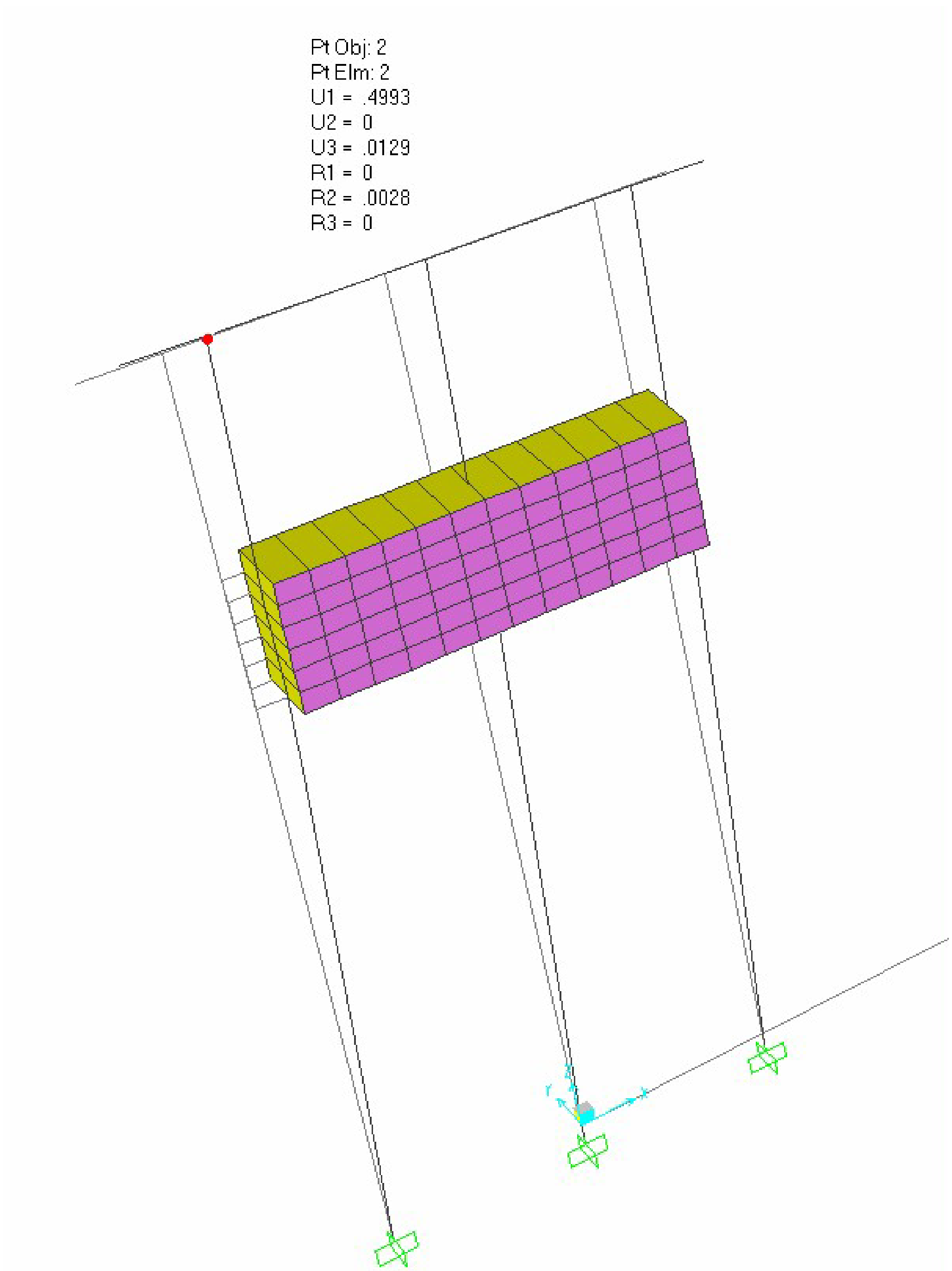


Figure VI.A.23 Response of Model 2 Subjected to 1000 Kips of Transverse Loading

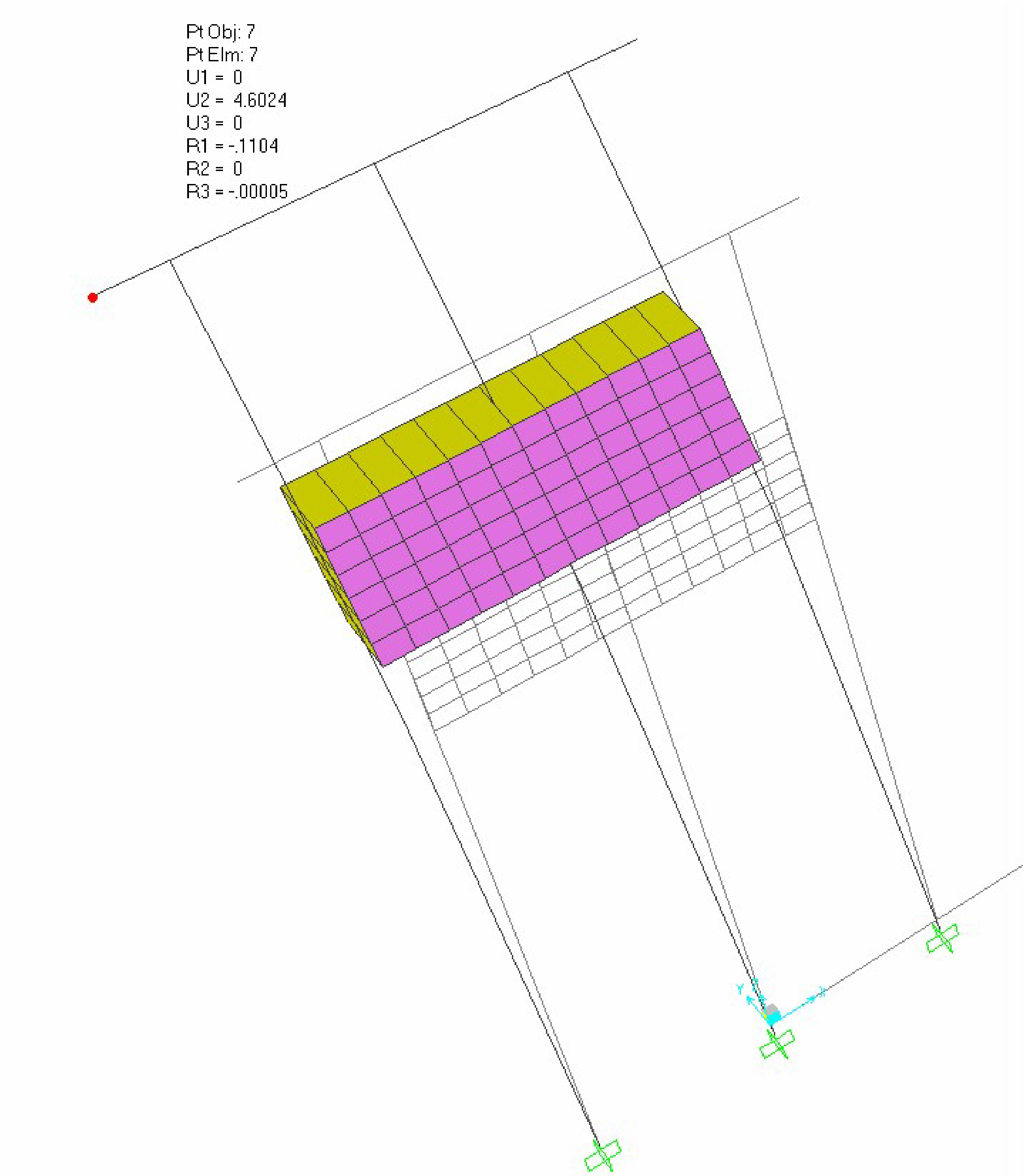


Figure VI.A.24 Response of Model 2 subjected to 1000 Kips of Longitudinal Loading

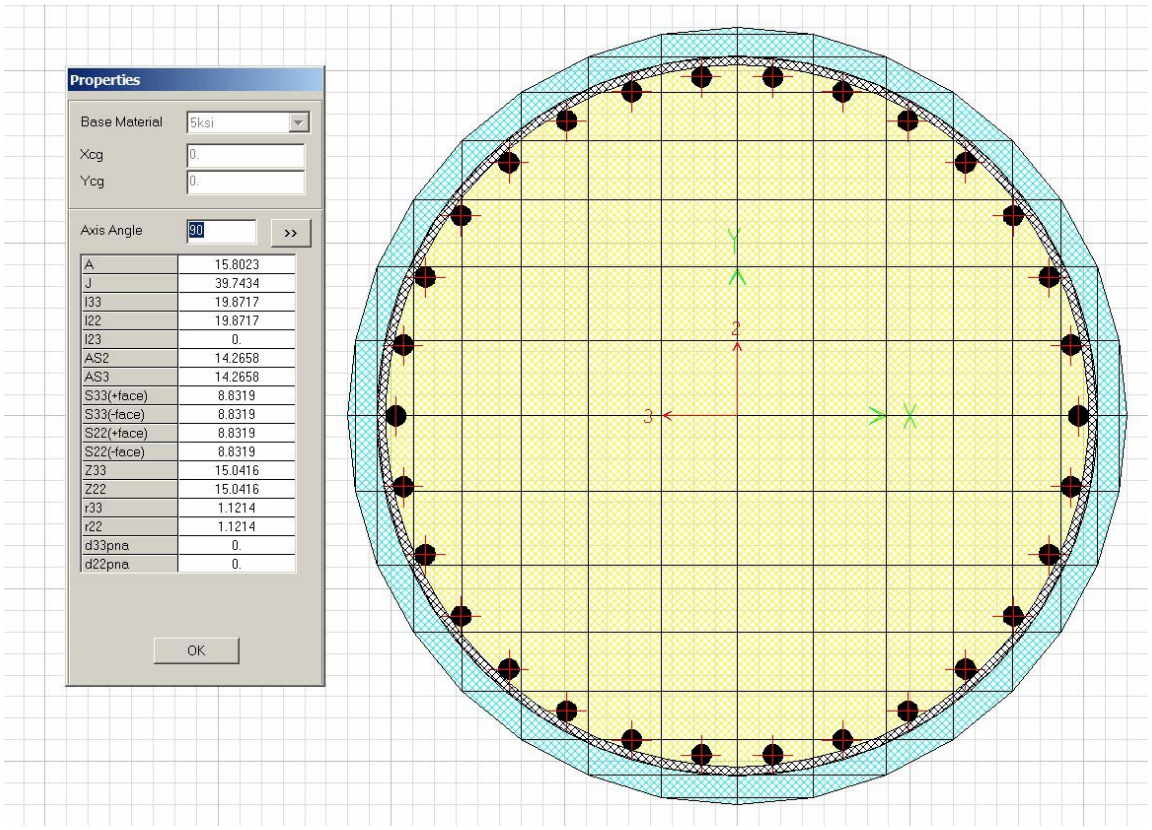


Figure VI.A.25 Column Gross Properties (CS3)

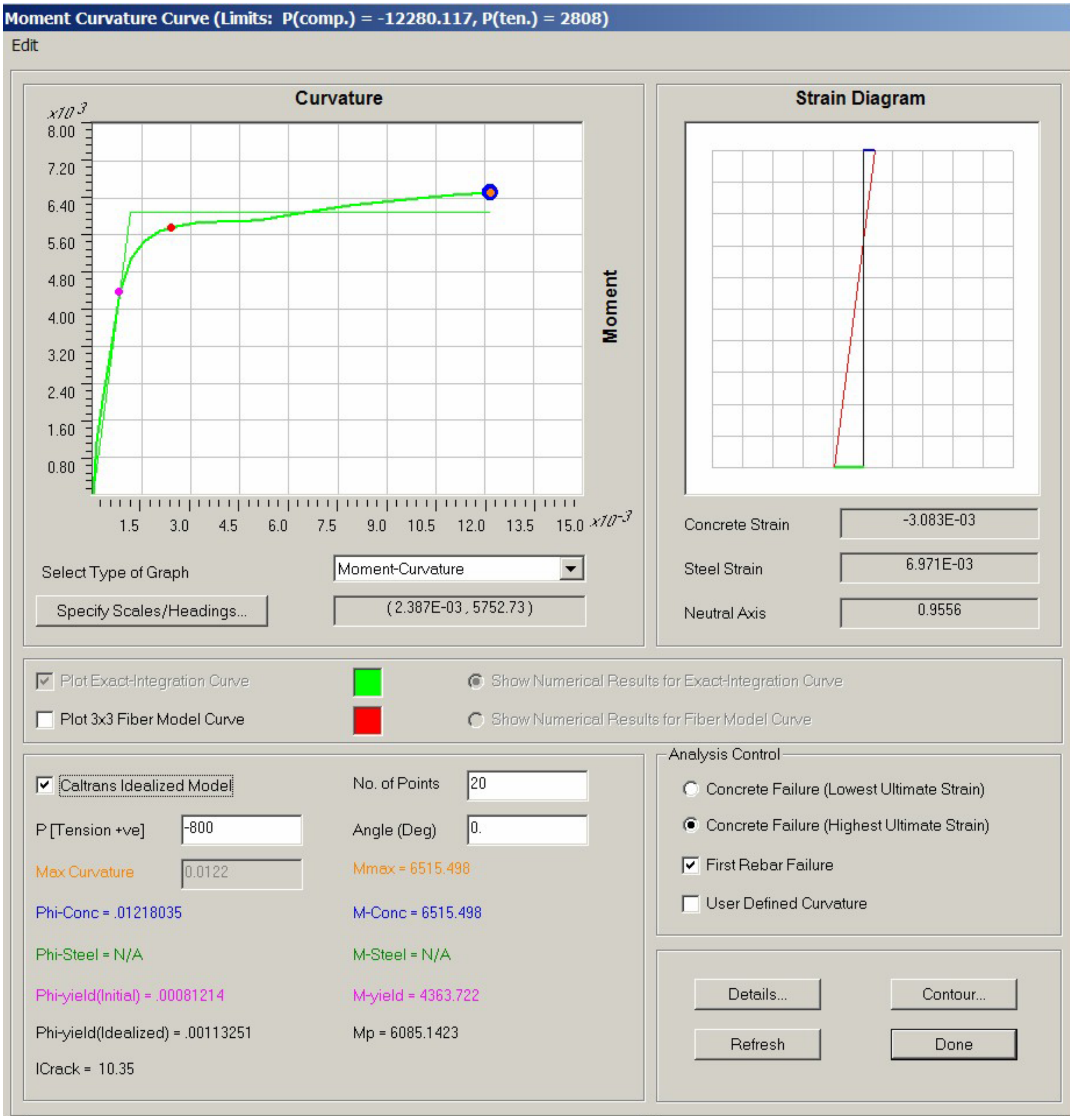


Figure VI.A.26 Moment Curvature Analysis (CS3)

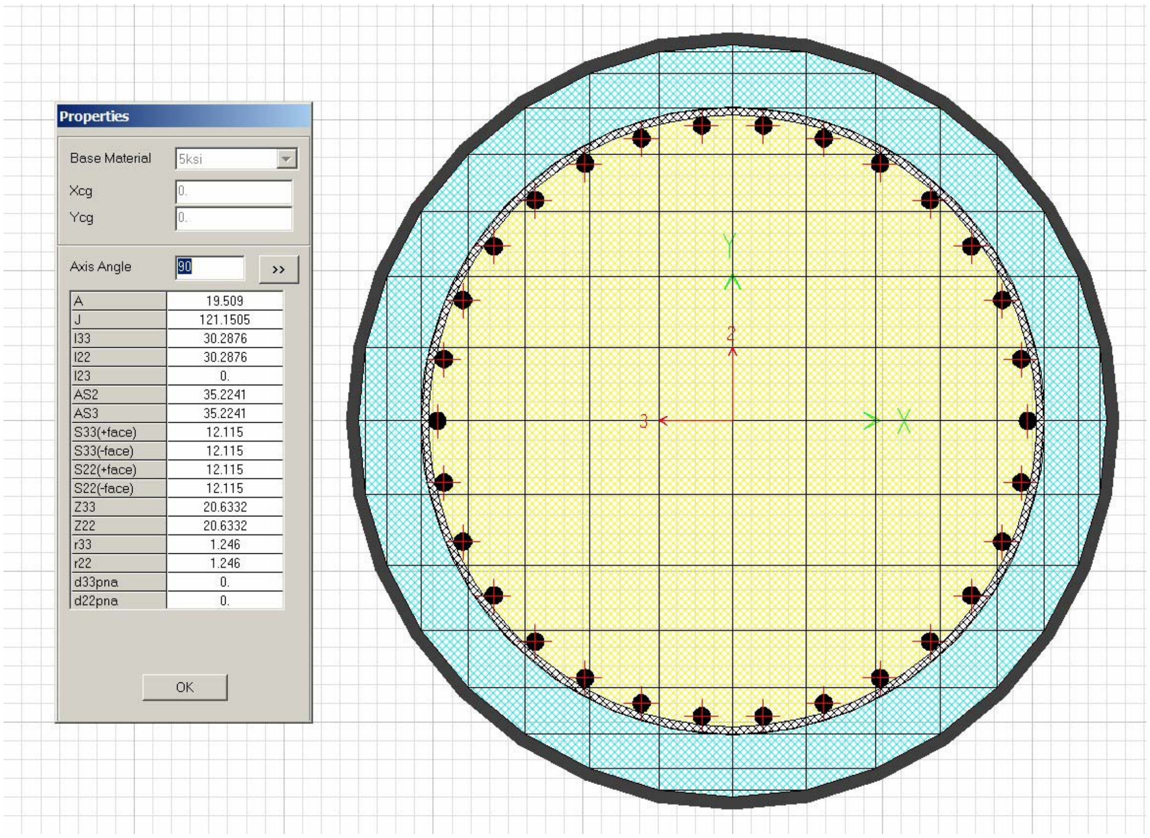


Figure VI.A.27 Column Casing Gross Properties (CSE3)

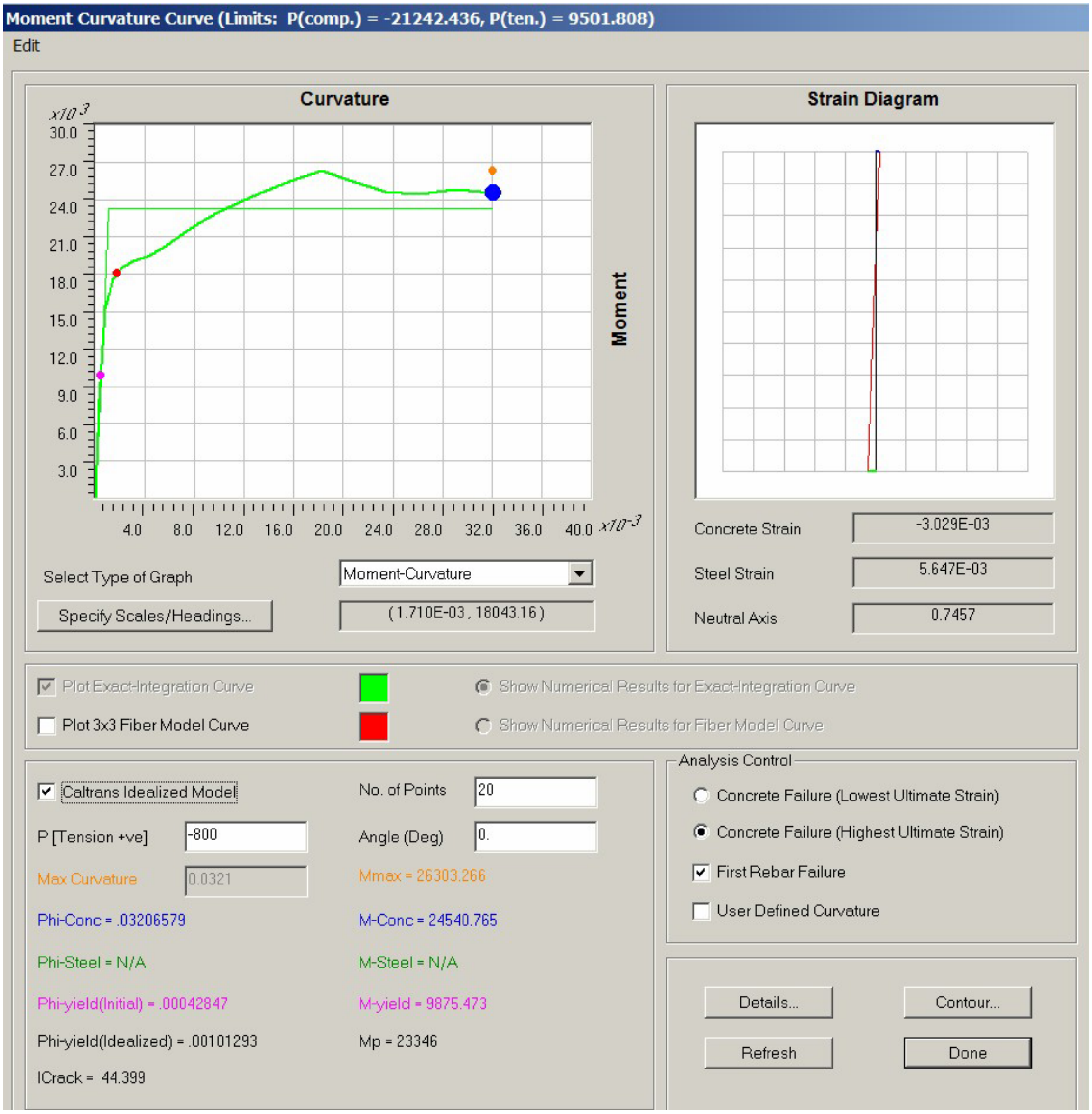


Figure VI.A.28 Moment Curvature Analysis (CSE3)

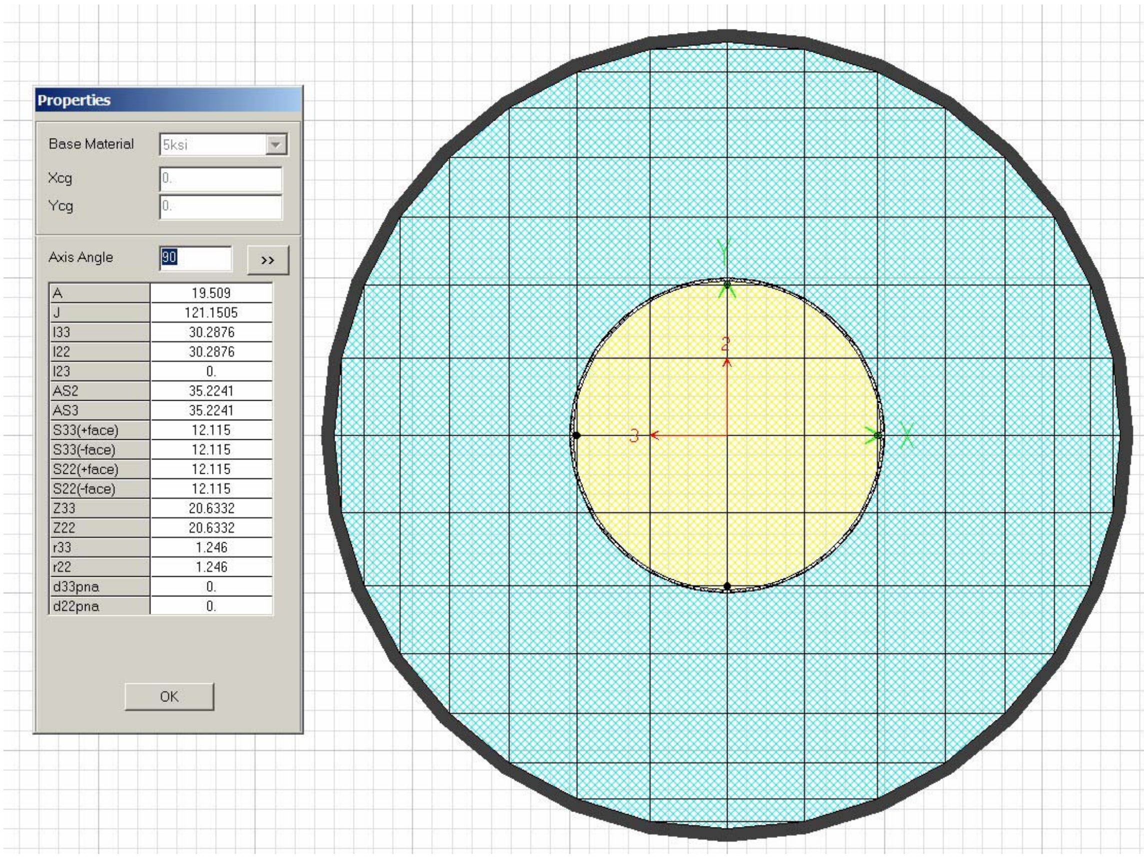


Figure VI.A.29 Shaft Casing Gross Properties (SC3)

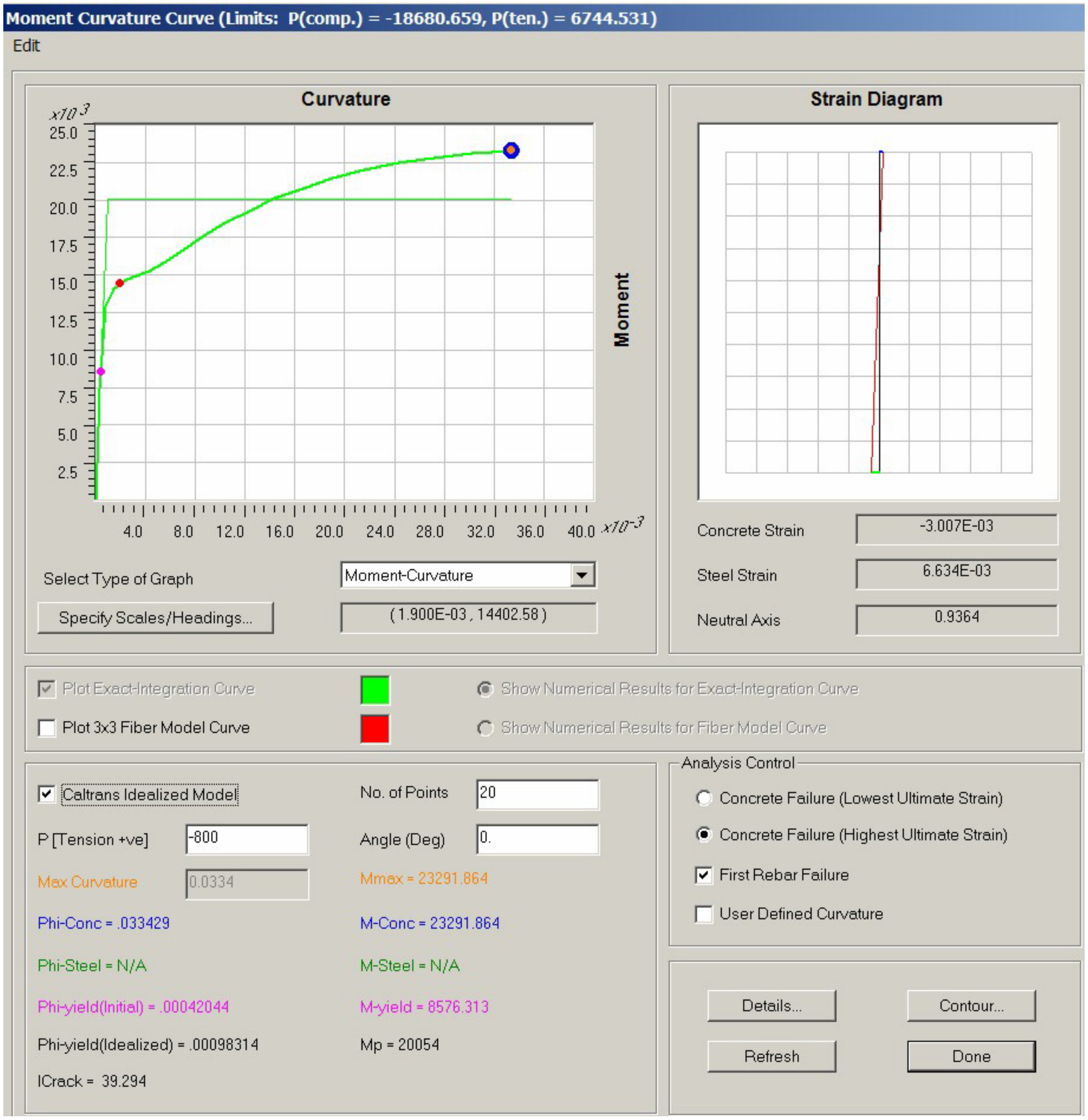


Figure VI.A.30 Moment Curvature Analysis (SC3)

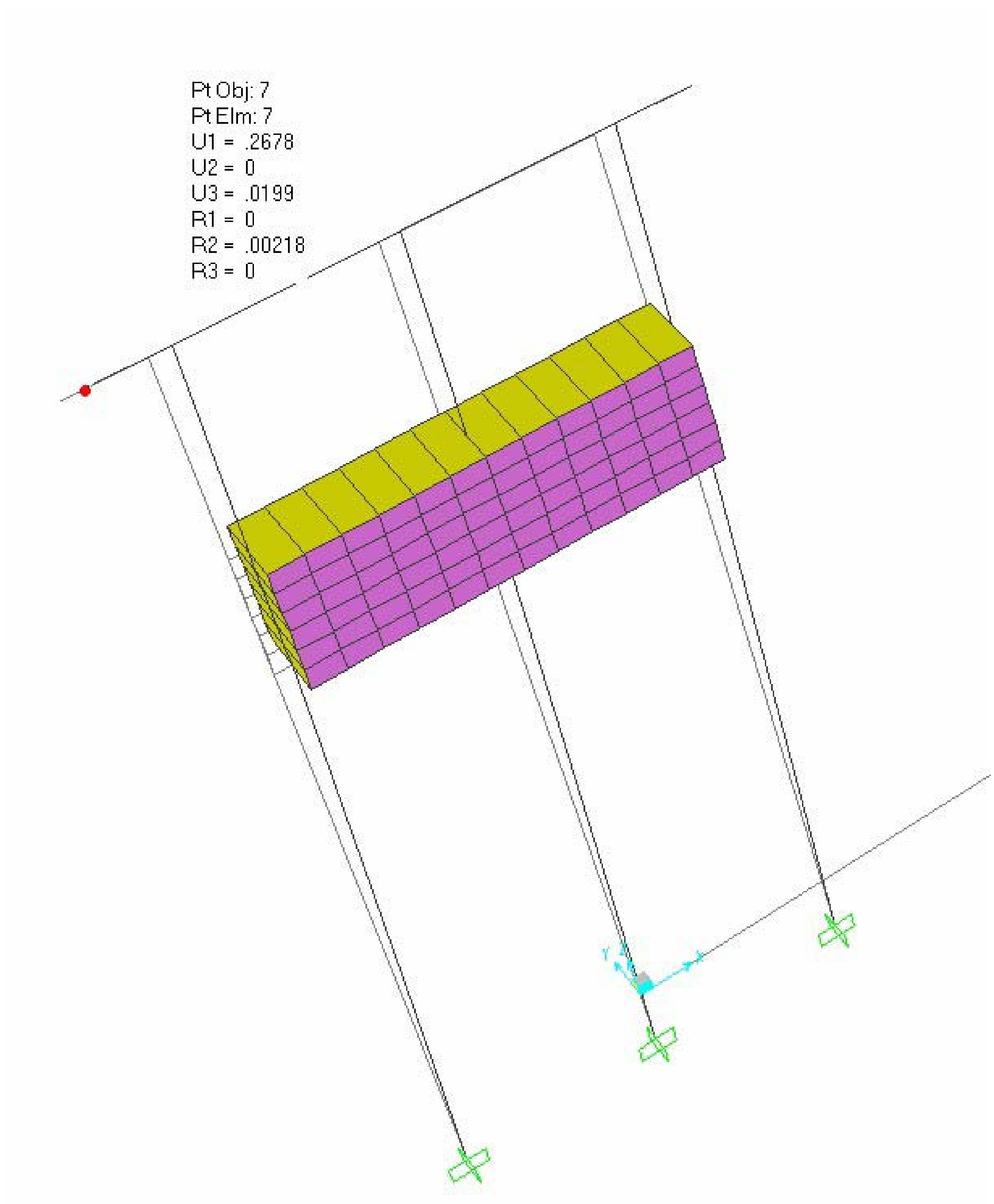


Figure VI.A.31 Response of Model 3 Subjected to 1000 Kips of Transverse Loading

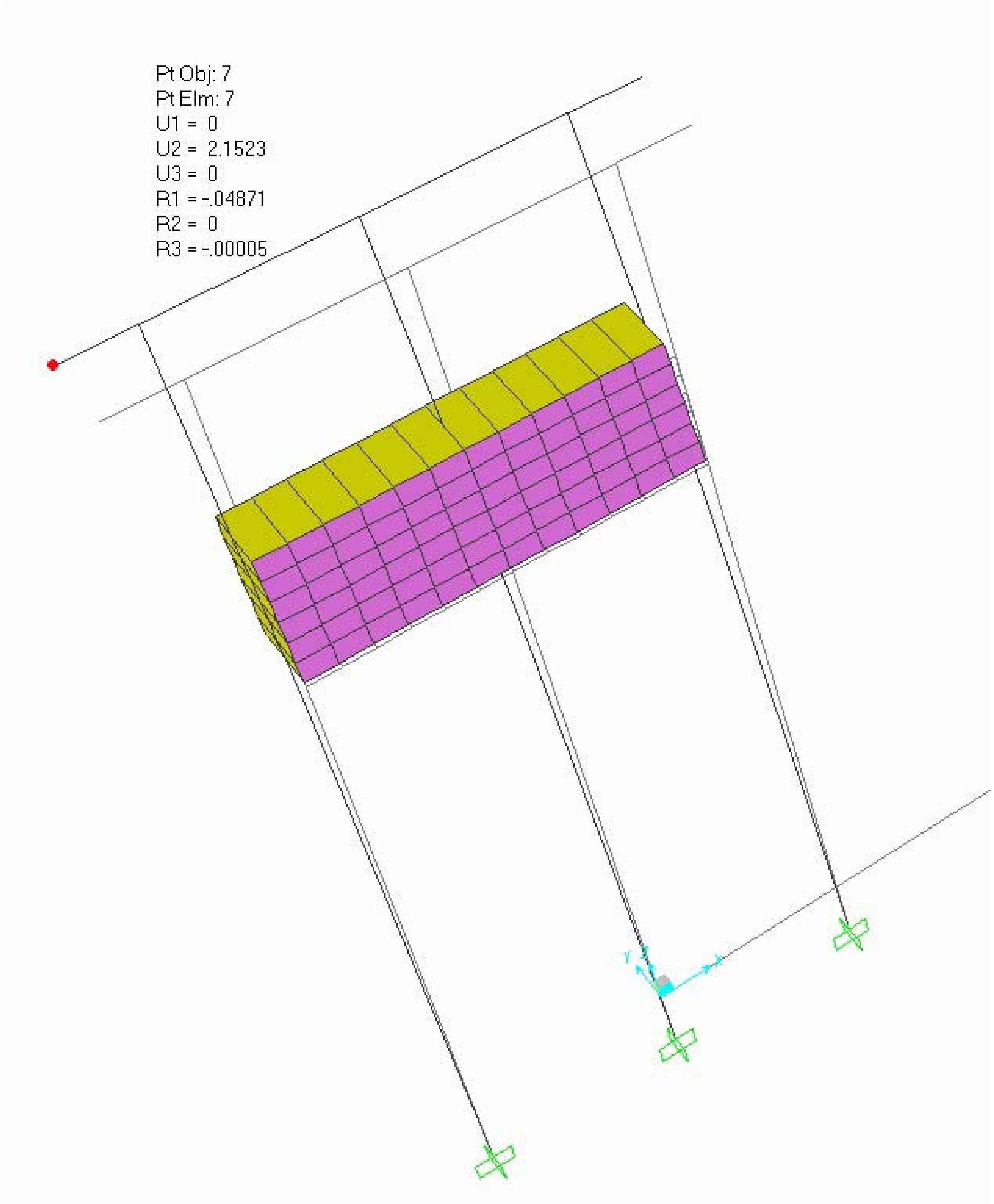


Figure VI.A.32 Response of Model 3 Subjected to 1000 Kips of Longitudinal Loading

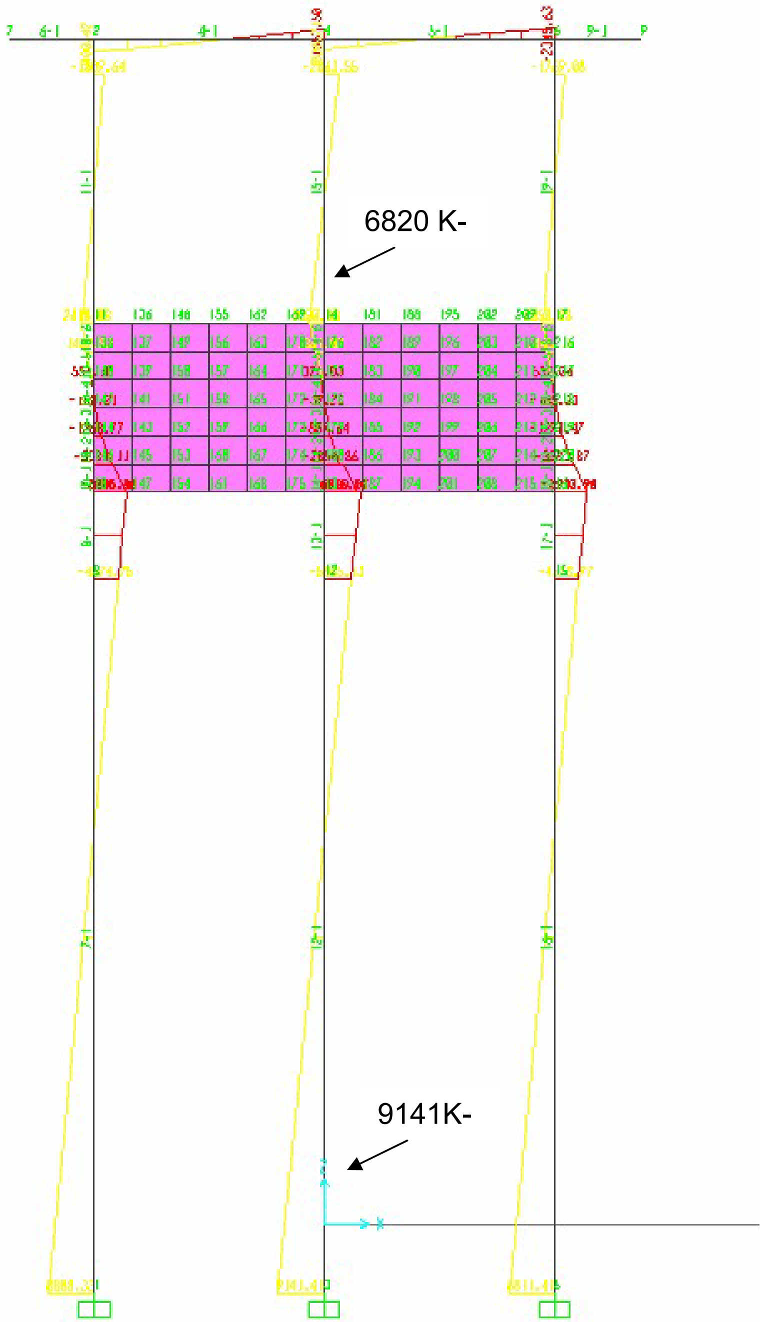


Figure VI.A.33 Model 3 Moment Distribution for Bent Subject to 1000 Kips of Transverse Loading

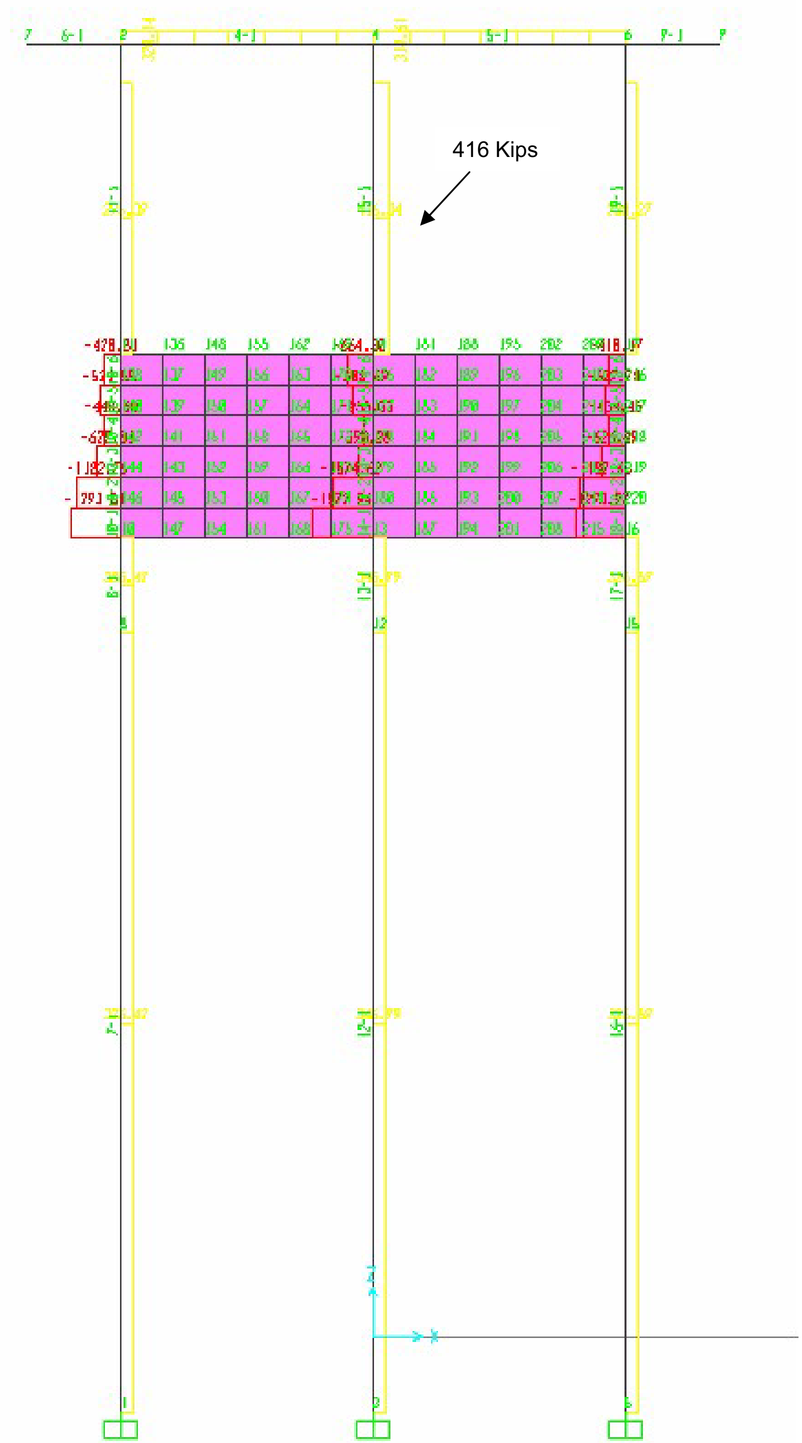


Figure VI.A.34 Model 3 Shear Distribution for Bent Subject to 1000 Kips of Transverse Loading

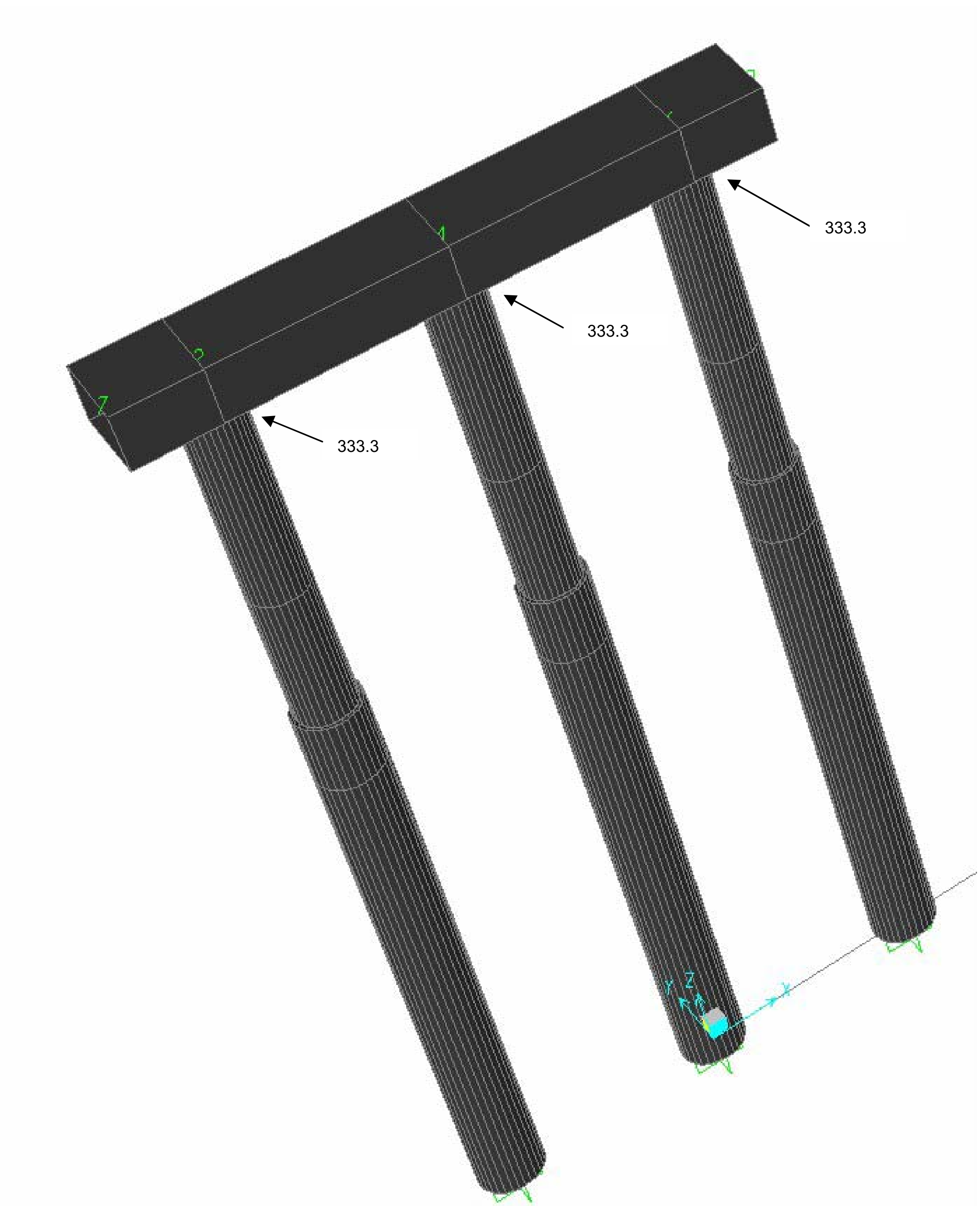


Figure VI.A.35 CSI-SAP Model 4 Subjected to 1000 Kips Longitudinal Loading

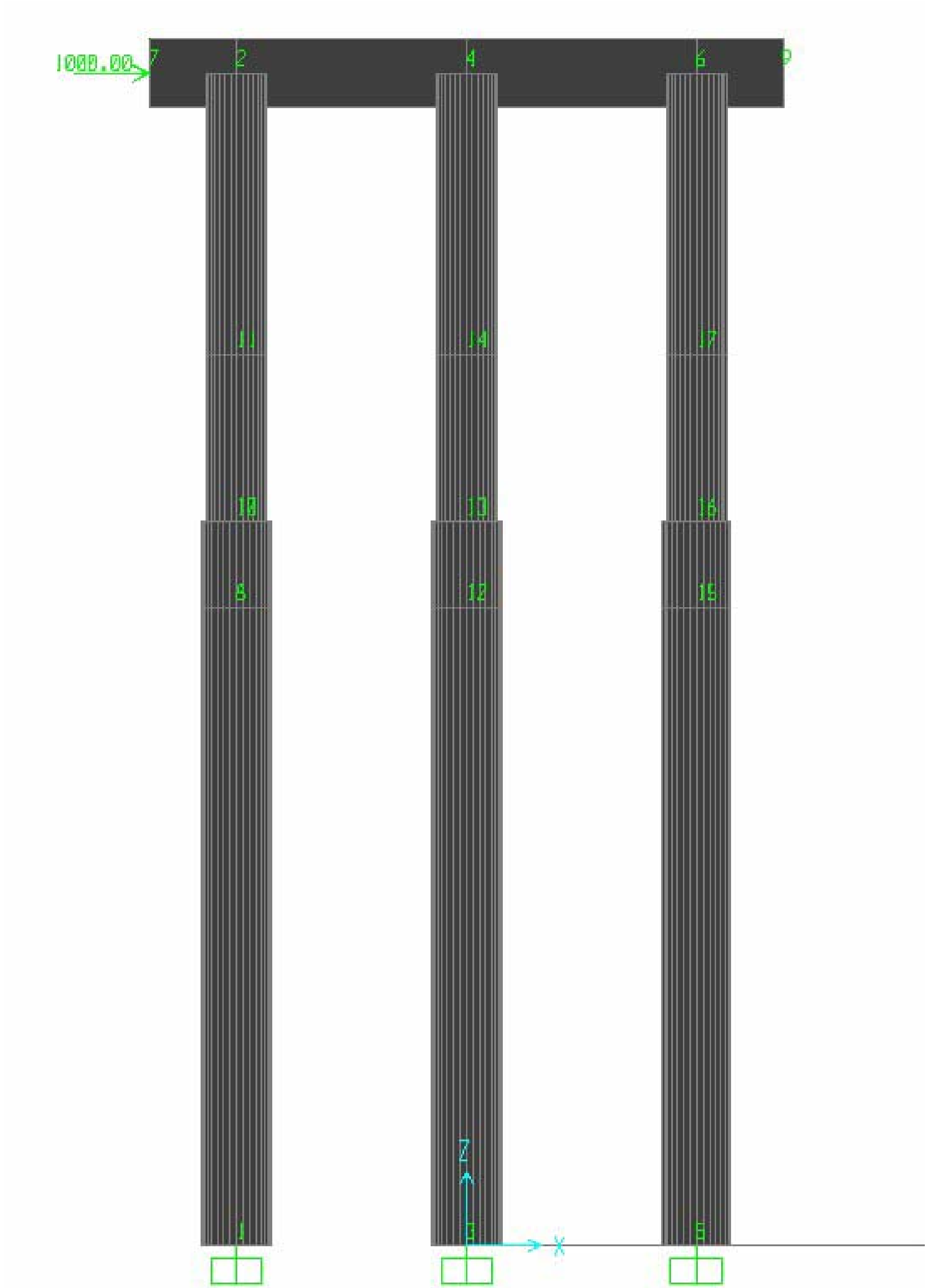


Figure VI.A.36 CSI-SAP Model 4 Subjected to 1000 Kips of Transverse Loading

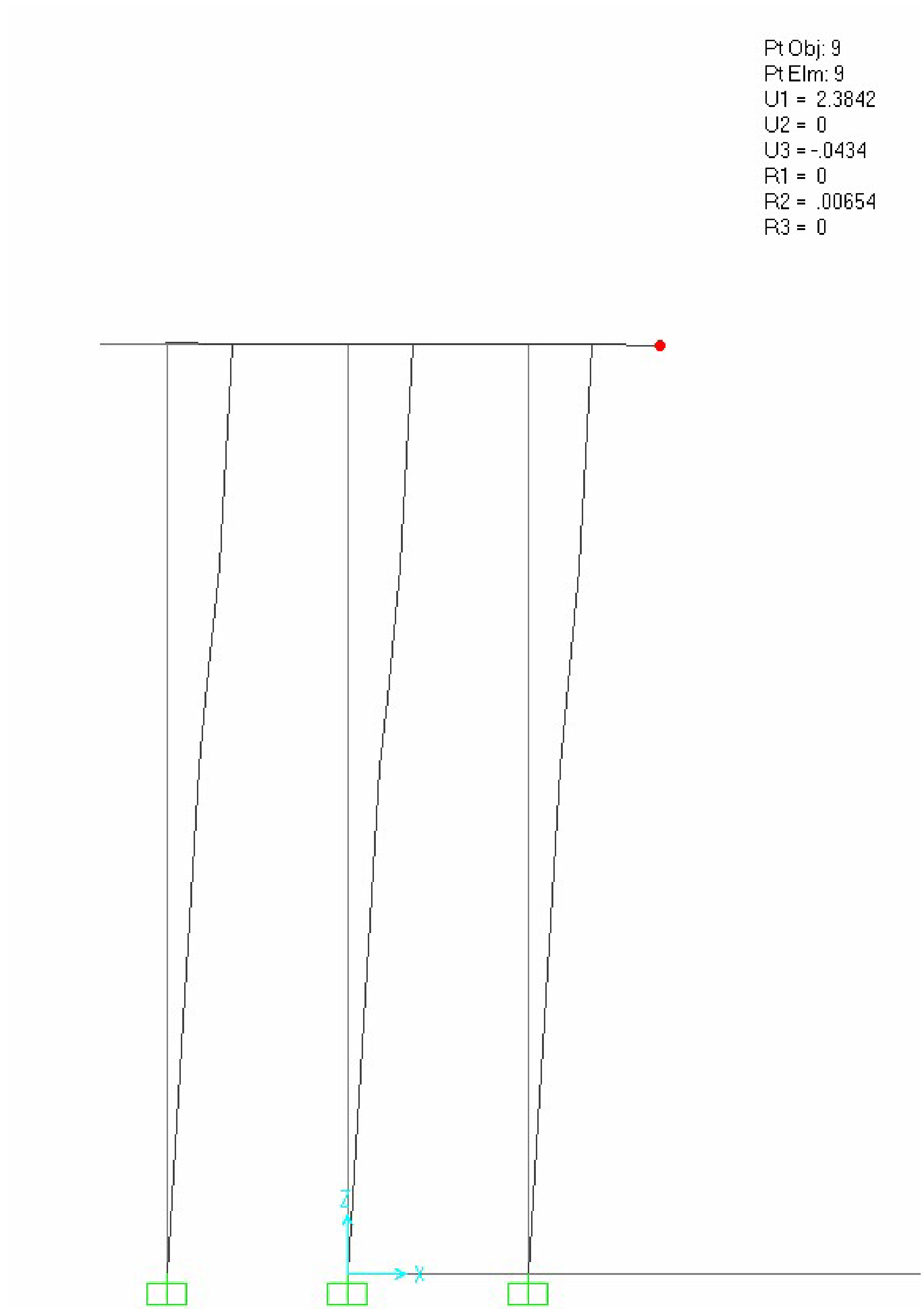


Figure VI.A.37 Response of Model 4 Subjected to 1000 Kips of Transverse Loading

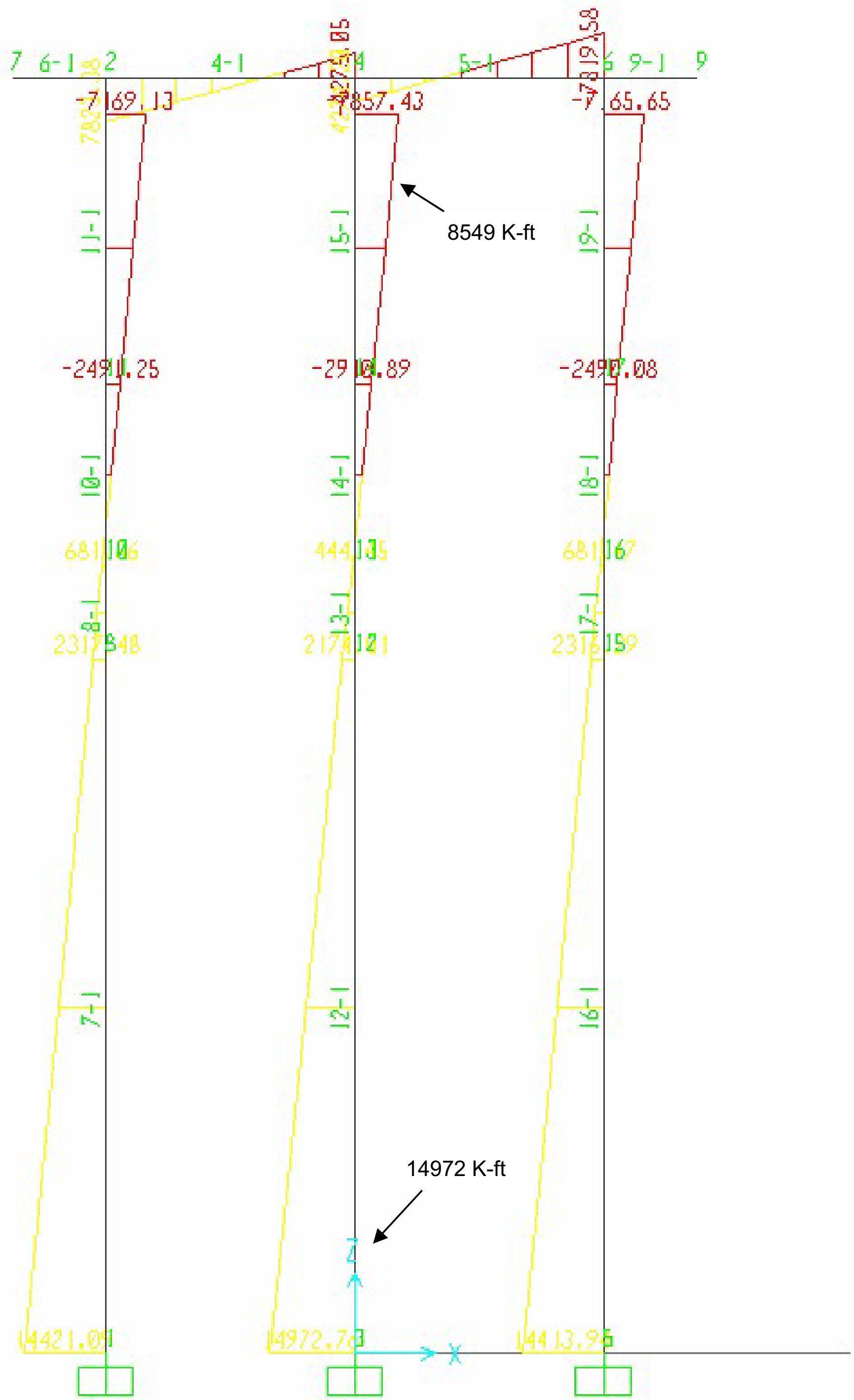


Figure VI.A.38 Model 4 Moment Distribution for Bent Subject to 1000 Kips of Transverse Loading

VI.B: Superstructure Details

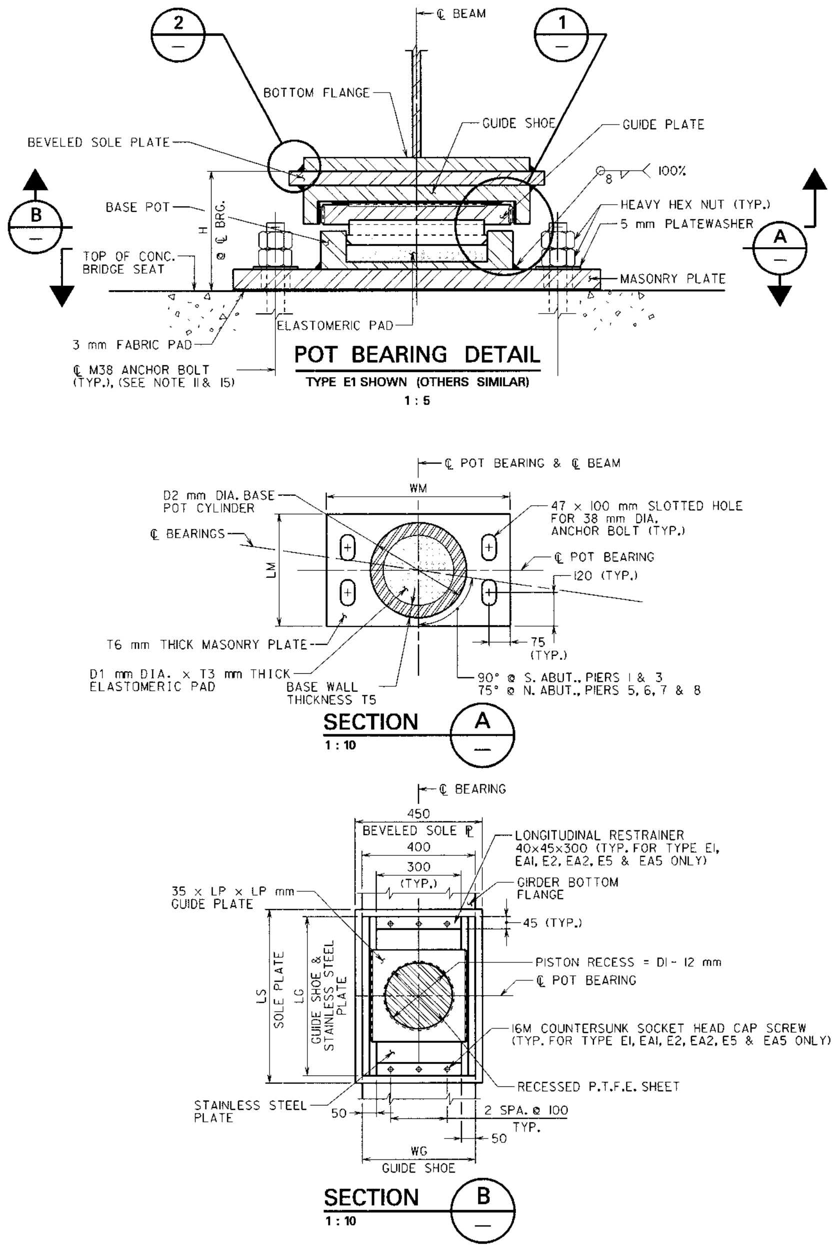
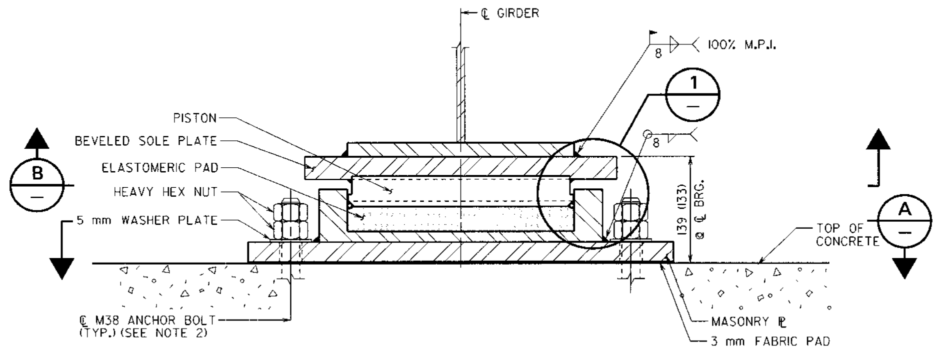


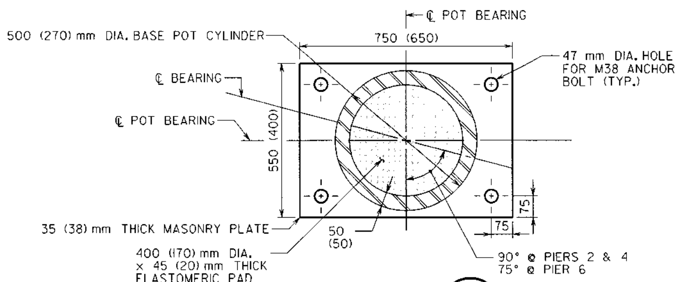
Figure VI.B.1Expansion Pot Bearing Details



POT BEARING DETAIL

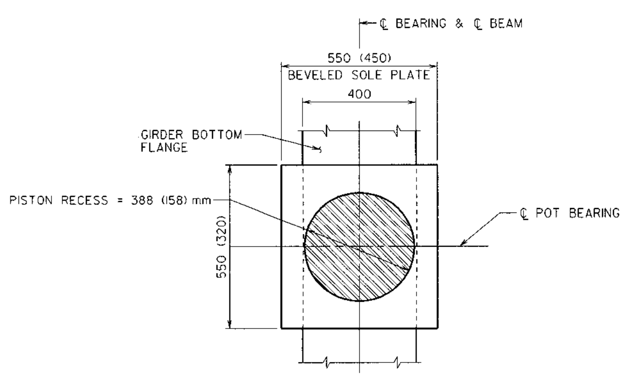
TYPE F1 (TYPE F2 SIMILAR AS NOTED)

1 : 5



SECTION A

1 : 10



SECTION B

1 : 10

Figure VI.B.2 Fixed Pot Bearing Details

TABLE OF DIMENSIONS

BEARING DESIGNATION	QTY	FACTORED MAXIMUM VERTICAL LOAD (kN)	FACTORED MAXIMUM HORIZONTAL FORCE (kN)	MOVEMENT ONE-WAY (MIN)	BEARING LOCATION	MASONRY PLATE (mm) A	B	C	D	E	SHEAR PIN (mm) H	I	J	K	UPPER BEARING PLATE (mm) L	M	N	O
E1	20	1700	485	40	PIER 3	655	450	500	160	45	70	15	60	45	285	335	125	40
E2	10	1650	416	75	PIER 6	655	475	500	160	45	70	15	60	45	285	335	125	40
E5	10	1650	416	65	N&S ABUT	655	535	500	160	50	70	15	60	45	285	335	125	40
E3	10	3000	285	45	PIER 7	705	550	550	210	45	70	15	75	60	350	370	110	35
E3	10	3000	285	45	PIER 8	705	550	550	210	45	70	15	75	60	350	370	110	35
E4	10	4200	355	40	PIER 5	750	685	600	310	65	85	15	80	65	415	415	145	35
E4	10	4200	355	40	PIER 1	750	685	600	310	65	85	15	80	65	415	415	145	35

BEARING DESIGNATION	PTFE (mm) Pa	Pb	Q	SPTFE (mm) R	S	STAINLESS PLATE (mm) T	U	Va	Vb	GUIDE BAR (mm) W	X	Y	SLIDE PLATE (mm) AA	BB	TH	HEIGHT (mm) HH	POLYTRON (mm) KK	LL
E1	250	250	5	325	35	260	380	40	455	480	35	50	510	420	40	168	230	35
E2	250	250	5	325	35	260	450	40	530	555	25	50	580	420	30	153	230	35
E5	250	250	5	325	35	260	430	40	510	535	25	50	560	420	30	160	230	35
E3	330	330	5	295	25	345	475	35	440	470	20	45	495	450	30	173	315	45
E3	330	330	5	295	25	345	475	35	440	470	20	45	495	450	40	170	315	45
E4	390	390	5	355	25	405	520	35	490	500	20	45	535	500	30	188	355	50
E4	390	390	5	355	25	405	520	35	490	500	20	45	535	500	30	185	355	50

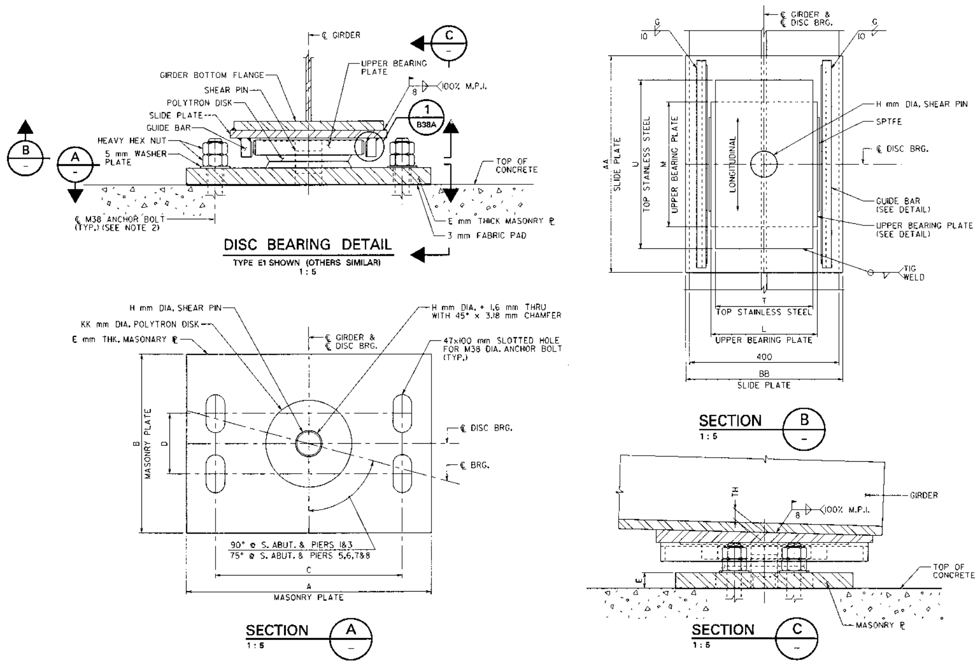


Figure VI.B.3 Expansion Disc Bearing Details

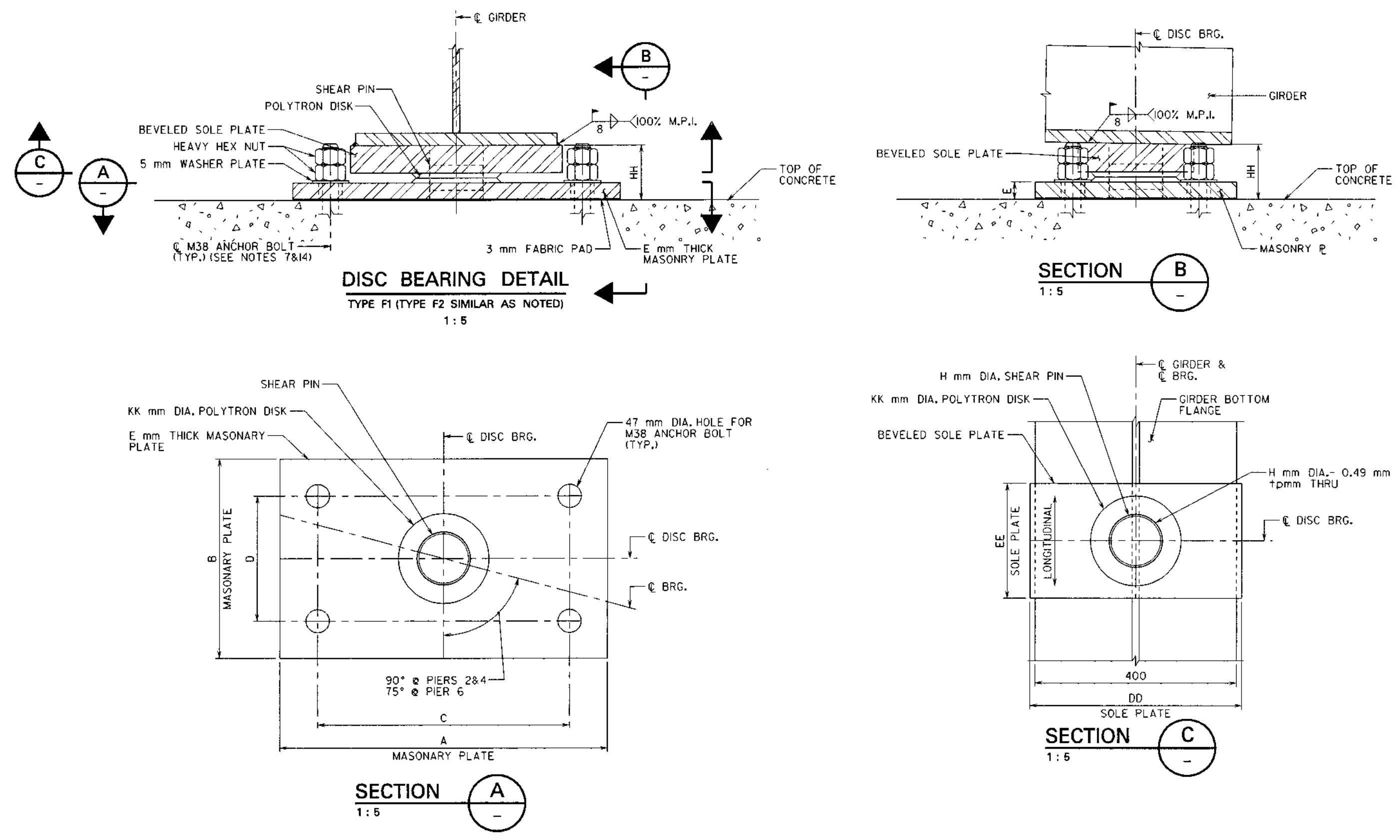
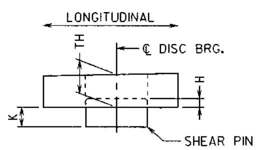
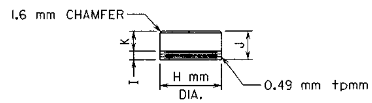


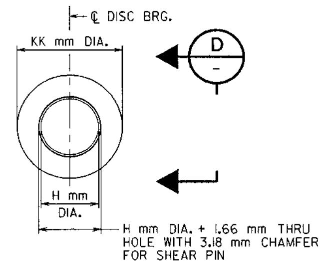
Figure VI.B.4 Fixed Disc Bearing Details 1



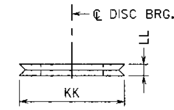
BEVELED SOLE PLATE DETAIL
1 : 5



SHEAR PIN
1 : 5



POLYTRON DISC
1 : 5



SECTION D
1 : 5

NOTE:
POLYTRON DISC TO BE INSTALLED w/ CHAMFER ON BOTTOM

TABLE OF DIMENSIONS

BEARING DESIGNATION	QTY	FACTORED MAXIMUM VERTICAL LOAD (kN)	FACTORED MAXIMUM HORIZONTAL FORCE (kN)	BEARING LOCATION	MASONRY PLATE (mm)					SHEAR PIN (mm)				SOLE PLATE (mm)			HEIGHT (mm)	POLYTRON (mm)	
					A	B	C	D	E	H	I	J	K	DD	EE	TH	HH	KK	LL
F1	10	4200	495	PIERS 2&4	755	695	600	400	65	115	15	80	65	425	425	25	139	355	50
F2	10	850	430	PIER 6	650	400	500	250	35	105	15	50	35	420	230	55	113	180	20

NOTE: THE ACTUAL HEIGHT VALUES, H, WILL BE LESS THAN THAT SHOWN DUE TO ROUNDING VARIATIONS FROM ENGLISH TO METRIC.

Figure VI.B.5 Fixed Disc Bearing Details 2

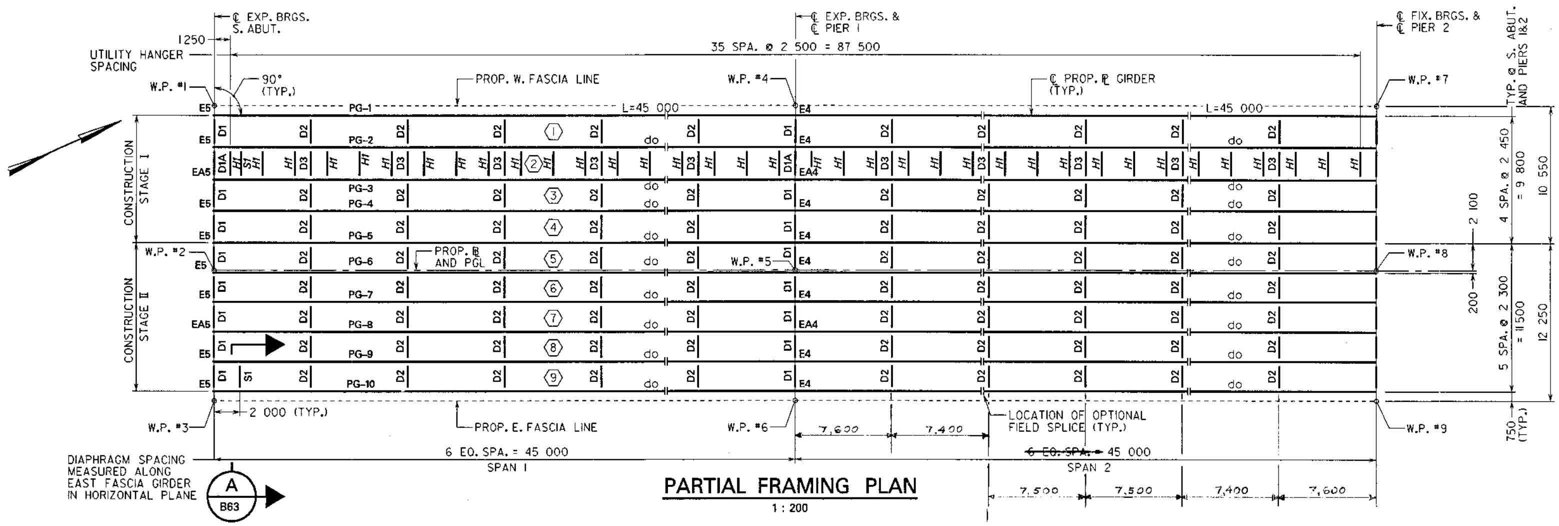


Figure VI.B.6 Partial Framing Plan Spans 1 and 2

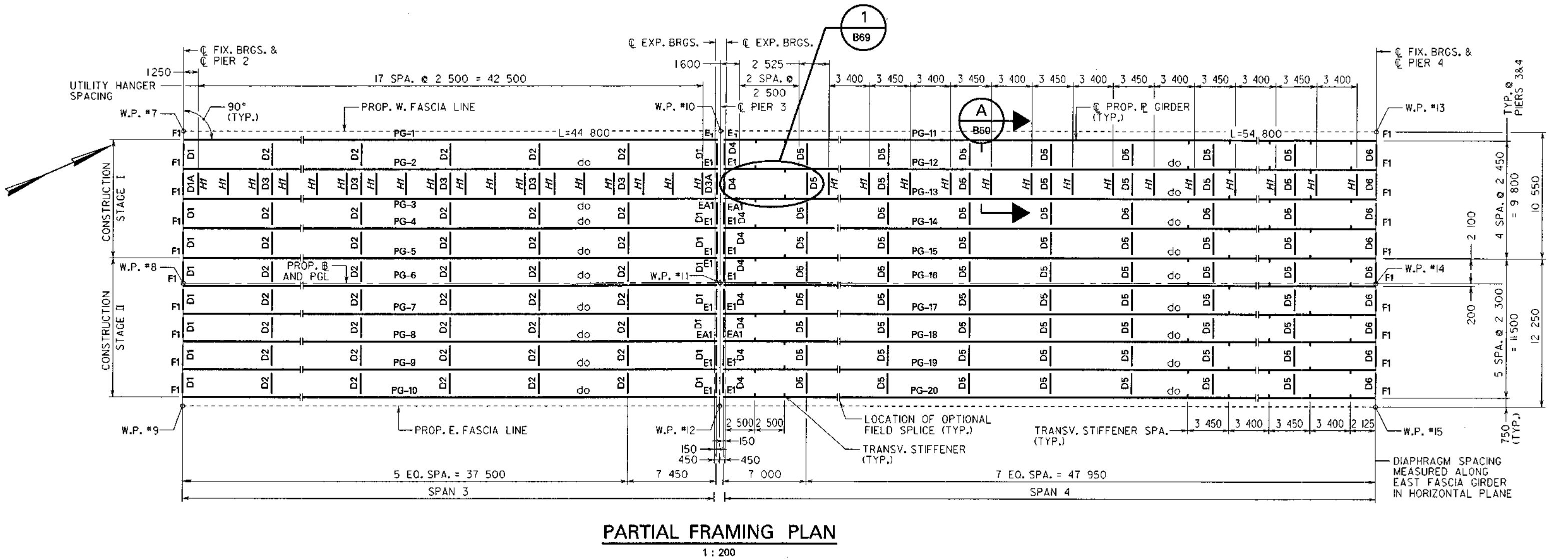


Figure VI.B.7 Partial Framing Plan Spans 3 and 4

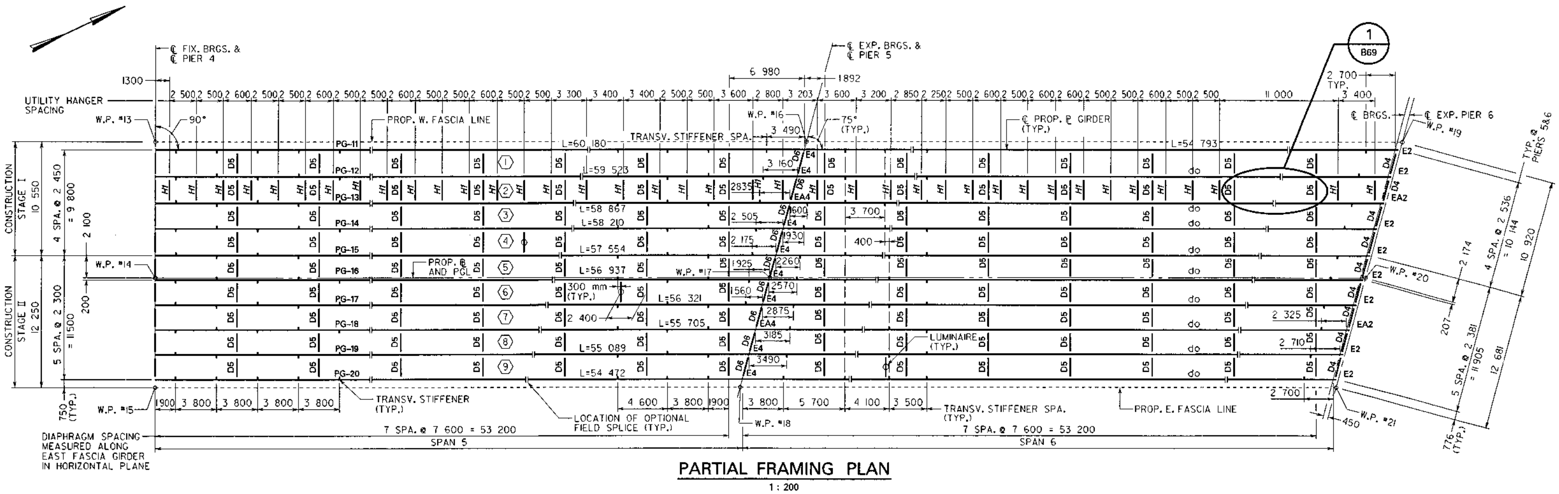


Figure VI.B.8 Partial Framing Plan Spans 5 and 6

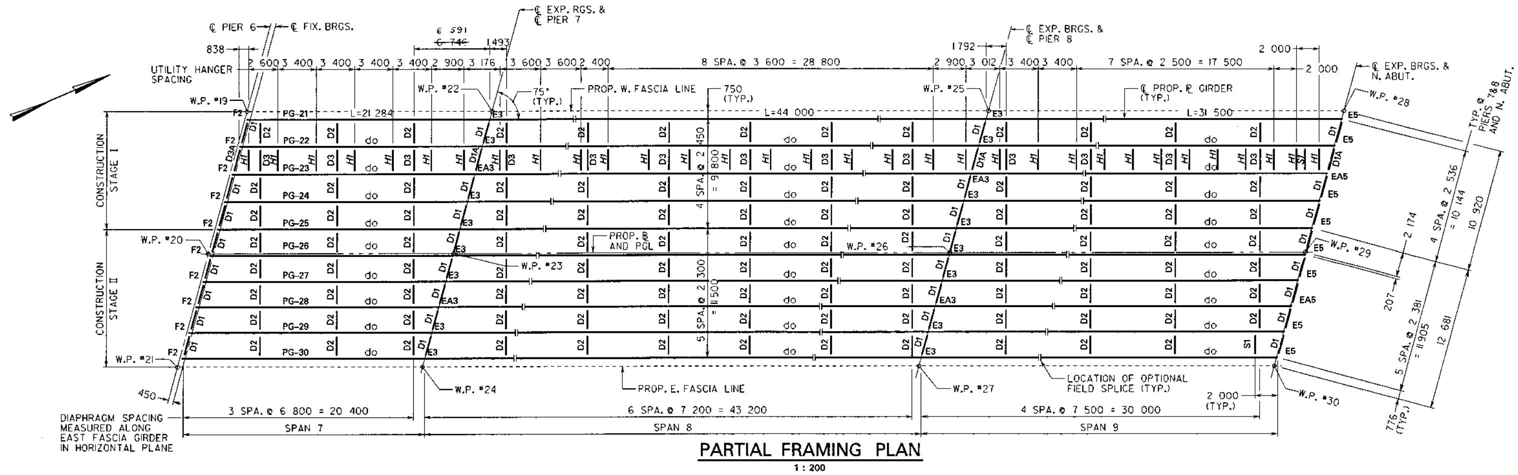


Figure VI.B.9 Partial Framing Plan Spans 7, 8, and 9

MOVEMENT REQUIREMENTS				
PIER NO.	JOINT OPENINGS			MOVEMENT CLASSIFICATION
	MIN.	AT 21°C	MAX.	
3	174	210	285	160
6	170	210	290	160

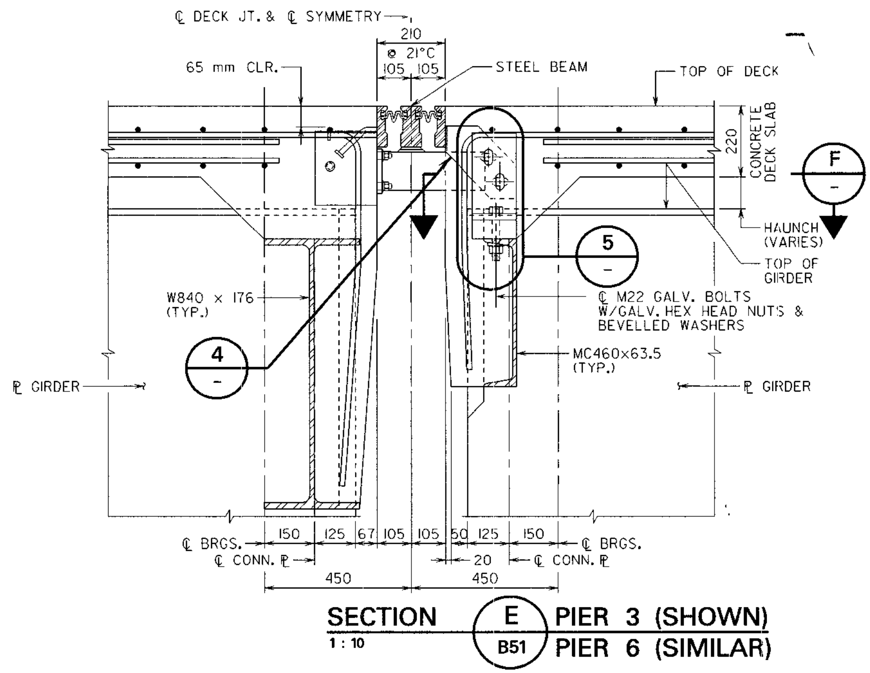


Figure VI.B.10 Expansion Joint Details at piers 3 and 6

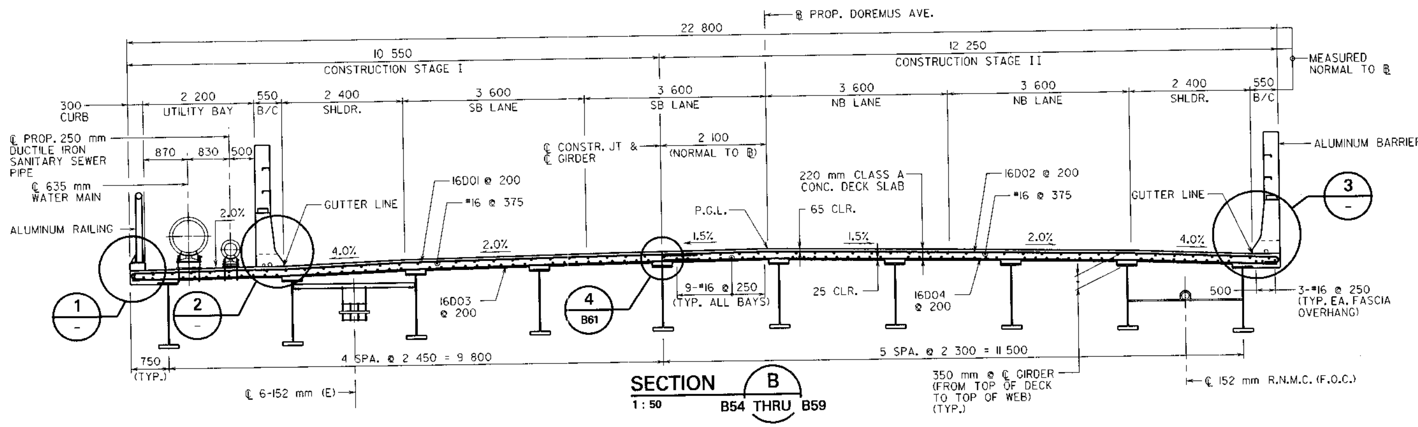


Figure VI.B.11 Superstructure Section

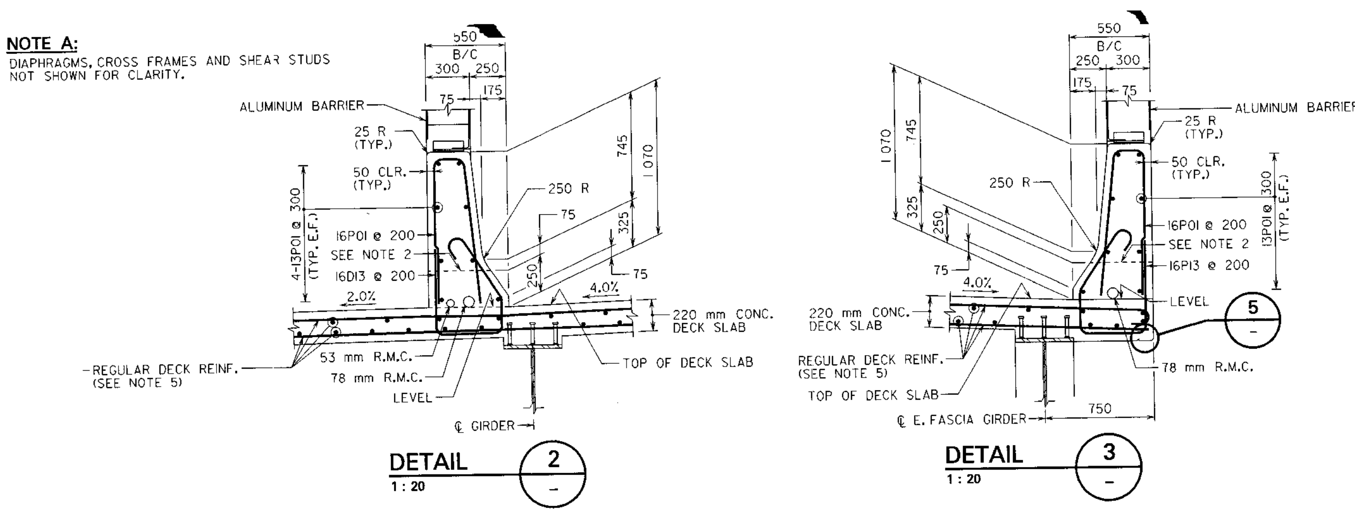


Figure VI.B.12 Barriers Section

VI.C: Substructure Details

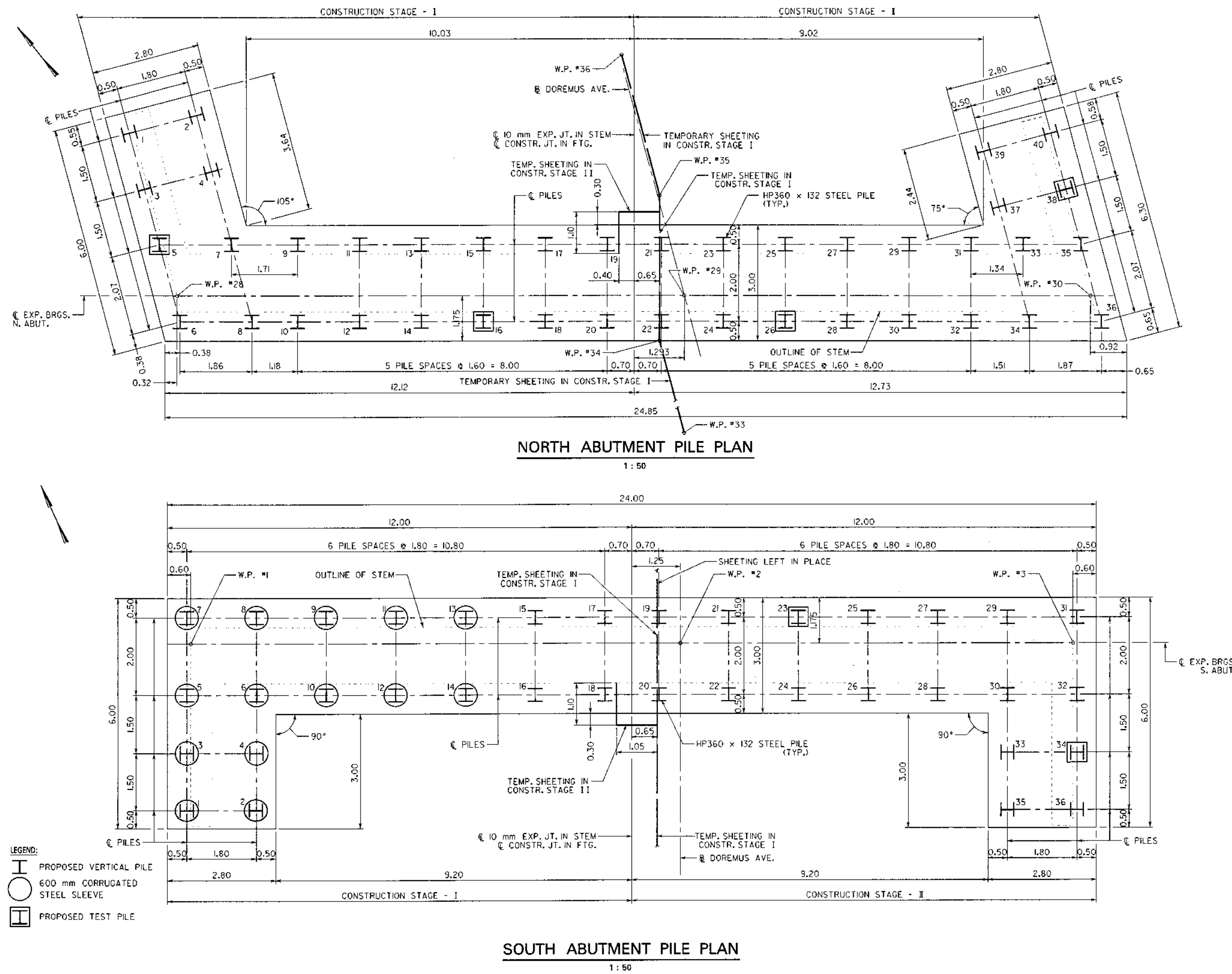


Figure VI.C.1 North and South Abutment Pile Plan

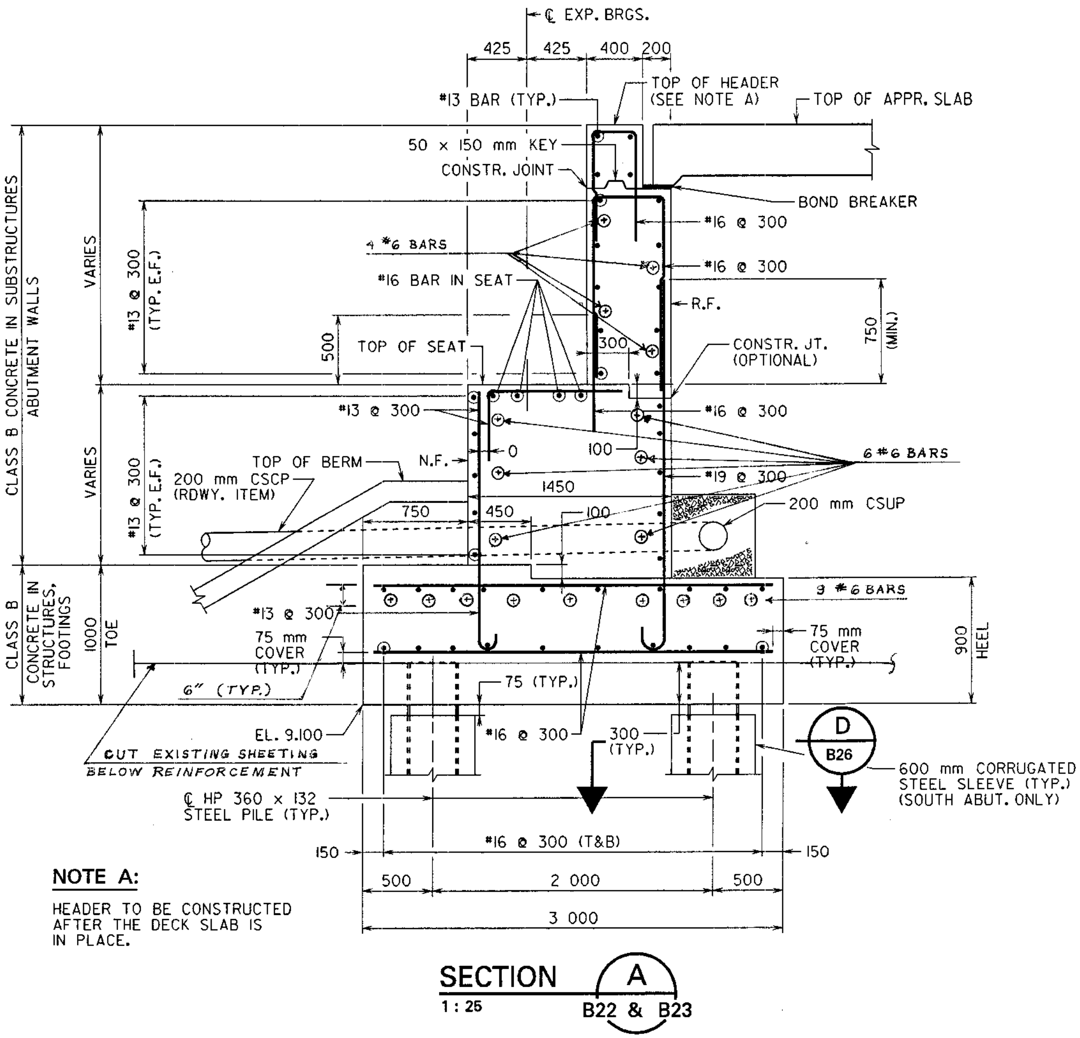
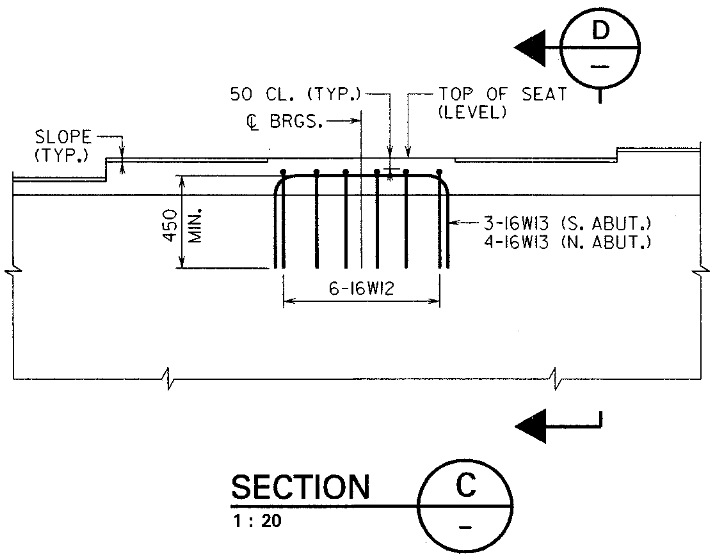


Figure VI.C.2 Typical Abutment Section



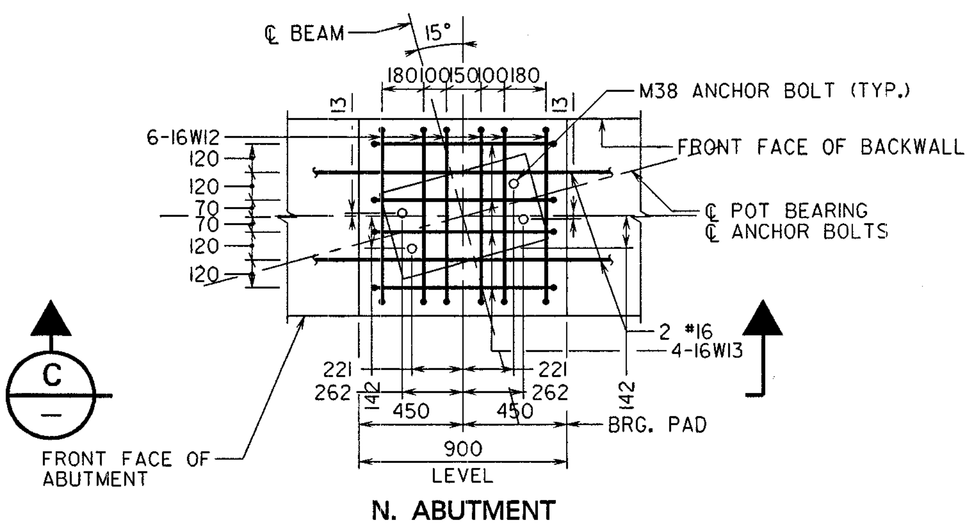


Figure VI.C.3 Bearing Pedestal Plan and Section

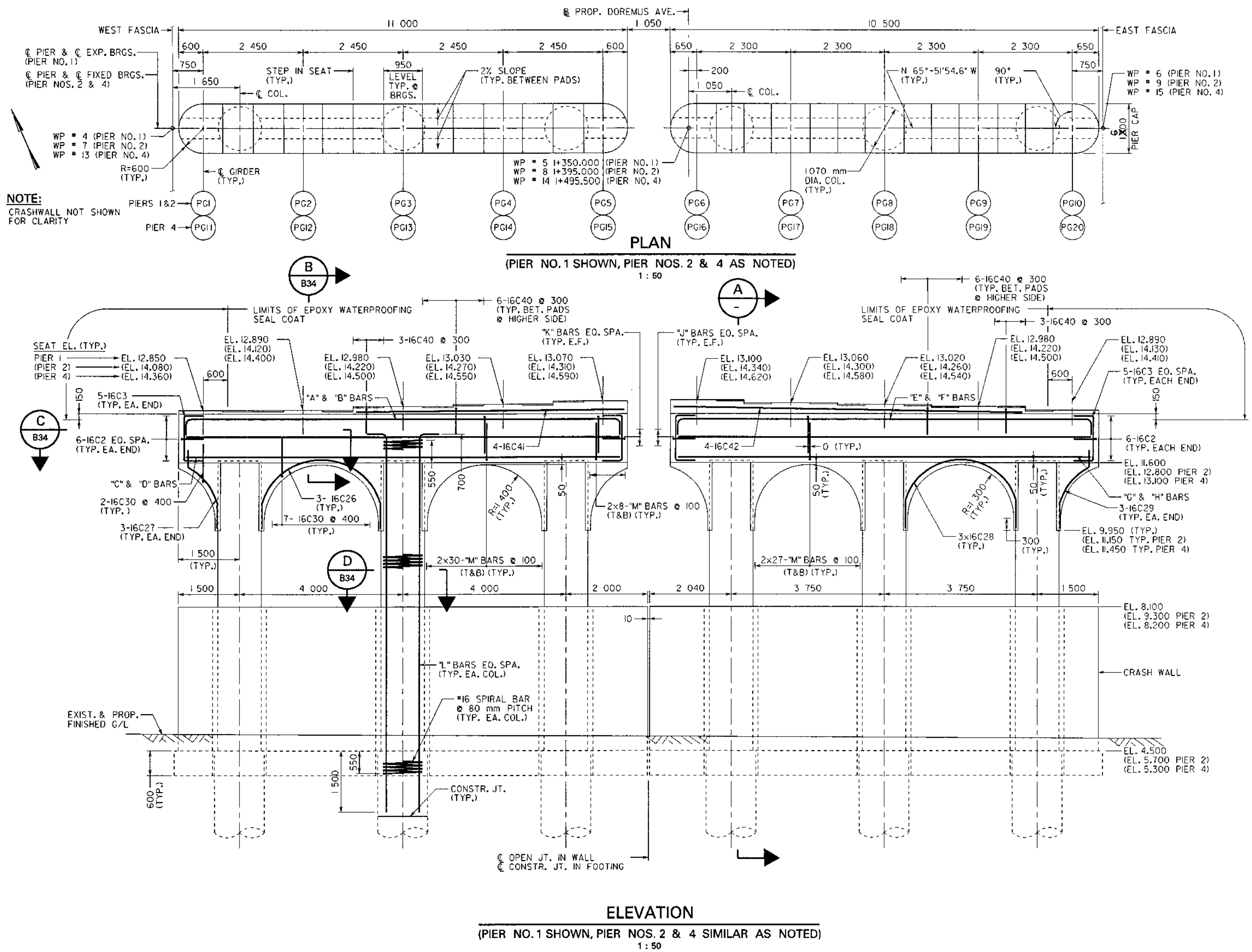


Figure VI.C.4 Elevation and Plan Pier 1, 2, and 4

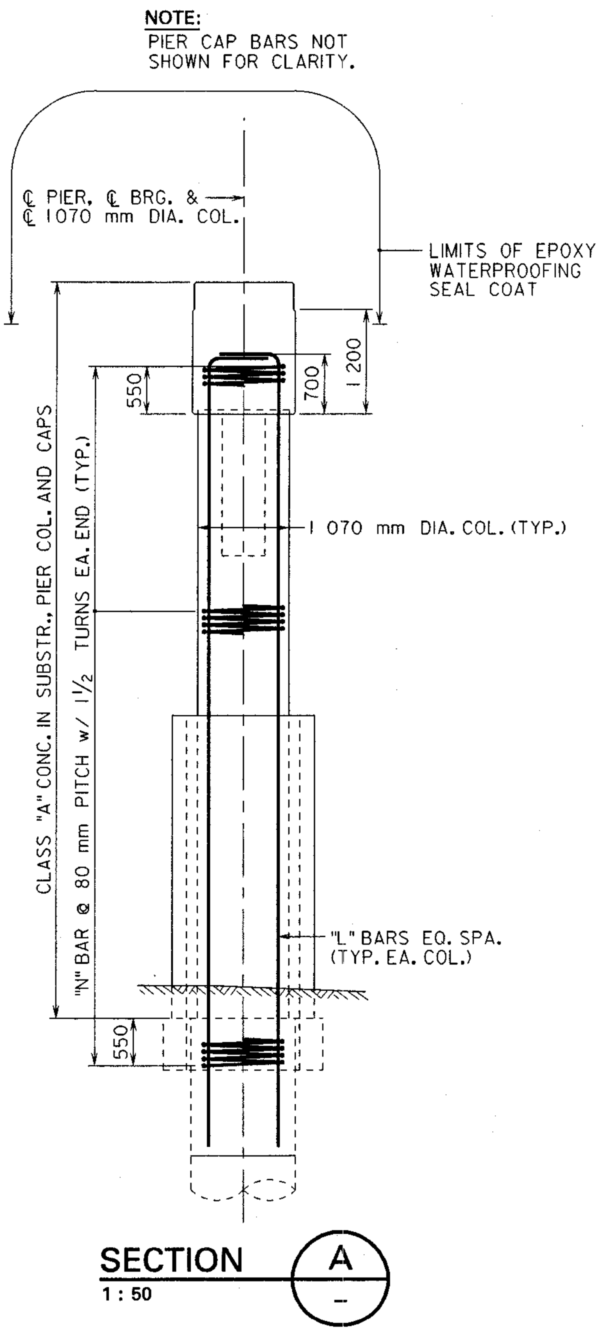


Figure VI.C.5 Section Pier 1, 2, and 4

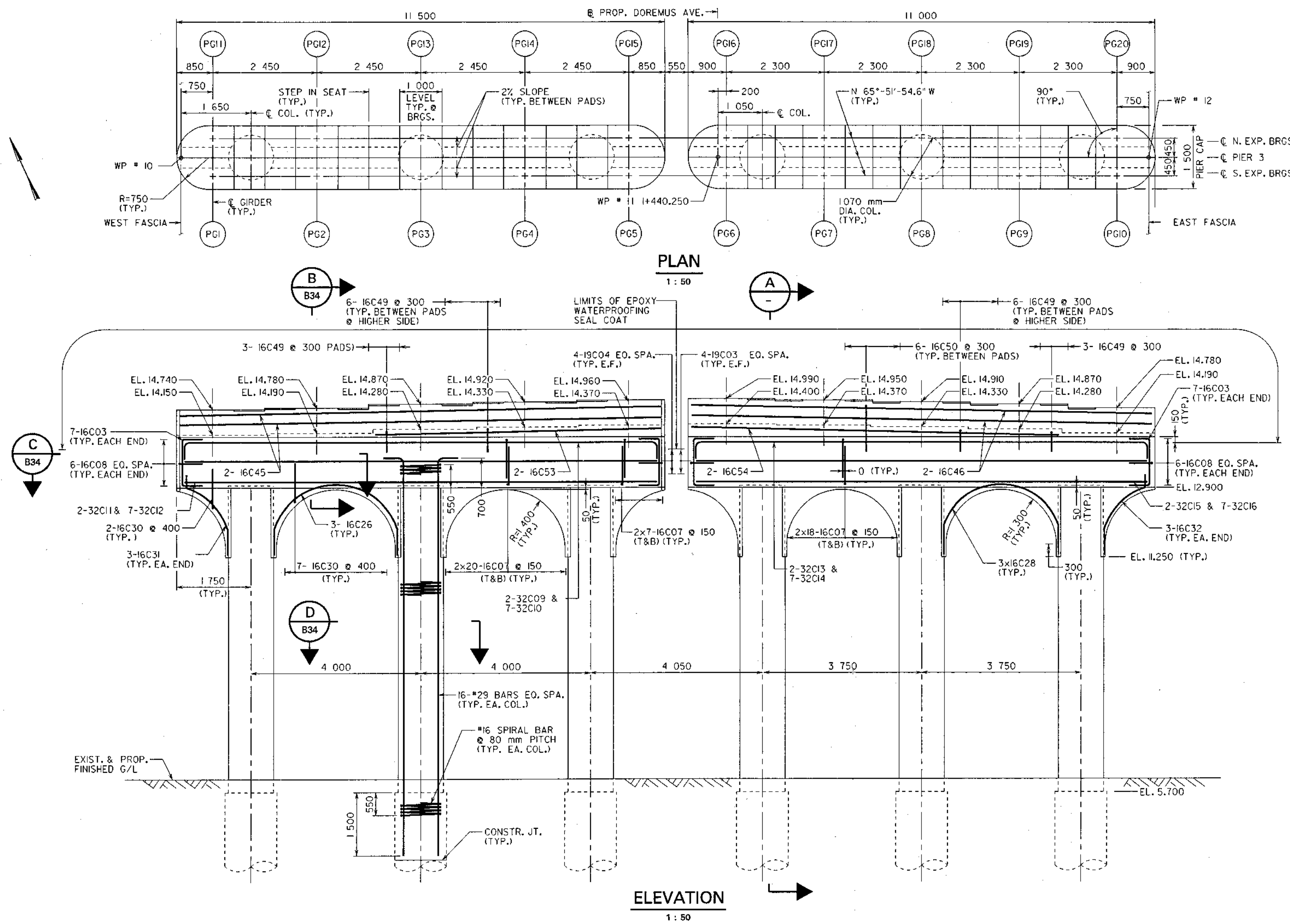


Figure VI.C.6 Elevation and Plan Pier 3

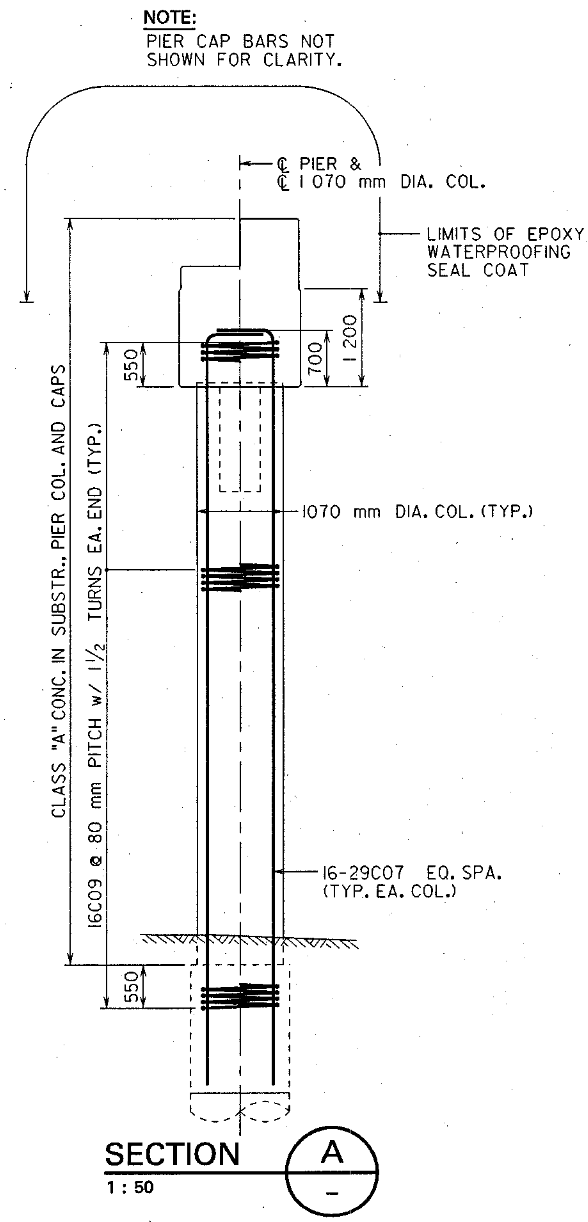


Figure VI.C.7 Section Pier 3

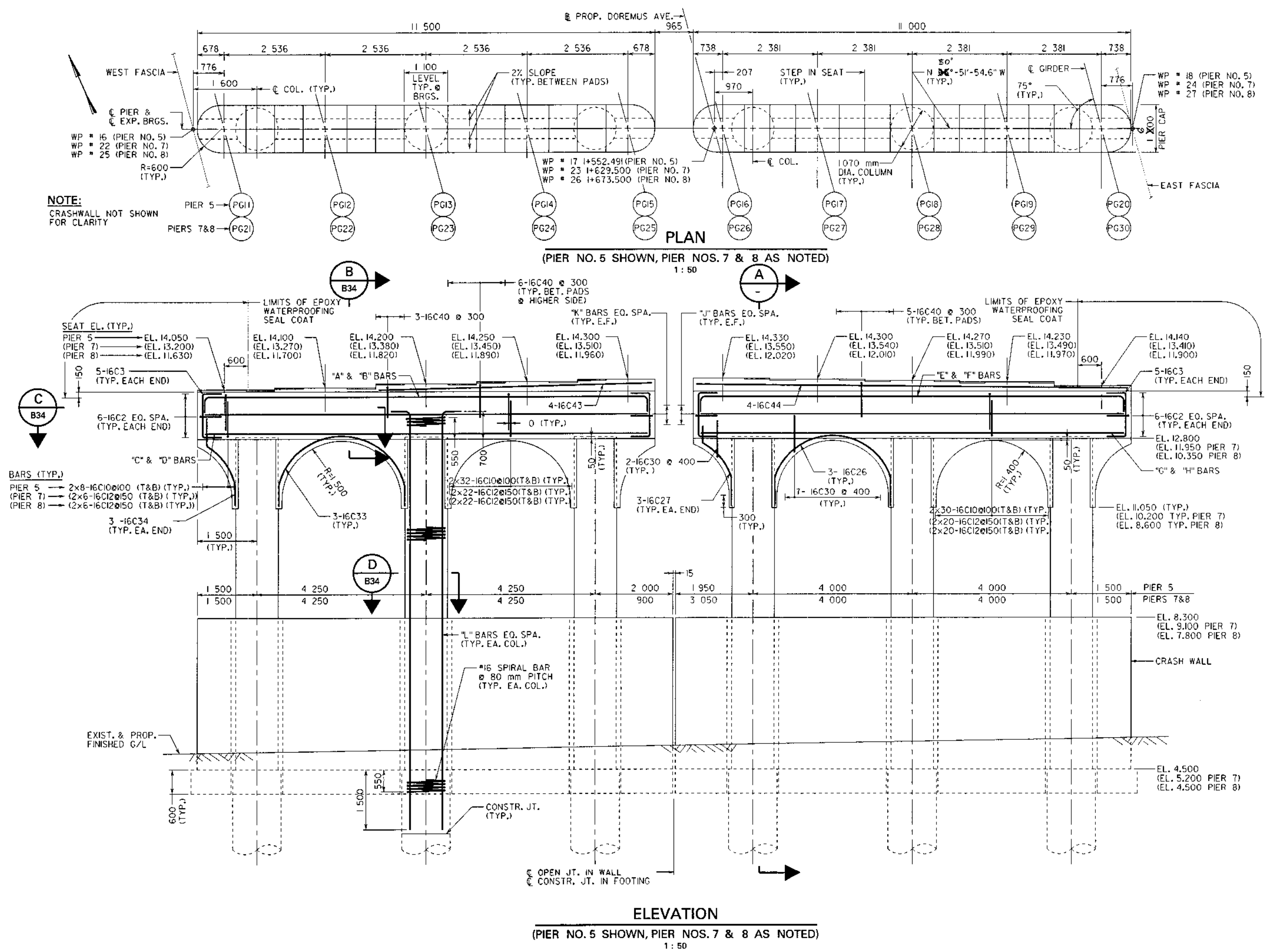


Figure VI.C.8 Elevation and Plan Pier 5, 7, and 8

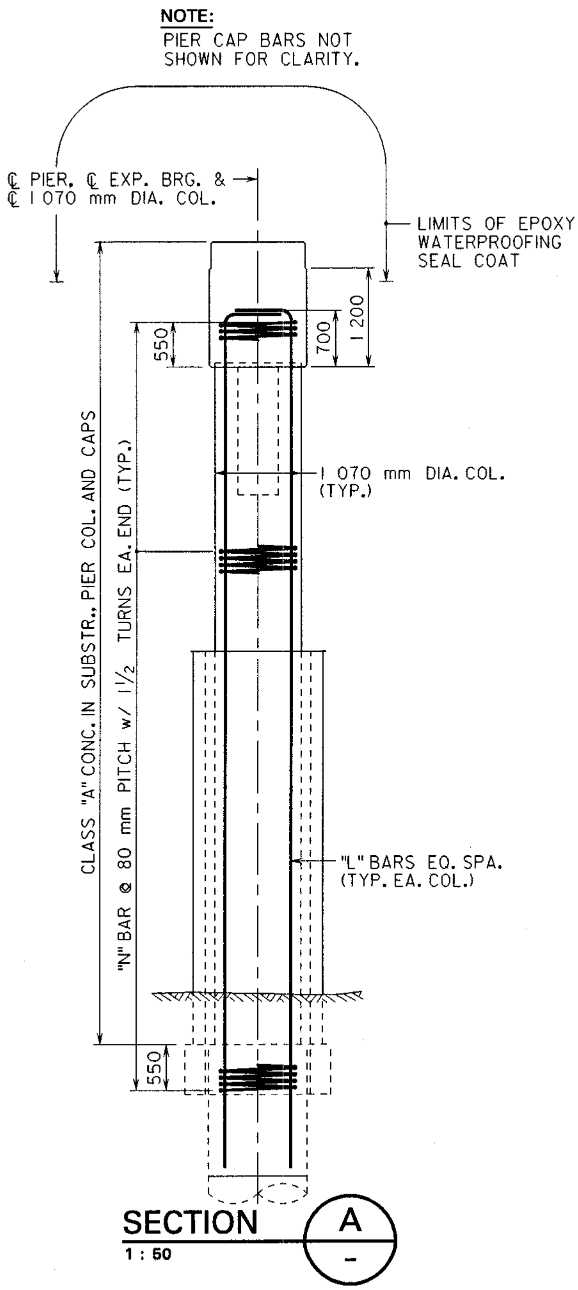


Figure VI.C.9 Section Pier 5, 7, and 8

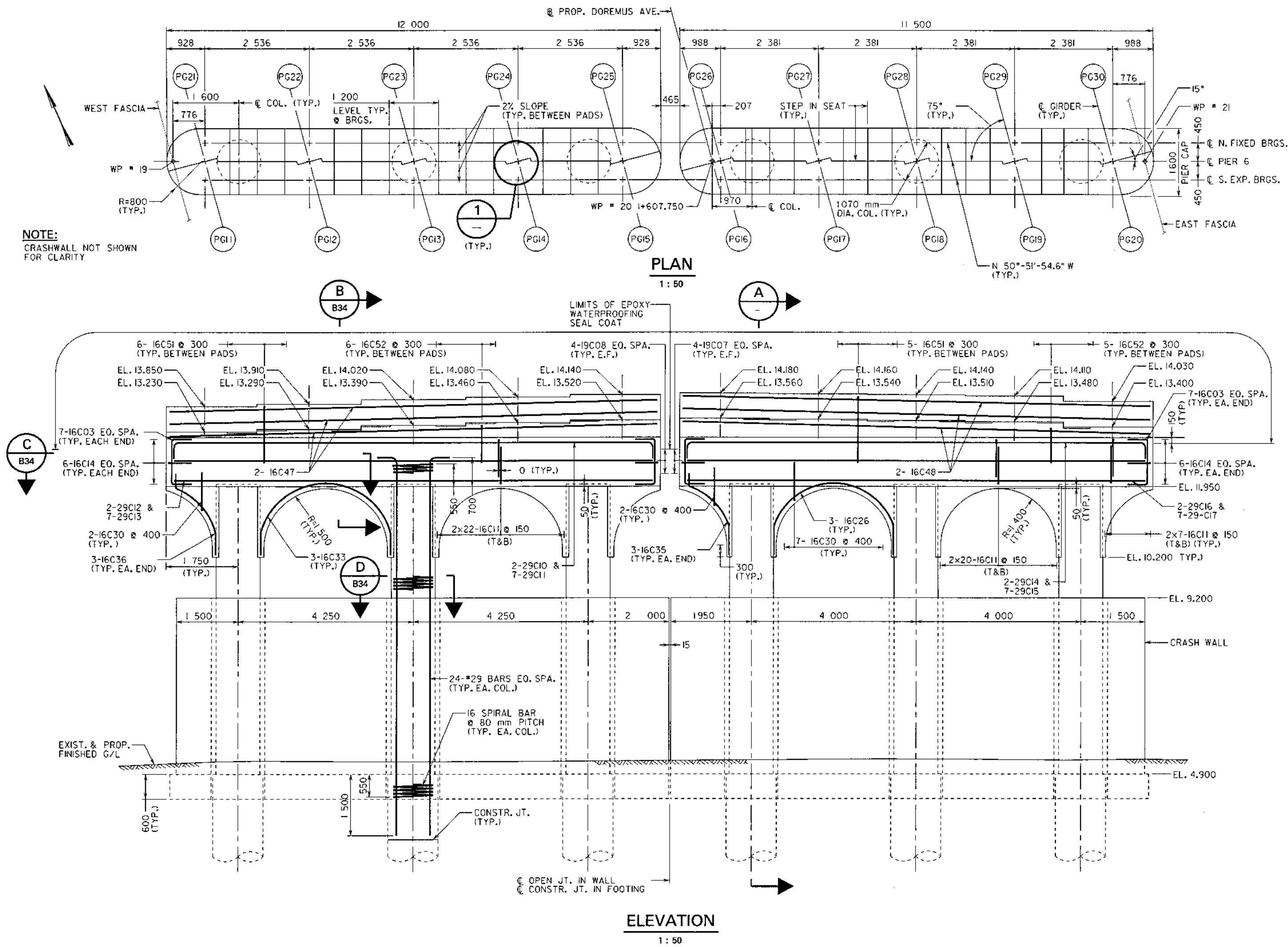


Figure VI.C.10 Elevation and Plan Pier 6

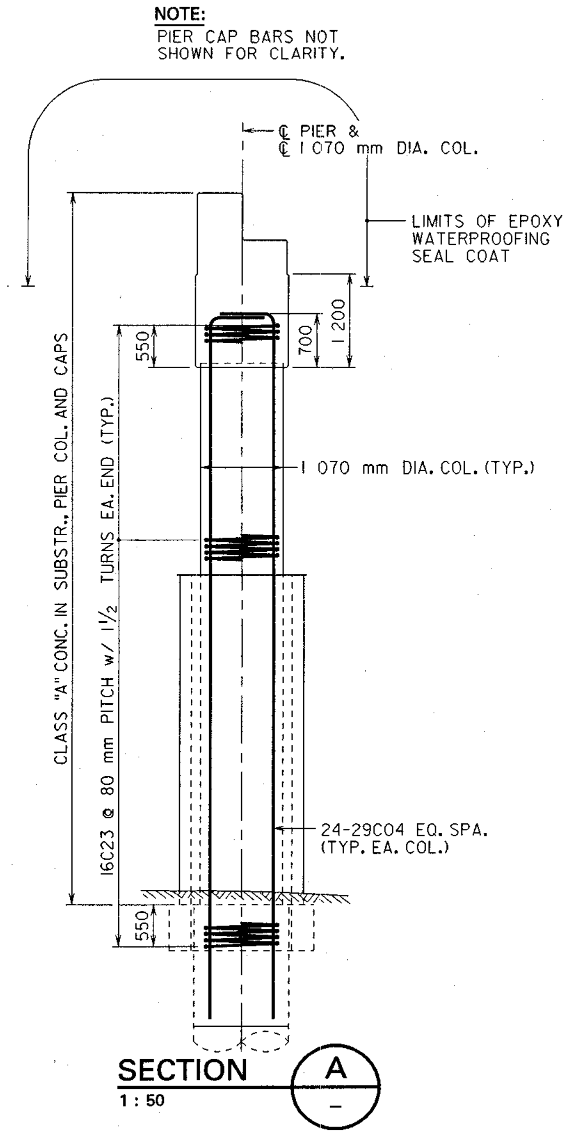


Figure VI.C.11 Section Pier 6

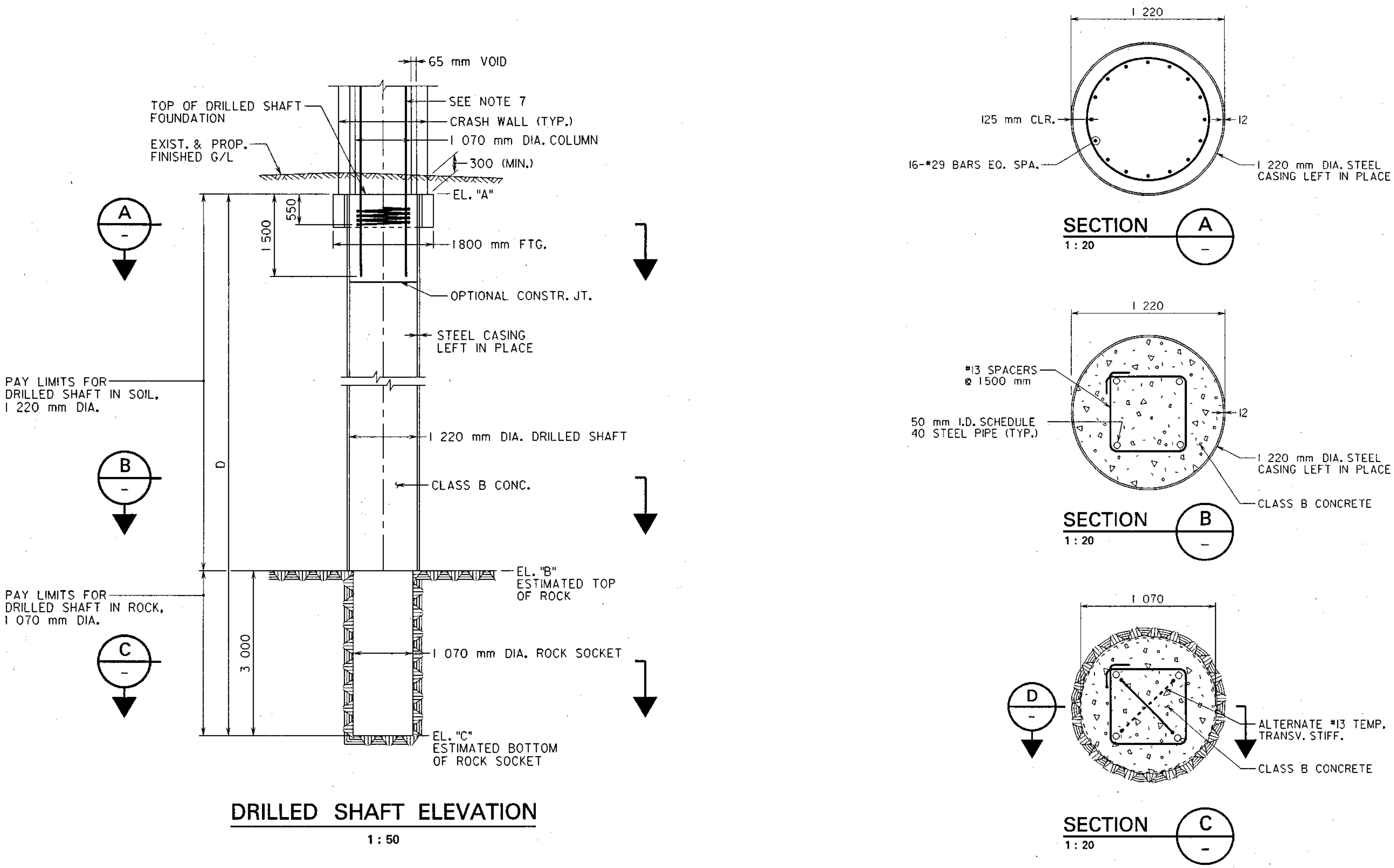


Figure VI.C.12 Drilled Shaft Elevation and Sections

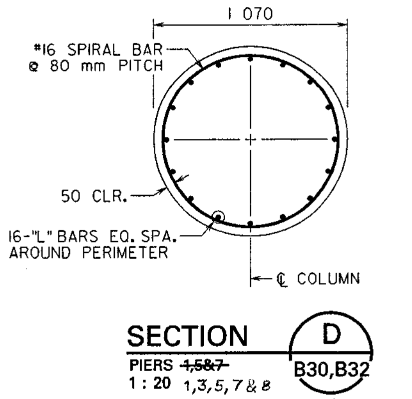


Figure VI.C.13 Column Section Pier 1, 3, 5, 7, and 8

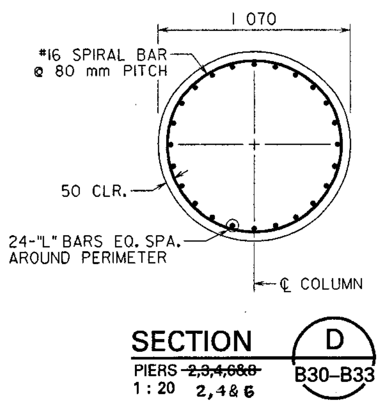
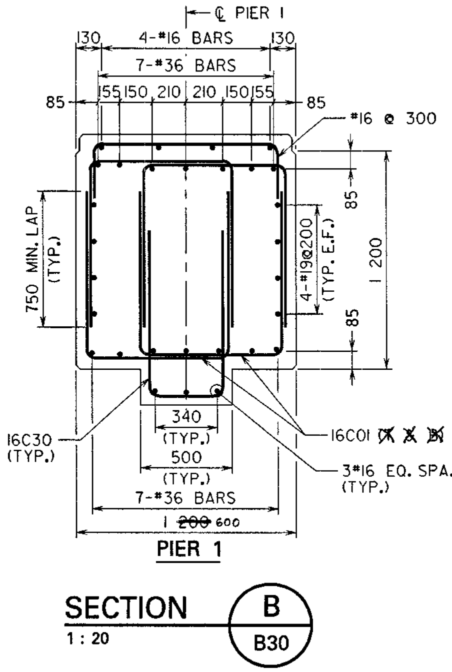
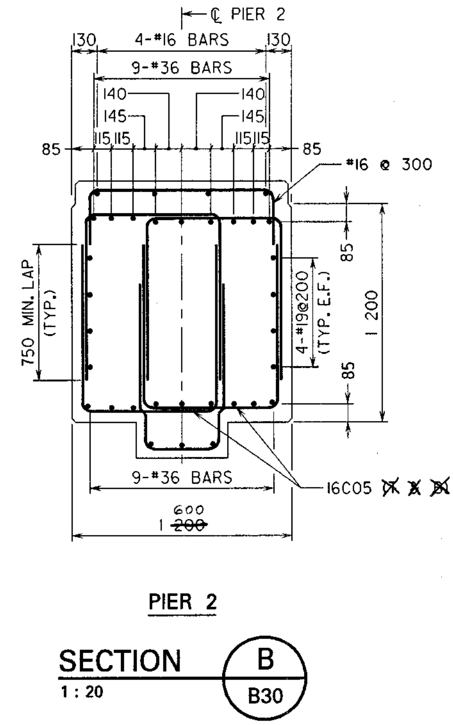
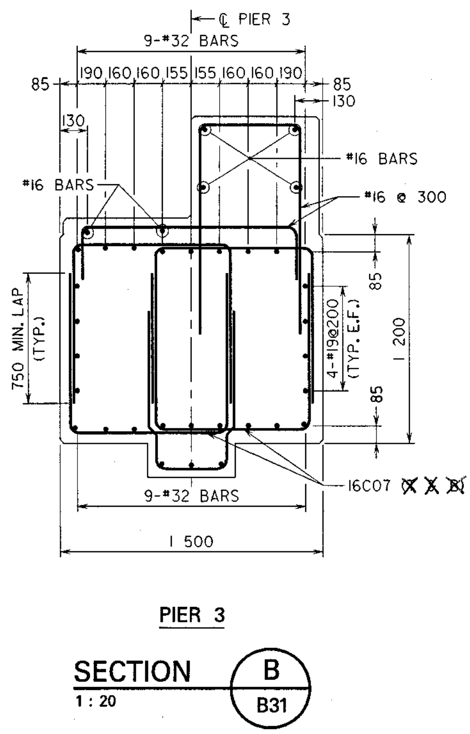


Figure VI.C.14 Column Section Pier 2, 4, and 6







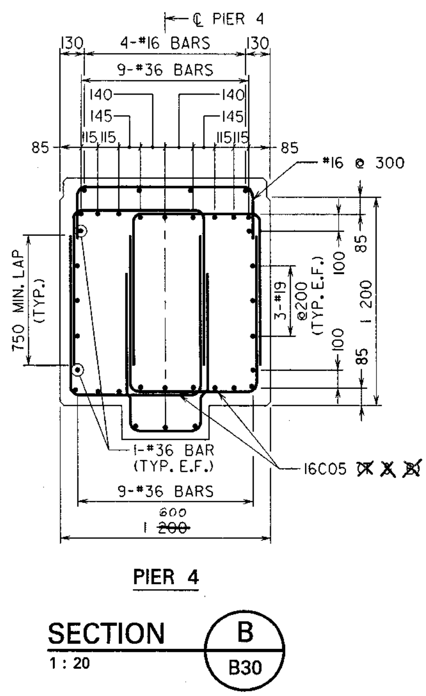


Figure VI.C.15 Bent Cap Section Piers 1, 2, 3, and 4

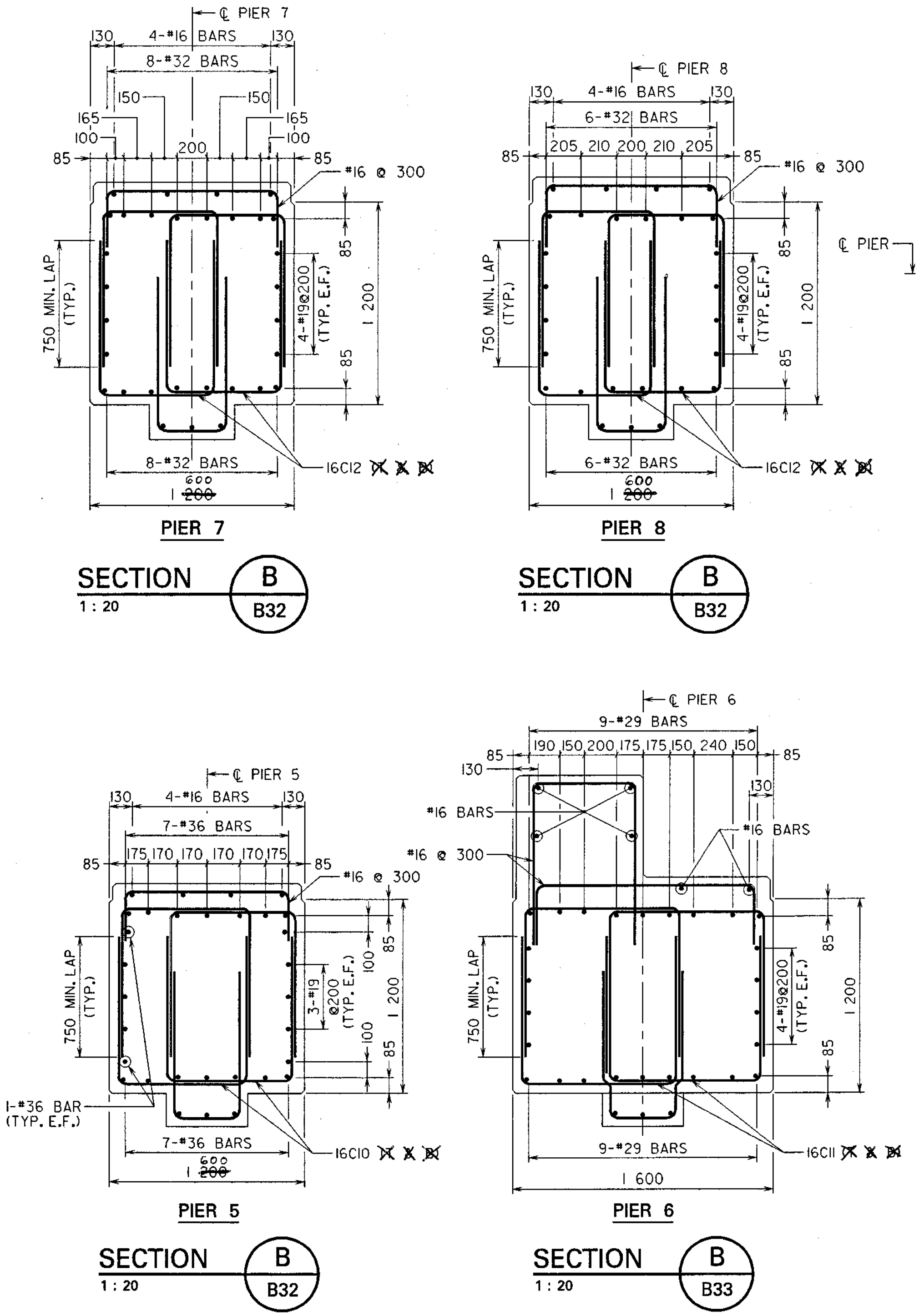


Figure VI.C.16 Bent Cap Section Piers 5, 6, 7, and 8

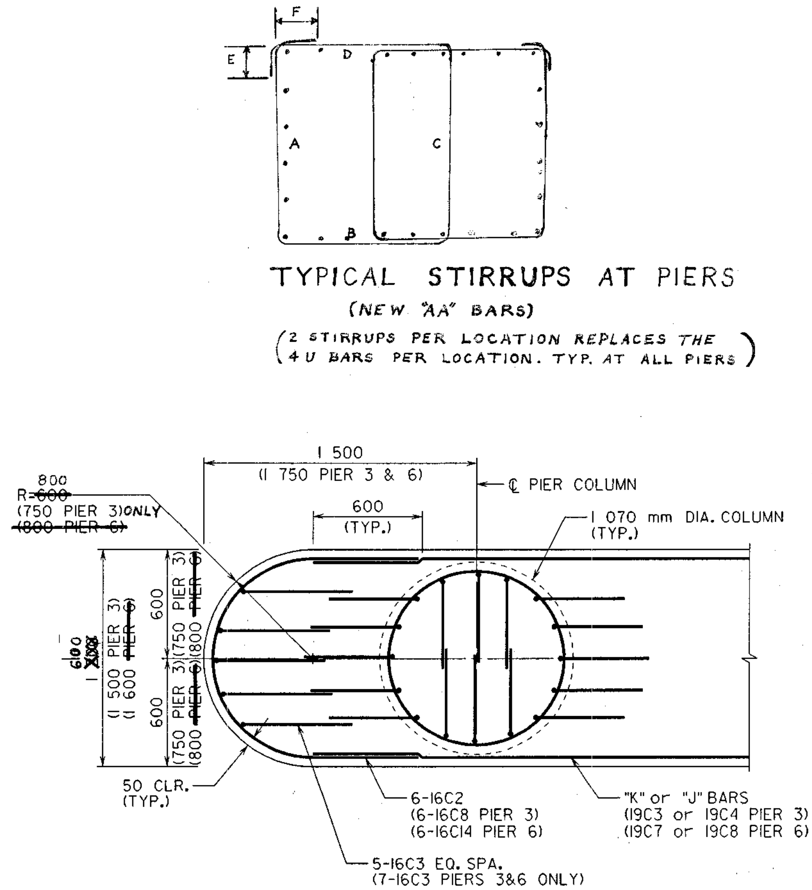


Figure VI.C.17 Typical Bent Cap Details

APPENDIX VII: EXAMPLE ON DESIGN OF SIX-SPAN CONCRETE BRIDGE

VII.A: Pier 3 Analysis

CSI –SAP version 12 is used to model the bent at Pier 3 shown in Figure VII.C.4 of Appendix VII.C. Four cross sections are considered to obtain gross and effective properties. The moment curvature relationship is also derived and plotted. For the outer column, the sections include:

CS-3 Dimension A referred to as (CS3A)

CS-4 Dimension B referred to as (CS4B)

CS-13 Dimension B referred to as (CS13B)

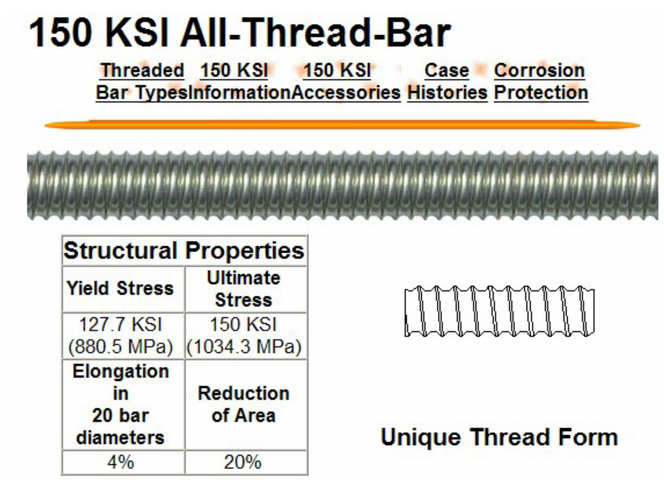
These sections are detailed in Figure VII.C.12 of Appendix VII.C showing the precast segment schedule. For the inner column, the section is prismatic as shown in Figure VII.C.8 and referred to as CS1. For the outer column, the section is not prismatic and shown in figure VII.C.11. Due to the lack of symmetry in geometry, prestressing or H.S bars reinforcement the moment curvature analysis needed to be run about:

1) Weak Axis
2) Strong Axis 1 corresponding to the smaller flexural moment in the strong direction
3) Strong Axis 2 corresponding to the larger flexural moment in the strong direction

In addition, as HS bars are not developed in top of the columns, the sections were run with prestressing shown only in one case and a second case that included high strength bars.

Materials:

The High Strength bars properties were reproduced from DYWIDAG Systems Information Data as shown in tables VII.A1 and VII.A2. Figure VII.A.1 shows the stress-strain curve used in modeling the cross section.



150 KSI All-Thread-Bar

Threaded Bar Types | 150 KSI Information | 150 KSI Accessories | Case Histories | Corrosion Protection

Structural Properties	
Yield Stress	Ultimate Stress
127.7 KSI (880.5 MPa)	150 KSI (1034.3 MPa)
Elongation in 20 bar diameters	Reduction of Area
4%	20%

Unique Thread Form

Table VII.A.1High Strength Bar Properties Data 1

R71 150 KSI All-Thread-Bar - ASTM A722

Nominal Bar Diameter	Minimum Net Area Thru Threads	Minimum Ultimate Strength	Minimum Yield Strength	Nominal Weight	Approx. Thread Major Dia.	Part Number
1" (25 mm)	0.85 in^2 (549 mm^2)	128 kips (567 kN)	102 kips (454 kN)	3.09 lbs./ft. (4.6 Kg/M)	1-1/8" (28.6 mm)	R71-08
1-1/4" (32 mm)	1.25 in^2 (807 mm^2)	188 kips (834 kN)	150 kips (667 kN)	4.51 lbs./ft. (6.71 Kg/M)	1-7/16" (36.5 mm)	R71-10

Notes:

Effective cross sectional areas shown are as required by ASTM A 722-98. Actual areas may exceed these values.
ACI 355.1R section 3.2.5.1 indicates an ultimate strength in shear has a range of .6 to .7 of the ultimate tensile strength. Designers should provide adequate safety factors for safe shear strengths based on the condition of use.
Per PTI Recommendations for Prestressed Rock and Soil Anchors section 6.6, anchors should be designed so that:
- The design load is not more than 60% of the specified minimum tensile strength of the prestressing steel.
- The lock-off load should not exceed 70% of the specified minimum tensile strength of the prestressing steel.
- The maximum test load should not exceed 80% of the specified minimum tensile strength of the prestressing steel.
** The 3" diameter bar is not covered under ASTM A722.

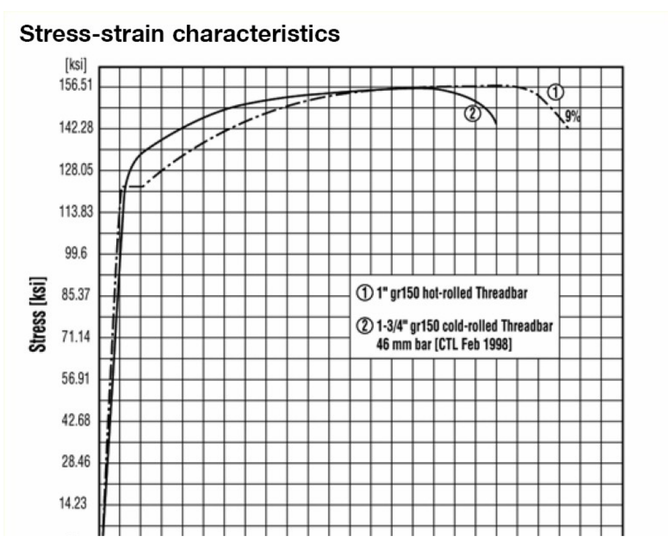


Figure VII.A.1 Stress-strain Curve for High Strength Bar

Prestressing Steel Modeling

Following LRFD-GS 8.4.3, prestressing steel is modeled with an idealized nonlinear stress-strain model. Figure VII.A.2 shows an idealized stress-strain model for 7-wire low-relaxation prestressing strand.

Essentially elastic prestress steel strain, $\varepsilon_{ps,EE}$, for 270 Ksi strands is taken as:

$\varepsilon_{ps,EE}$ = 0.0086

Reduced ultimate prestress steel strain is taken as:

$\varepsilon^{R}_{ps,u}$ = 0.03

The stress, f_{ps}, in the prestressing steel is taken as:

$$f_{ps} = 28{,}500\varepsilon_{ps} \text{ when } \varepsilon_{ps} \leq 0.0086$$

$$f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \text{ when } \varepsilon_{ps} > 0.0086$$

where:

ε_{ps} = strain in prestressing steel

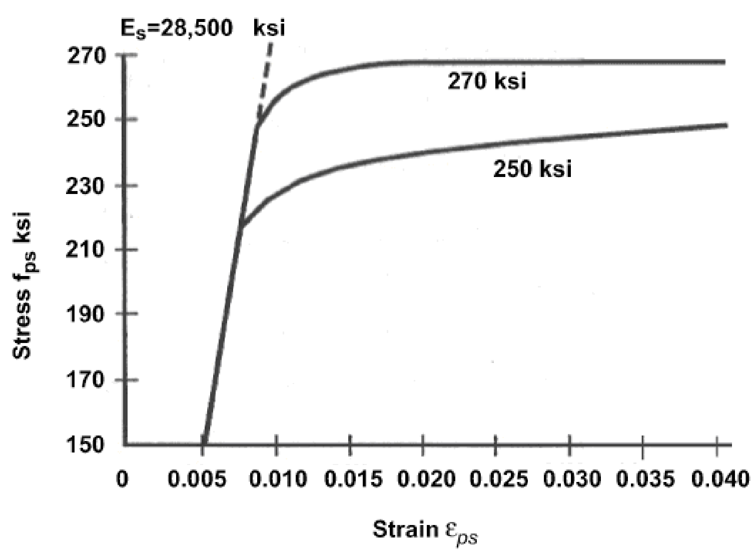


Figure VII.A.2 Prestressing Strand Stress-Strain Model.

Concrete:

CSI SAP Mander's unconfined concrete model is used as the nominal moment of the cross section is of interest.

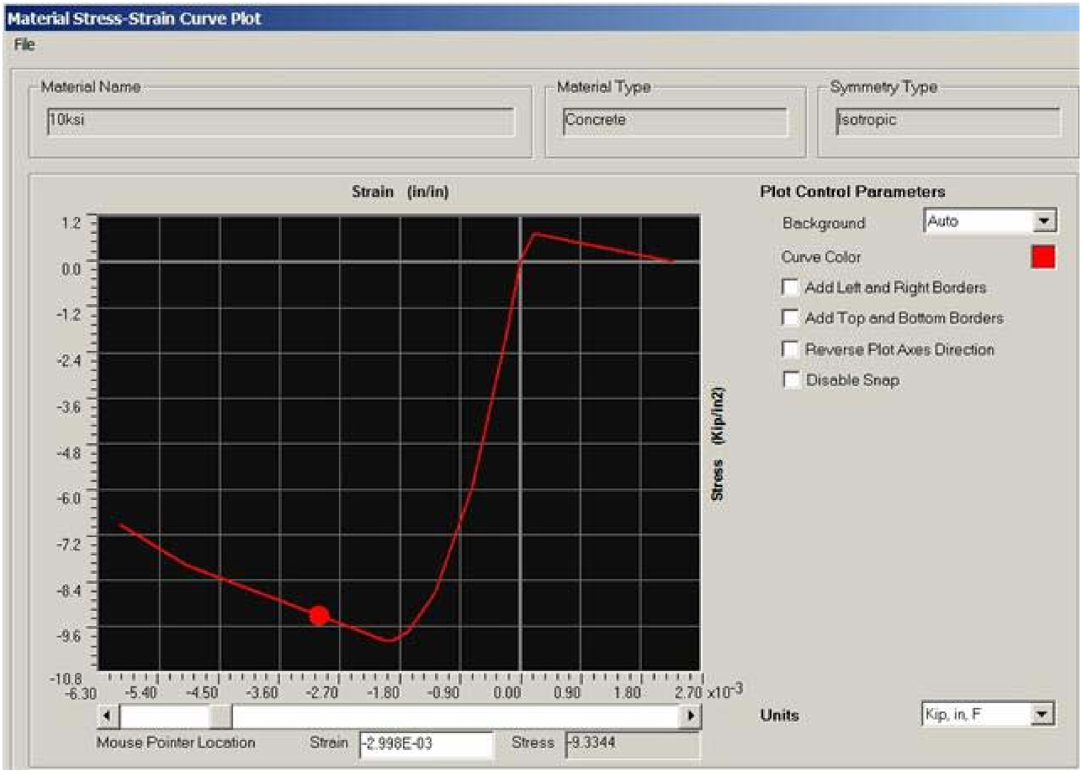


Figure VII.A.3 Concrete Stress-Strain Model

The results for cross-section properties, weak axis bending, strong axis 1 and 2 bending are shown in Figures VII.A.4- VII.A.35 and are listed below. Figure, VII.A.36- VII.A.37 show properties of the bent cap.

Figure VII.A.4: CS1-PS CSI-SAP Properties

Figure VII.A.5: CS1-PS-HS CSI SAP Properties

Figure VII.A.6: CS1-PS Weak Axis Moment Curvature Curve

Figure VII.A.7: CS1-PS Strong Axis 1 Moment Curvature Curve

Figure VII.A.8: CS1-PS Strong Axis 2 Moment Curvature Curve

Figure VII.A.9: CS1-PS-HS Weak Axis Moment Curvature Curve

Figure VII.A.10: CS1-PS-HS Strong Axis 1 Moment Curvature Curve

Figure VII.A.11: CS1-PS-HS Strong Axis 2 Moment Curvature Curve

Figure VII.A.12: CS3A-PS CSI-SAP Properties

Figure VII.A.13: CS3A-PS-HS CSI SAP Properties

Tables VII.A.3 and VII.A.4 summarize the results of the column sections pertaining to the nominal flexural capacity and the effective flexural inertia.

Table VII.A.2 Nominal Flexural Capacity of Column Section (K-in)

Flexural Capacity	Weak Axis	Strong Axis 1	Strong Axis 2
CS1-PS	86068	108993	116008
CS1-PS-HS	96471	119132	132022
CS3A-PS	85216	103640	112819
CS3A-PS-HS	96081	113642	121370
CS4B-PS	91436	157523	172315
CS4B-PS-HS	103163	174783	189088
CS13B-PS	95600	221000	243038
CS13B-PS-HS	107980	248263	269733

Table VII.A.3 Effective Flexural Inertia of Column Section (in^4)

Flexural Inertia	Weak Axis	Strong Axis 1	Strong Axis 2
CS1-PS	95980	158382	177154
CS1-PS-HS	144389	169057	271678
CS3A-PS	92501	143682	155991
CS3A-PS-HS	140882	155007	165912
CS4B-PS	101099	331580	358443
CS4B-PS-HS	153411	361680	389982
CS13B-PS	106242	654292	716277
CS13B-PS-HS	161730	727521	784365

The results for cross section area, weak axis bending curvature, strong axis 1 and 2 bending curvature are normalized against values of properties of section CS3A. The following plots are shown:

Figure VII.A.38: Weak Axis Flexural Capacity CS3A, CS4B, and CS13B

Figure VII. A.39: Strong Axis 1 Flexural Capacity CS3A, CS4B, and CS13B

Figure VII.A.40: Strong Axis 2 Flexural Capacity CS3A, CS4B, and CS13B

Figure VII.A.41: Normalized Strong Axis Flexural Capacity

Figure VII.A.42: Normalized Weak Axis Flexural Capacity

Figure VII.A.43: Normalized X-Section Area

Figure VII.A.44: Normalized Strong Axis Flexural Inertia 1 and 2

Figure VII.A.45: Normalized Weak Axis Flexural Inertia

The normalized functions are used to derive properties used in modeling Pier 3 using CSI-SAP. Pier 3 is analyzed for Longitudinal and Transverse push in reversed directions to capture the effect of the sloping non-prismatic outer column. The results of the analysis are shown graphically in the following figures:

Figure VII.A.46: Pier 3 Longitudinal Loading

Figure VII.A.47: Pier 3 Outer Column Longitudinal Displacement

Figure VII.A.48: Pier 3 Inner Column Longitudinal Displacement

Figure VII.A.49: Pier 3 Transverse Loading Lateral 1

Figure VII.A.50: Pier 3 Transverse Displacement Lateral 1

Figure VII.A.51: Pier 3 Transverse Reactions Lateral 1

Figure VII.A.52: Pier 3 Transverse Members Shear Lateral 1

Figure VII.A.53: Pier 3 Transverse Members Moment Lateral 1

Figure VII.A.54: Pier 3 Transverse Loading Lateral 2

Figure VII.A.55: Pier 3 Transverse Displacement Lateral 2

Figure VII.A.56: Pier 3 Transverse Reactions Lateral 2

Figure VII.A.57: Pier 3 Transverse Members Shear Lateral 2

Figure VII.A.58: Pier 3 Transverse Members Moment Lateral 2

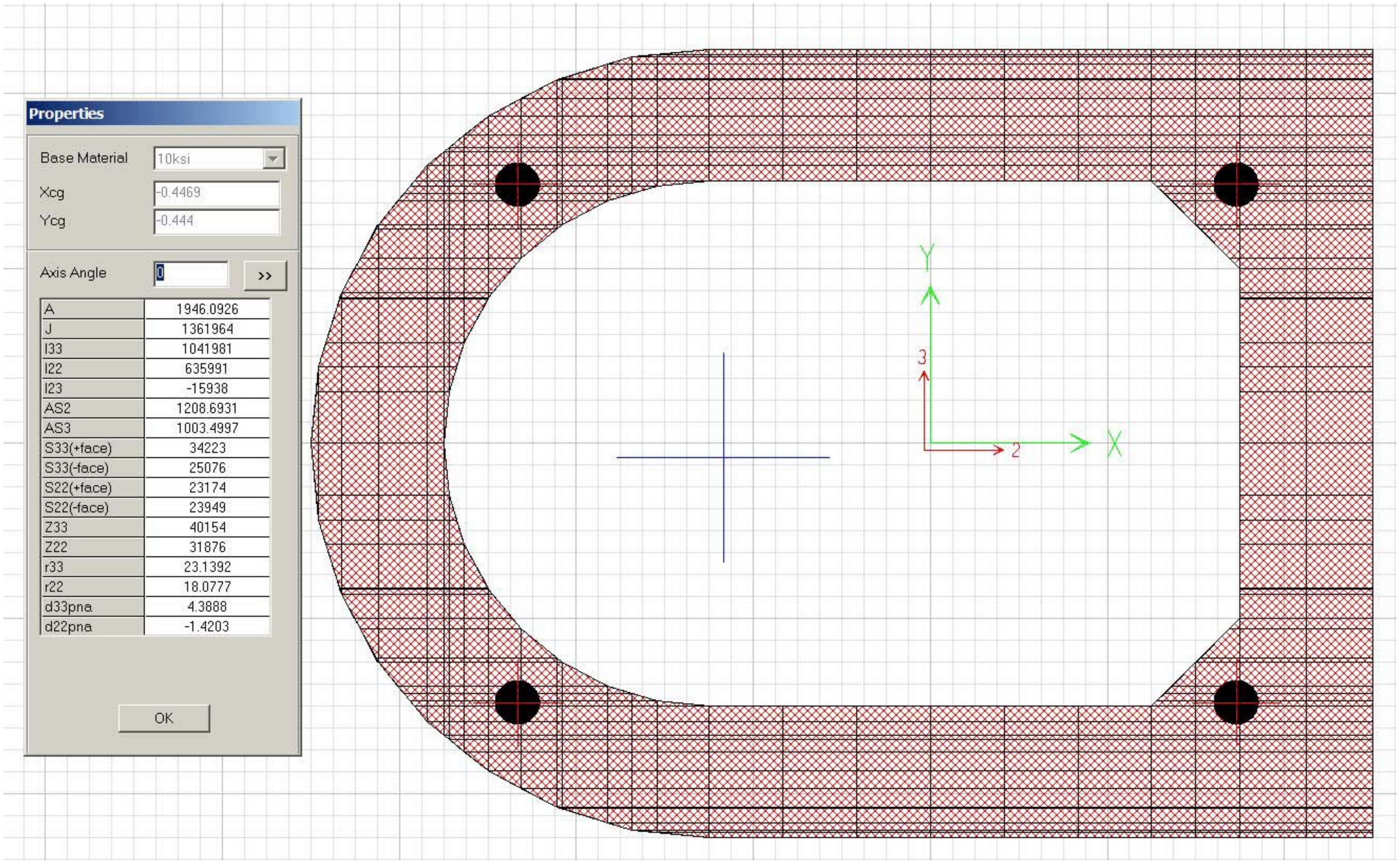


Figure VII.A.4 CS1-PS CSI-SAP Properties

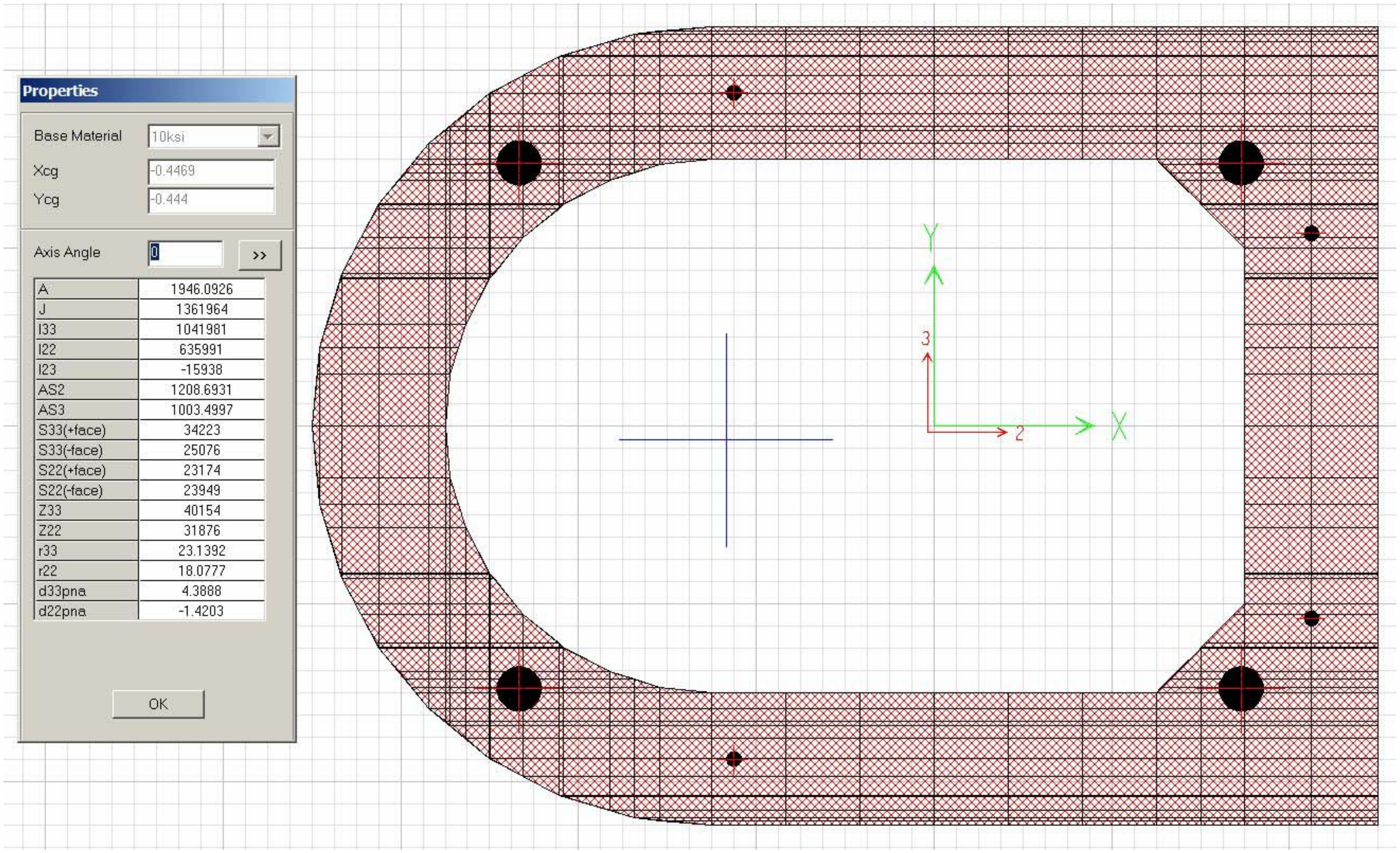


Figure VII.A.5 CS1-PS-HS CSI SAP Properties

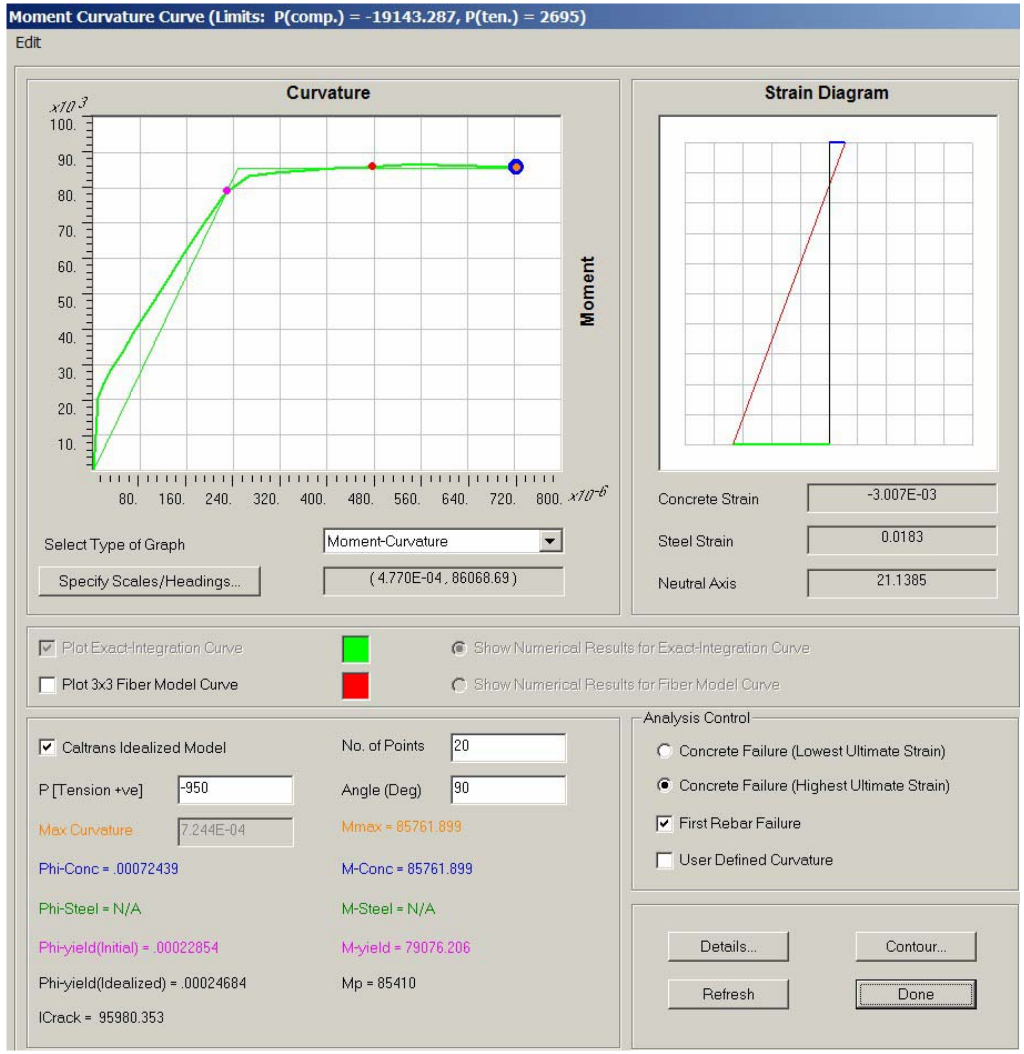


Figure VII.A.6 CS1-PS Weak Axis Moment – Curvature Curve

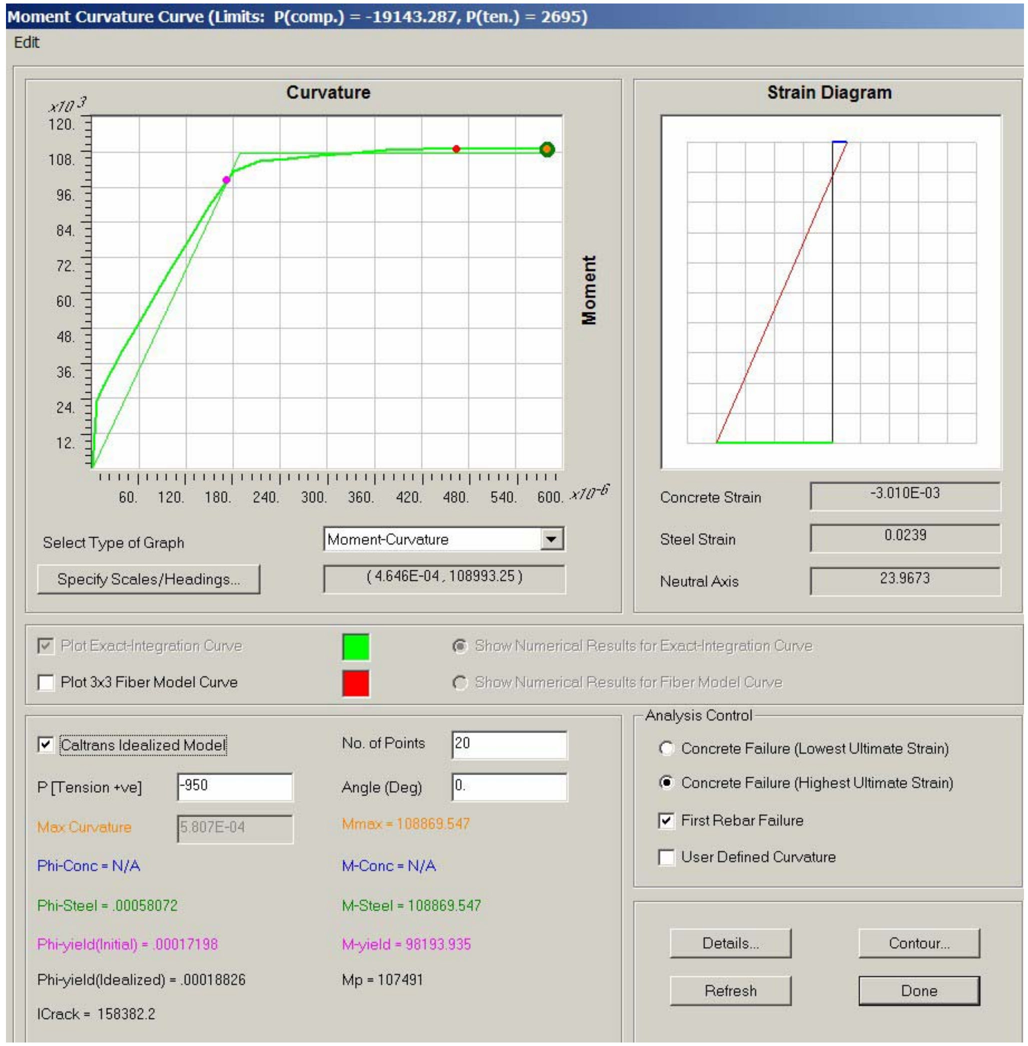


Figure VII.A.7 CS1-PS Strong Axis 1 Moment Curvature Curve

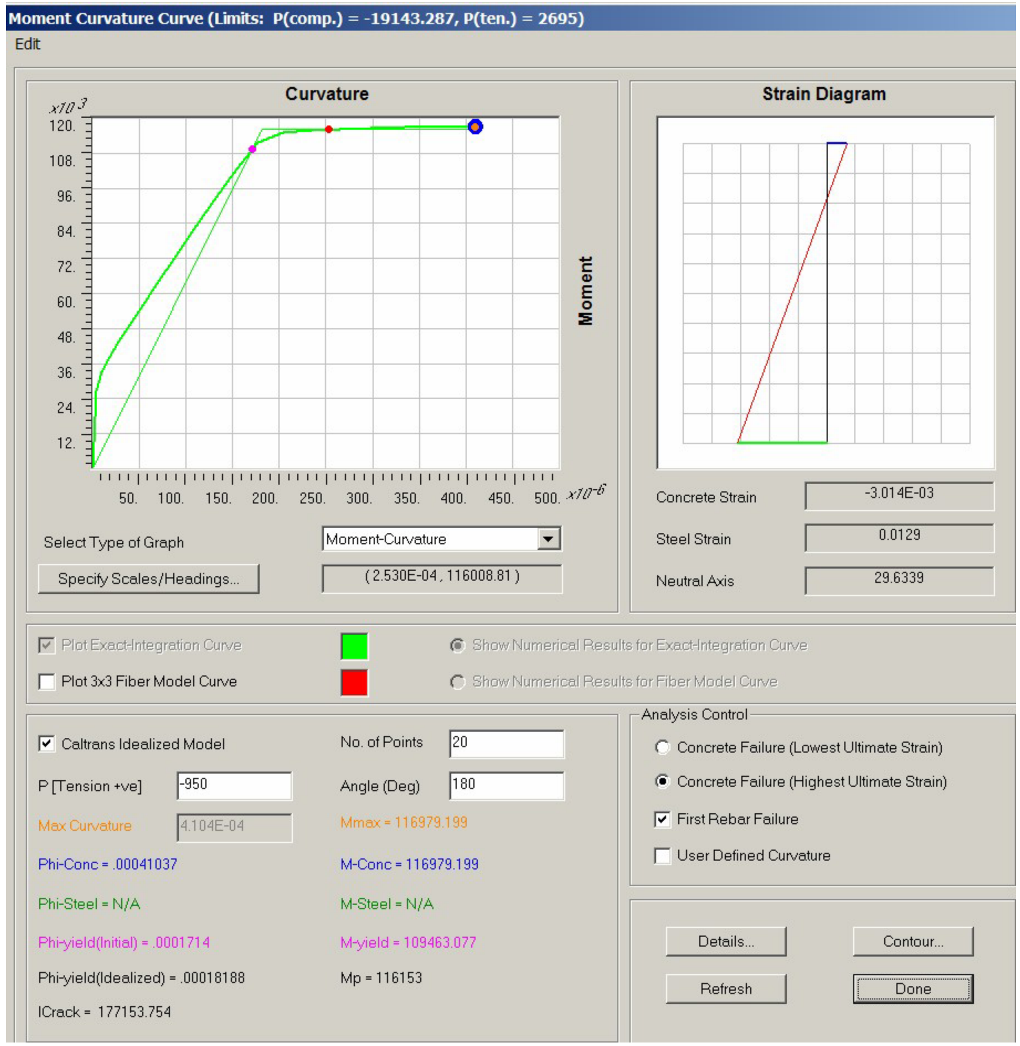


Figure VII.A.8 CS1-PS Strong Axis 2 Moment Curvature Curve

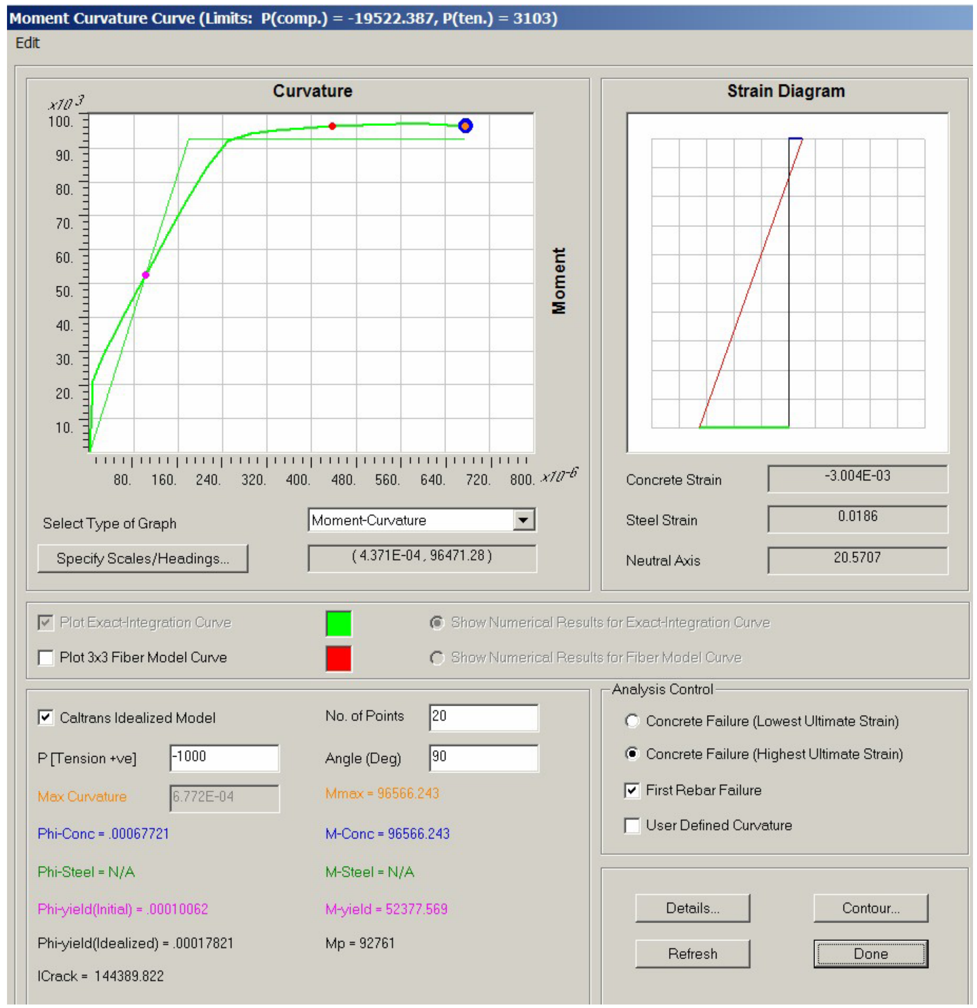


Figure VII.A.9 CS1-PS-HS Weak Axis Moment Curvature Curve

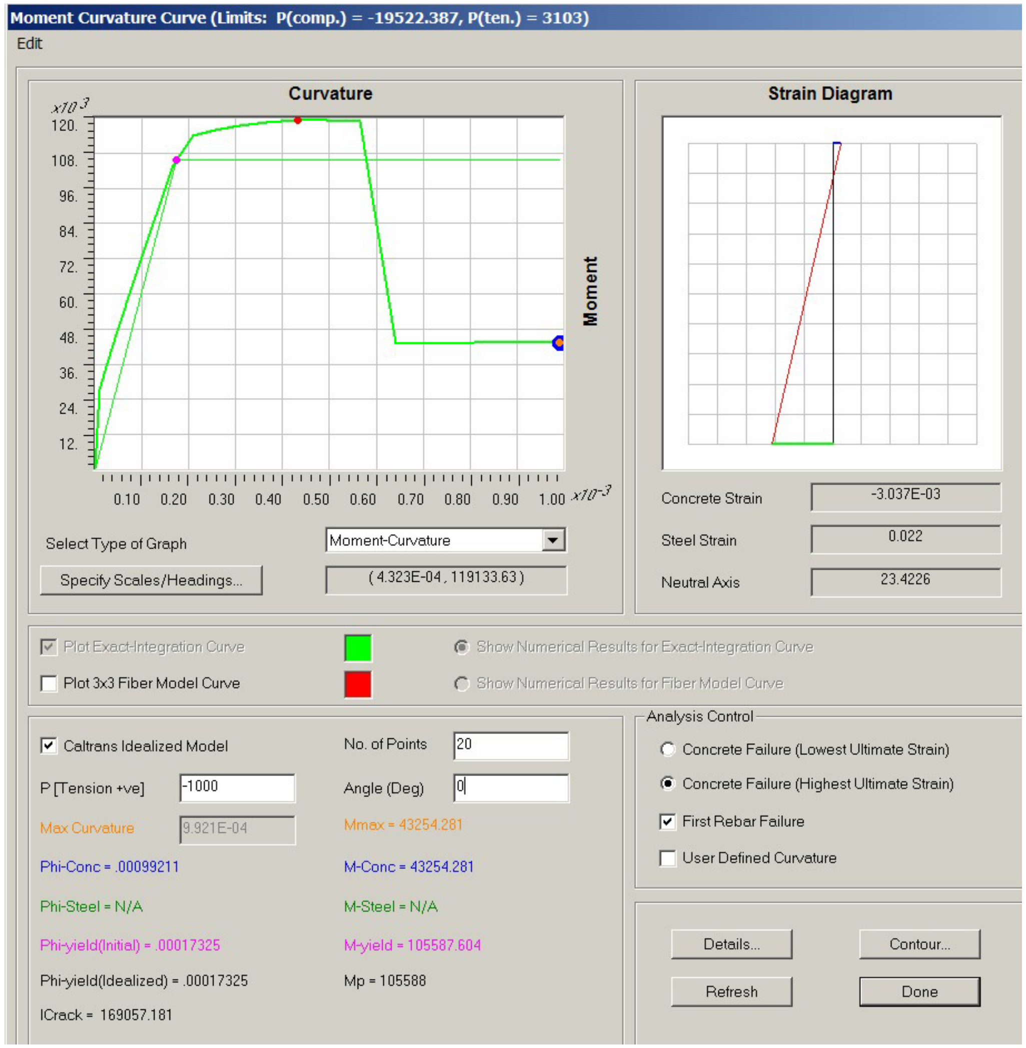


Figure VII.A.10 CS1-PS-HS Strong Axis 1 Moment Curvature Curve

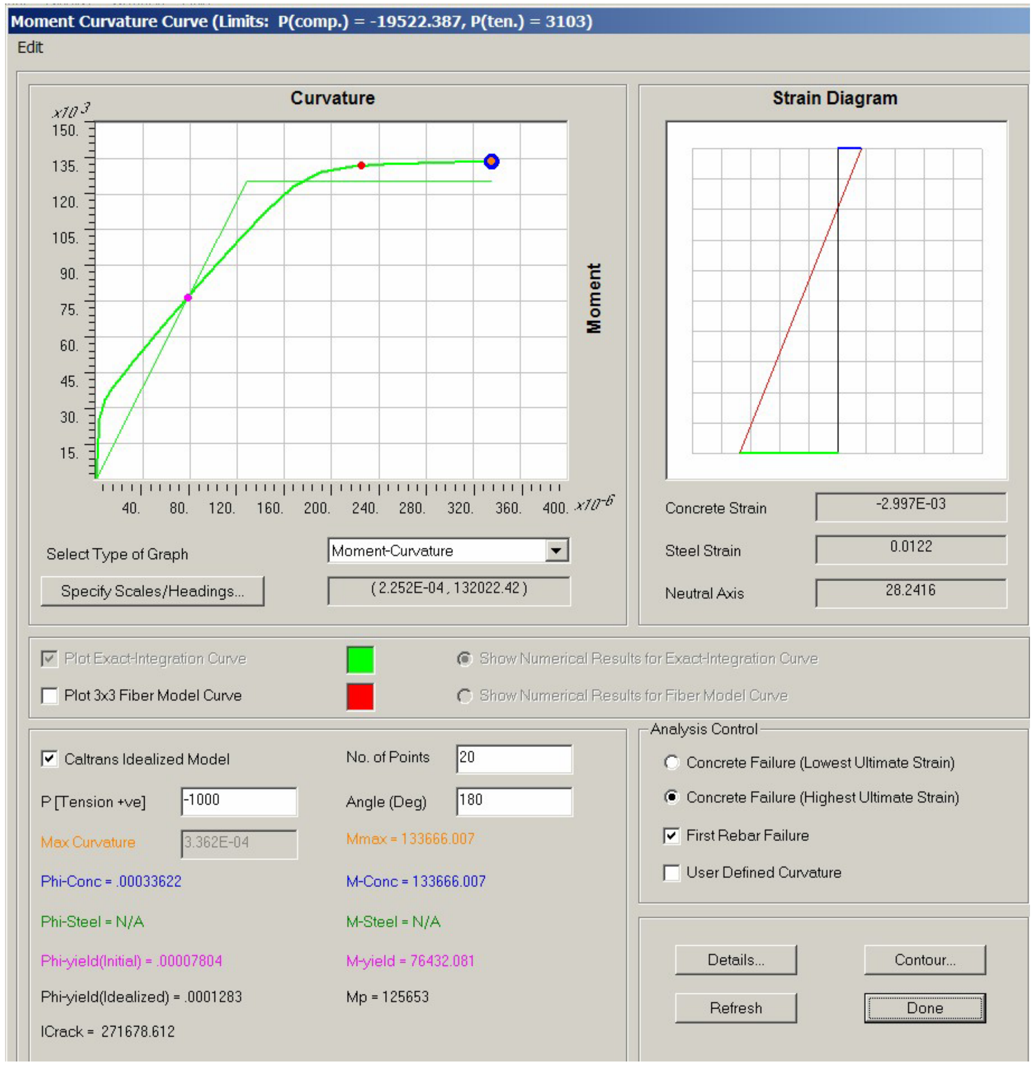


Figure VII.A.11CS1-PS-HS Strong Axis 2 Moment Curvature Curve

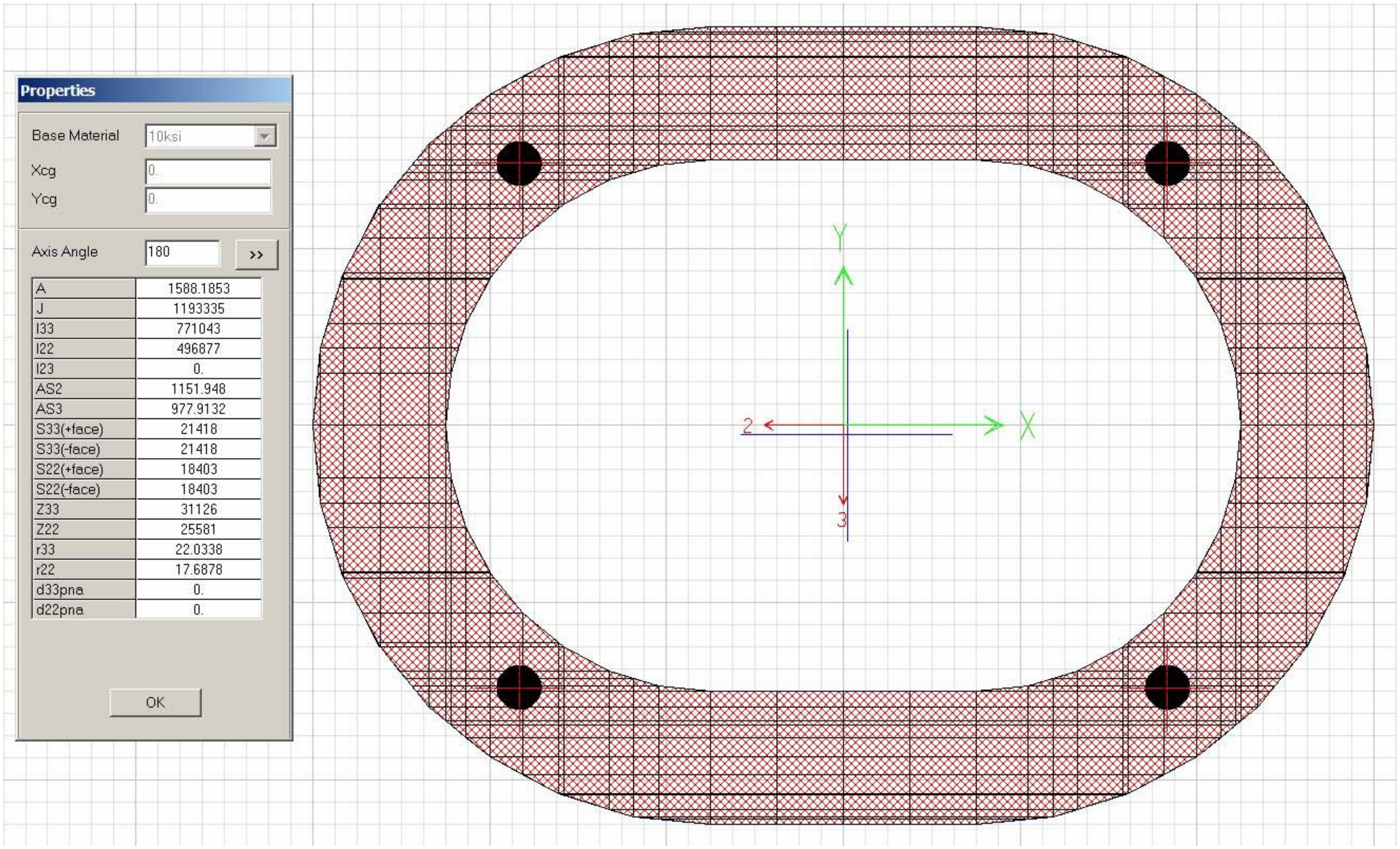


Figure VII.A.12 CS3A-PS CSI SAP Properties

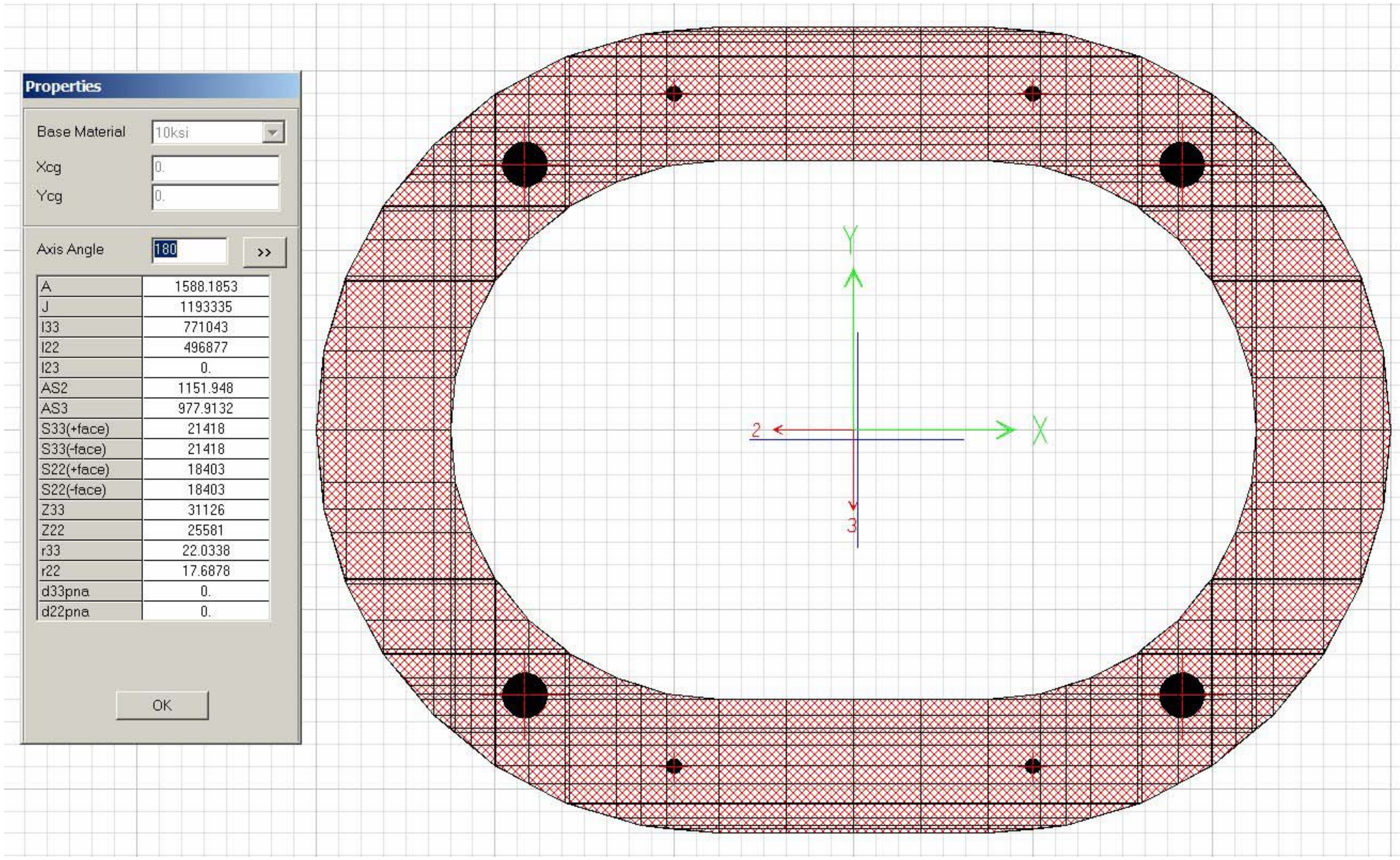


Figure VII.A.13 CS3A-PS-HS CSI SAP Properties

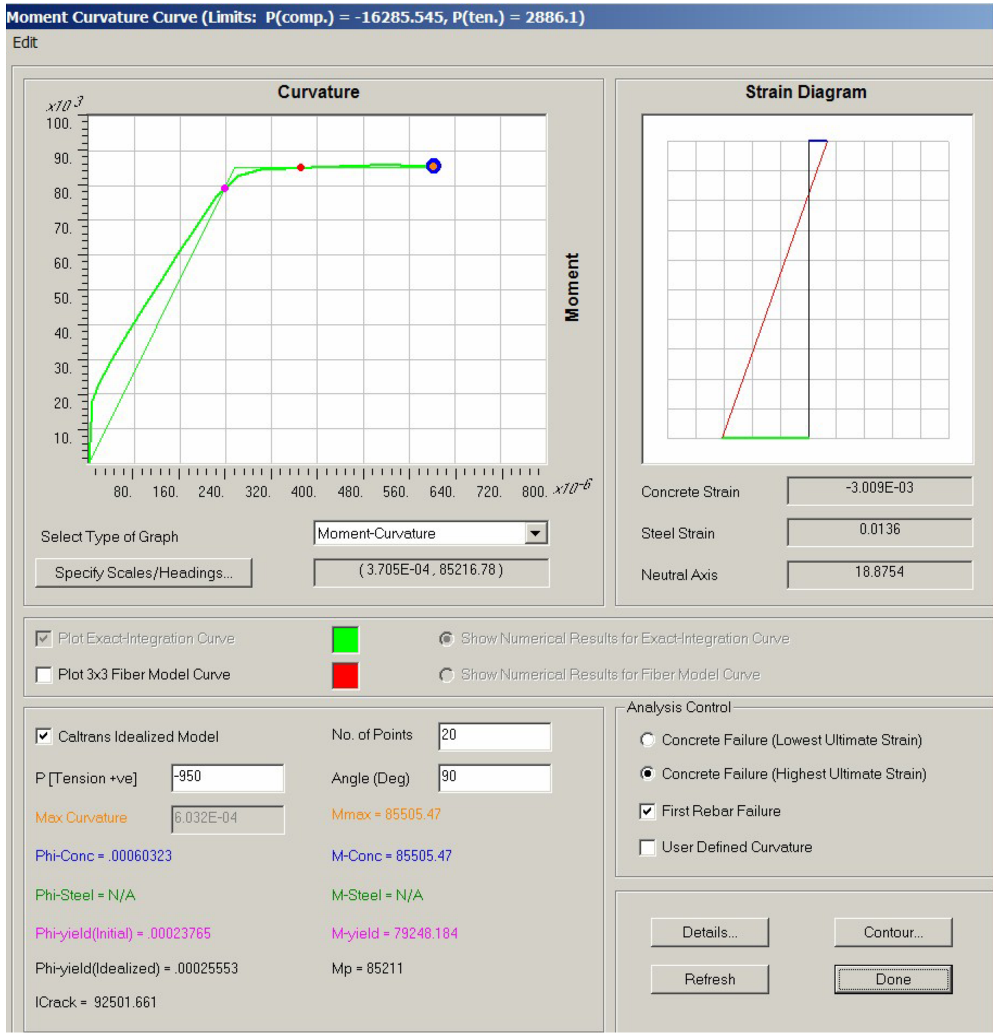


Figure VII.A.14 CS3A-PS Weak Axis Moment-Curvature Curve

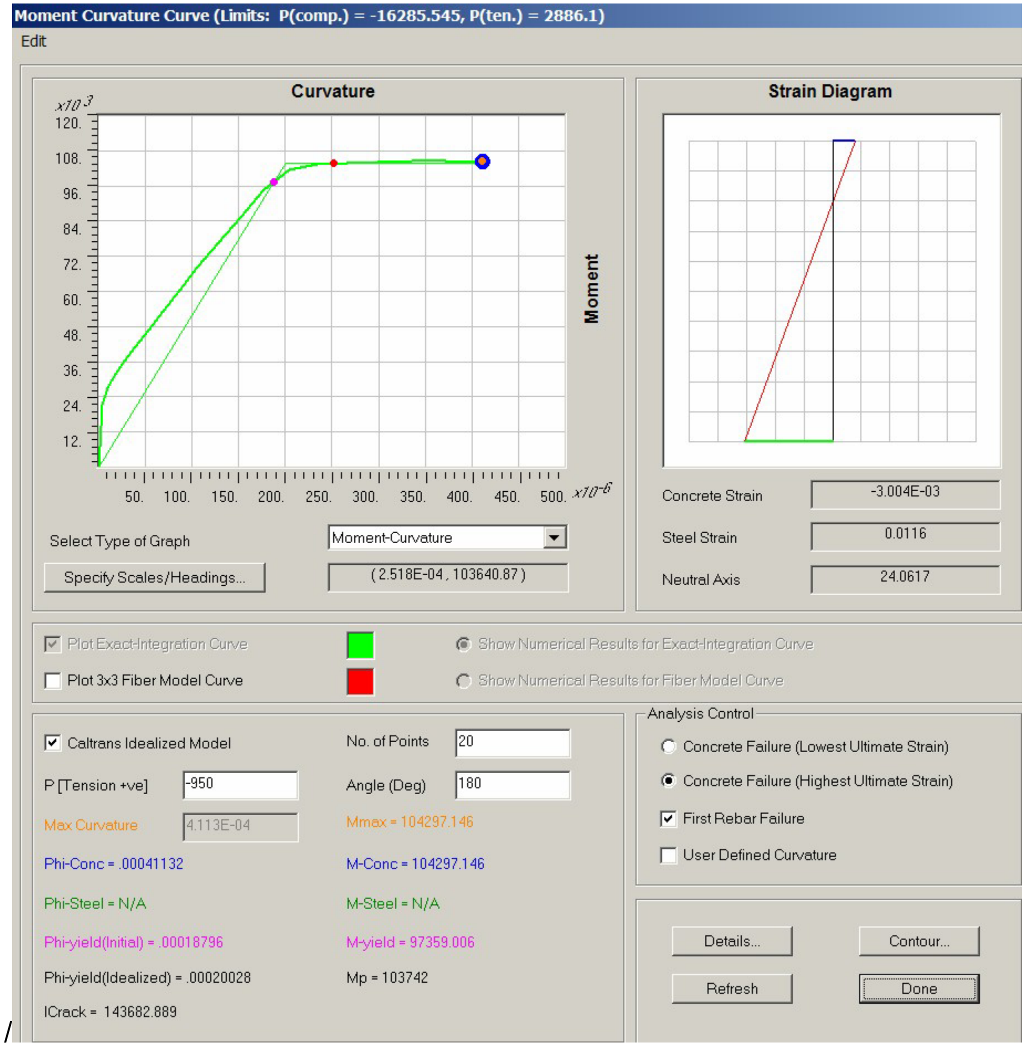


Figure VII.A.15 CS3A-PS Strong Axis 1 Moment Curvature Curve

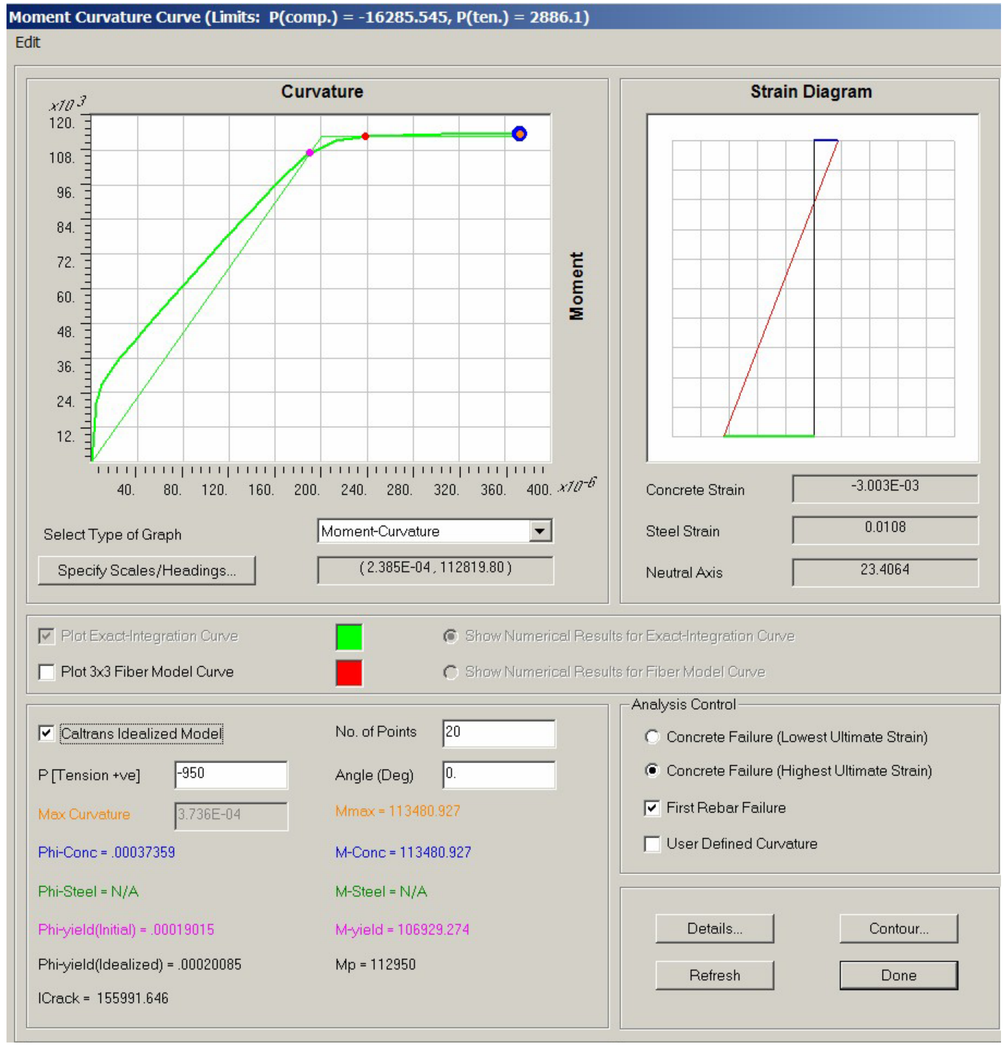


Figure VII.A.16 CS3A-PS Strong Axis 2 Moment Curvature Curve

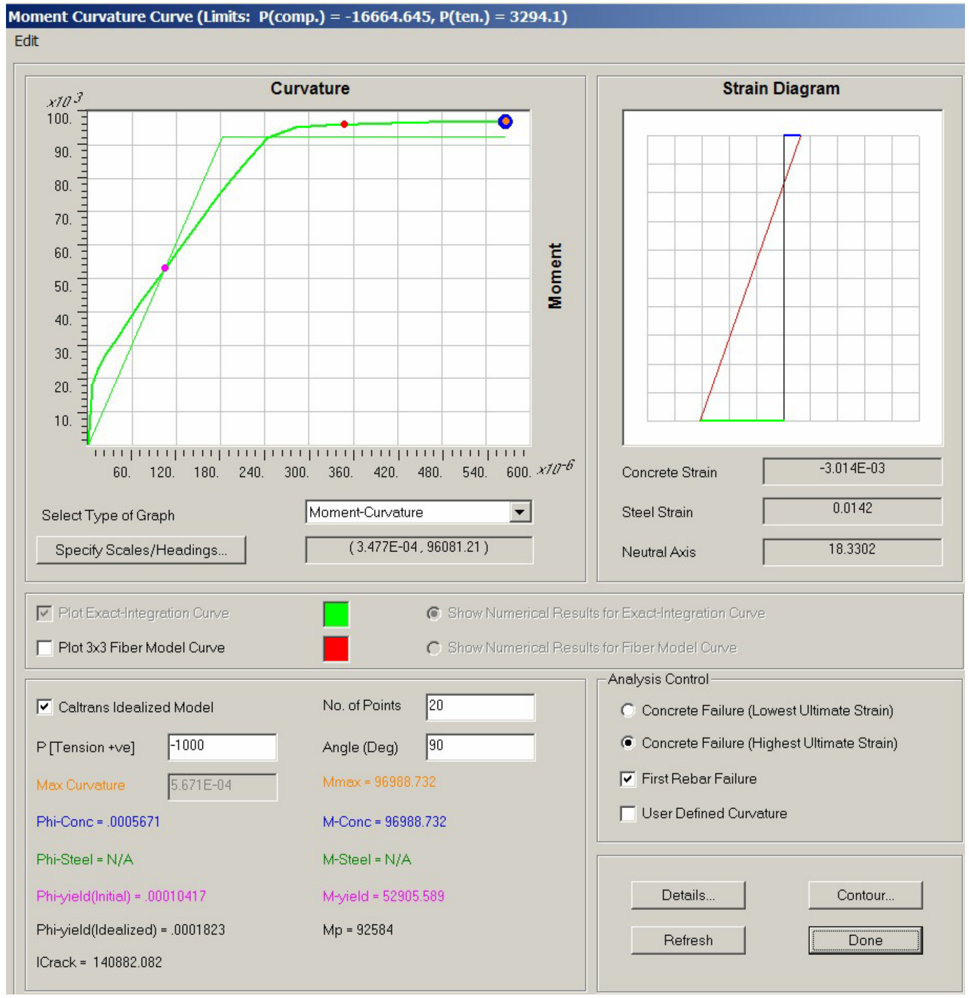


Figure VII.A.17 CS3A-PS-HS Weak Axis Moment Curvature Curve

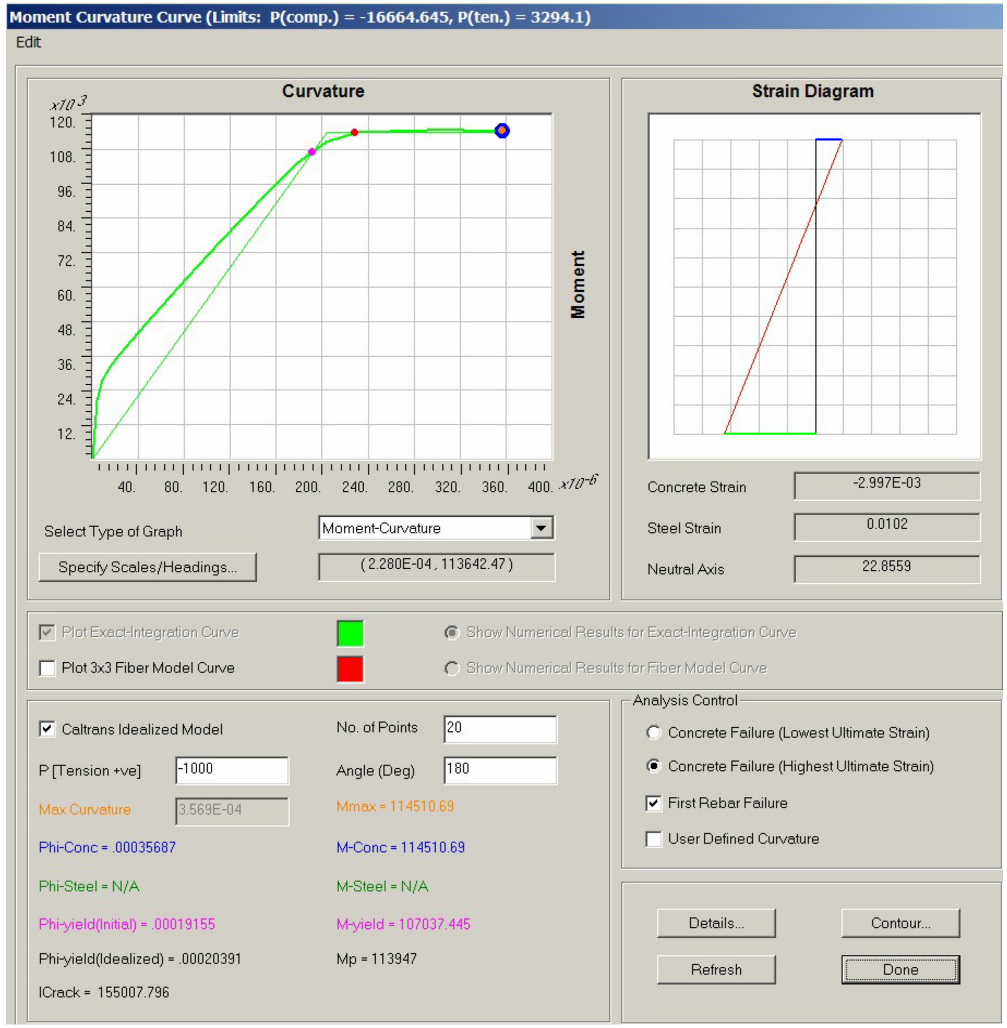


Figure VII.A.18 CS3A-PS-HS Strong Axis1 Moment Curvature Curve

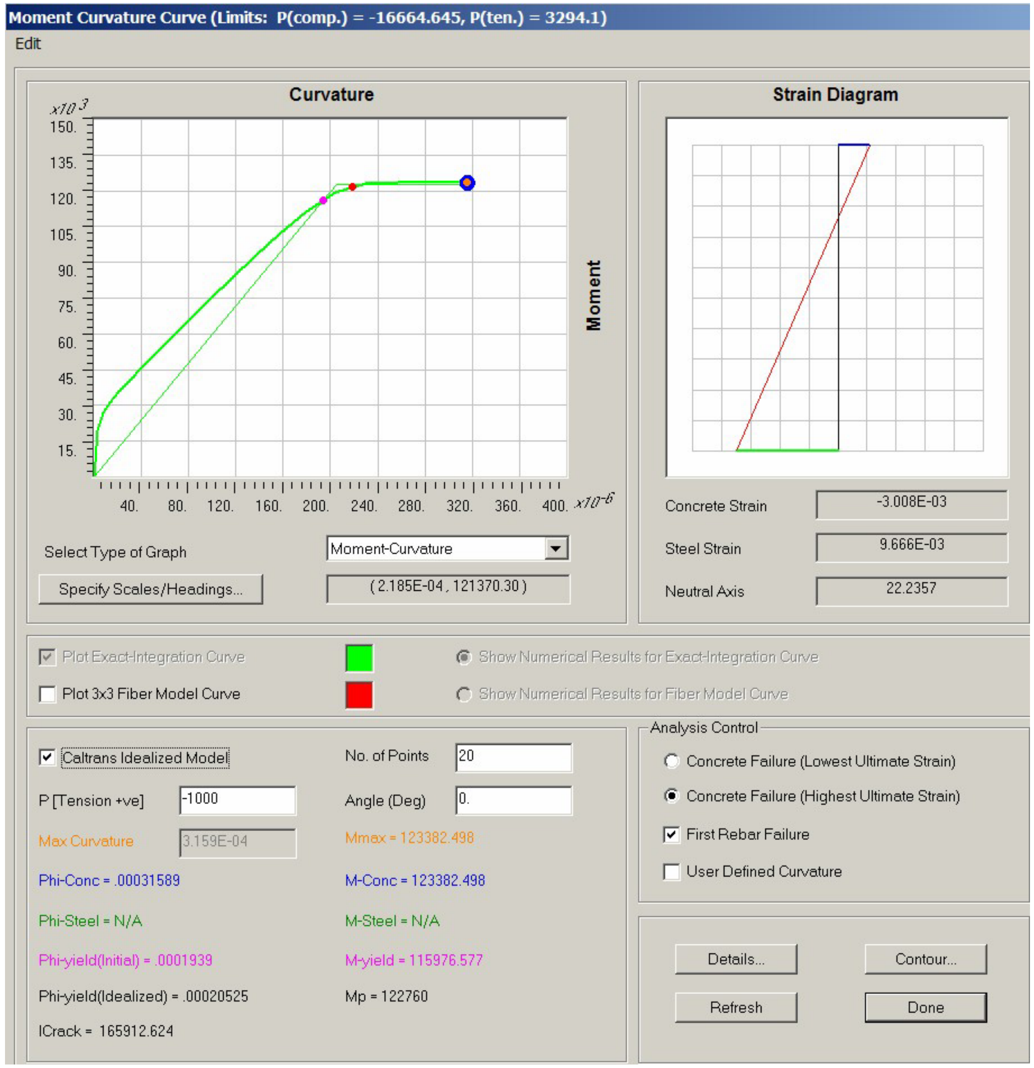


Figure VII.A.19 CS3A-PS-HS Strong Axis 2 Moment Curvature Curve

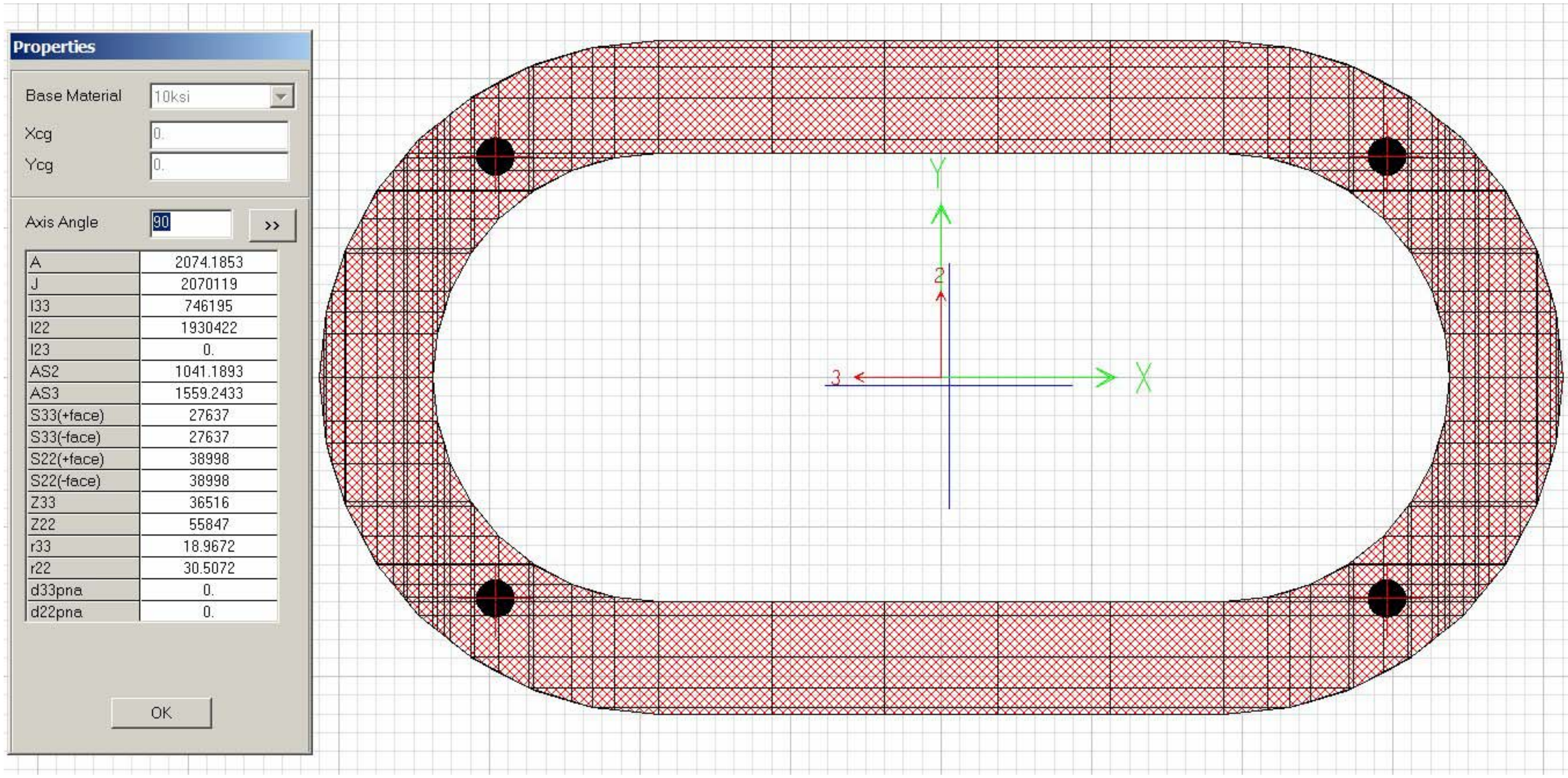


Figure VII.A.20 CS4B-PS SCI-SAP Properties

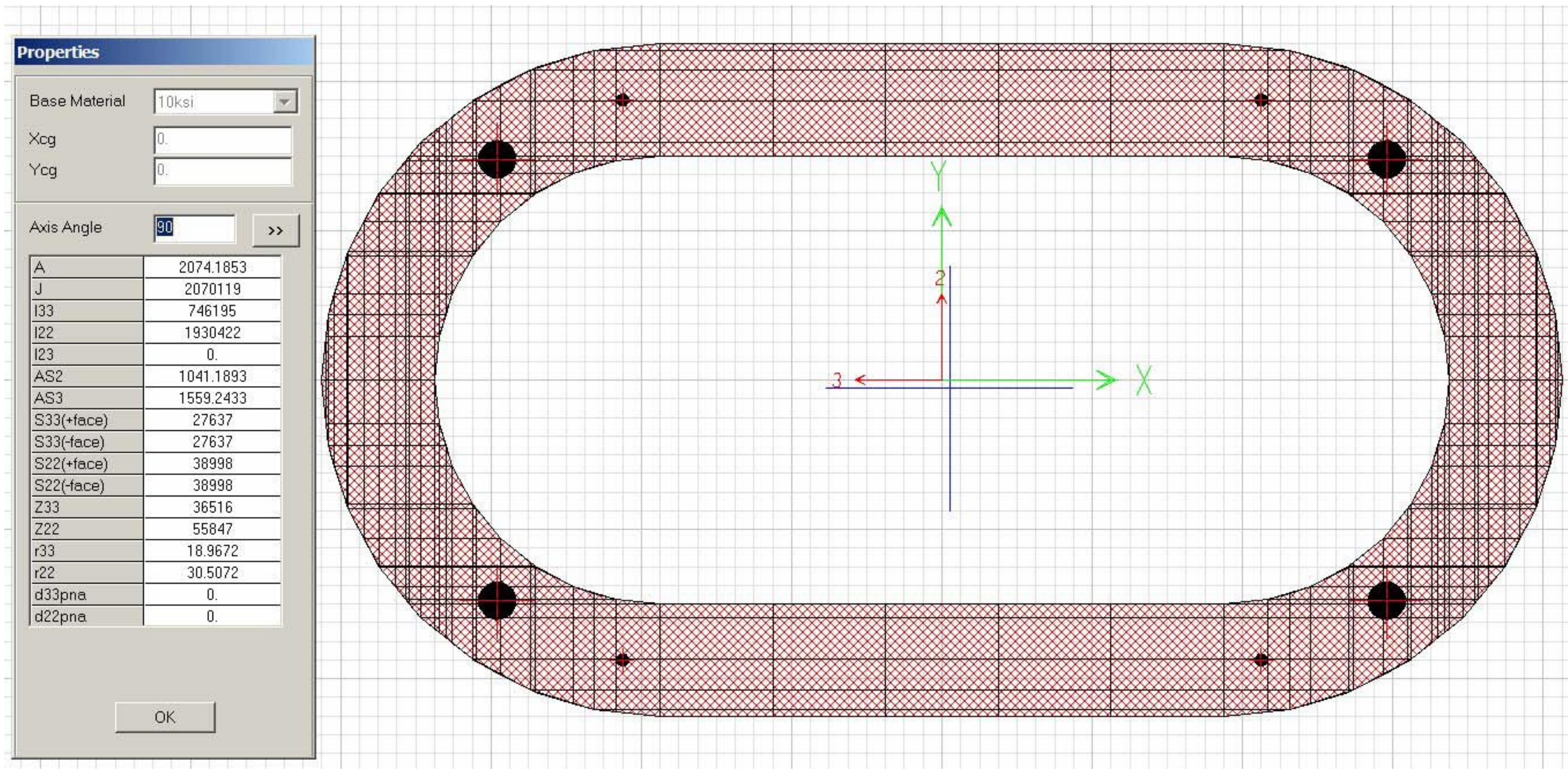


Figure VII.A.21 CS4B-PS-HS CSI-SAP Properties

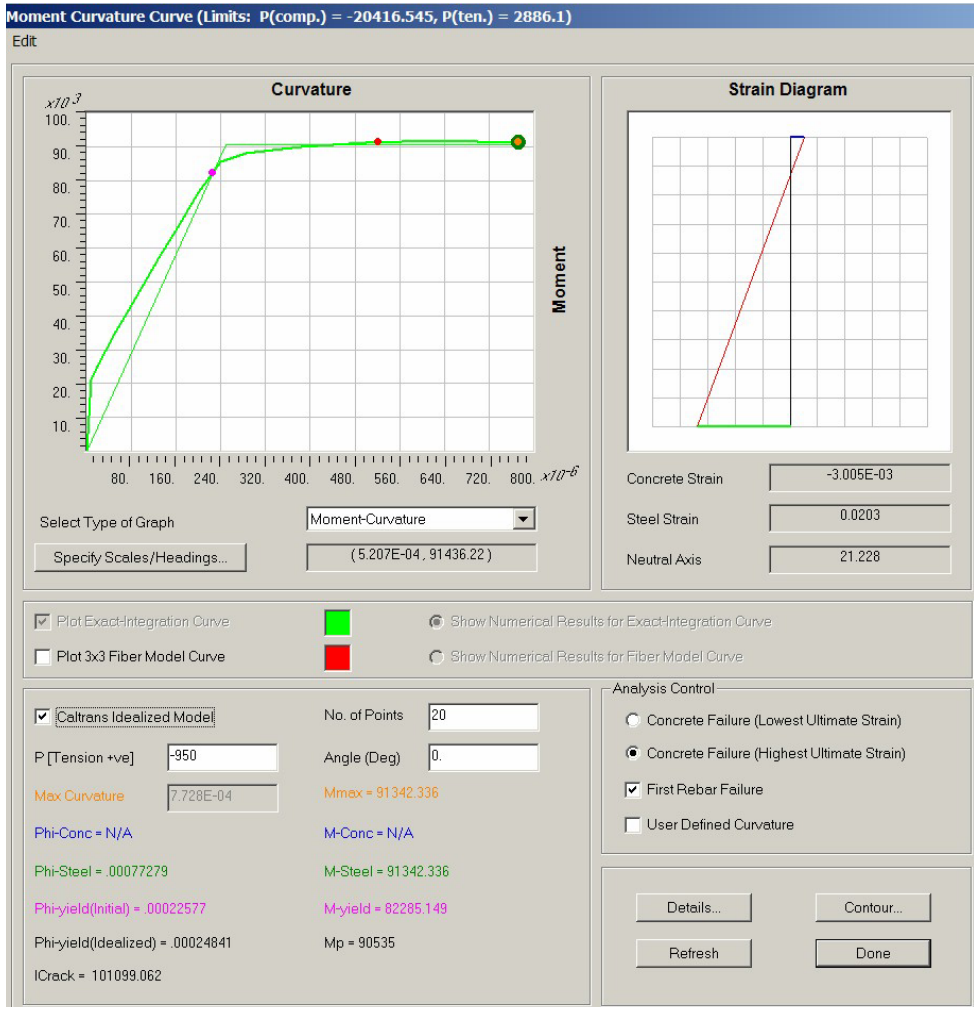


Figure VII.A.22 CS4B-PS Weak Axis Moment-Curvature Curve

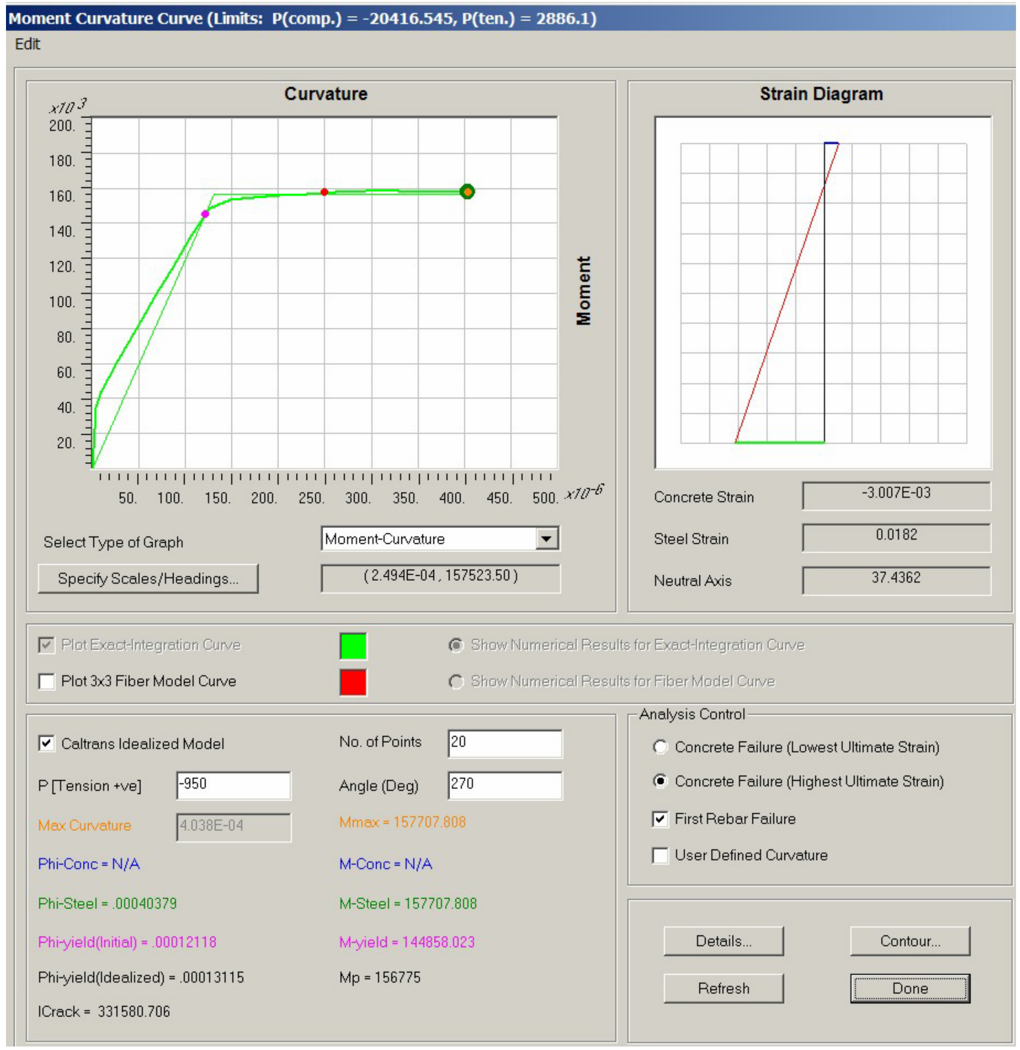


Figure VII.A.23 CS4B-PS Strong Axis 1 Moment Curvature Curve

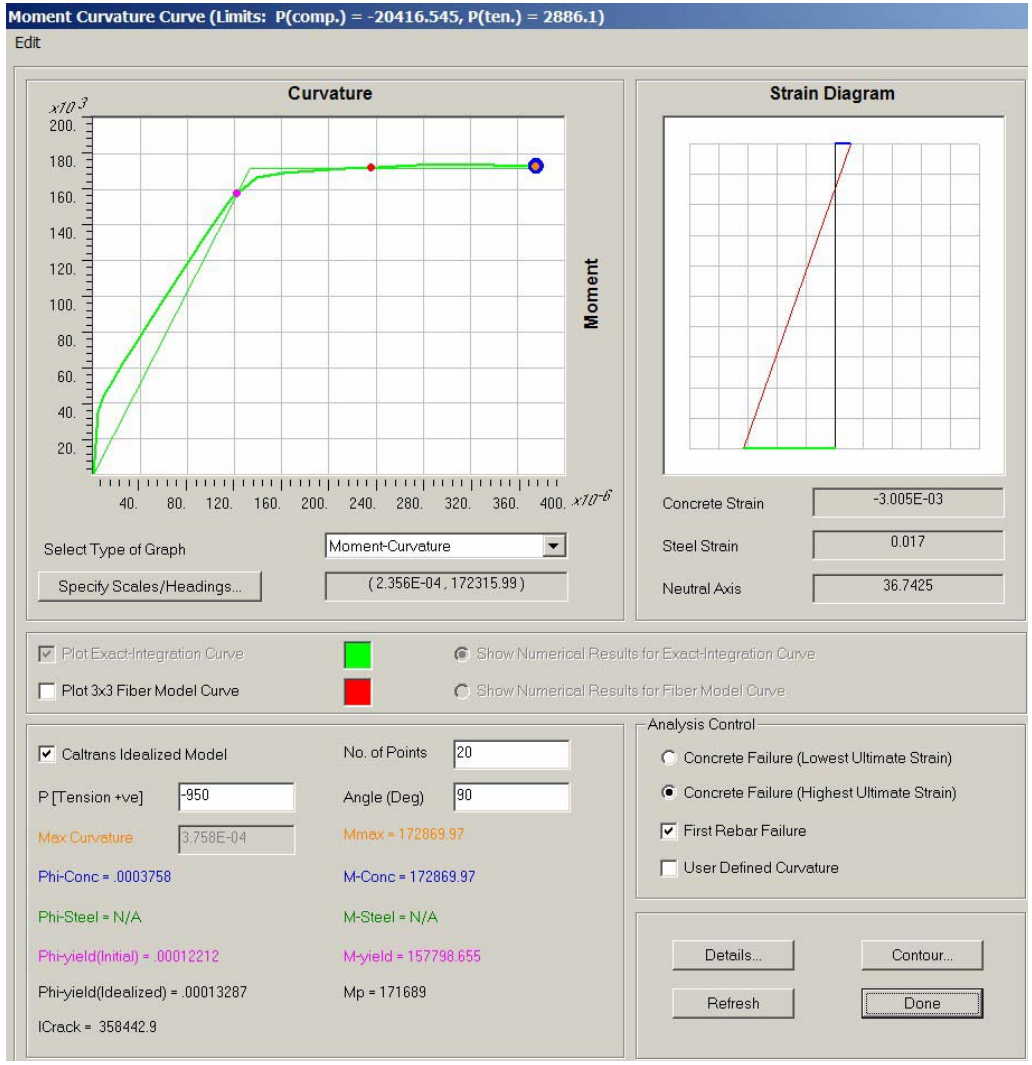


Figure VII.A.24 CS4B-PS Strong Axis 2 Moment Curvature Curve

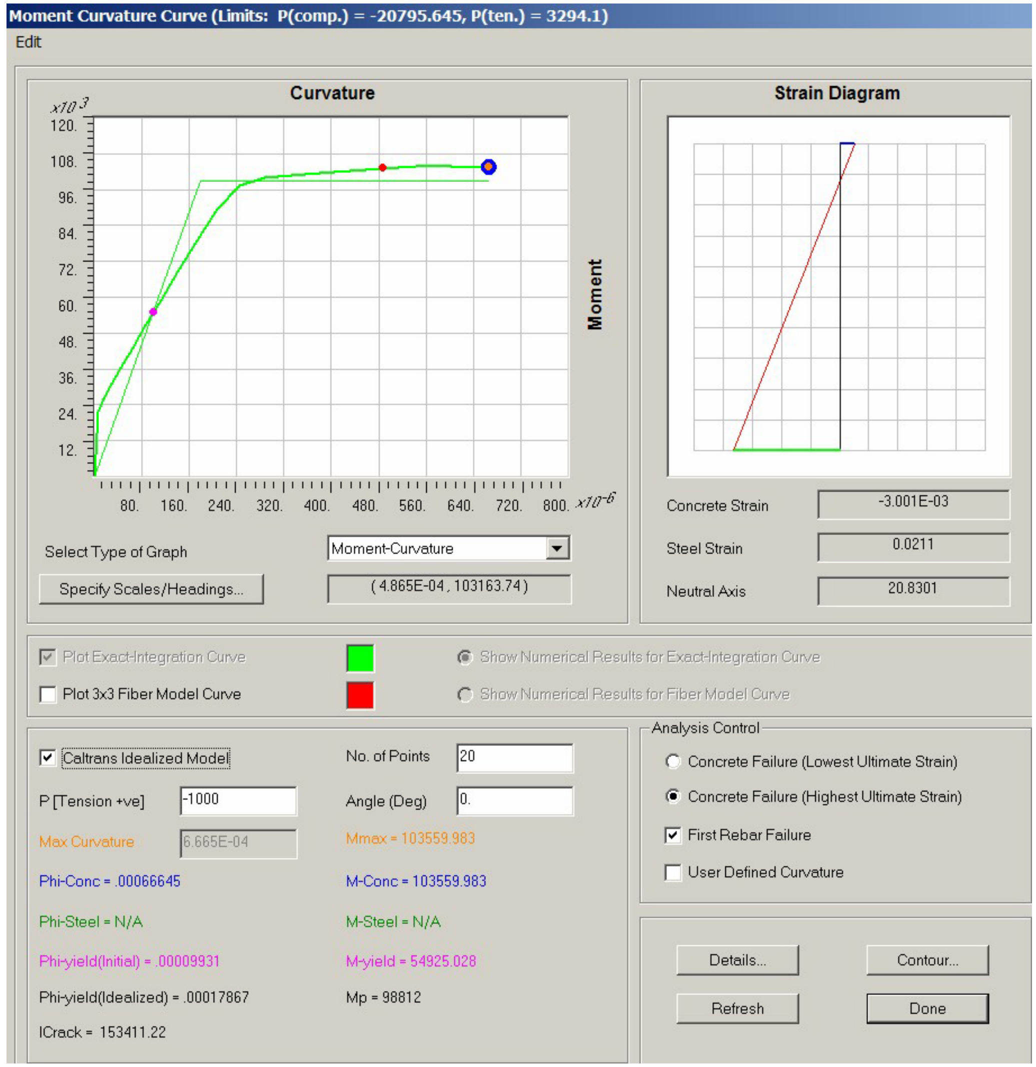


Figure VII.A.25 CS4B-PS-HS Weak Axis Moment Curvature Curve

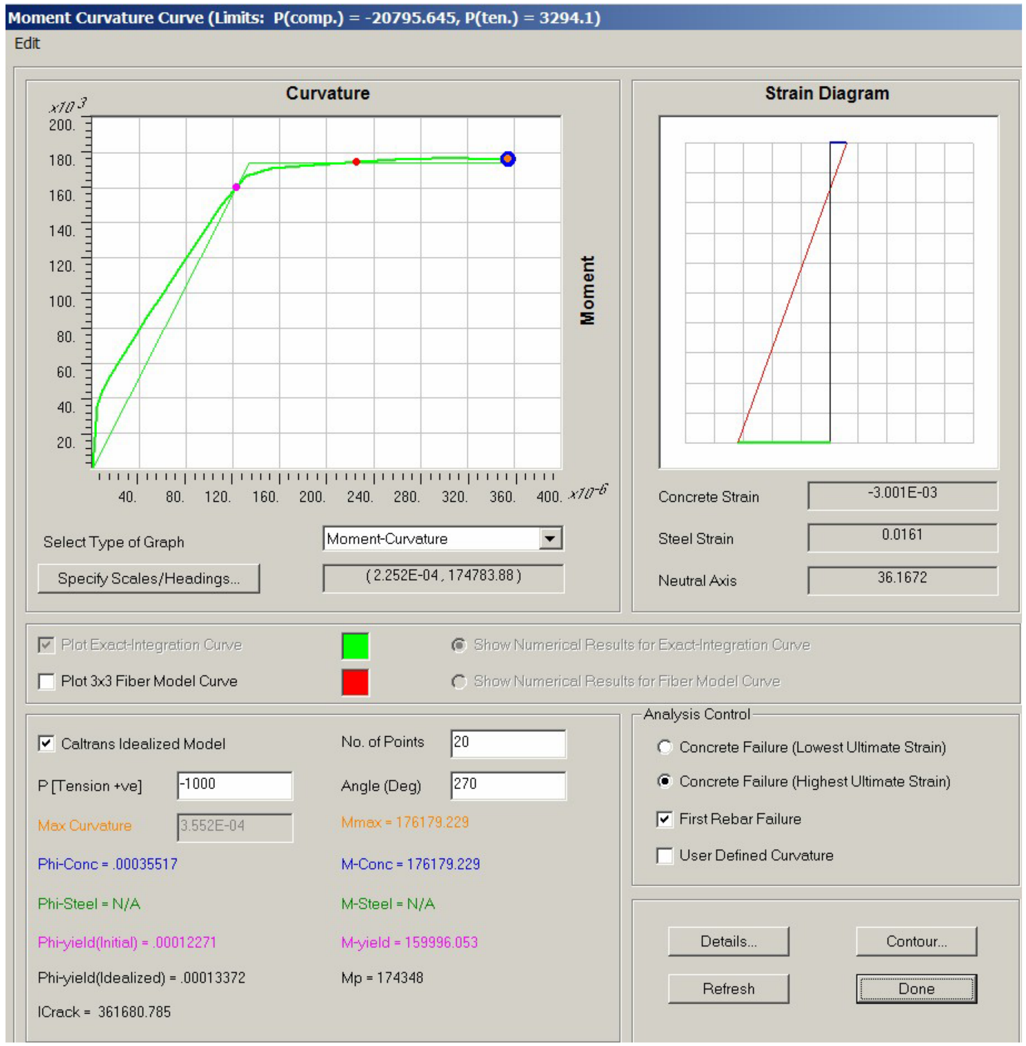


Figure VII.A.26 CS4B-PS-HS Strong Axis 1 Moment Curvature Curve

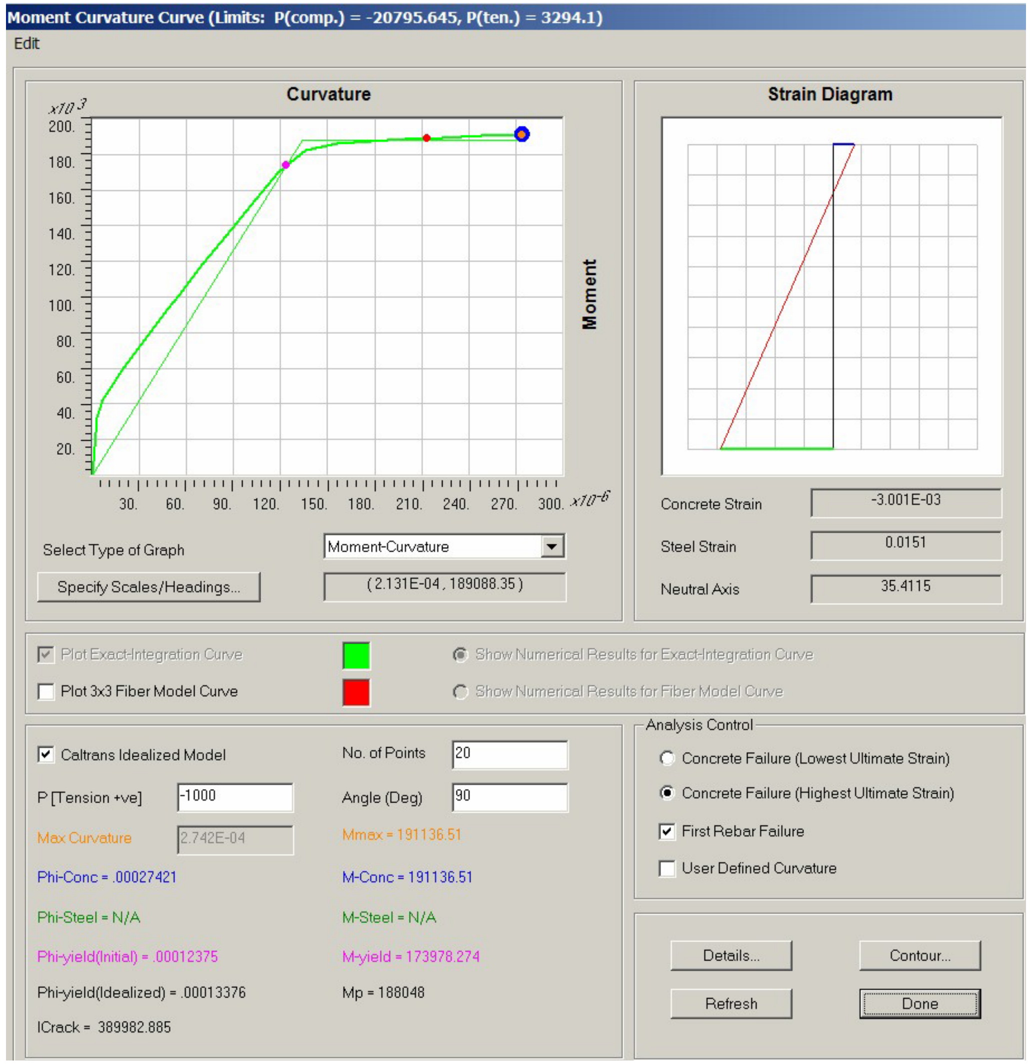


Figure VII.A.27 CS4B-PS-HS Strong Axis 2 Moment Curvature Curve

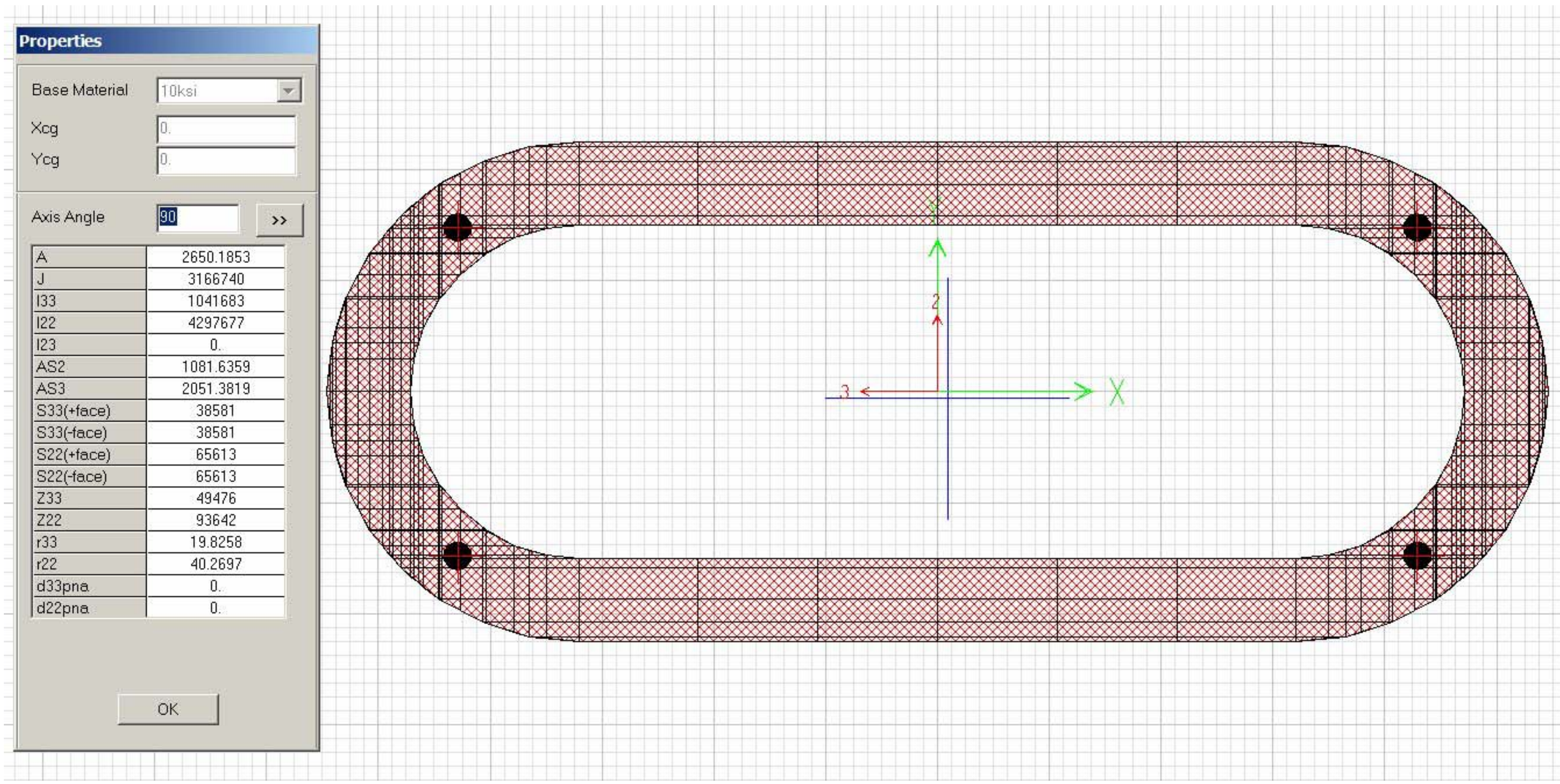


Figure VII.A.28 CS13B-PS CSI-SAP Properties

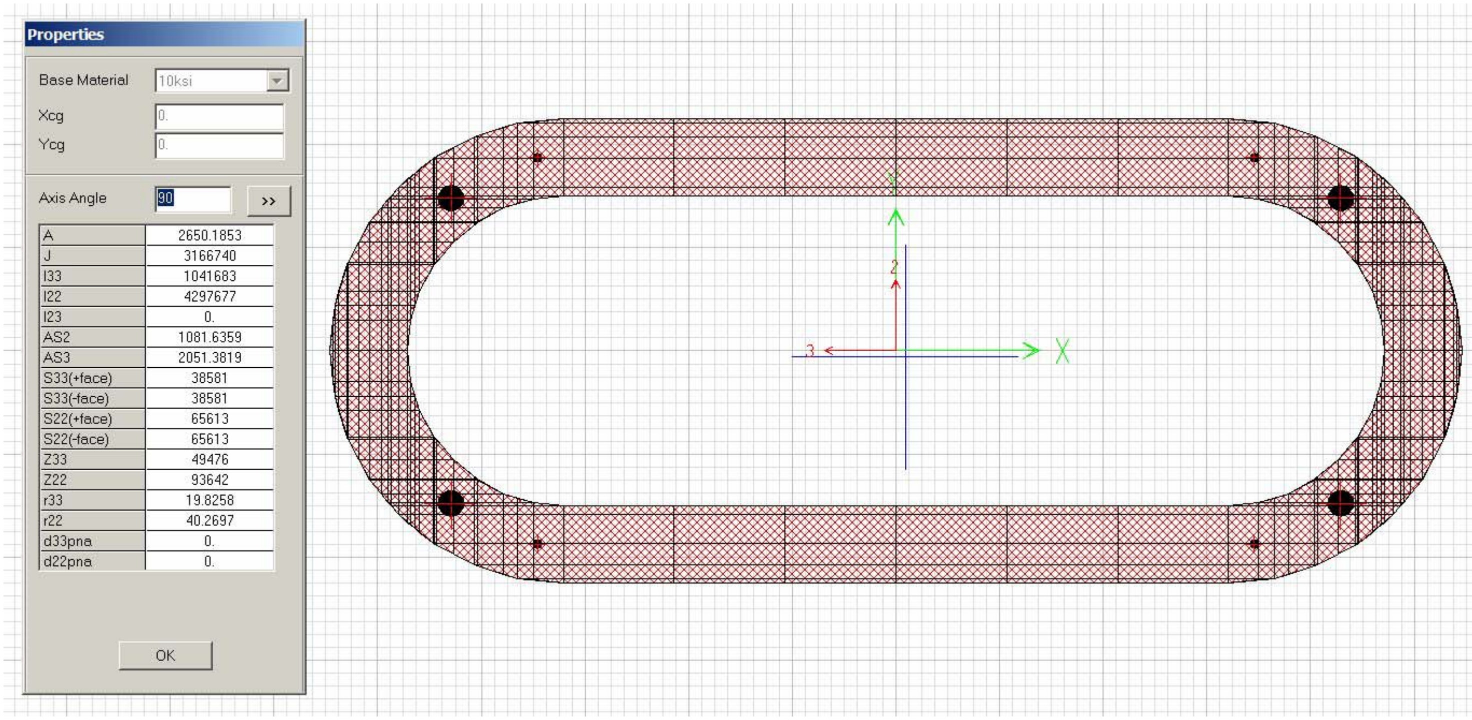


Figure VII.A.29 CS13B-PS-HS CSI-SAP Properties

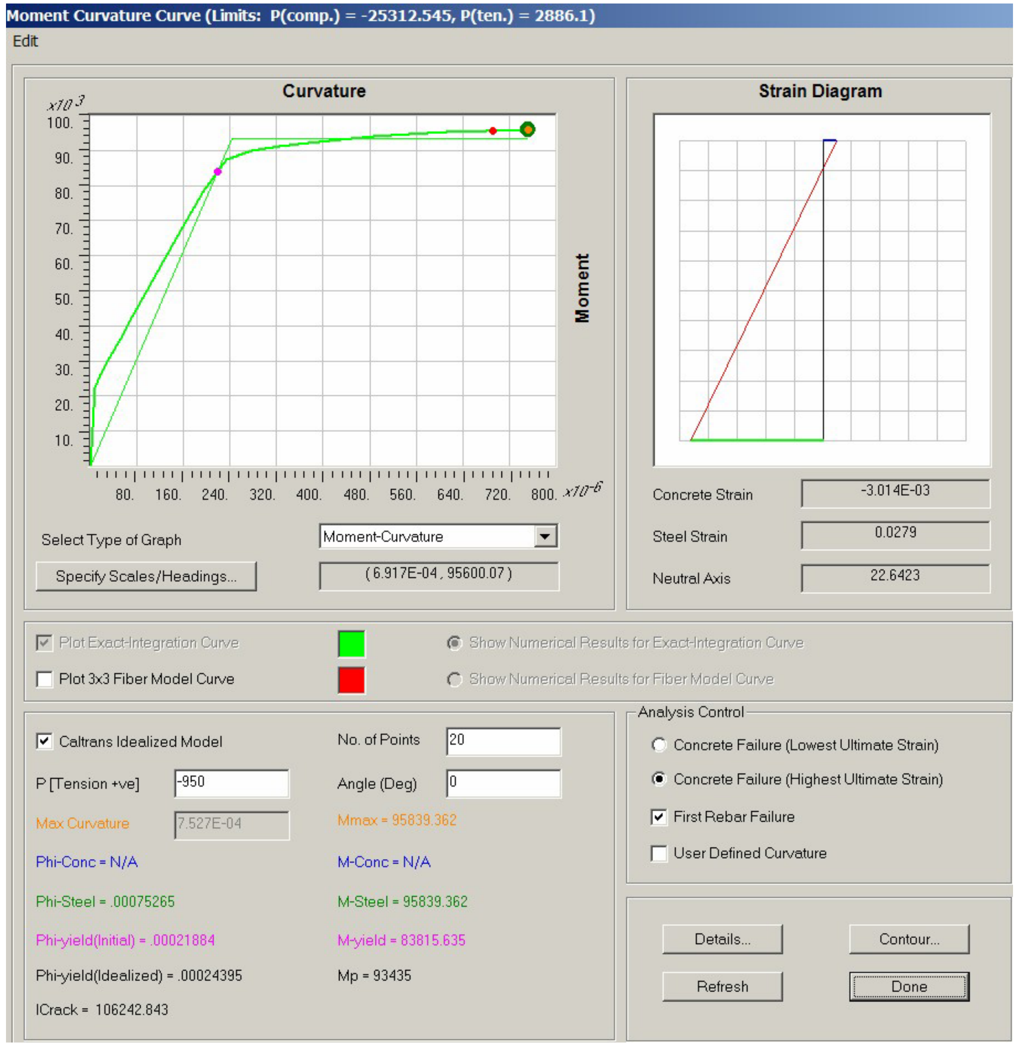


Figure VII.A.30 CS13B-PS Weak Axis Moment Curvature Curve

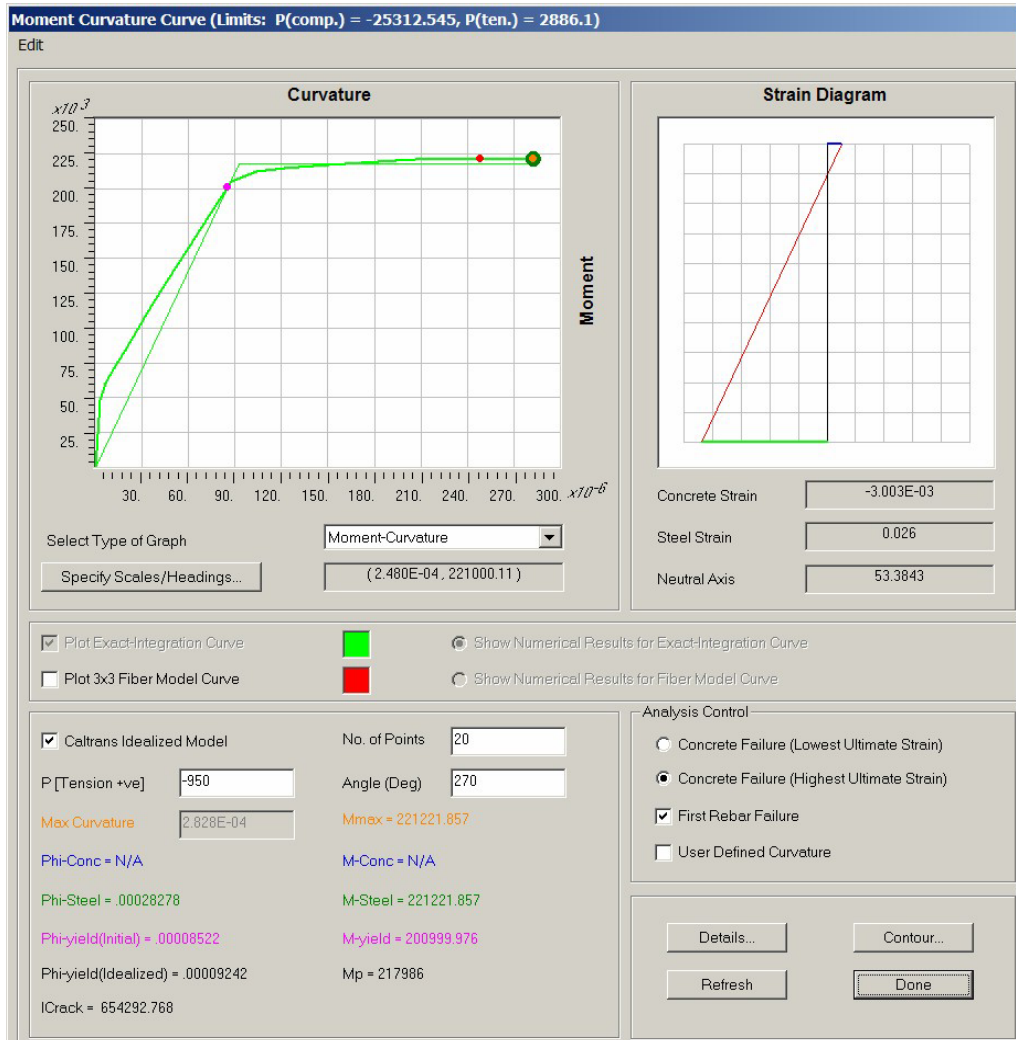


Figure VII.A.31 CS13B-PS Strong Axis 1 Moment Curvature Curve

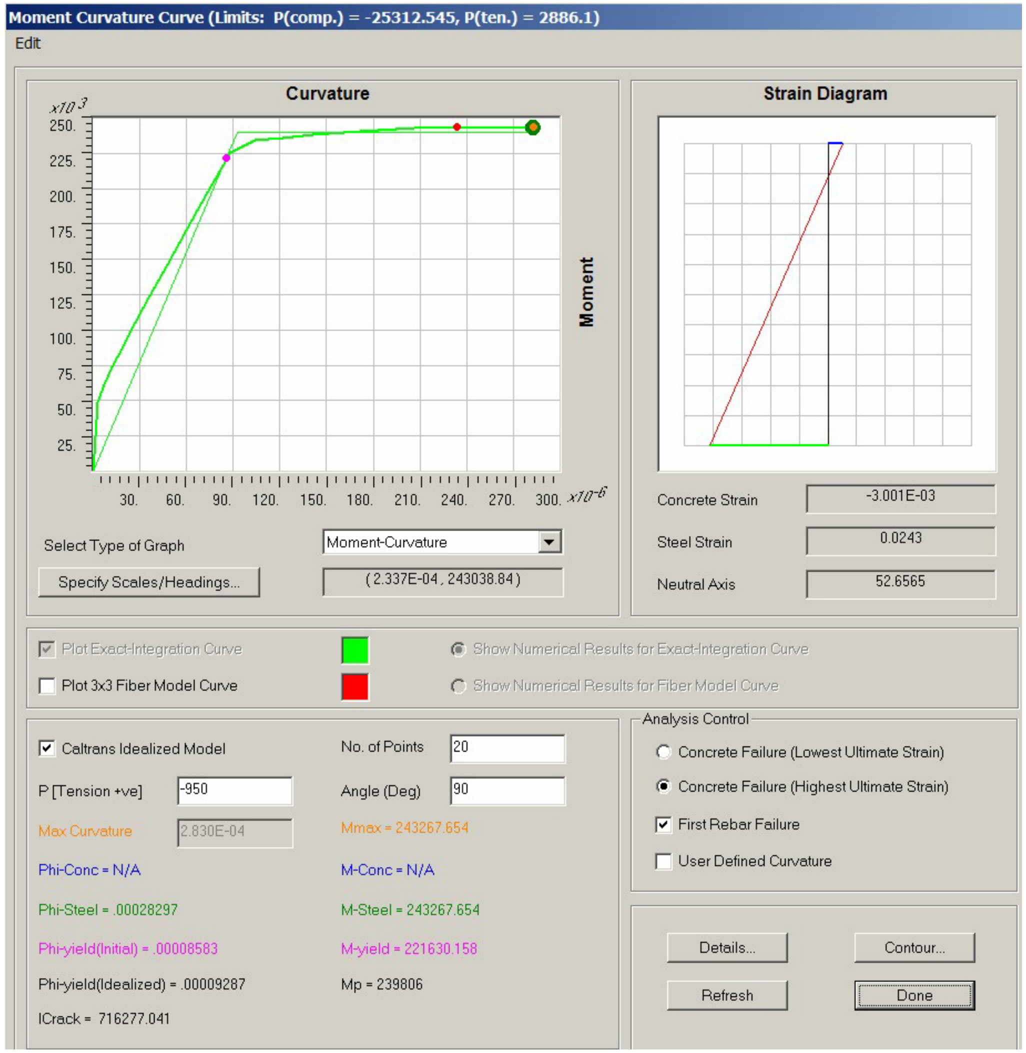


Figure VII.A.32 CS13B-PS Strong Axis 2 Moment Curvature Curve

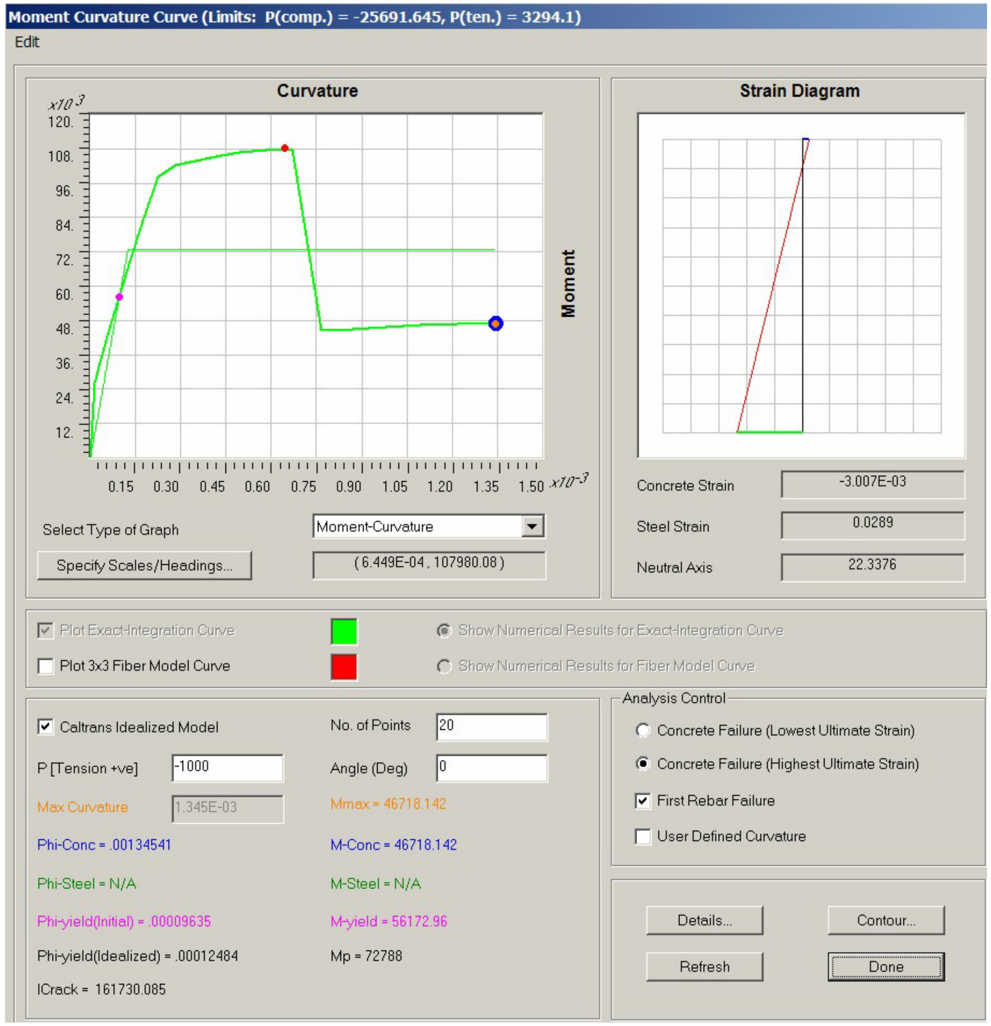


Figure VII.A.33 CS13B-PS-HS Weak Axis Moment Curvature Curve

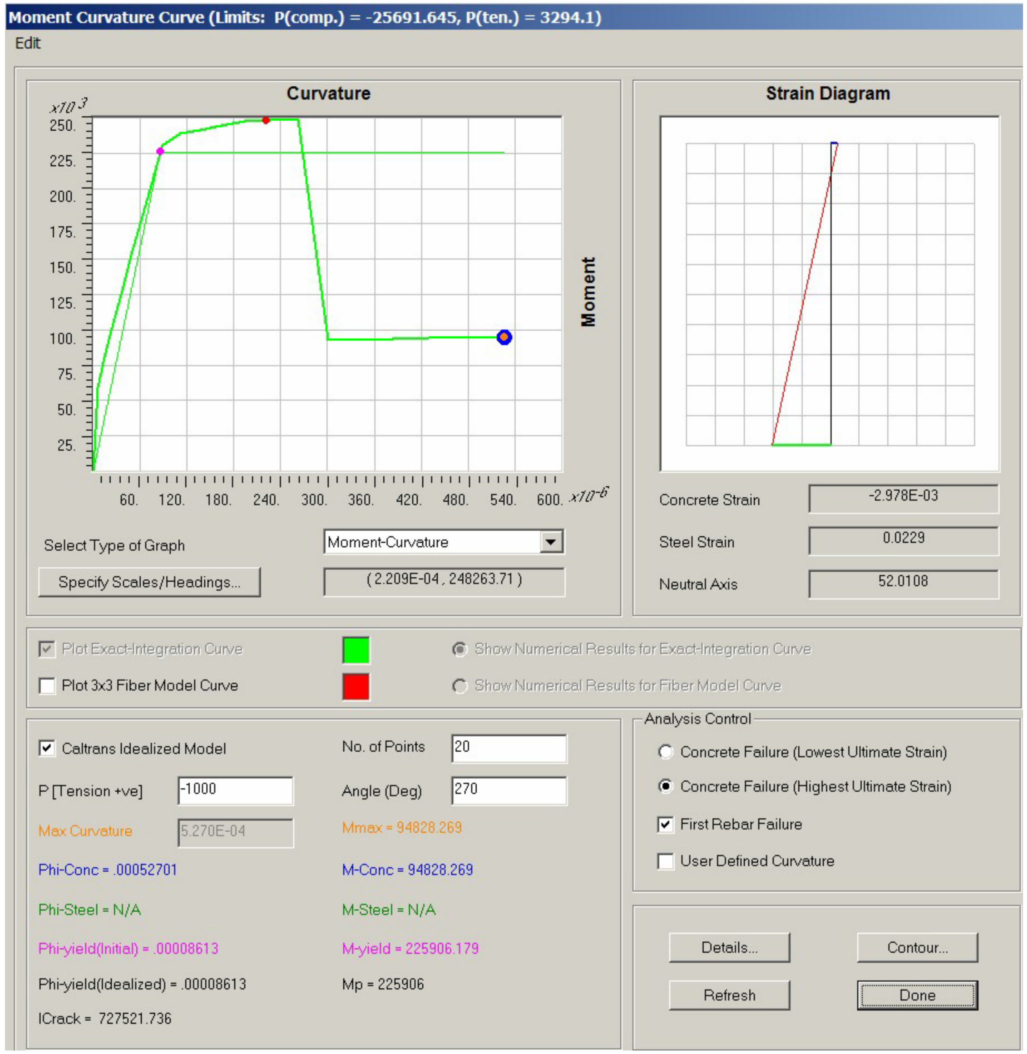


Figure VII.A.34 CS13B-PS-HS Strong Axis 1 Moment Curvature Curve

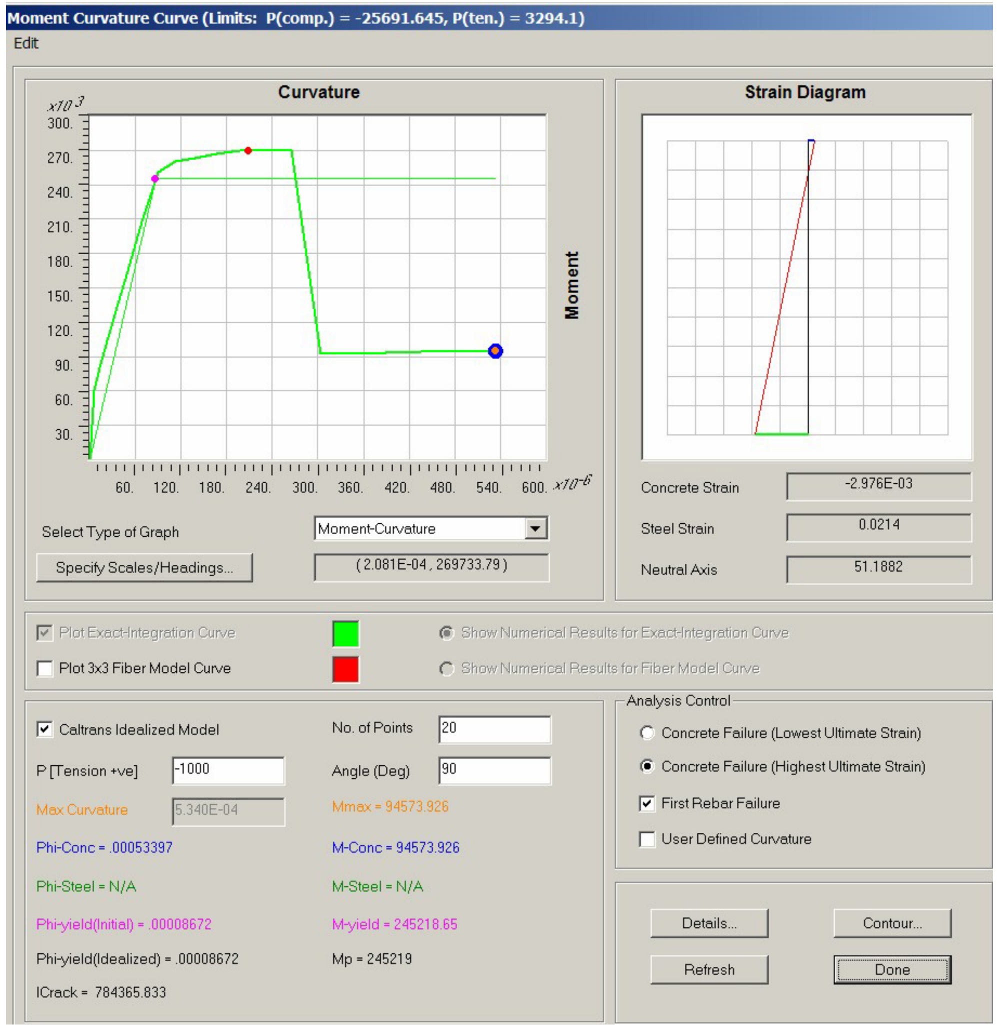


Figure VII.A.35 CS13B-PS-HS Strong Axis 2 Moment Curvature Curve

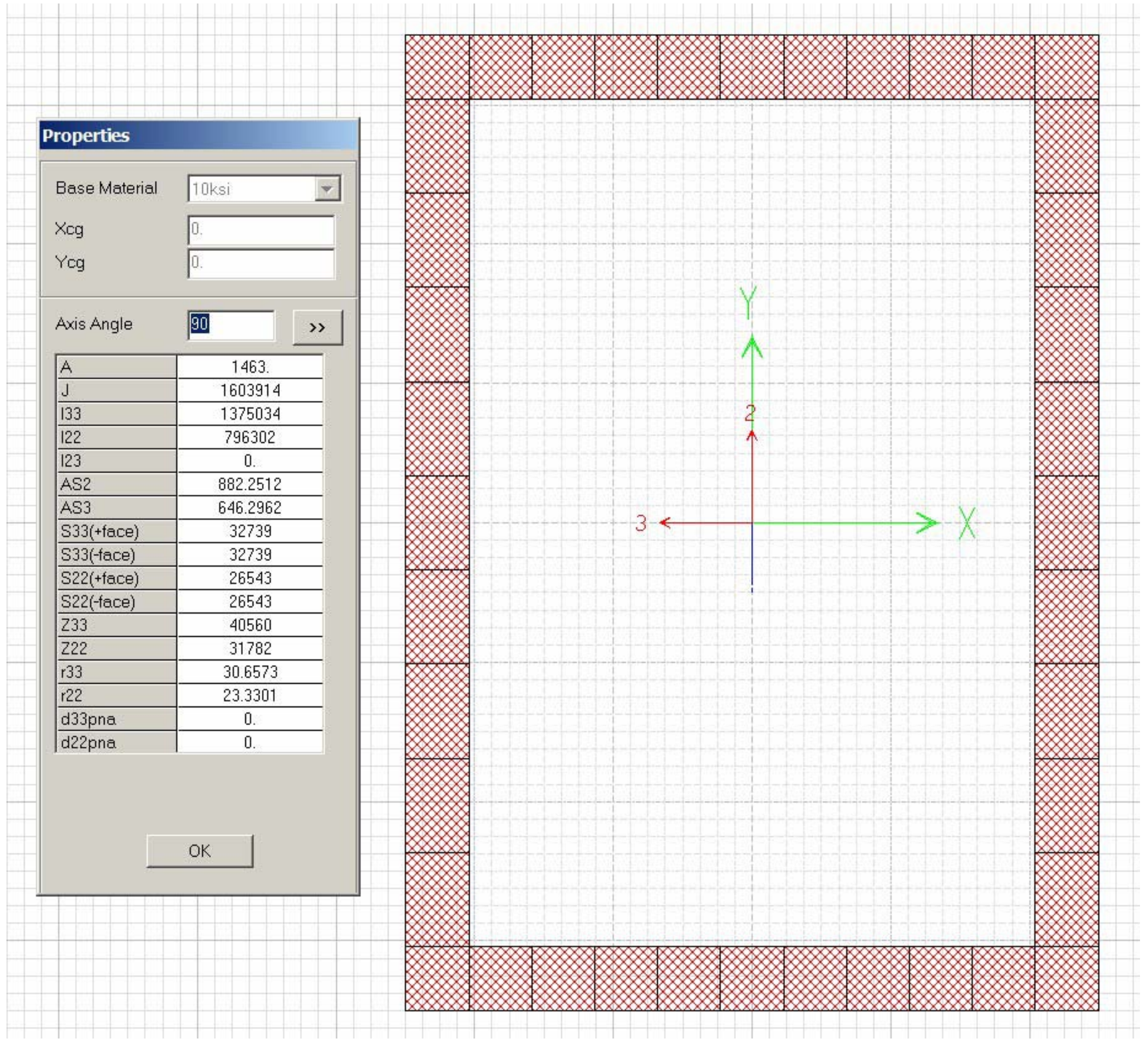


Figure VII.A.36 Bent Cap Section A-A Hollow Core

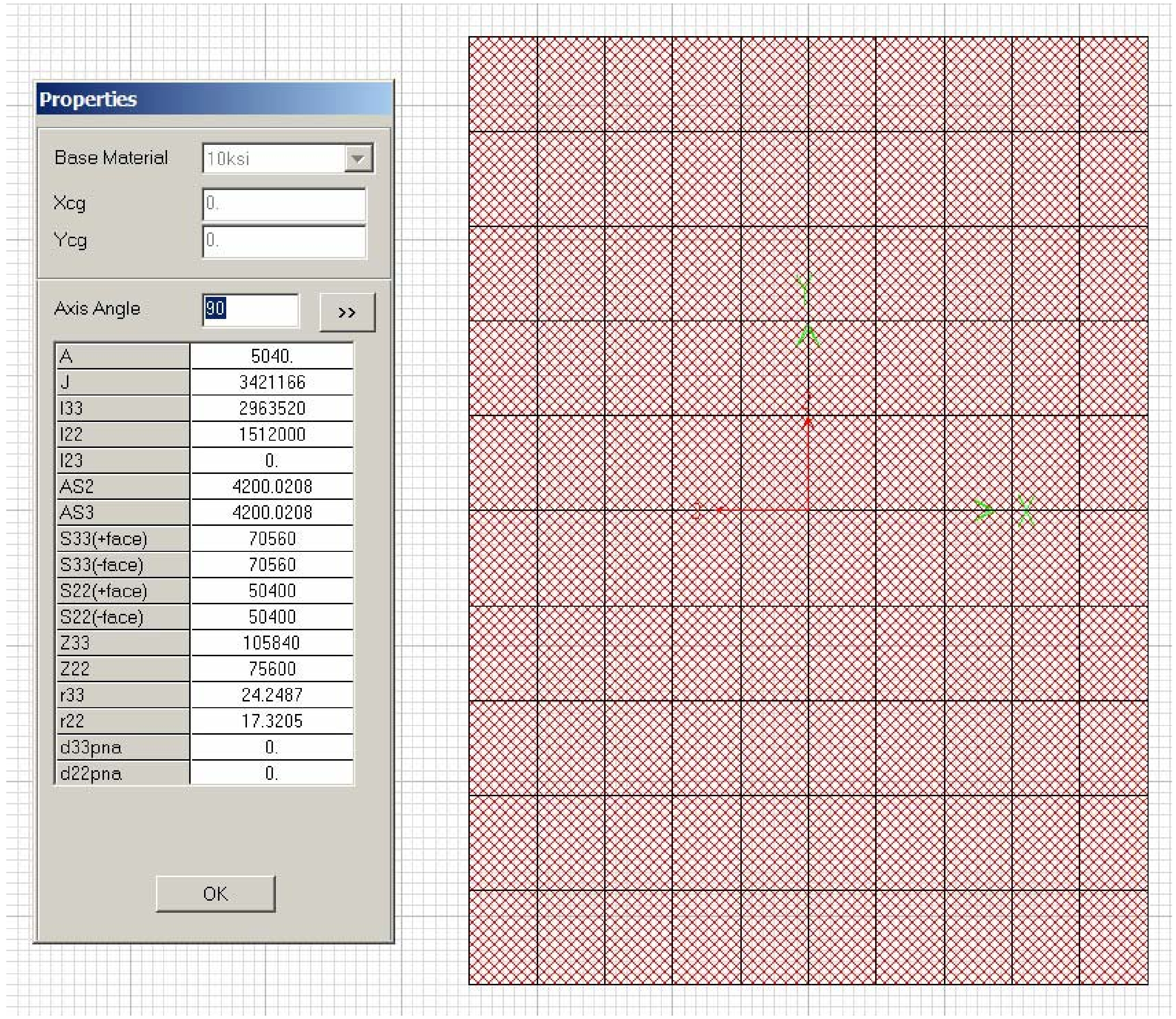


Figure VII.A.37 Bent Cap Section B-B Solid Section

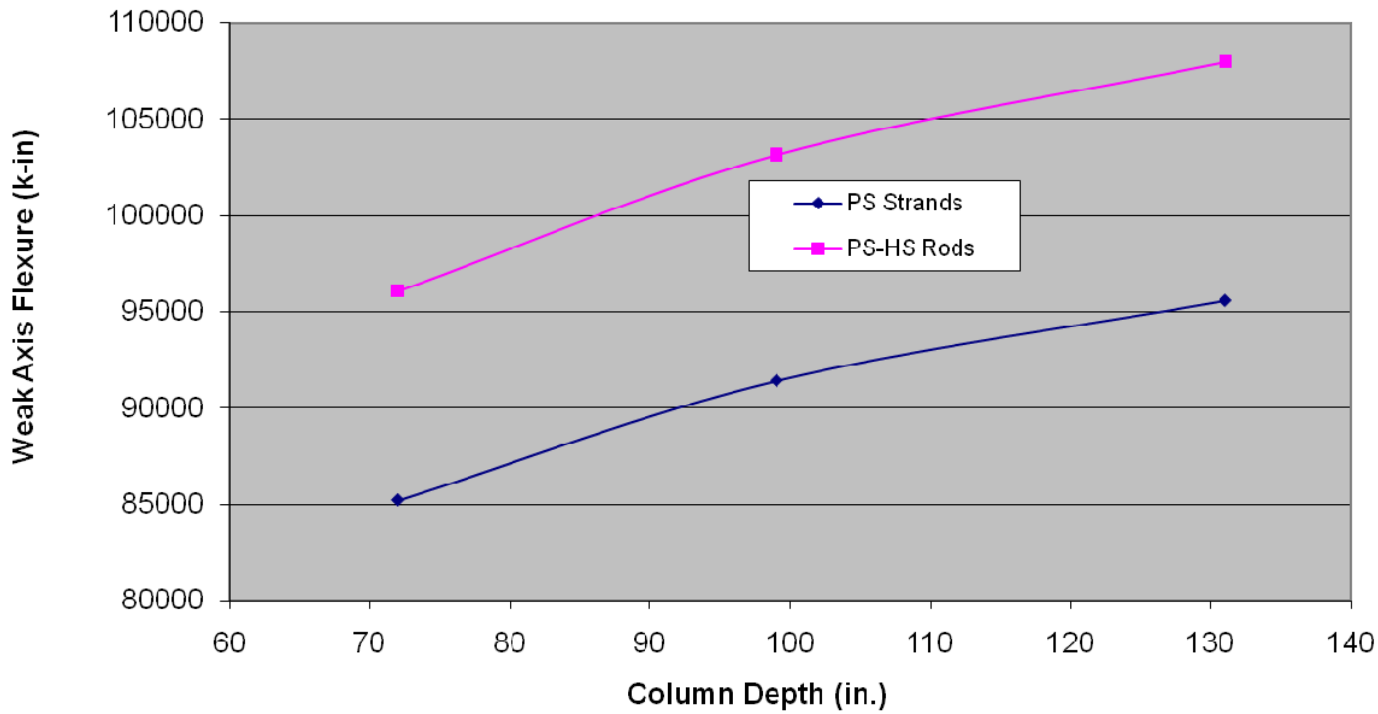


Figure VII.A.38 Weak Axis Flexural Capacity CS3A, CS4B, and CS13B

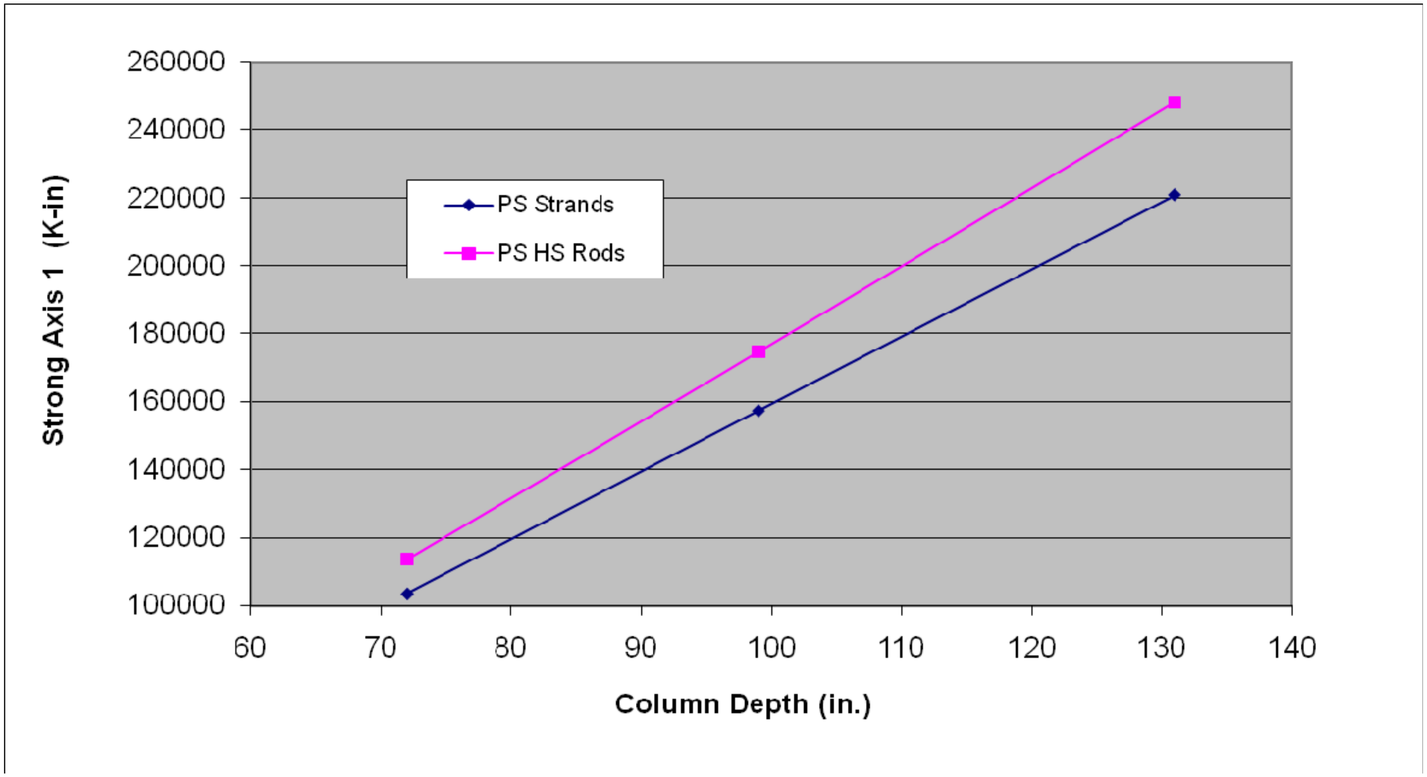


Figure VII.A.39 Strong Axis 1 Flexural Capacity CS3A, CS4B, and CS13B

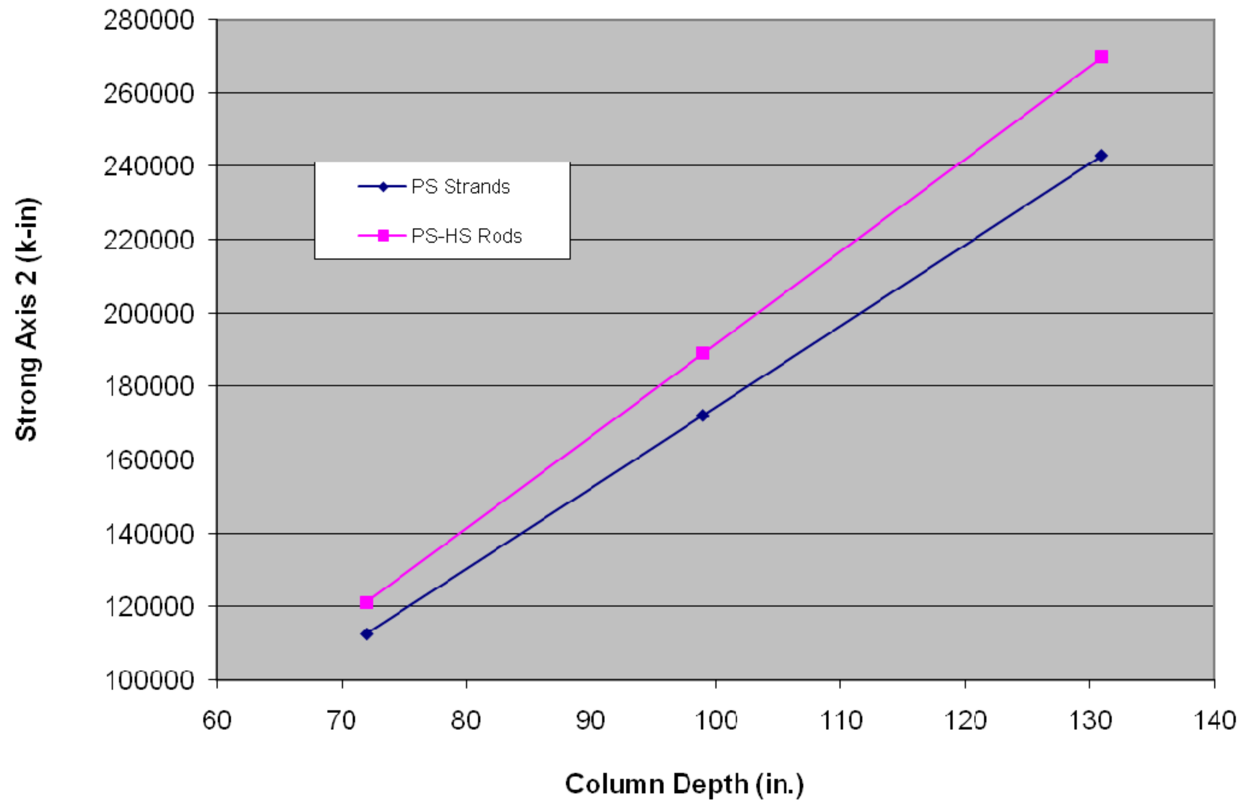


Figure VII.A.40 Strong Axis 2 Flexural Capacity CS3A, CS4B, and CS13B

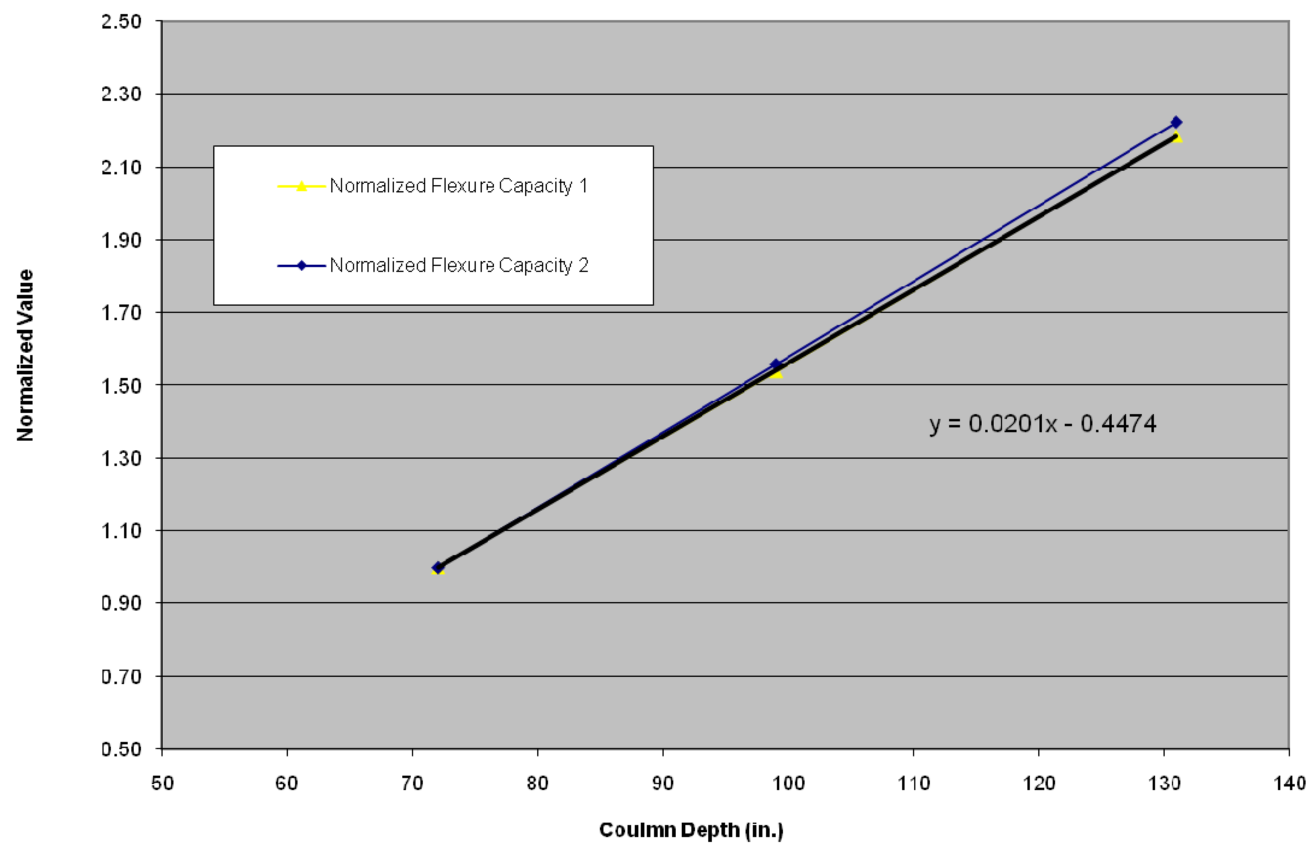


Figure VII.A.41 Normalized Strong Axis Flexural Capacity

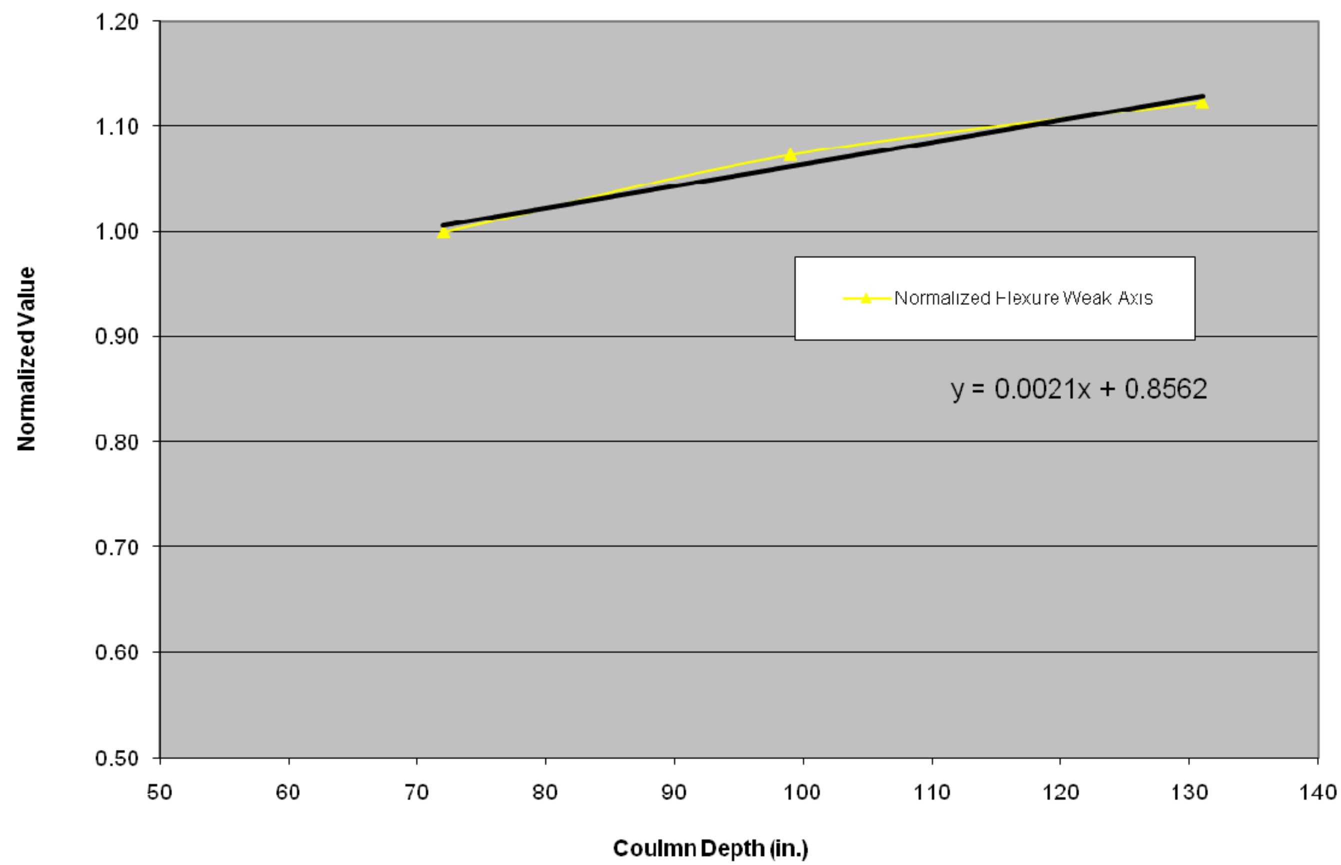


Figure VII.A.42 Normalized Weak Axis Flexural Capacity

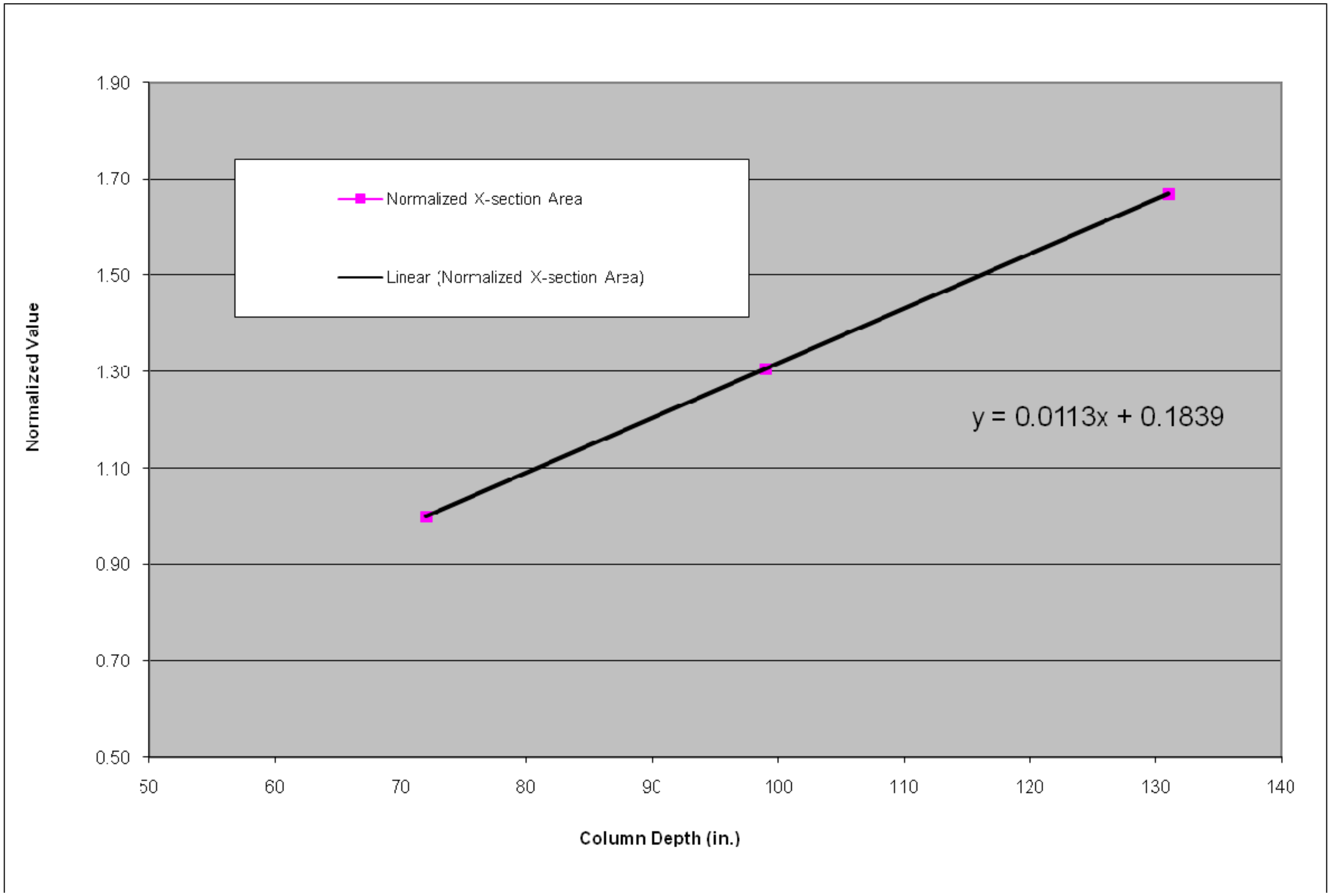


Figure VII.A.43 Normalized X-Section Area

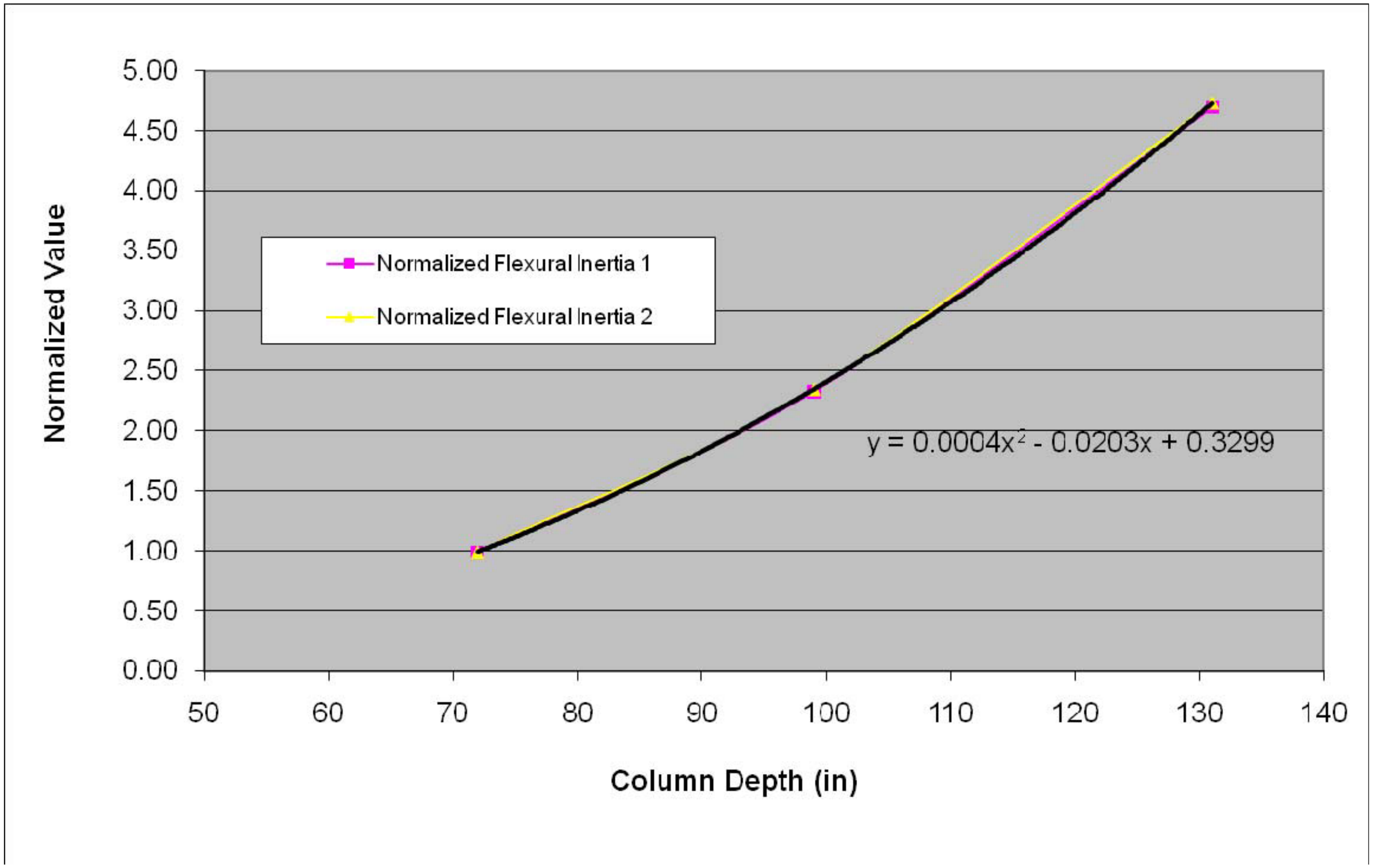


Figure VII.A.44 Normalized Strong Axis Flexural Inertia 1 and 2

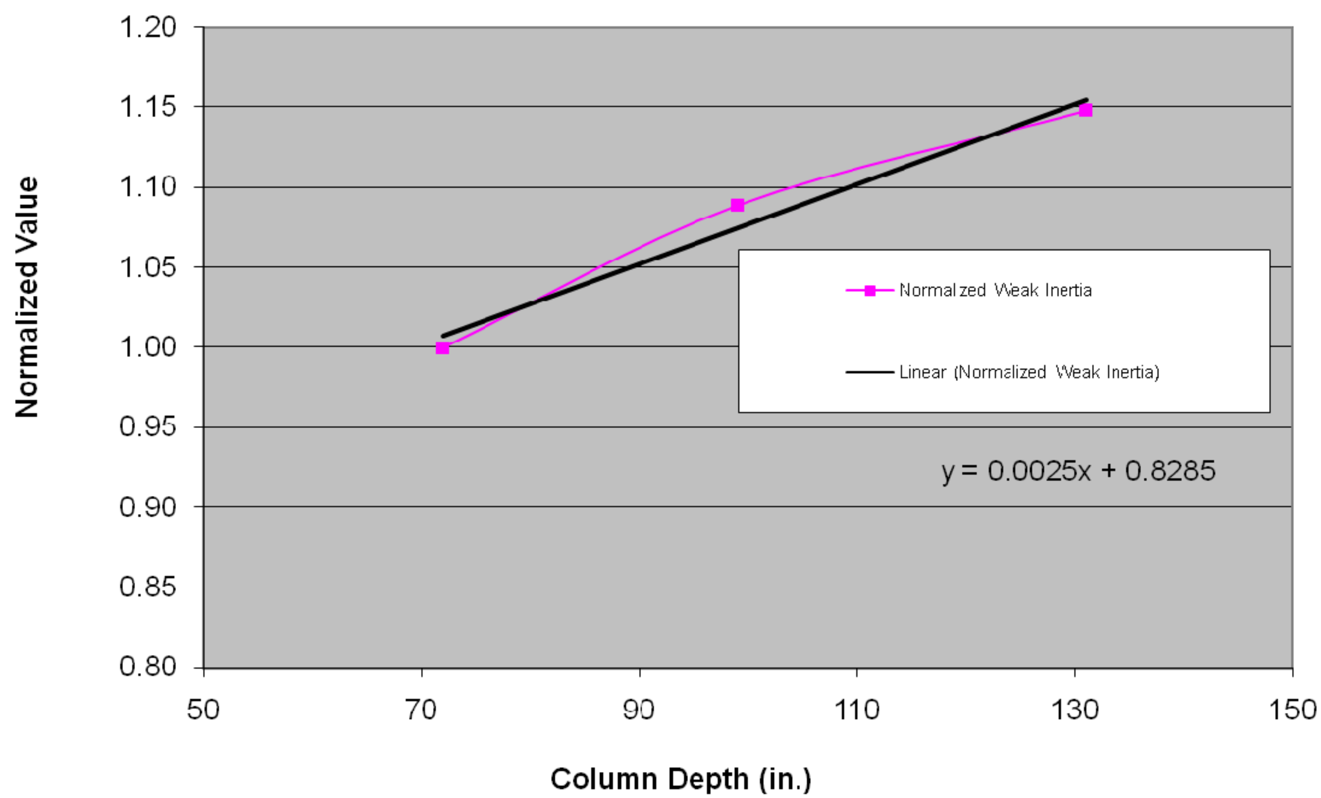


Figure VII.A.45 Normalized Weak Axis Flexural Inertia

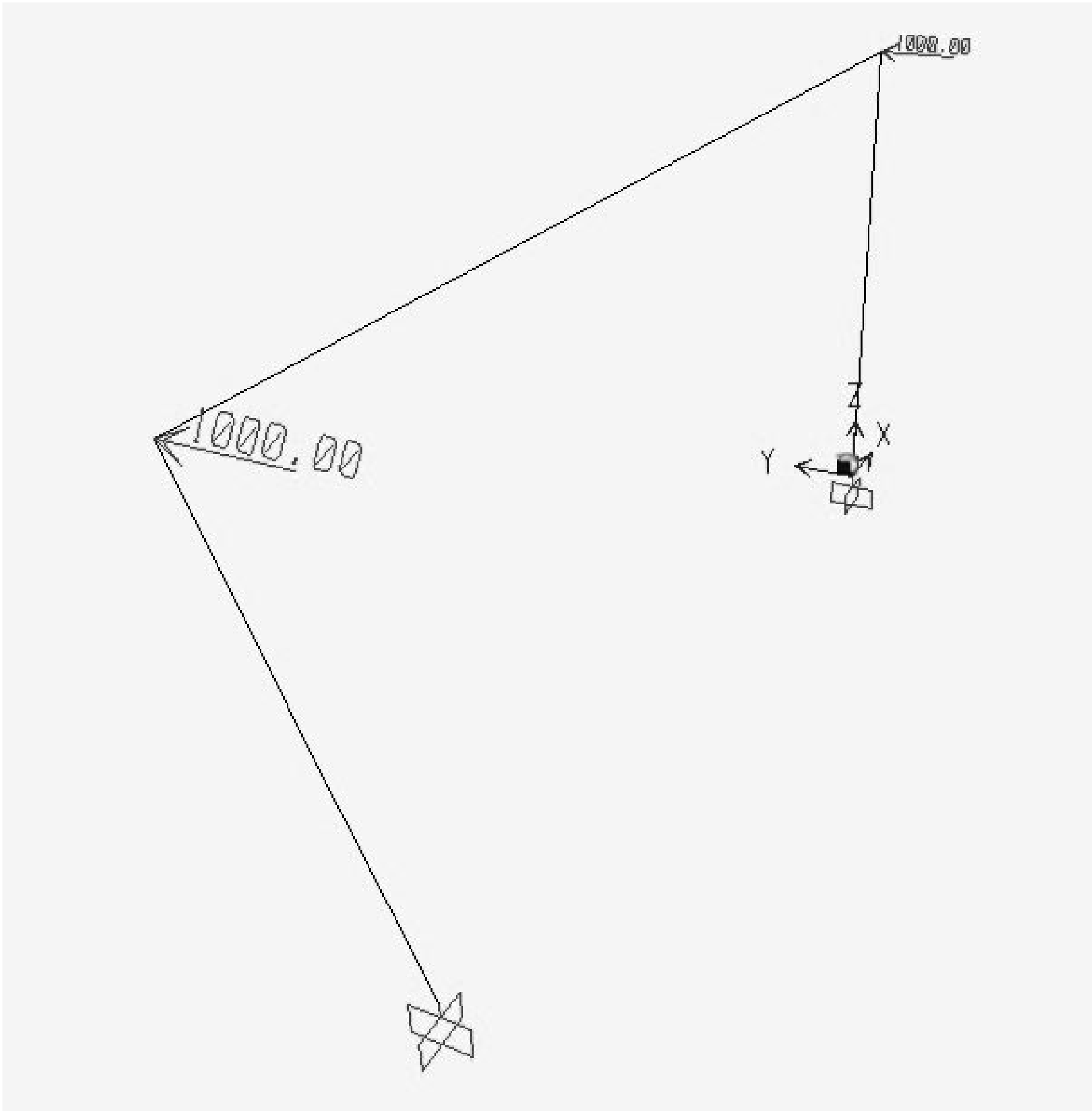


Figure VII.A.46 Pier 3 Longitudinal Loading

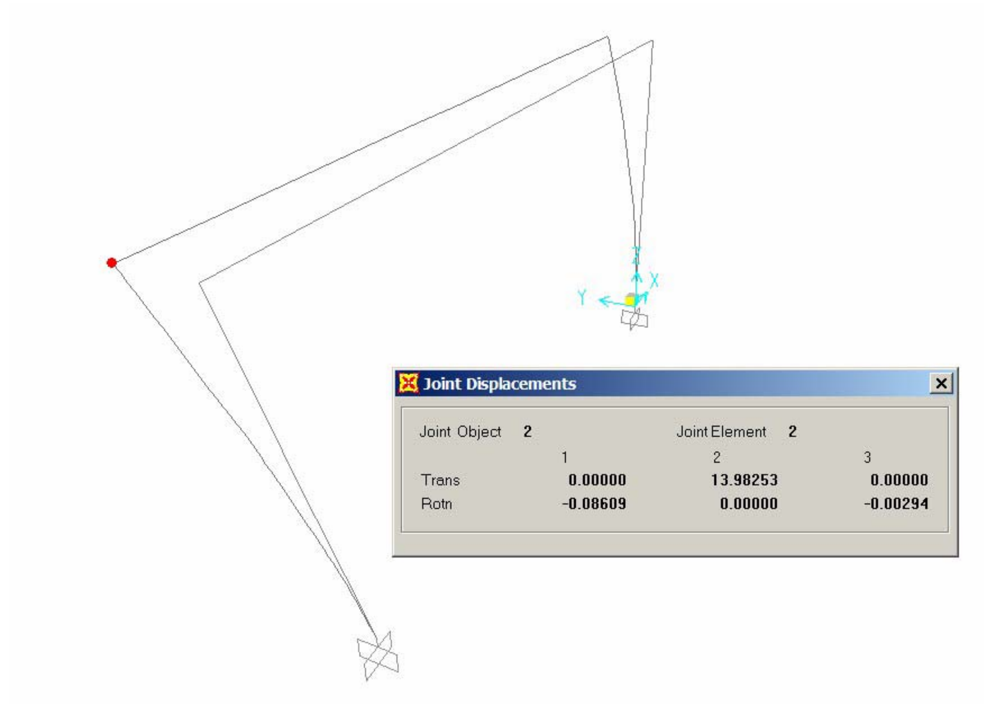


Figure VII.A.47 Pier 3 Outer Column Longitudinal Displacement

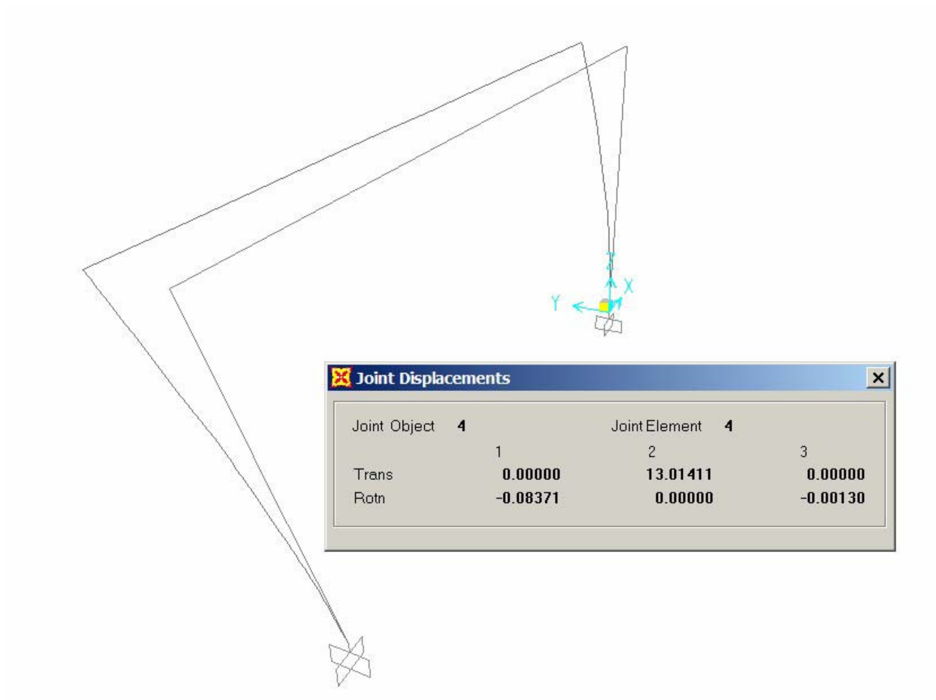


Figure VII.A.48 Pier 3 Inner Column Longitudinal Displacement

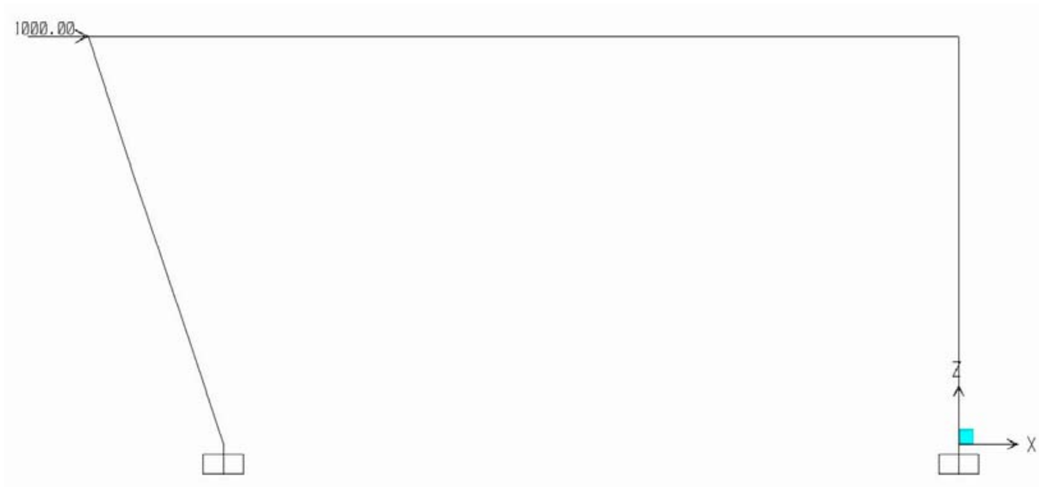


Figure VII.A.49 Pier 3 Transverse Loading Lateral 1

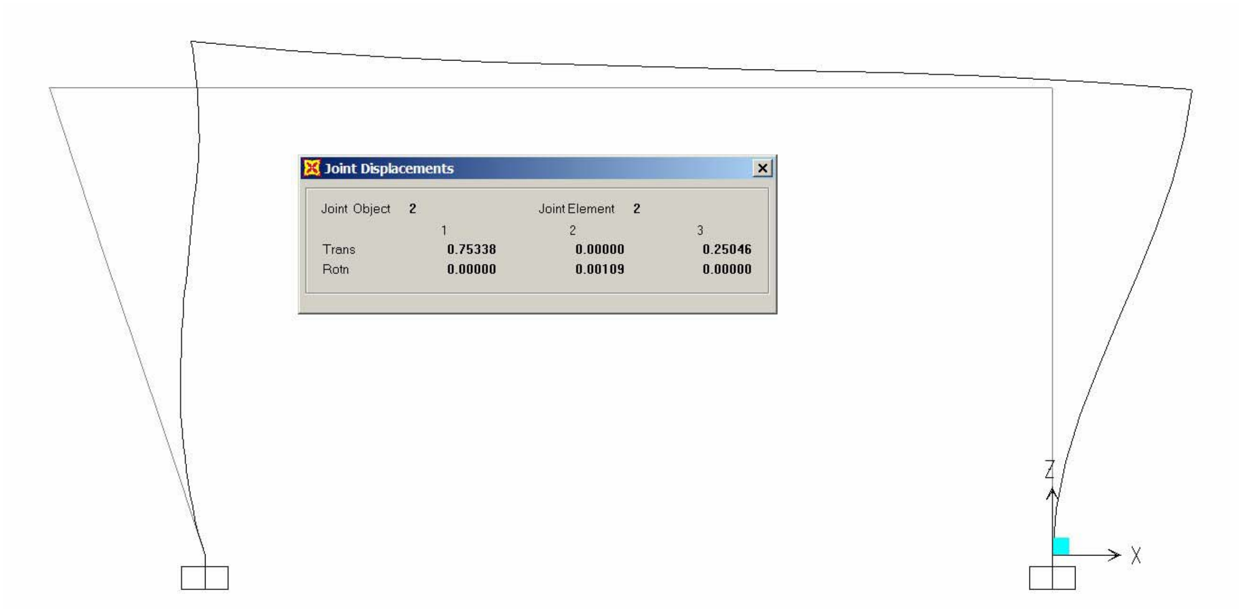


Figure VII.A.50 Pier 3 Transverse Displacement Lateral 1

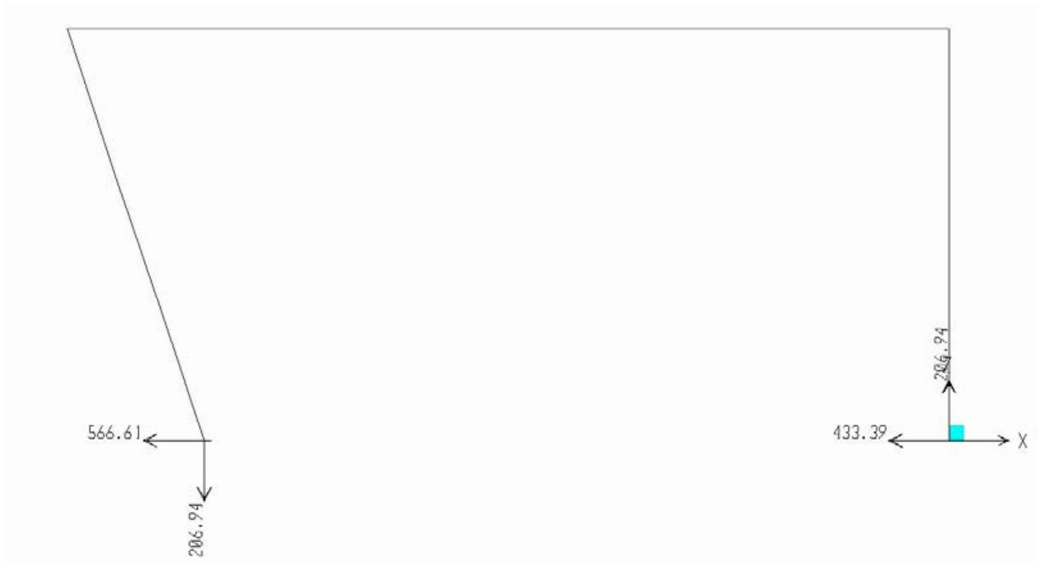


Figure VII.A.51 Pier 3 Transverse Reactions Lateral 1

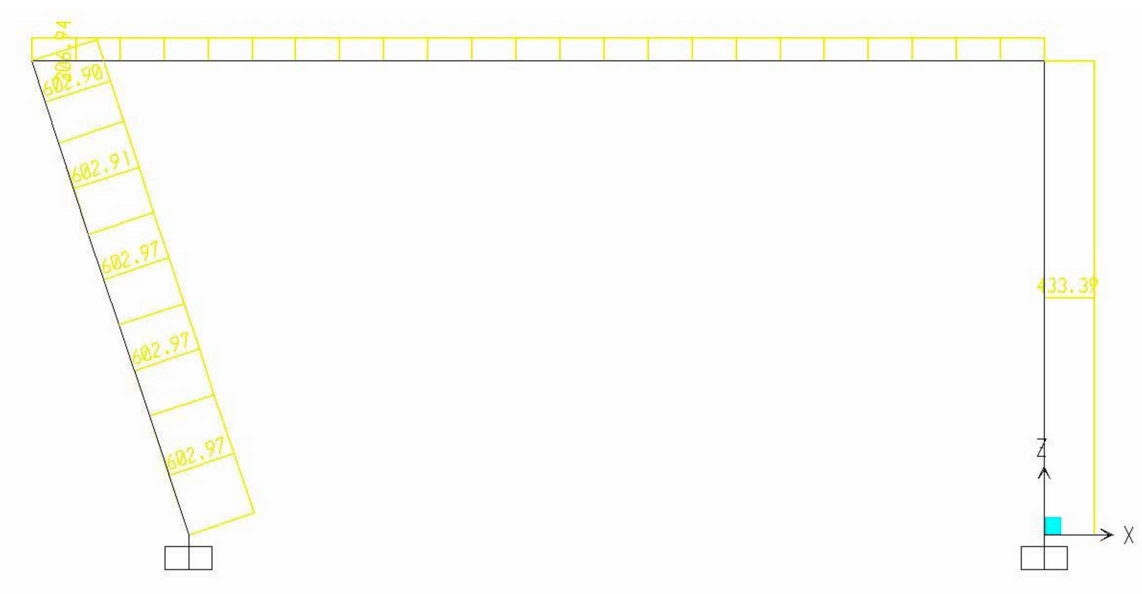


Figure VII.A.52 Pier 3 Transverse Members Shear Lateral 1

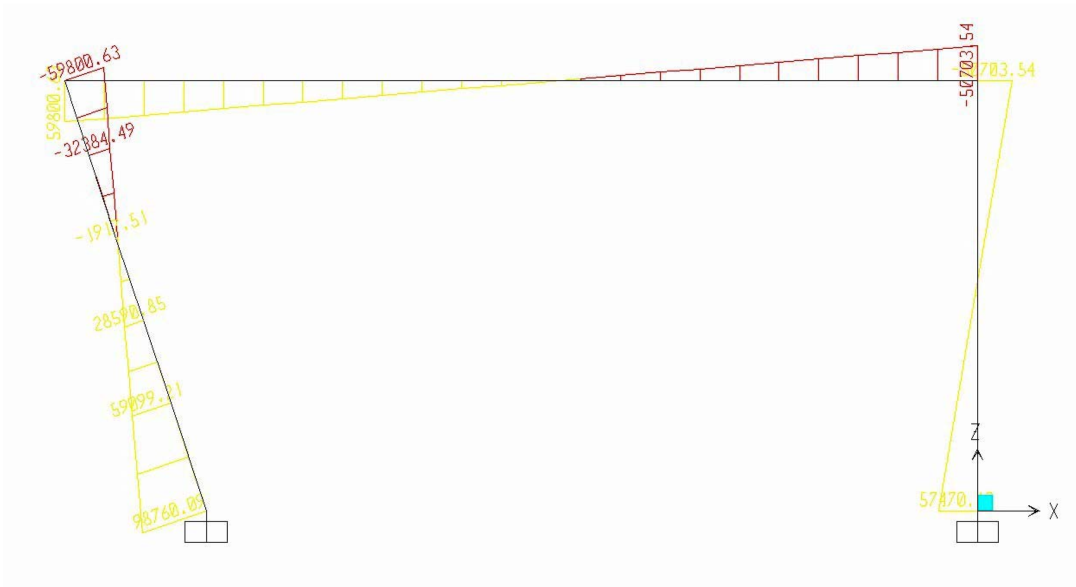


Figure VII.A.53 Pier 3 Transverse Members Moment Lateral 1

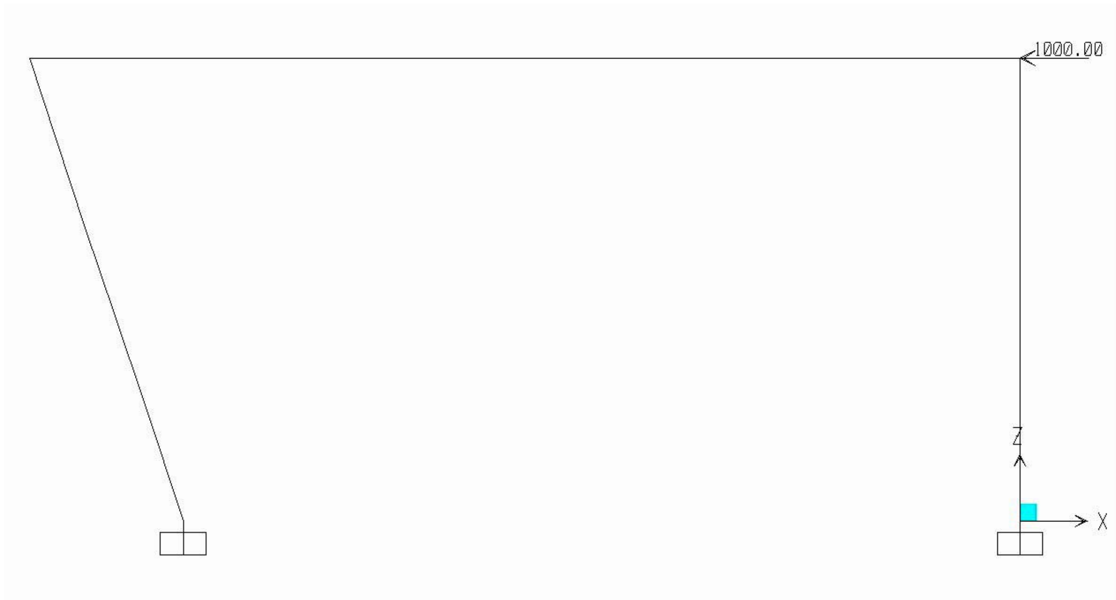


Figure VII.A.54 Pier 3 Transverse Loading Lateral 2

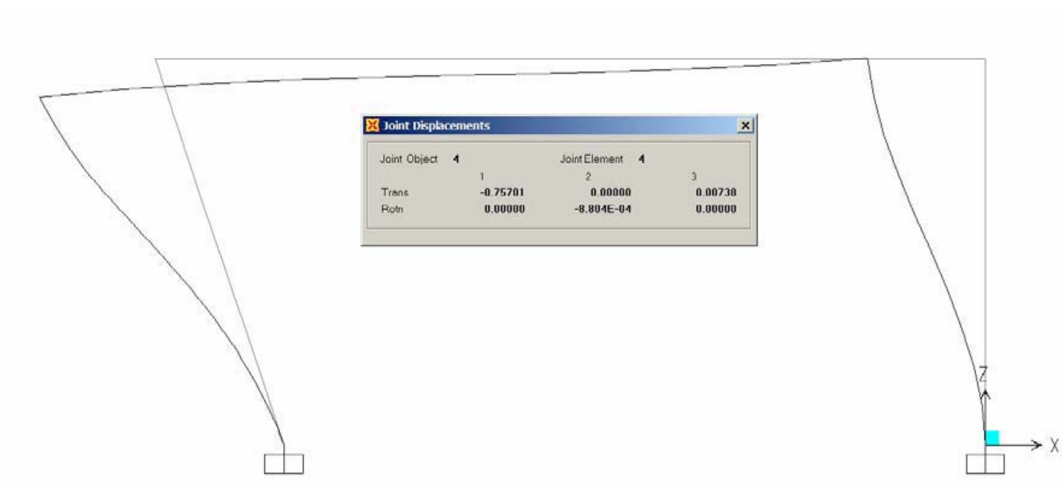


Figure VII.A.55 Pier 3 Transverse Displacement Lateral 2

Figure VII.A.56 Pier 3 Transverse Reactions Lateral 2

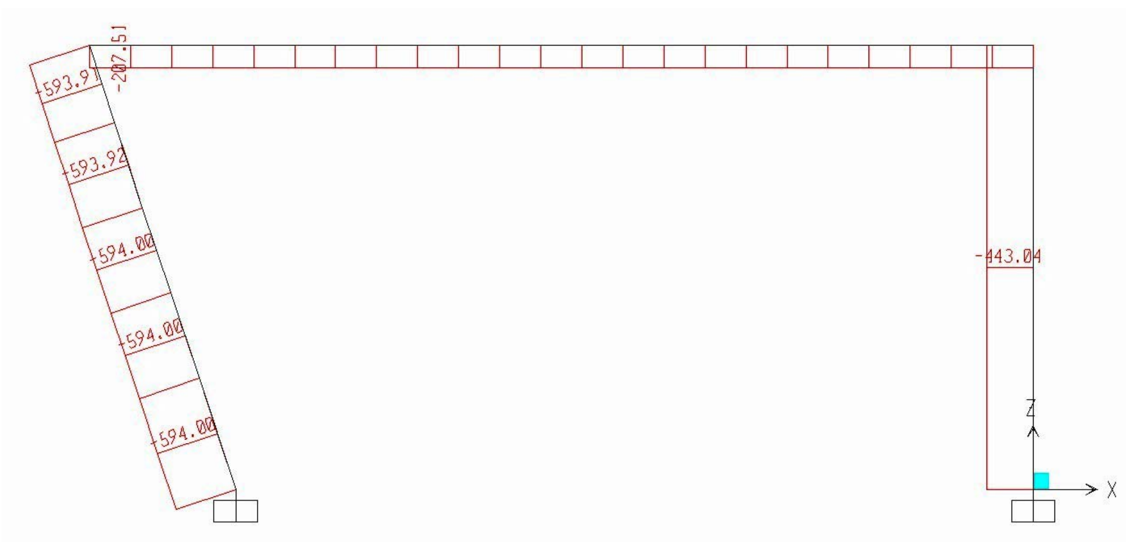
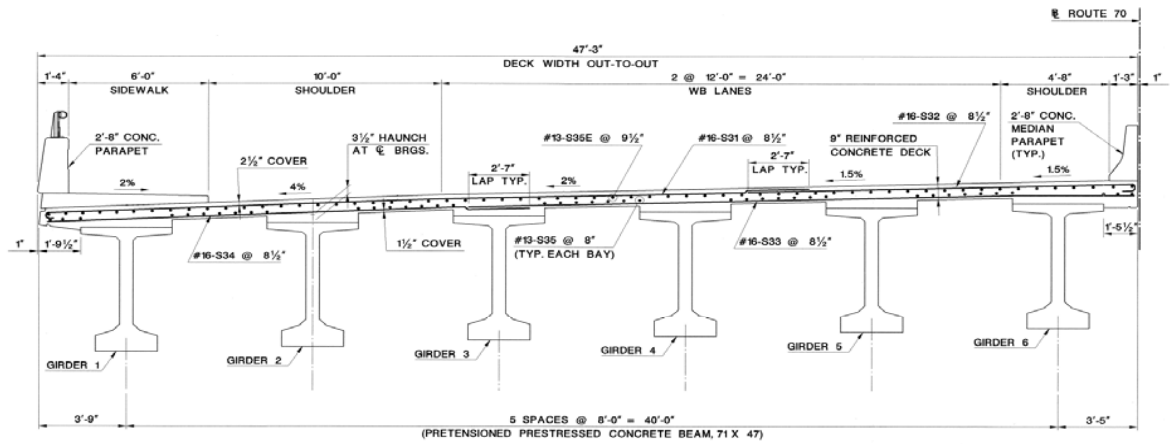


Figure VII.A.57 Pier 3 transverse Members Shear Lateral 2

Figure VII.A.58 Pier 3 Transverse Members Moment Lateral 2



TYPICAL DECK SECTION (ROUTE 70 WESTBOUND)

SCALE: 3/8" = 1'-0"

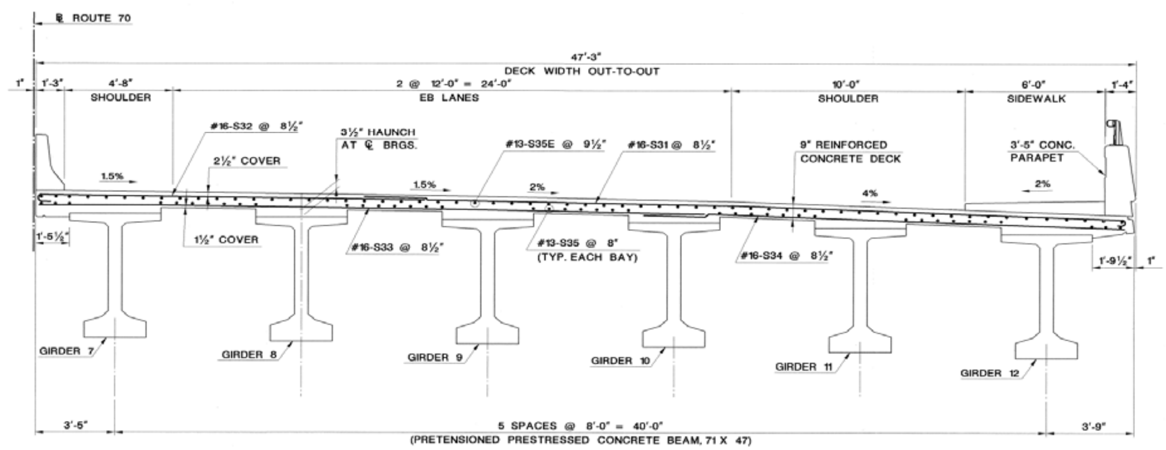


Figure VII.B.1 Typical Deck Section (Route 70 Eastbound)

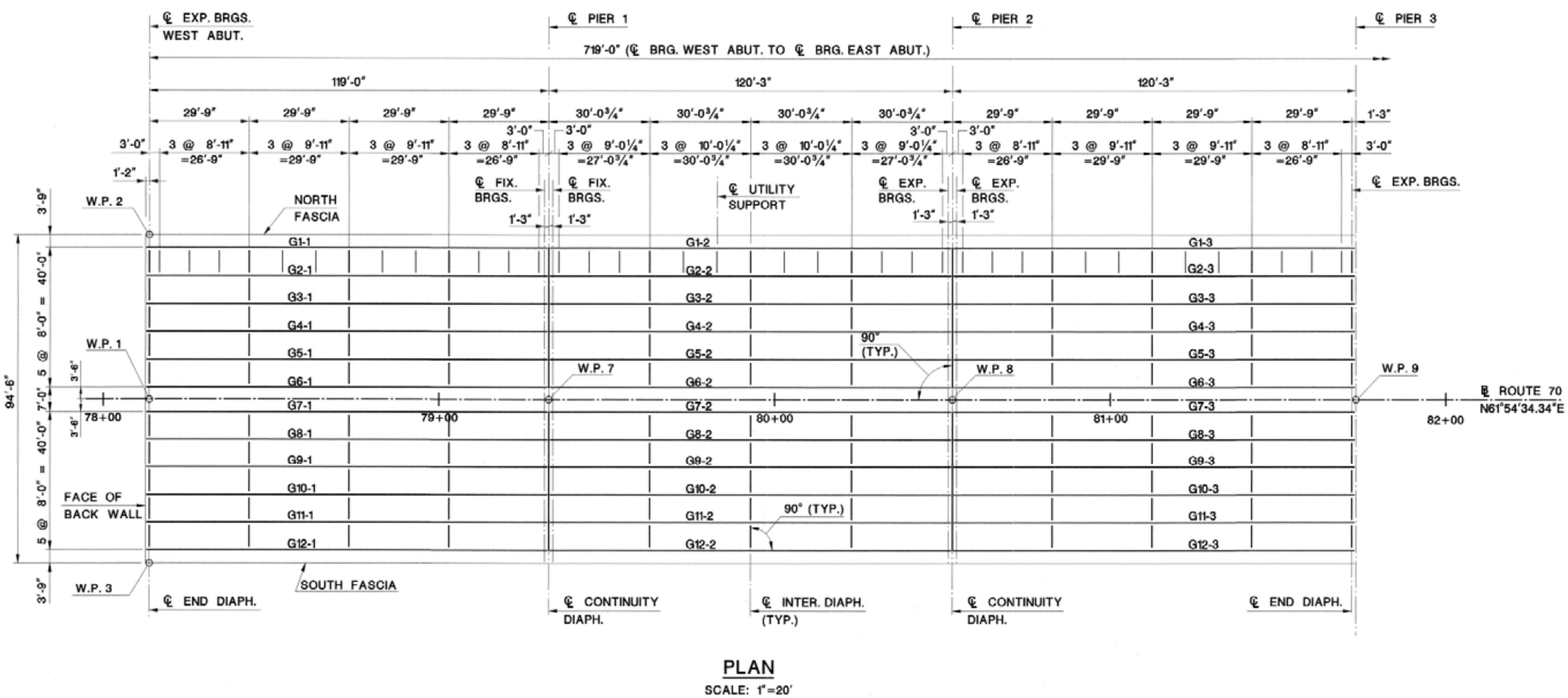
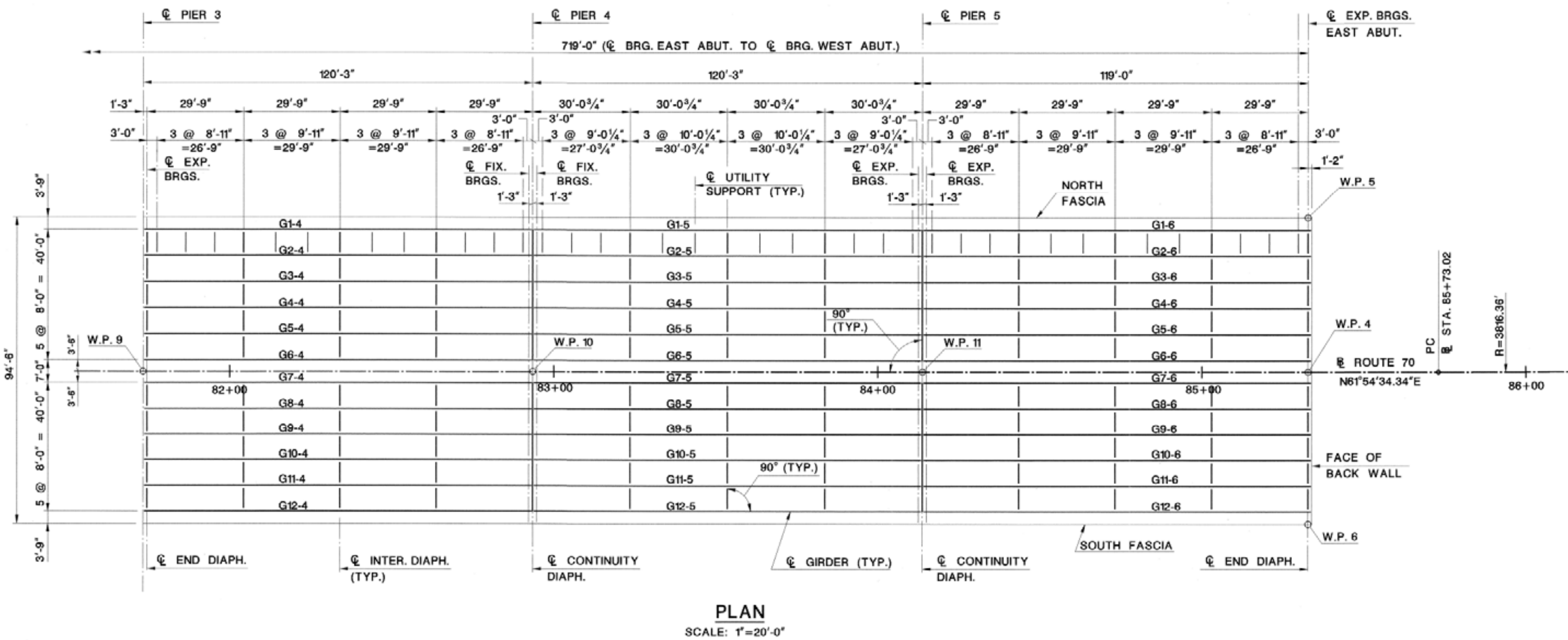
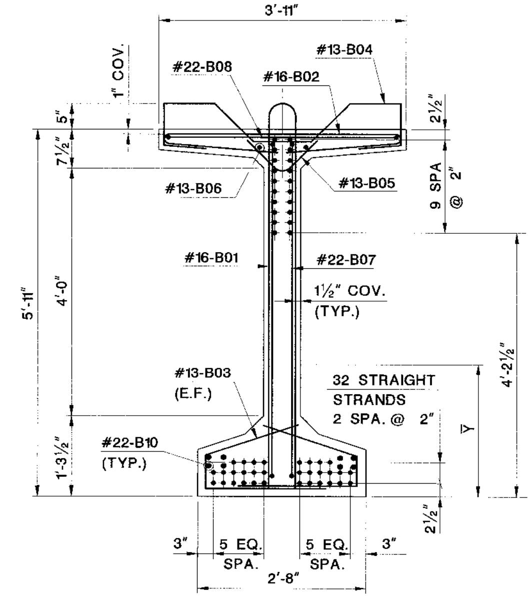


Figure VII.B.2 Framing Plan -1



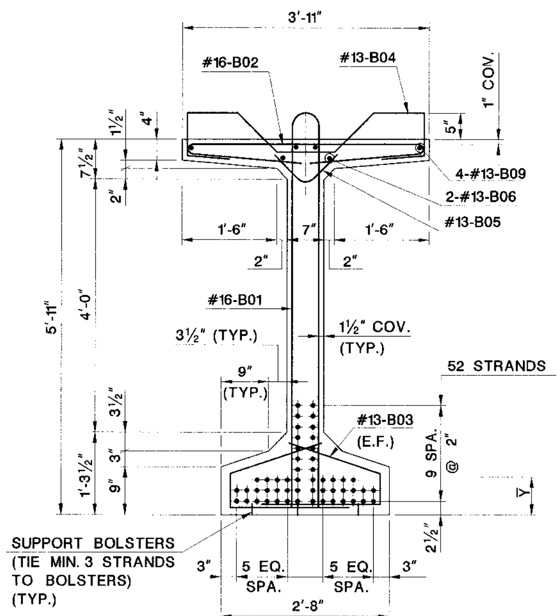
PLAN
SCALE: 1"=20'-0"

Figure VII.B.3 Framing Plan-2



VIEW A-A

SCALE: ¾"=1'-0"



VIEW B-B

SCALE: ¾"=1'-0"

Figure VII.B.4 Girder Cross Section

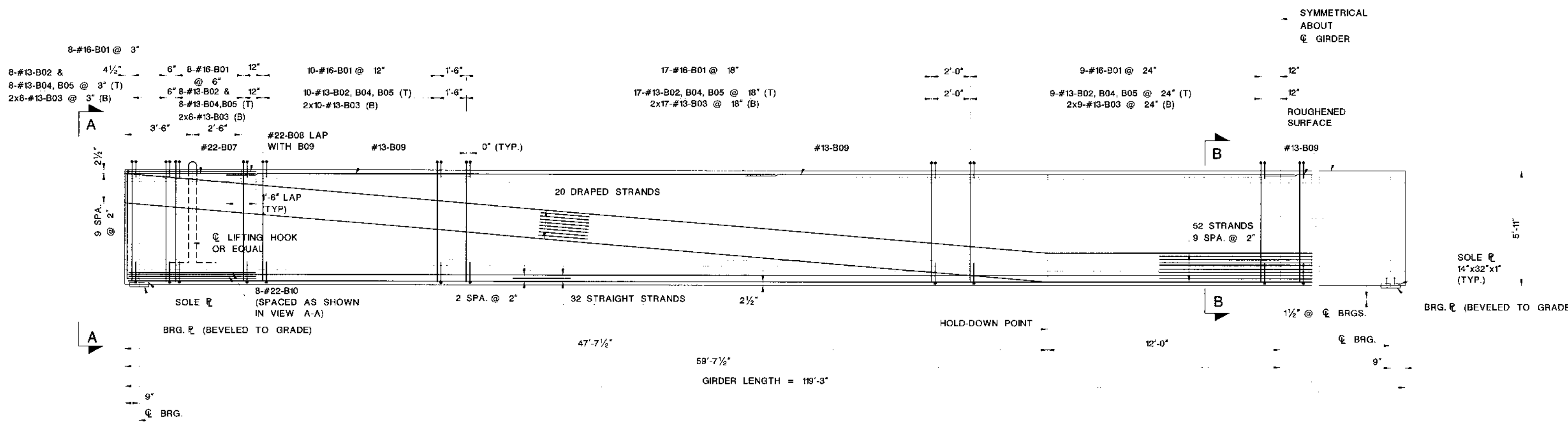


Figure VII.B.5 Girder Plan

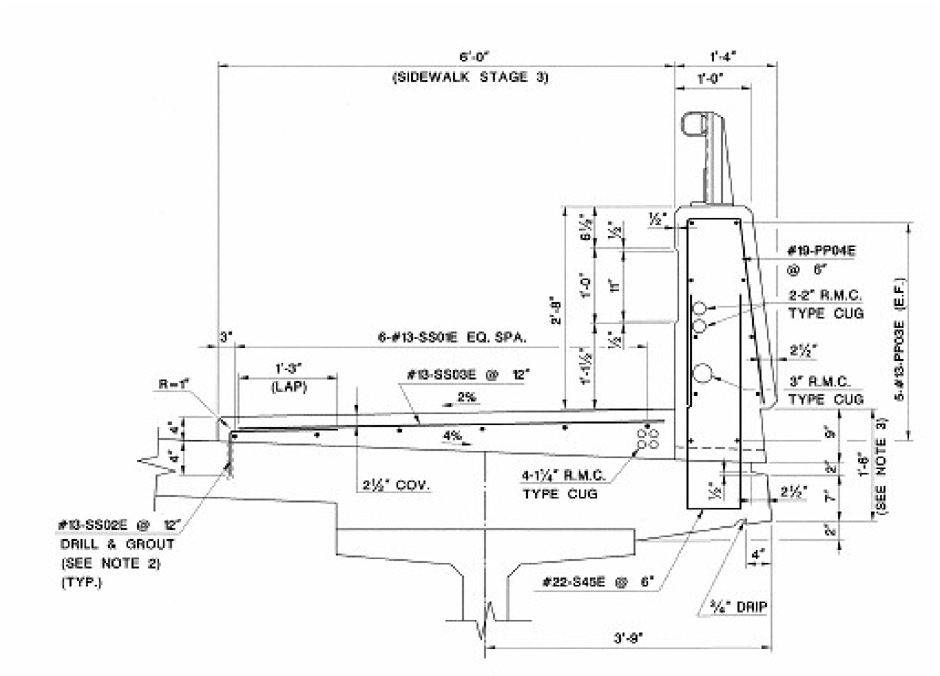


Figure VII.B.6 3'-5" High Parapet EB Sidewalk (South)

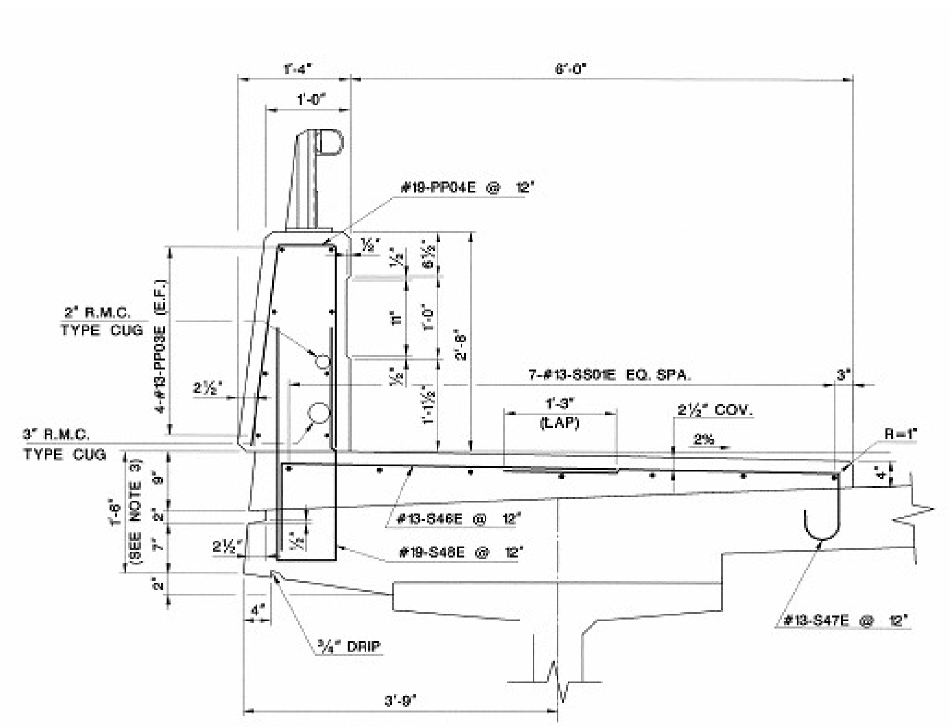


Figure VII.B.7 2'-8" Parapet WB Sidewalk (North)

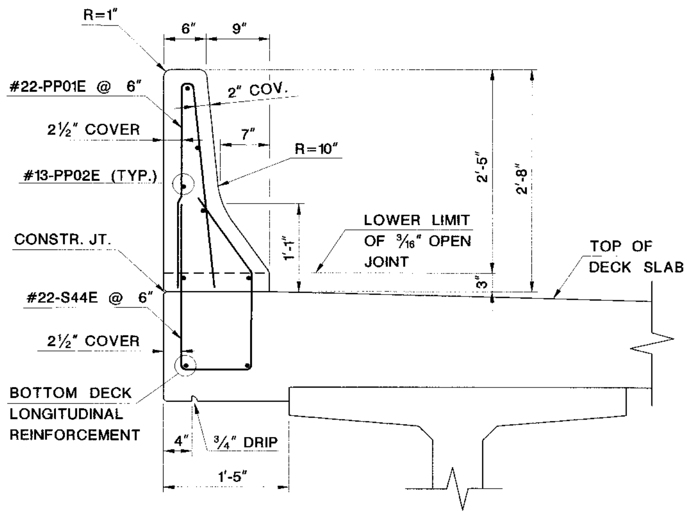
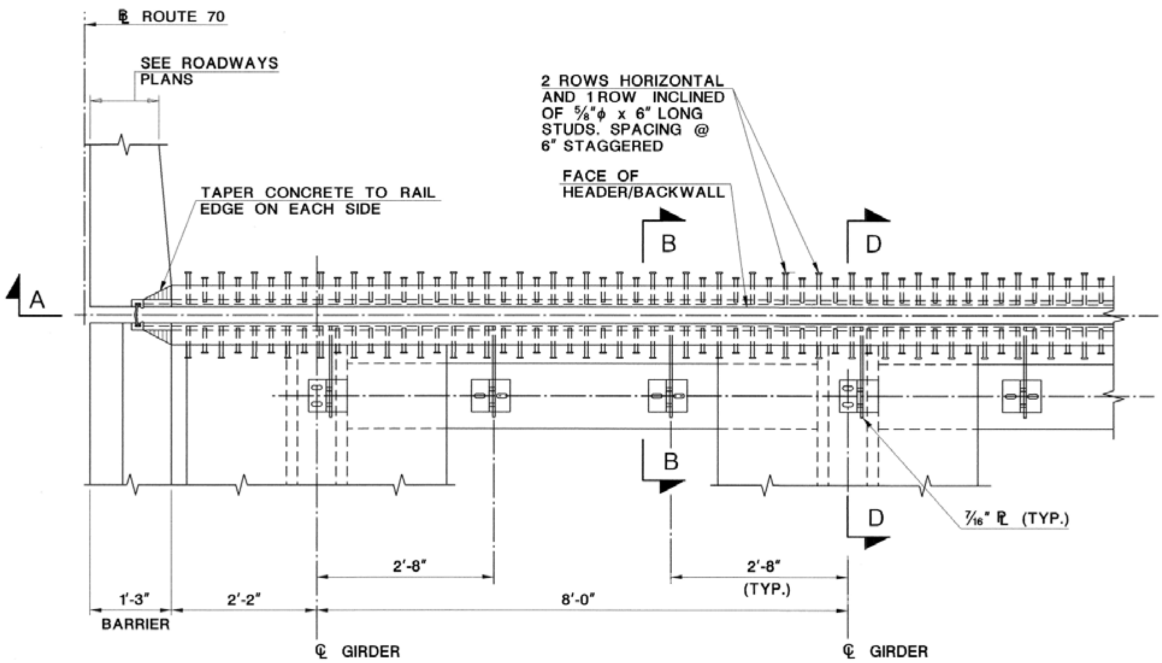


Figure VII.B.8 Typical Median Barrier Section



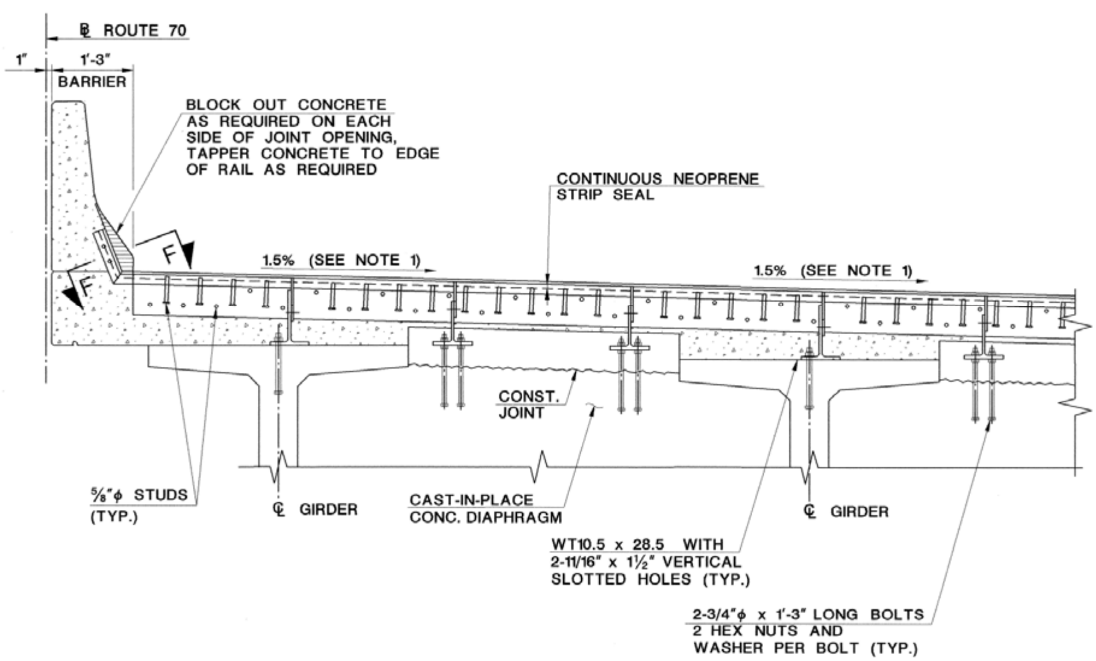
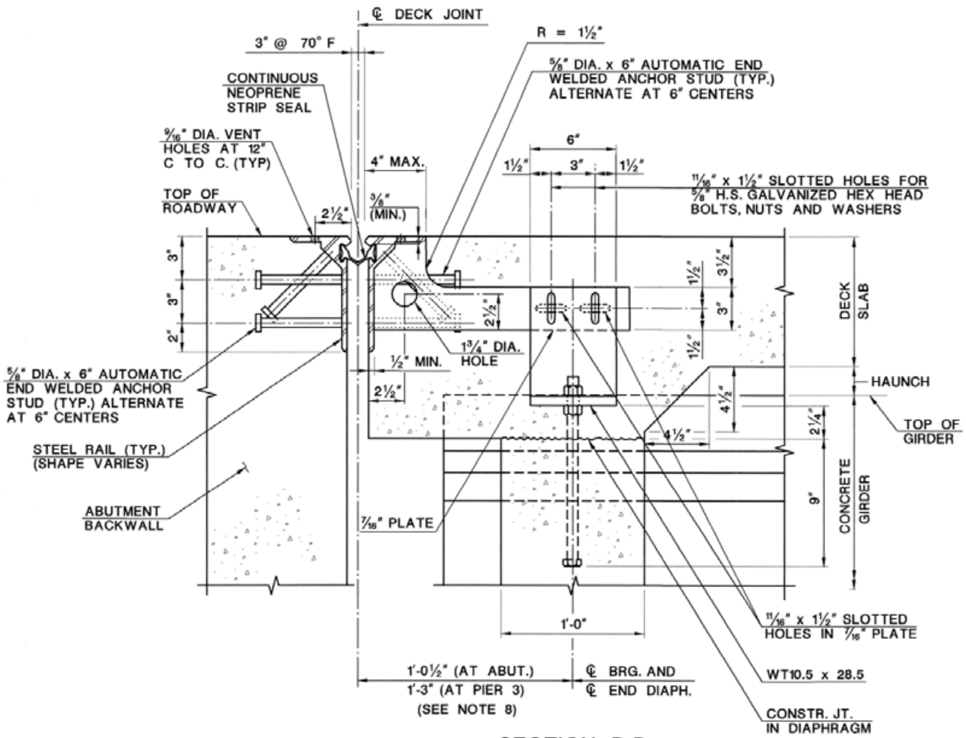


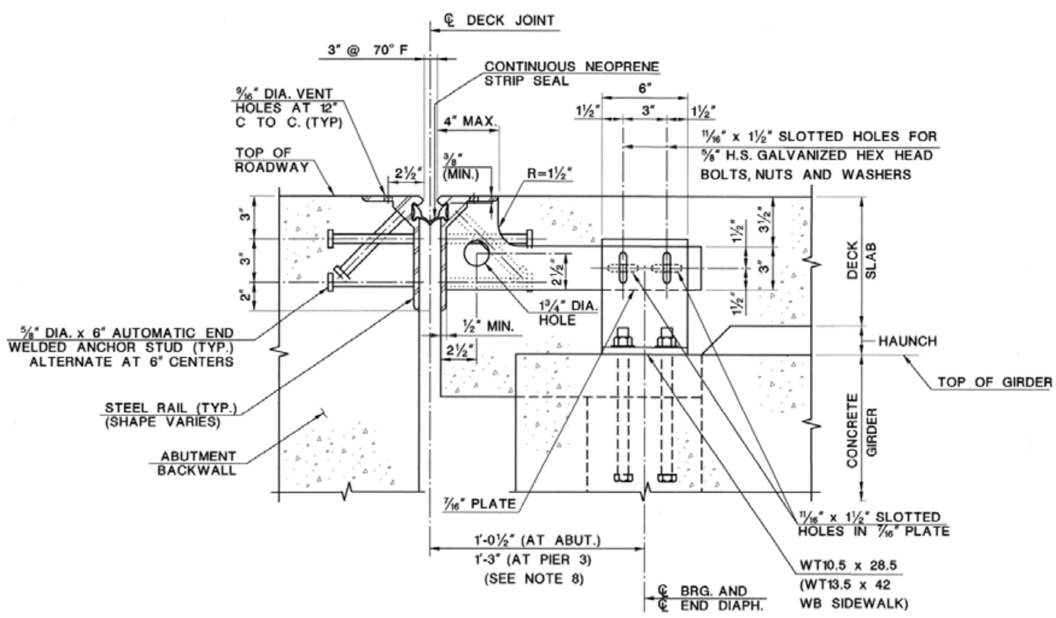
Figure VII.B.9 Expansion Joint – Plan and Elevation



SECTION B-B

(AT END DIAPHRAGM)

SCALE: 3"=1'-0"



SECTION D-D

(AT ℄ GIRDER)

SCALE: 3"=1'-0"

Figure VII.B.10 Expansion Joint Sections at End Diaphragm and Girder

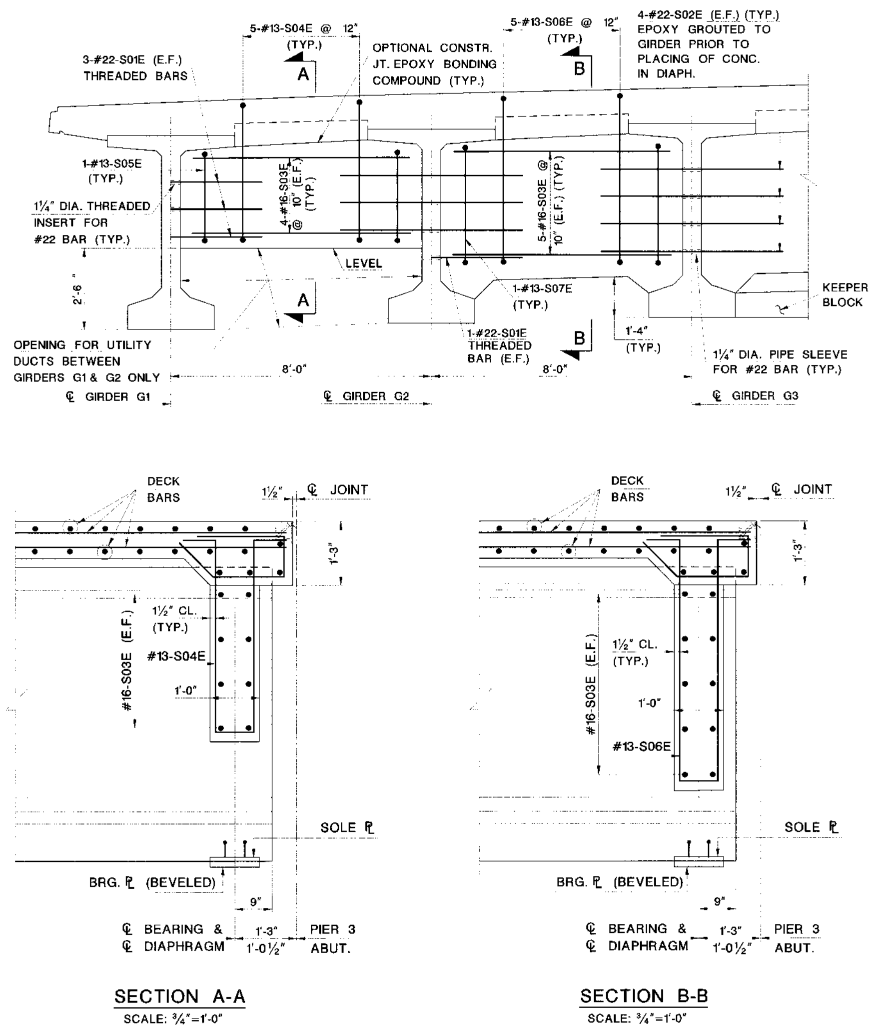


Figure VII.B.11 End Diaphragm at Pier 3 and Abutments

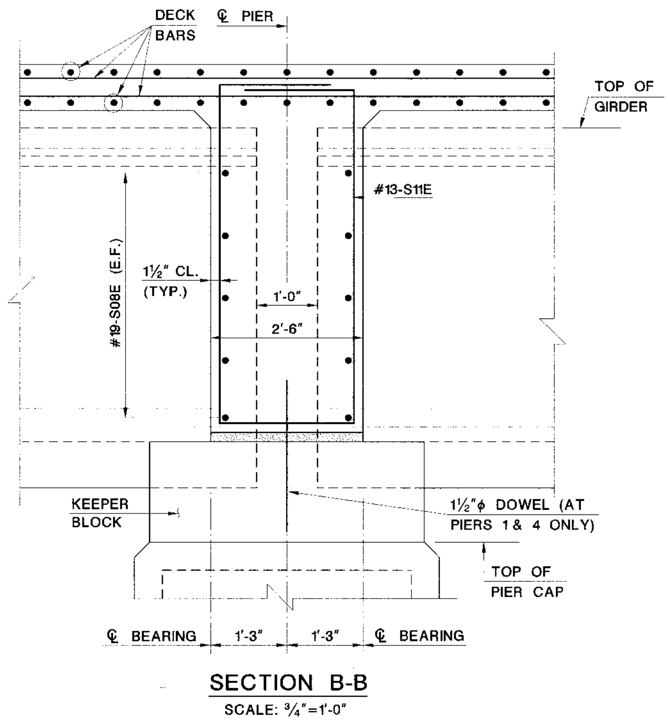


Figure VII.B.12 Diaphragm at Piers 1 and 4

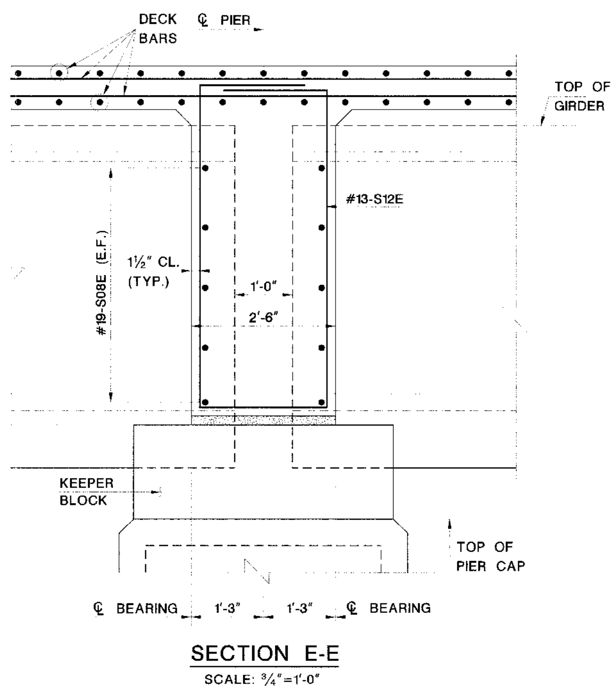


Figure VII.B.13 Diaphragm at Piers 2 and 5

VII.C: Substructure Details

PIER	DIMENSIONS						
	A	B	C	D	E	F	G
1 WB & EB	21'-1⅛"	9'-11⅞"	2'-8"	24'-1¾"	6'-11⅜"	25'-9⅜"	5'-3⅜"
2 WB & EB	19'-6⅛"	10'-8⅝"	3'-6⅜"	22'-6¾"	7'-8⅛"	24'-2¼"	6'-0⅛"
3 WB & EB	18'-8¼"	11'-1⅛"	3'-11⅝"	21'-8⅞"	8'-0⅝"	23'-4⅜"	6'-4⅝"
4 WB & EB	18'-7⅝"	11'-1⅜"	4'-0"	21'-8¼"	8'-1"	23'-3¾"	6'-4⅞"
5 WB & EB	19'-4¼"	10'-9½"	3'-7⅜"	22'-4⅞"	7'-9"	24'-0⅜"	6'-1"

DIMENSIONS G AND E ARE TAKEN AT BOTTOM OF RECESS, 6" BELOW TOP OF 1ST FOOTING POUR.

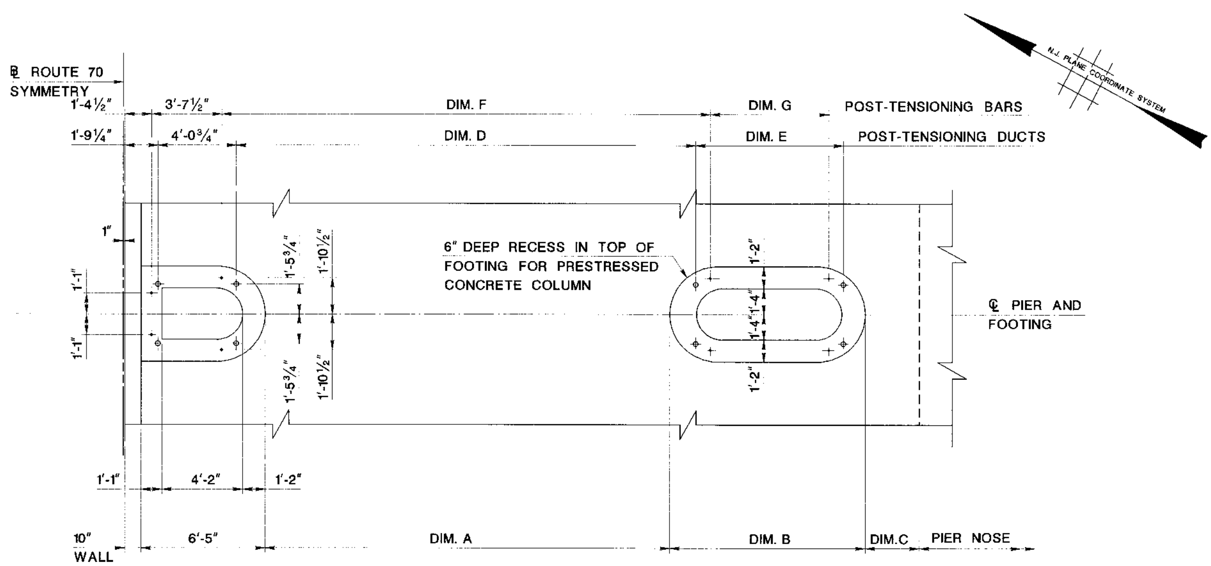


Figure VII.C.1 Pier Post - Tensioning Bars and Ducts Layout

SCALE: 1/4"=1'-0"

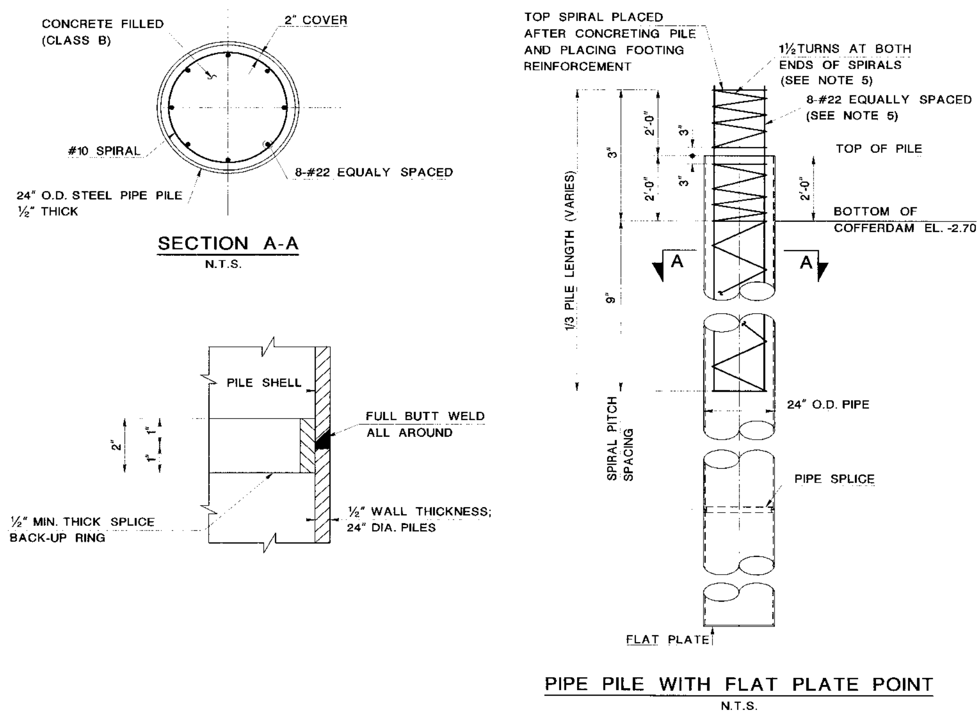
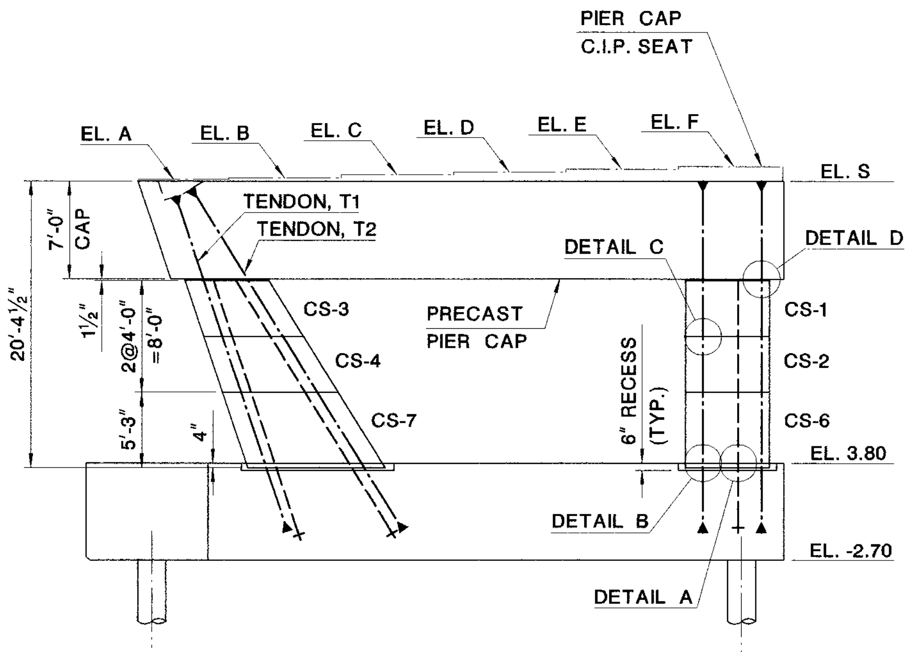
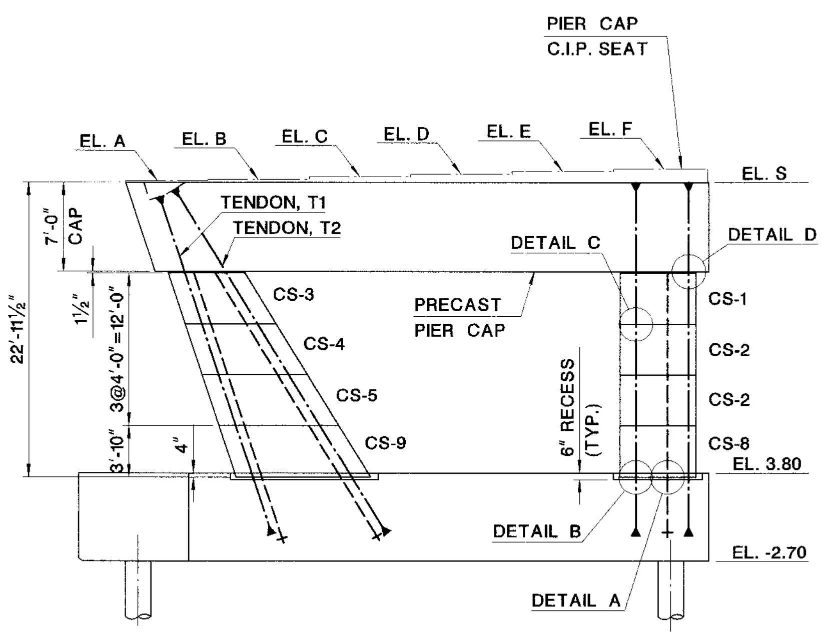


Figure VII.C.2 Pipe Pile Details



PIER 1

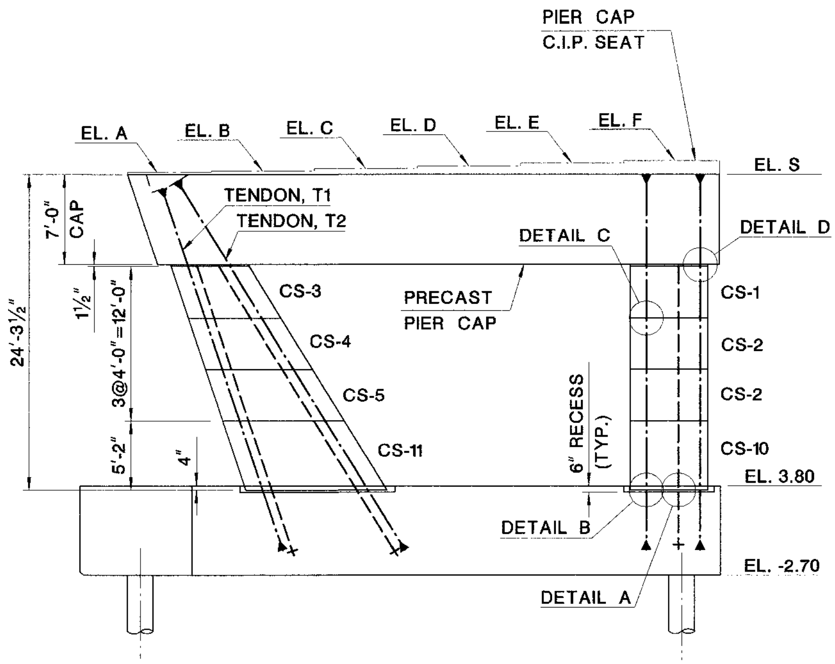
SCALE: ⅛"=1'-0"



PIER 2

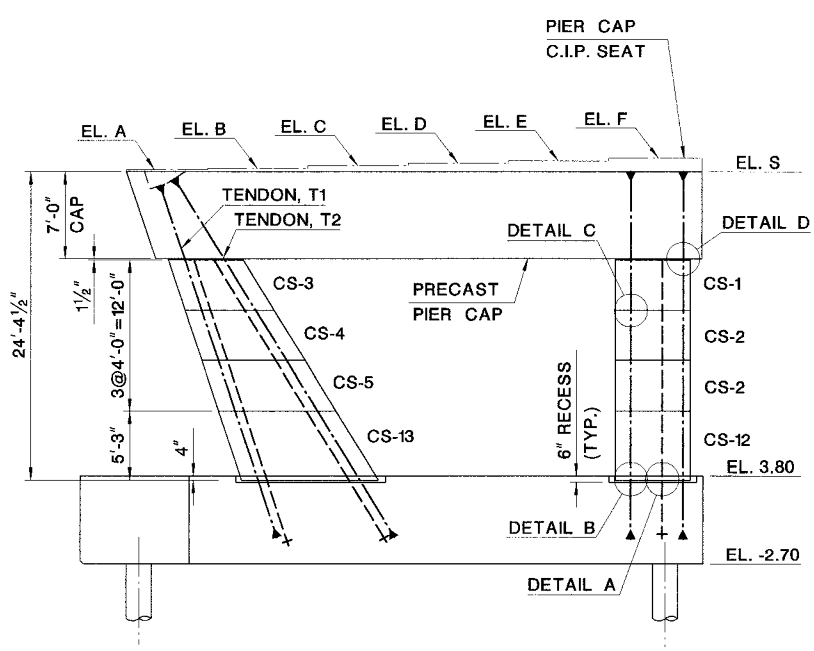
SCALE: ⅛"=1'-0"

Figure VII.C.3 Piers 1 and 2 Precast Segment Elevation



PIER 3

SCALE: ⅛"=1'-0"



PIER 4

SCALE: ⅛"=1'-0"

Figure VII.C.4 Piers 3 and 4 Precast Segment Elevation

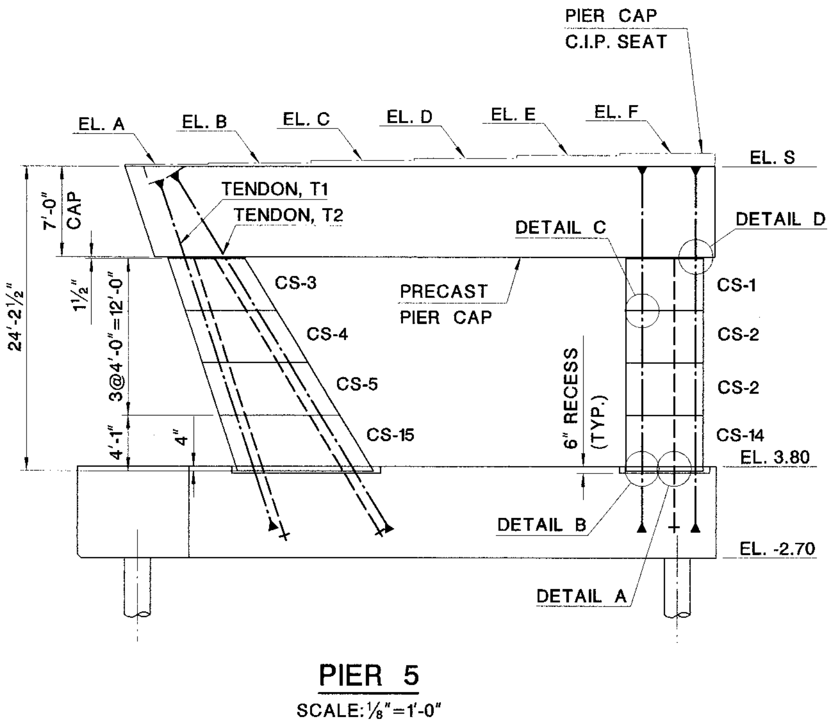


Figure VII.C.5 Pier 5 Precast Segment Elevation

PIER NO.	BEARING SEAT ELEVATIONS						EL. S
	A	B	C	D	E	F	
1 EB	24.09	24.41	24.68	24.84	24.96	25.08	23.84
	24.15	24.47	24.74	24.90	25.03	25.15	
2 EB	26.67	26.99	27.26	27.42	27.55	27.67	26.42
	26.71	27.03	27.30	27.46	27.59	27.71	
3 EB	28.01	28.33	28.60	28.76	28.88	29.00	27.76
	28.02	28.34	28.61	28.77	28.90	29.02	
4 EB	28.09	28.41	28.68	28.84	28.97	29.09	27.83
	28.08	28.40	28.67	28.83	28.95	29.07	
5 EB	26.95	27.27	27.53	27.67	27.79	27.91	26.66
	26.91	27.23	27.50	27.63	27.75	27.87	
1 WB	24.09	24.41	24.68	24.84	24.96	25.08	23.84
	24.15	24.47	24.74	24.90	25.03	25.15	
2 WB	26.67	26.99	27.26	27.42	27.55	27.67	26.42
	26.71	27.03	27.30	27.46	27.59	27.71	
3 WB	28.01	28.33	28.60	28.76	28.88	29.00	27.76
	28.02	28.34	28.61	28.77	28.90	29.02	
4 WB	28.09	28.41	28.68	28.84	28.97	29.09	27.83
	28.08	28.40	28.67	28.83	28.95	29.07	
5 WB	26.91	27.23	27.50	27.66	27.79	27.91	26.63
	26.88	27.20	27.47	27.63	27.75	27.87	

Figure VII.C.6 Cap and Bearing Seat Elevation

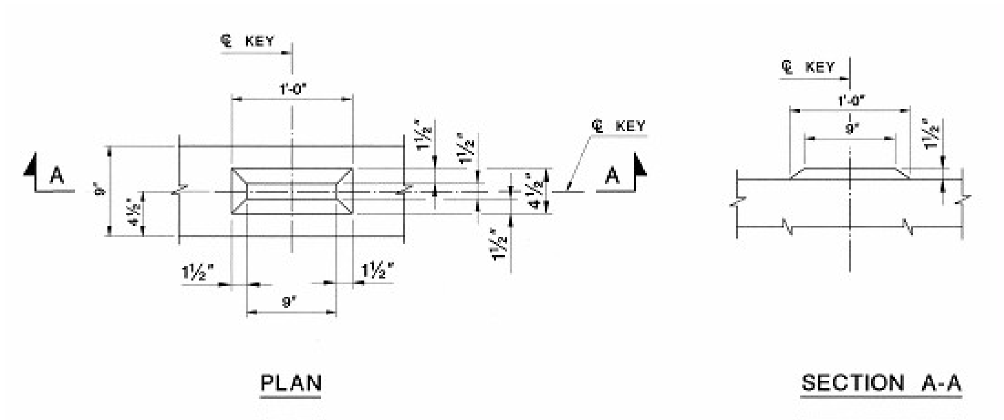


Figure VII.C.7 Typical Shear Key Detail

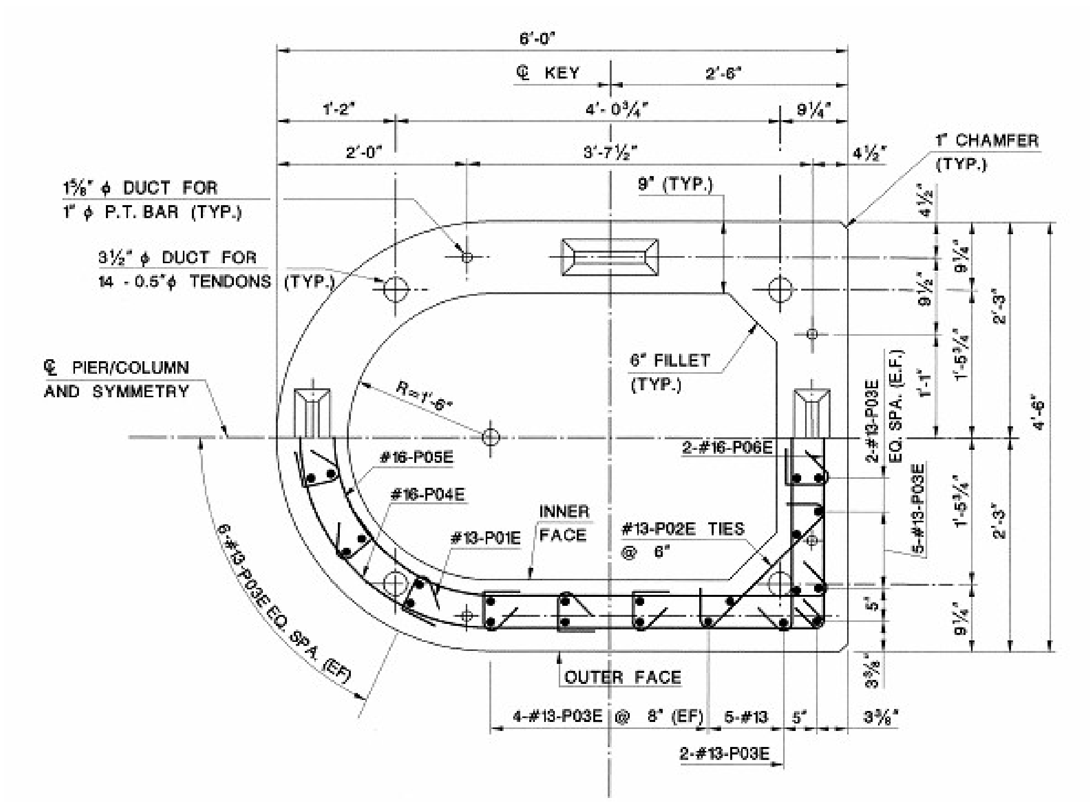


Figure VII.C.8 Inner Column Section

PRECAST COLUMN SEGMENT	DIM. C	NO. OF ROWS N	CONCRETE VOLUME (C.Y.)	WEIGHT OF REINFORCEMENT (LB.)
CS-1	4′-0″	8	1.60	540
CS-2	4′-0″	8	1.60	540
CS-6	5′-3″	10	2.14	685
CS-8	3′-10″	8	1.57	535
CS-10	5′-2″	10	2.10	680
CS-12	5′-3″	11	2.14	740
CS-14	4′-1″	8	1.67	540

QUANTITIES SHOWN ARE FOR EACH TYPICAL VERTICAL COLUMN SEGMENT

Figure VII.C.9 Inner Column Precast Segment Schedule

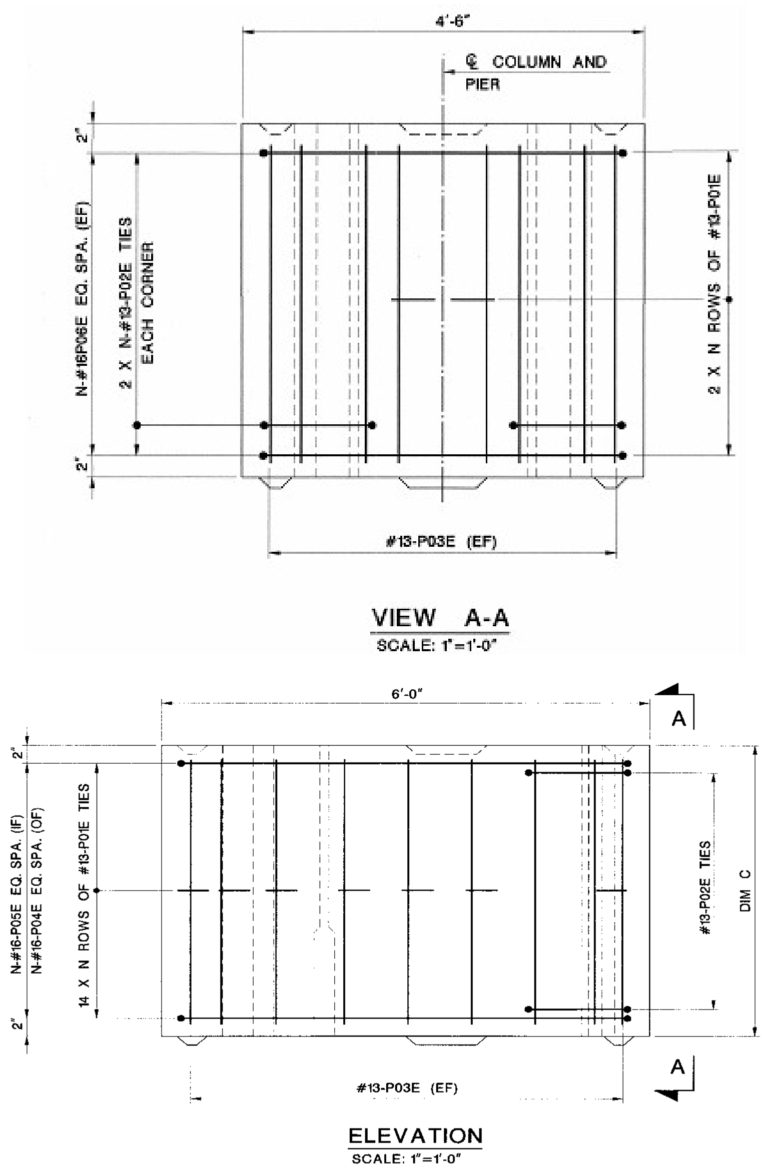


Figure VII.C.10 Inner Column Elevation and View

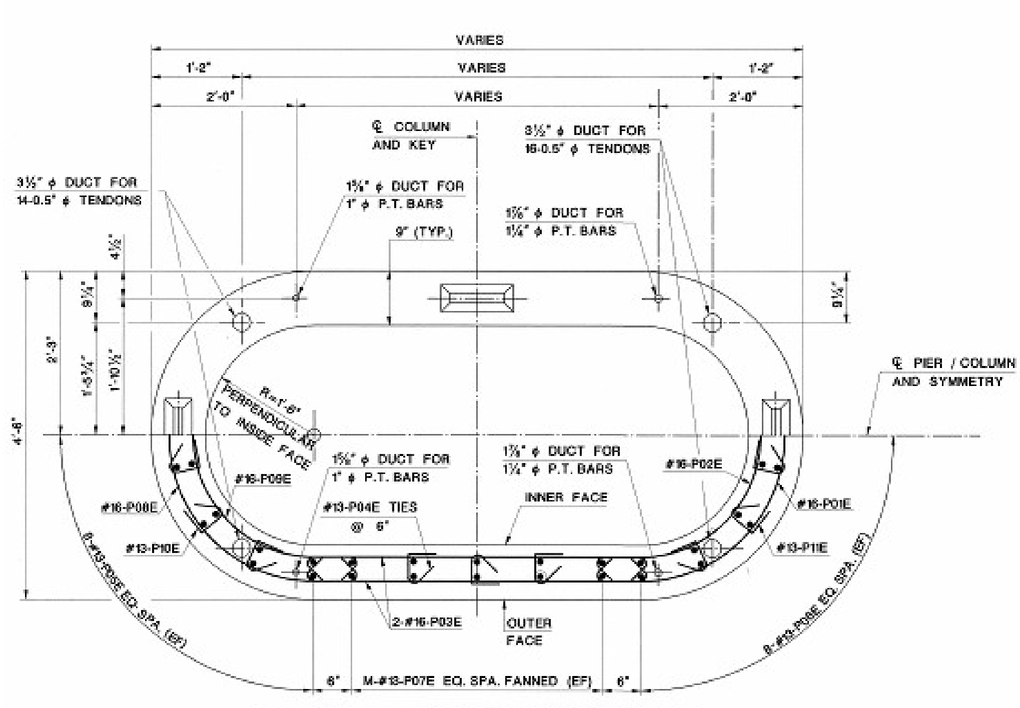
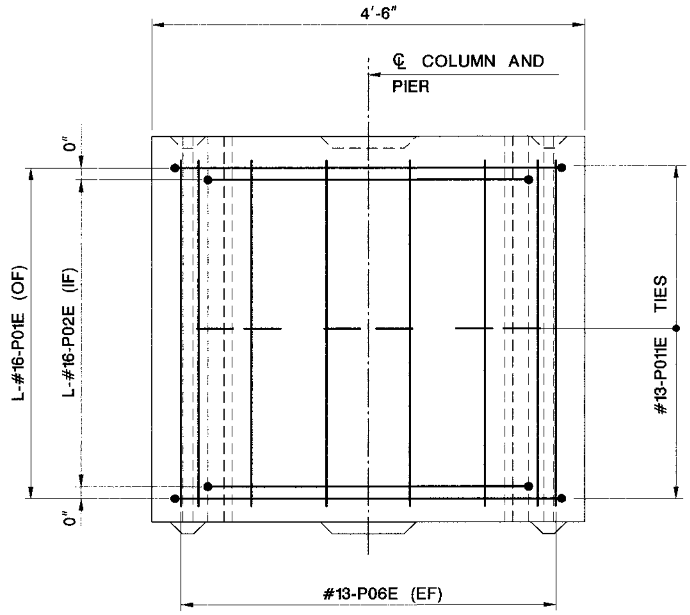


Figure VII.C.11 Outer Column Section

PRECAST COLUMN SEGMENT	DIMENSION			NO. OF ROWS/BARS			CONC. VOLUME (C.Y.)	WEIGHT OF REINF. (LB.)
	A	B	C	K	L	M		
CS-3	6'-0"	7'-1½"	4'-0"	8	8	3	1.80	565
CS-4	7'-1½"	8'-3"	4'-0"	48	8	5	2.05	680
CS-5	8'-3"	9'-4⅝"	4'-0"	80	8	7	2.30	765
CS-7	8'-3"	9'- 8⅞"	5'-3"	132	11	8	3.12	1050
CS-9	9'-4⅝"	10'-5½"	3'-10"	112	8	9	2.50	815
CS-11	9'-4⅝"	10'-10⅛"	5'-2"	176	11	10	3.40	1175
CS-13	9'-4⅝"	10'-10⅜"	5'-3"	176	11	10	3.46	1165
CS-15	9'-4⅝"	10'-6½"	4'-1"	128	8	10	2.66	850

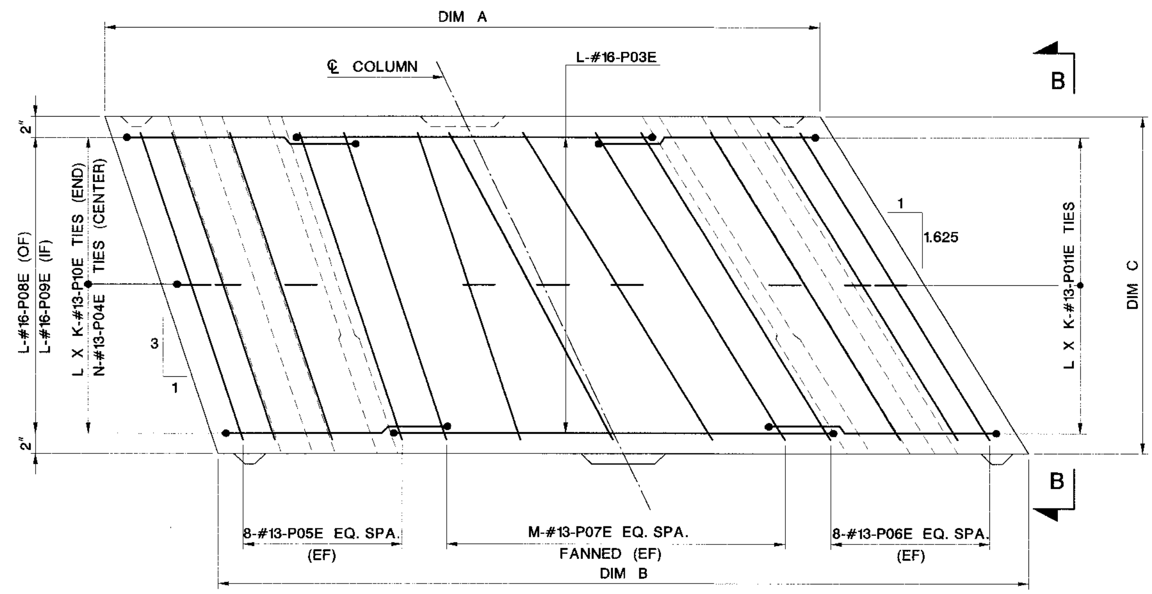
QUANTITIES SHOWN ARE FOR EACH TYPICAL SLOPED COLUMN SEGMENT

Figure VII.C.12 Outer Column Precast Segment Schedule



VIEW B-B

SCALE: 1"=1'-0"



ELEVATION

SCALE: 1"=1'-0"

Figure VII.C.13 Outer Column Elevation and View

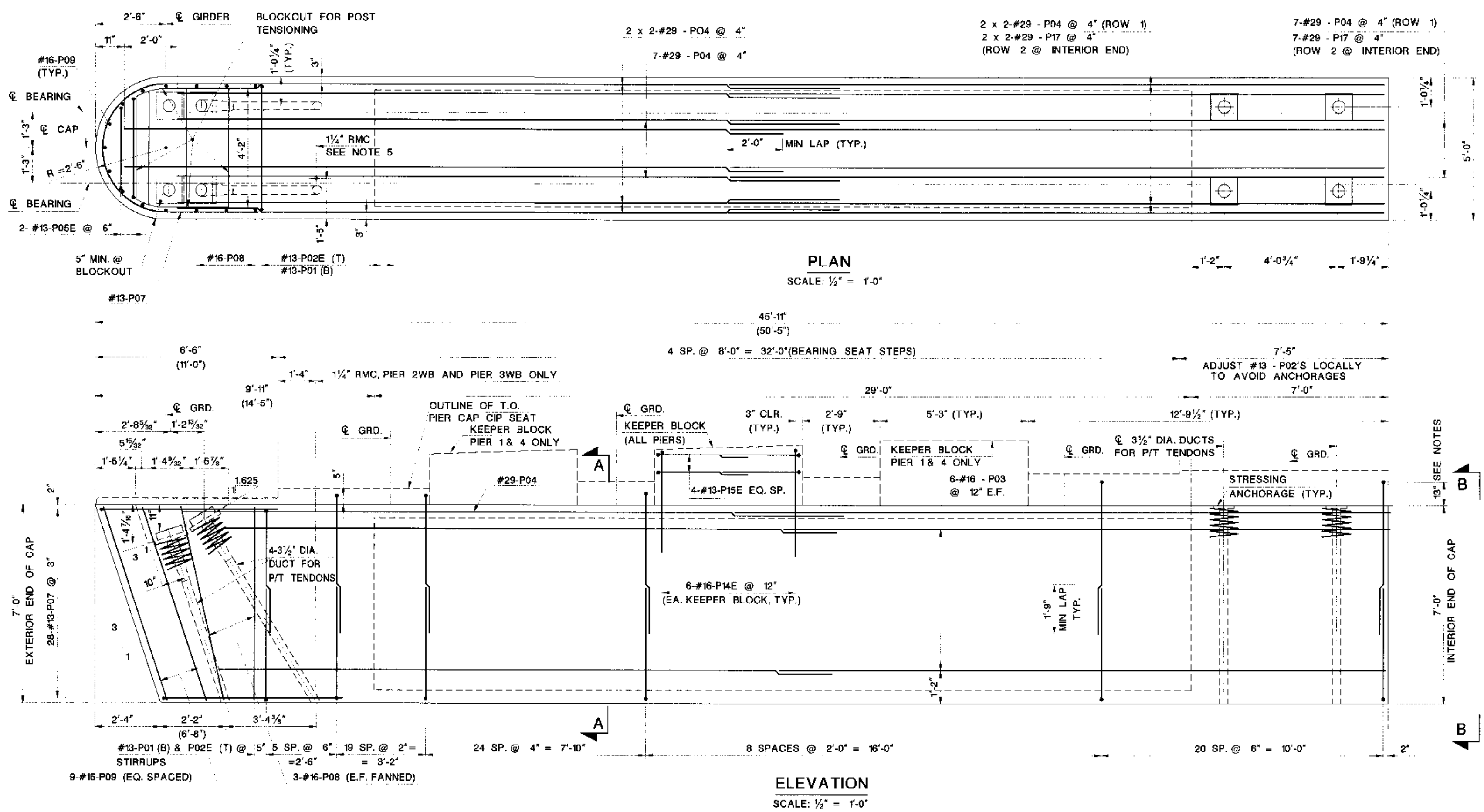
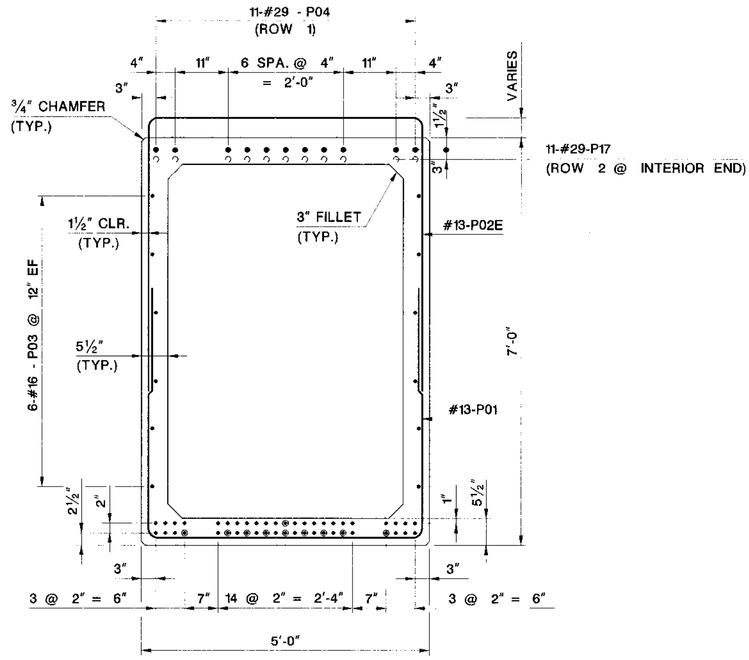
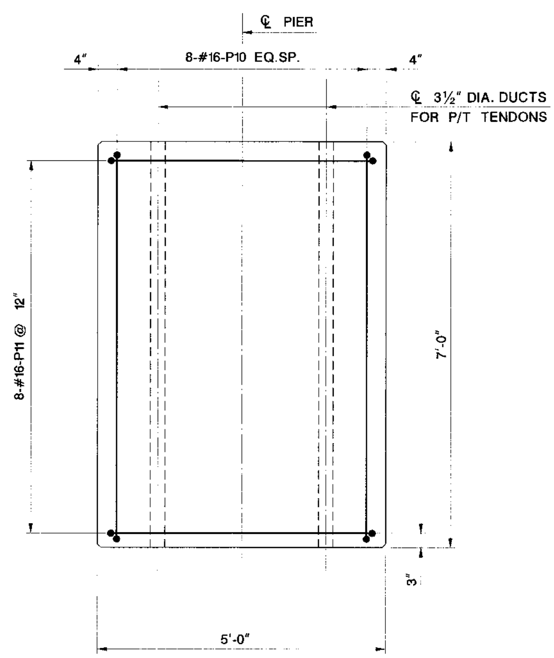


Figure VII.C.14 Bent Cap Elevation and Plan



SECTION A-A

SCALE: 3/4" = 1'-0"



SECTION B-B

(INTERIOR END)

SCALE: 3/4" = 1'-0"

Figure VII.C.15 Bent Cap Section A-A and B-B

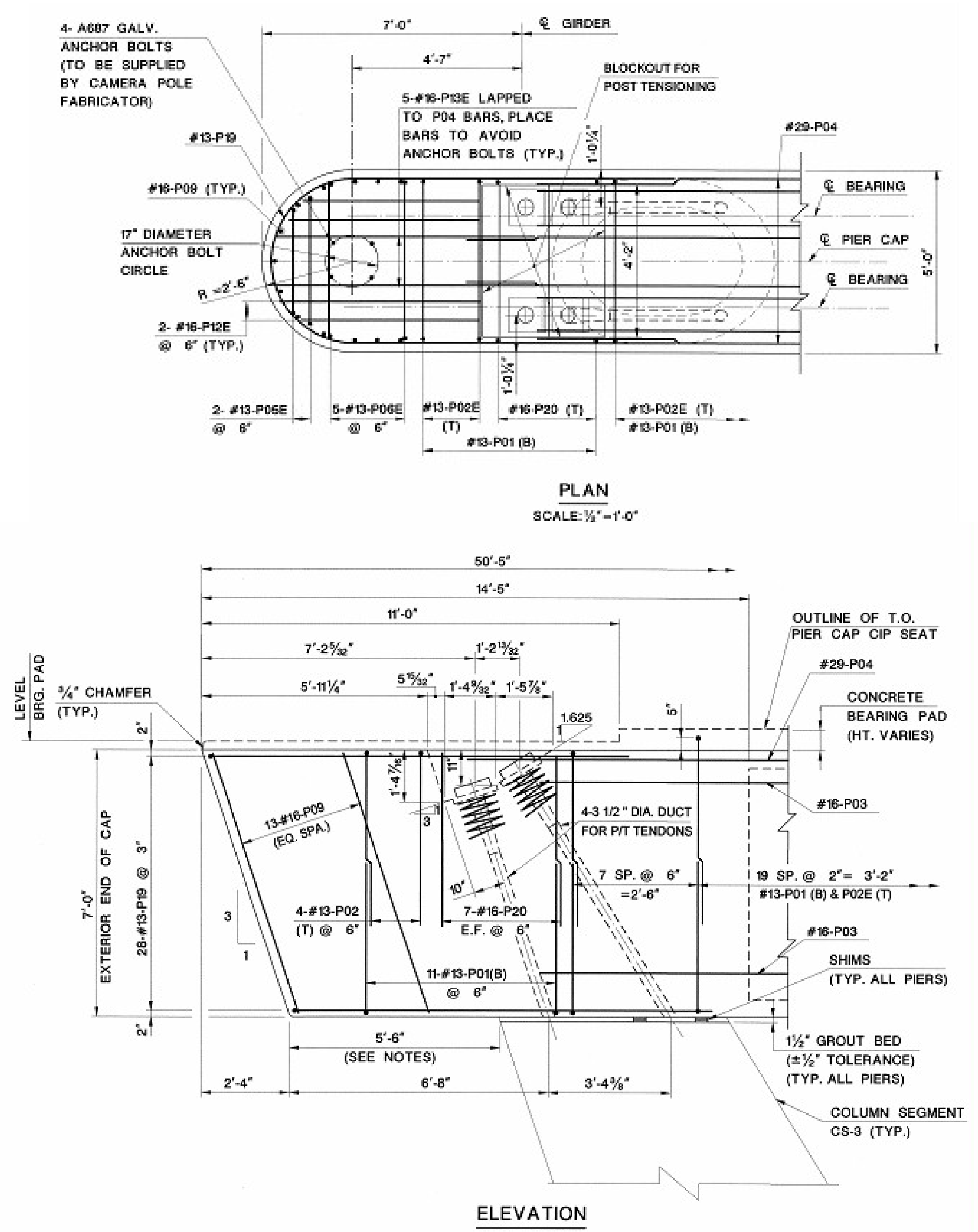


Figure VII.C.16 Pier 3 EB Cap End Details

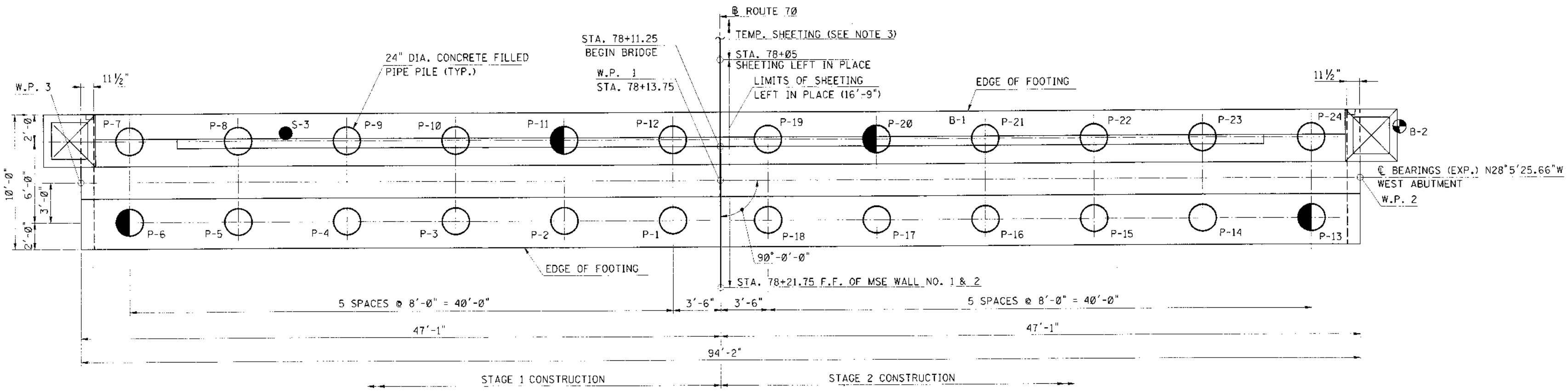


Figure VII.C.17 West Abutment Pile Layout

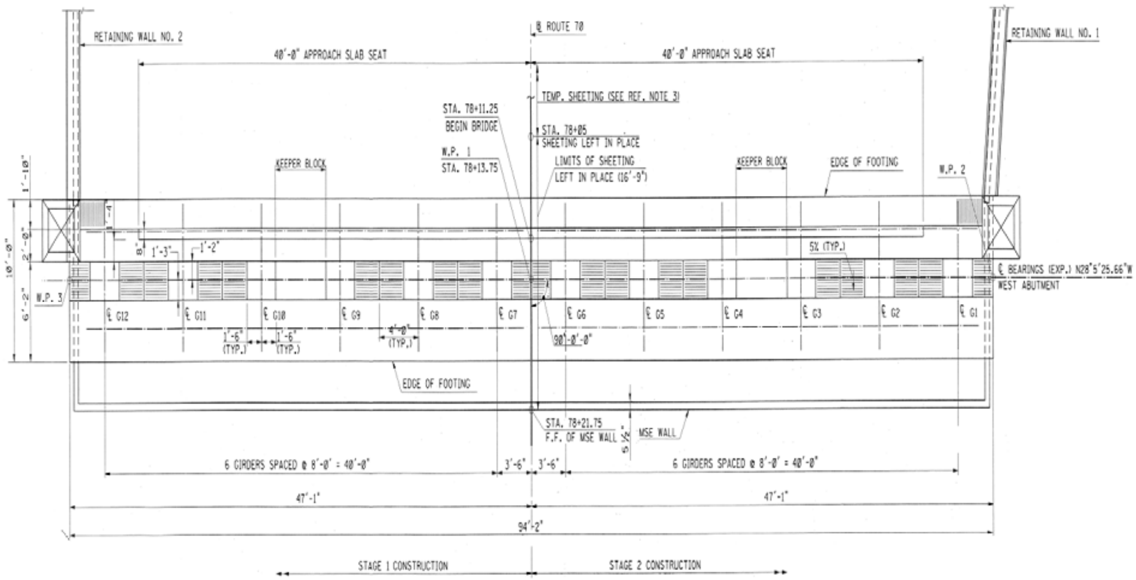


Figure VII.C.18 West Abutment Bearing Layout Plan

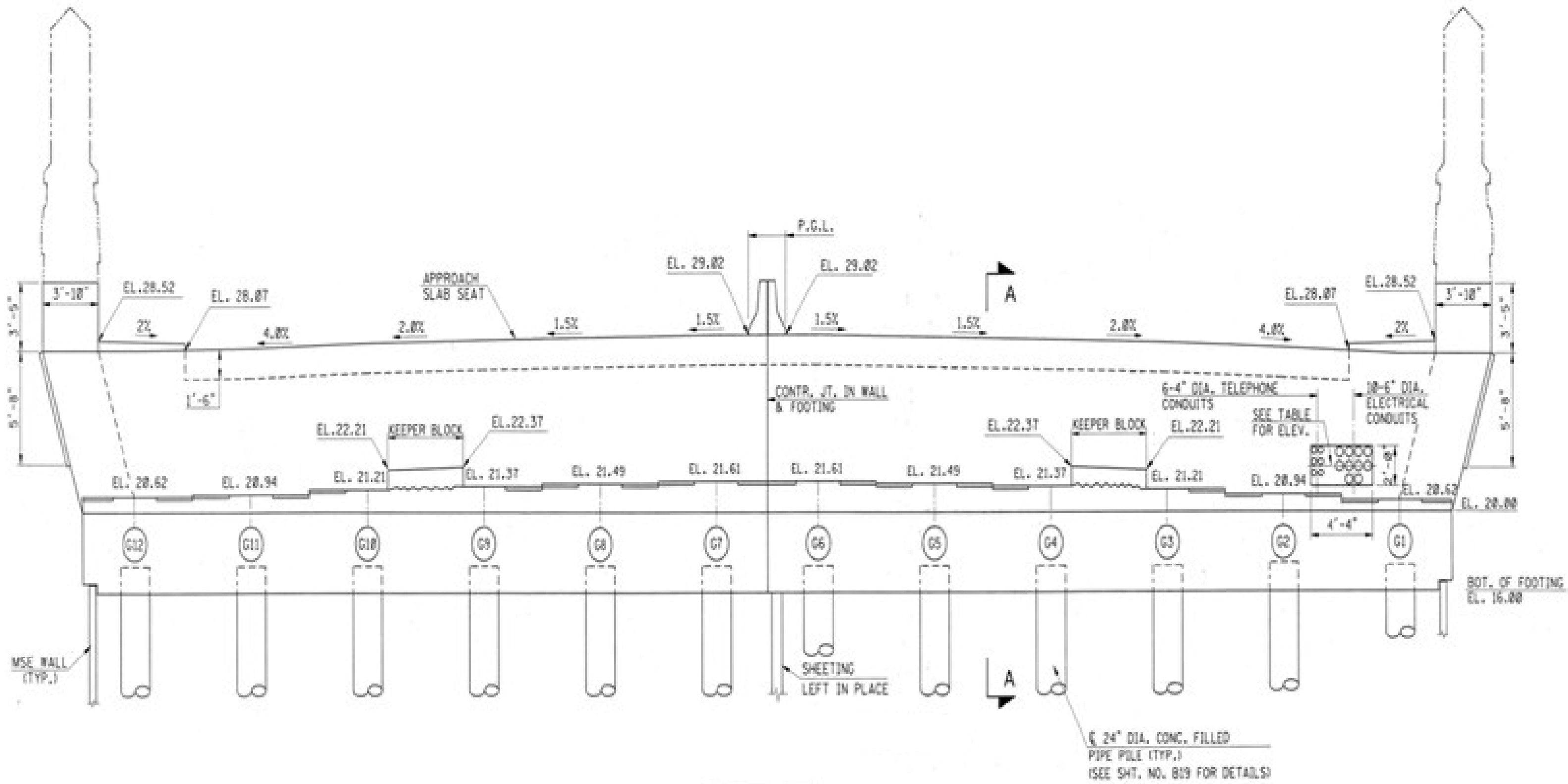


Figure VII.C.19 West Abutment Elevation

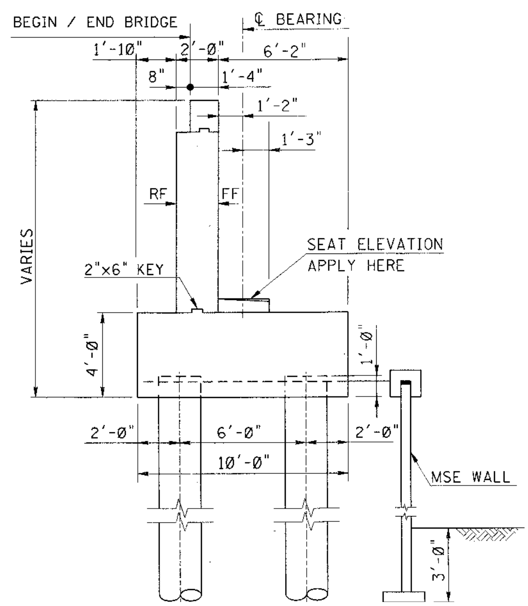


Figure VII.C.20 Abutment Section A-A

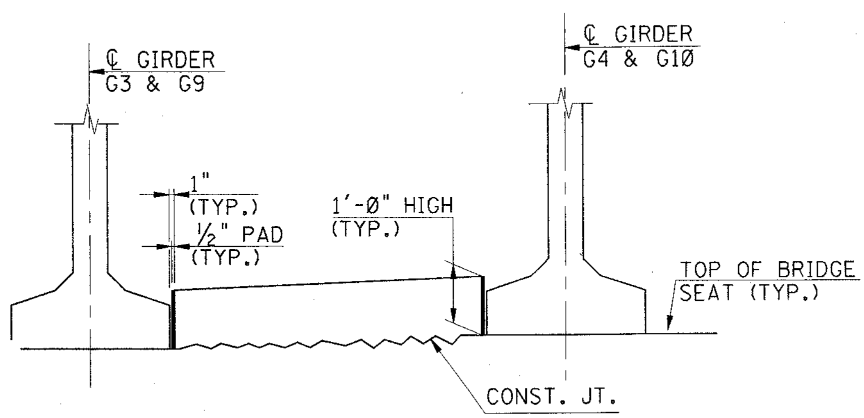


Figure VII.C.21 Keeper Block

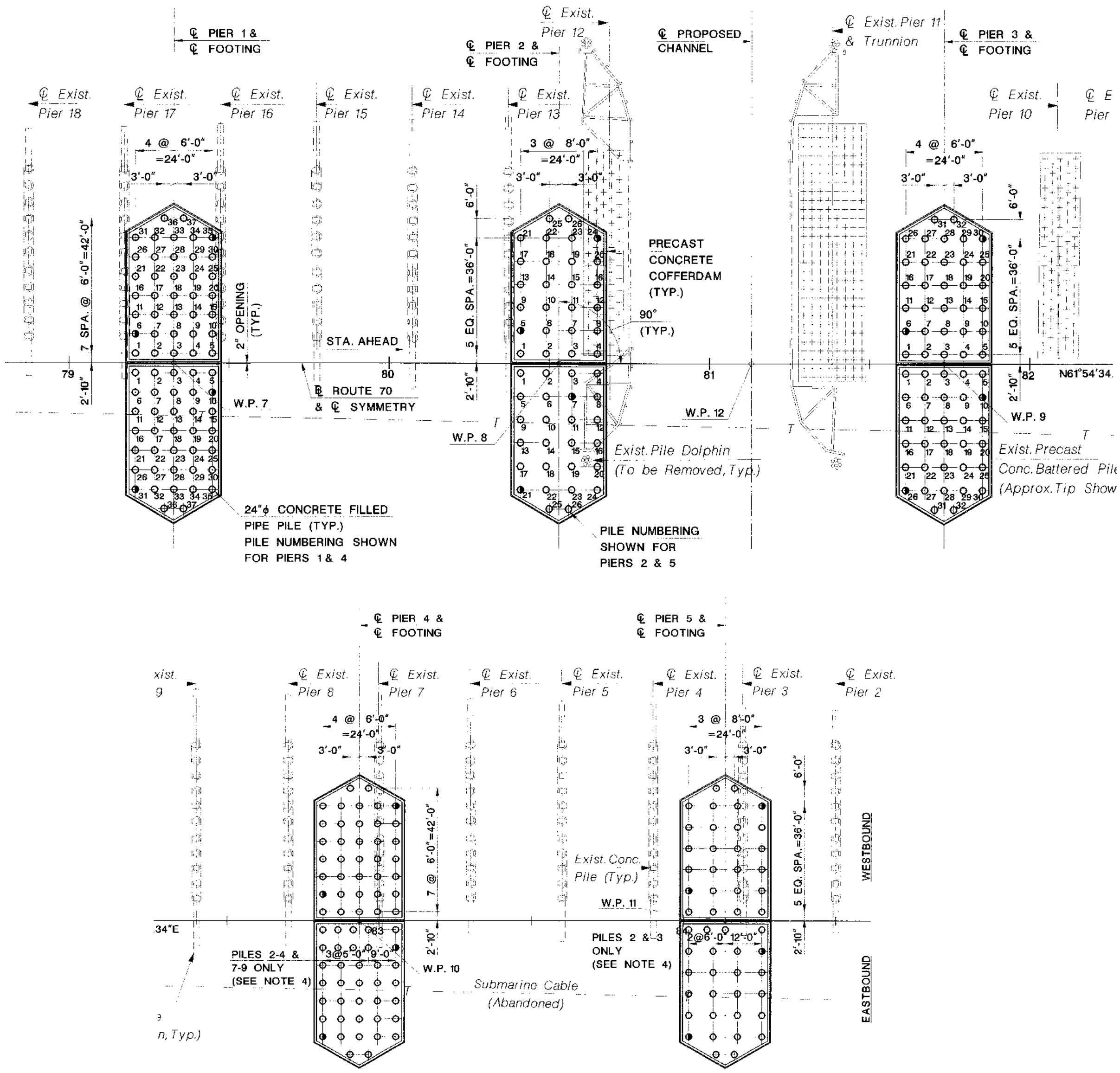


Figure VII.C.22 Footing Layout for Piers 1,2,3,4,5

APPENDIX VIII: EXAMPLE ON DESIGN OF NINE-SPAN CONCRETE BRIDGE

VIII.A: Pier Analysis

CSI-SAP12 is used to model the bent at pier 5 shown in Figure VIII.C.8, VIII.C.9, and VIII.C.10 of Appendix VIII.C. Three cross sections are considered to obtain the gross and effective properties. The moment curvature relationship is also derived and plotted. The three cross sections refer to :

1) The top portion of the pile equal to 1.3m length (PT)
2) The lower protion of the pile reaching to the river bottom (PB)

Pier 5 is the longitudinal seismic collector bent for Frame 2 that include spans 4, 5, and 6 . Pier 5 having fixed bearings is a collector in the longitudinal direction for a total bridge length of 103.75m or 344 ft. In the transverse direction, the seismic load is shared by all piers.

The model developed by SCI-SAP 12 has the following characteristics:

a.) The point of fixity of the pile is considered at a depth of 3 times the equivalent pile diameter below river bottom.
b.) The cross sections material modeling based on LRFD-GS 8.4 include:
    - An unconfined concrete model for the pile concrete cover.
    - A confined concrete model fo the core concete surrounded by the transverse spiral reinforcement shown in Figures VIII.A.2, and VIII.A.3 for sections PT and PB respectively.

For SDCs B and C, the expected material properties shall be used to determine the pile section stiffness and overstrength capacities. The stress-strain properties are used as shown in Table VIII.A.1. The top pile connection reinforcement consists of ASTM A615 Grade 60 #9 longitudinal reinforcement with material properties as follows:

#9	$f_y = 60\,Ksi$	$\varepsilon_{sh} = 0.0125$
	$f_{ye} = 68\,Ksi$	$\varepsilon_{su}^{R} = 0.06$
	$f_{ue} = 95 Ksi$	$\varepsilon_{su} = 0.09$

Table VIII.A.1 Stress Properties of Reinforcement Steel Bars.

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3 - #18	60	60
Expected yield stress (ksi)	f_{ye}	#3 - #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3 - #18	95	95
Expected yield strain	ε_{ye}	#3 - #18	0.0023	0.0023
Onset of strain hardening		#3 - #8	0.0150	0.0150
		#9	0.0125	0.0125
	ε_{sh}	#10 - #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^{R}	#4 - #10	0.090	0.060
		#11 - #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4 - #10	0.120	0.090
		#11 - #18	0.090	0.060

Following LRFD-GS 8.4.3, prestressing steel is modeled with an idealized nonlinear stress-strain model. Figure VIII.A.1 shows an idealized stress-strain model for 7-wire low-relaxation prestressing strand.

Essentially elastic prestress steel strain, $\varepsilon_{ps,EE}$, for 270 Ksi strands is taken as:

$$\varepsilon_{ps,EE} = 0.0086$$

Reduced ultimate prestress steel strain is taken as:

$$\varepsilon_{ps,u}^{R} = 0.03$$

The stress, f_{ps}, in the prestressing steel is taken as:

$$f_{ps} = 28{,}500\varepsilon_{ps} \text{ when } \varepsilon_{ps} \le 0.0086$$

$$f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \text{ when } \varepsilon_{ps} > 0.0086$$

where:

. ε_{ps} = strain in prestressing steel

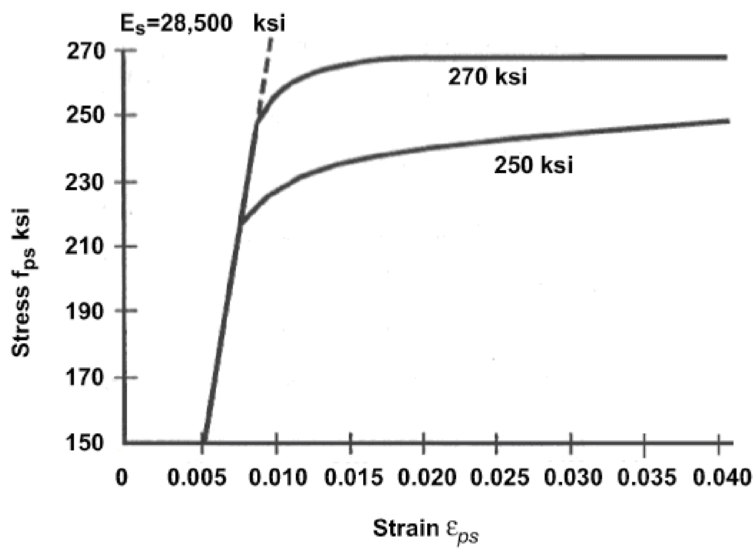


Figure VIII.A.1 Prestressing Strand Stress-Strain Model.

The gross and effective properties are generated for the following:

- Pile section (PT) (See Figure VIII.A.4, VIII.A.5)
- Pile Section (BP) (See Figures VIII.A.6, VIII.A.7)

The Transverse and Longitudinal Loadings of the model are shown in Figures VIII.A.8 and VIII.A.9. Figure VIII.A.10 shows the transverse response for the bent subject to 1200 kips of loading at the cap elevation.

Figure VIII.A.11 shows the longitudinal response for the bent subject to a total of 1200 kips of loading distributed equally to the 12 pile locations and imposed at the cap elevation.

Figures VIII.A.12 and VIII.A.13 shows the moment distribution and axial force distribution subject to the 1200 Kips Transverse loading described above. Figures VIII.A.14 and VIII.A.15 shows the moment distribution and axial force distribution subject to the 1200 Kips Longitudinal loading described above.

The Following is a list of figures associated with the CSI-SAP12 Model:

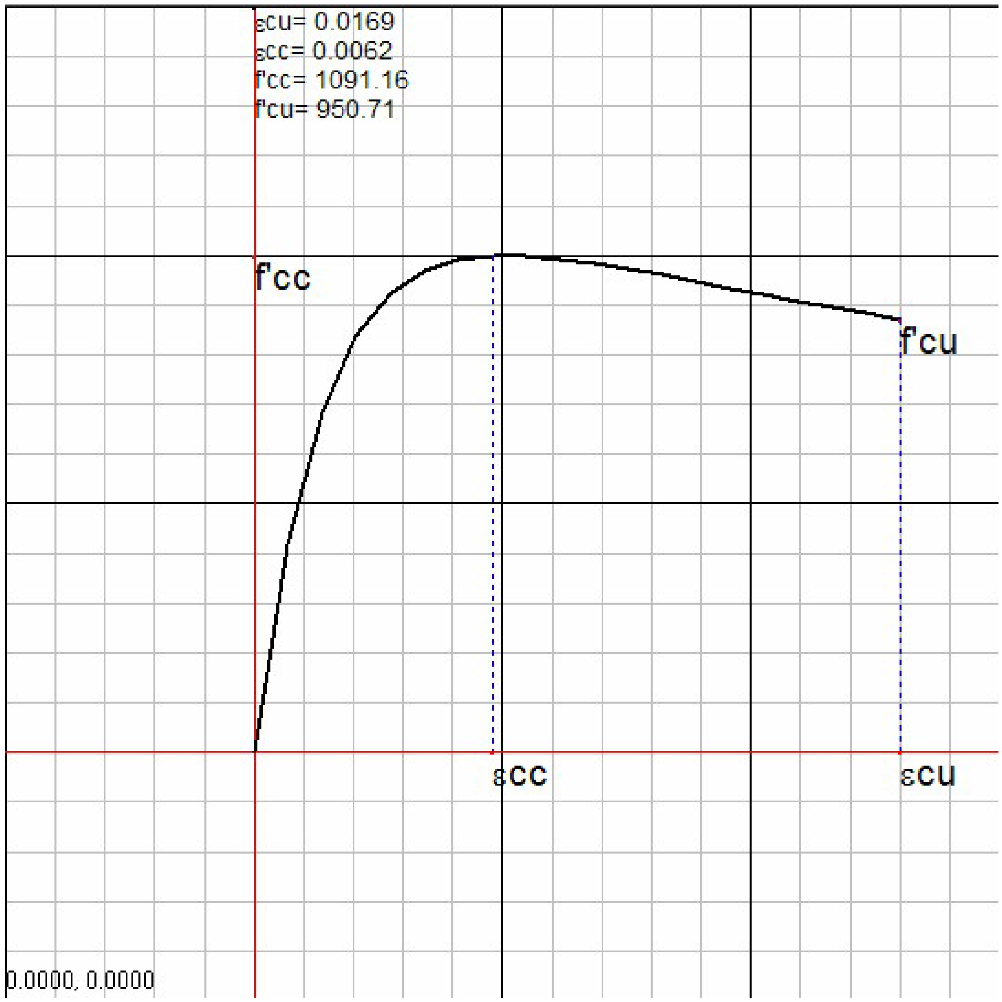


Figure VIII.A.2 Confined Concrete Stress Strain Model Section (PT)

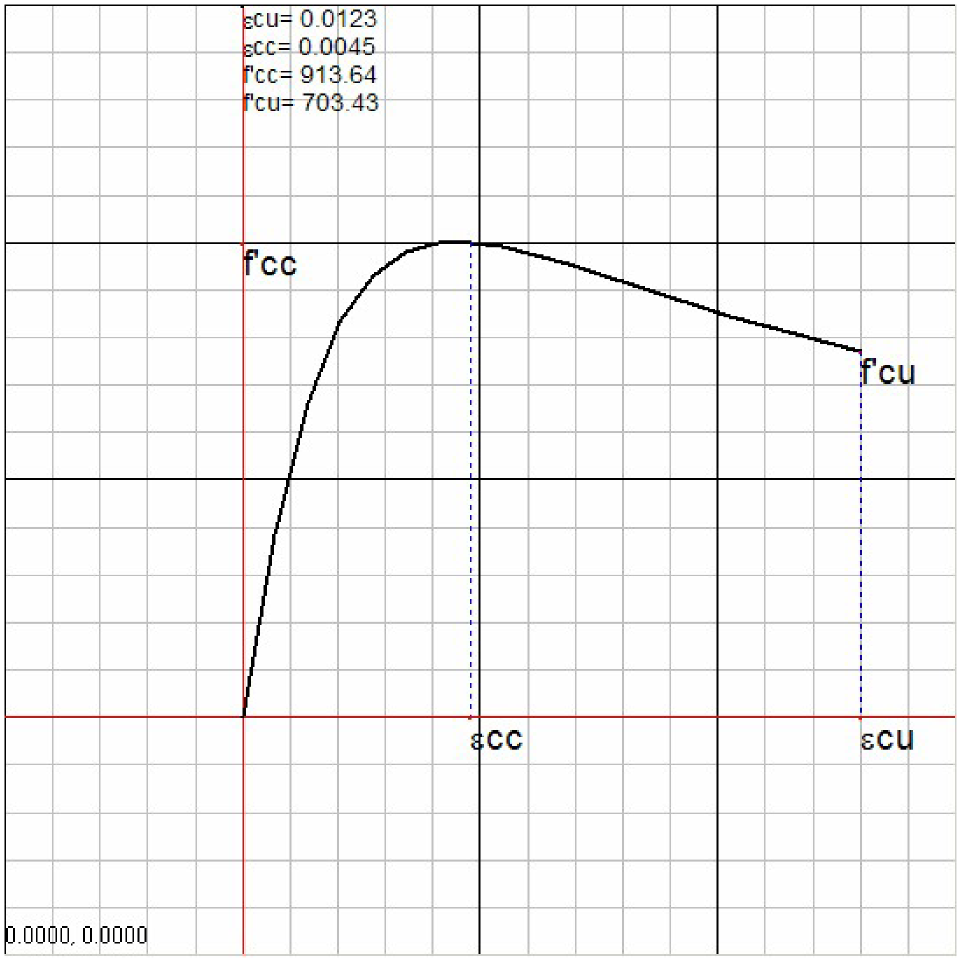


Figure VIII.A.3 Confined Concrete Stress Strain Model Section (PB)

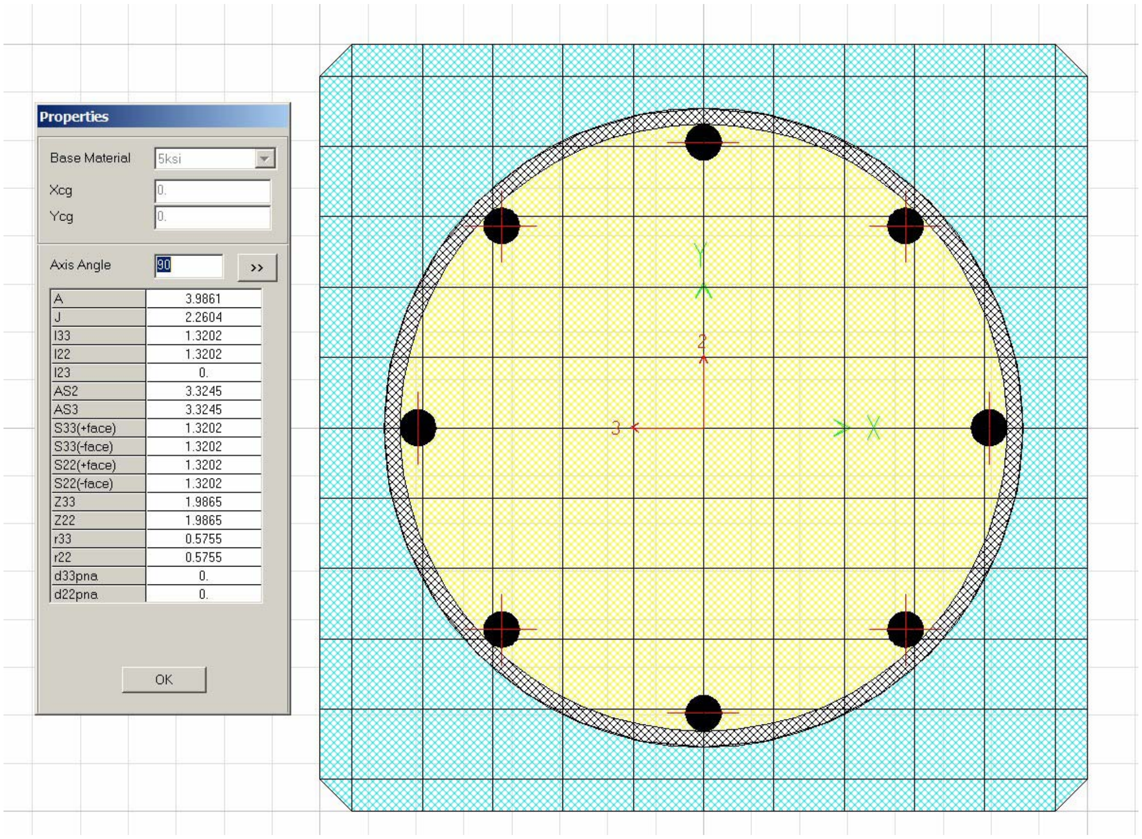


Figure VIII.A.4 Pile Gross Properties (PT)

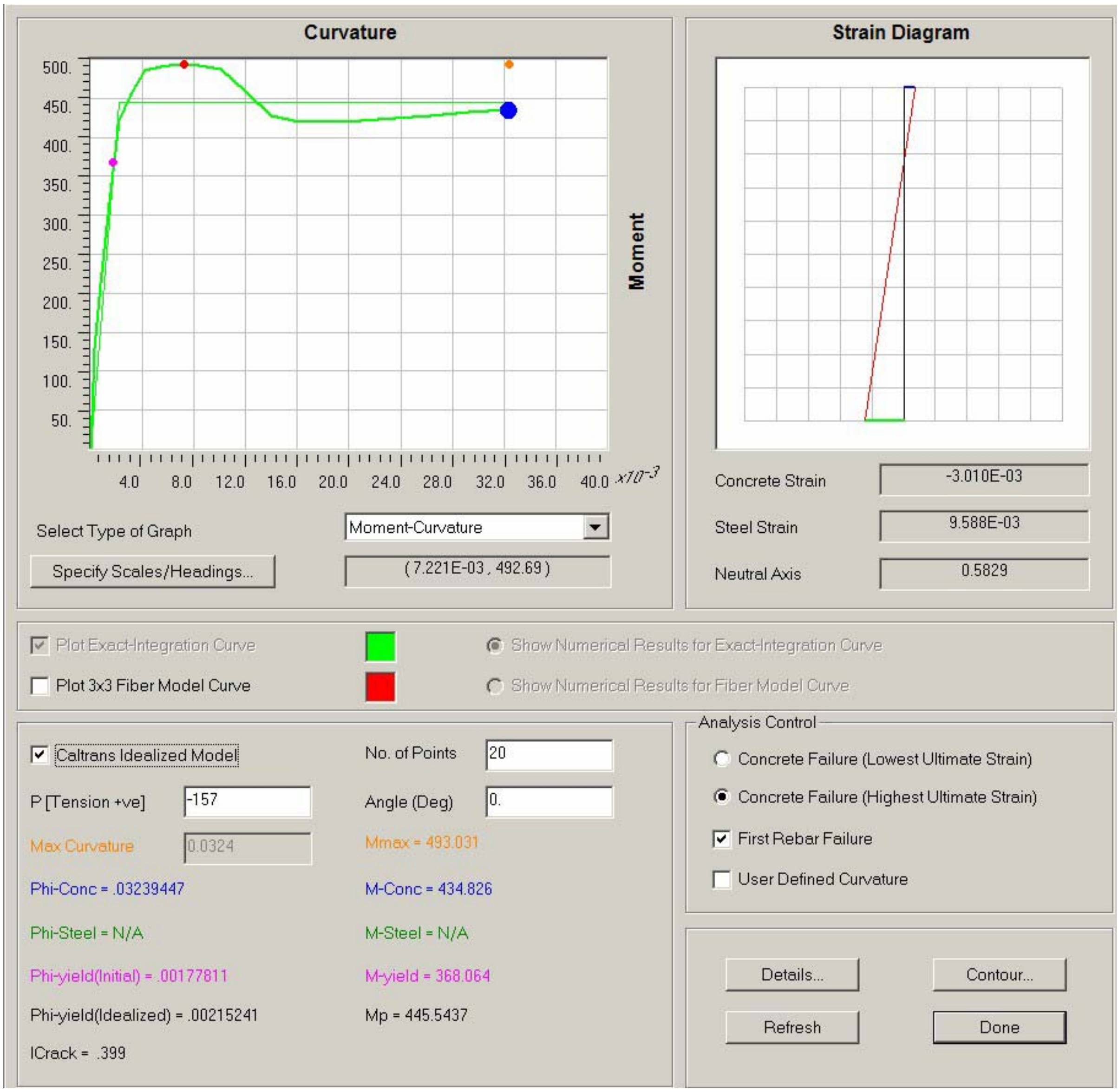


Figure VIII.A.5 Moment Curvature Analysis (PT)

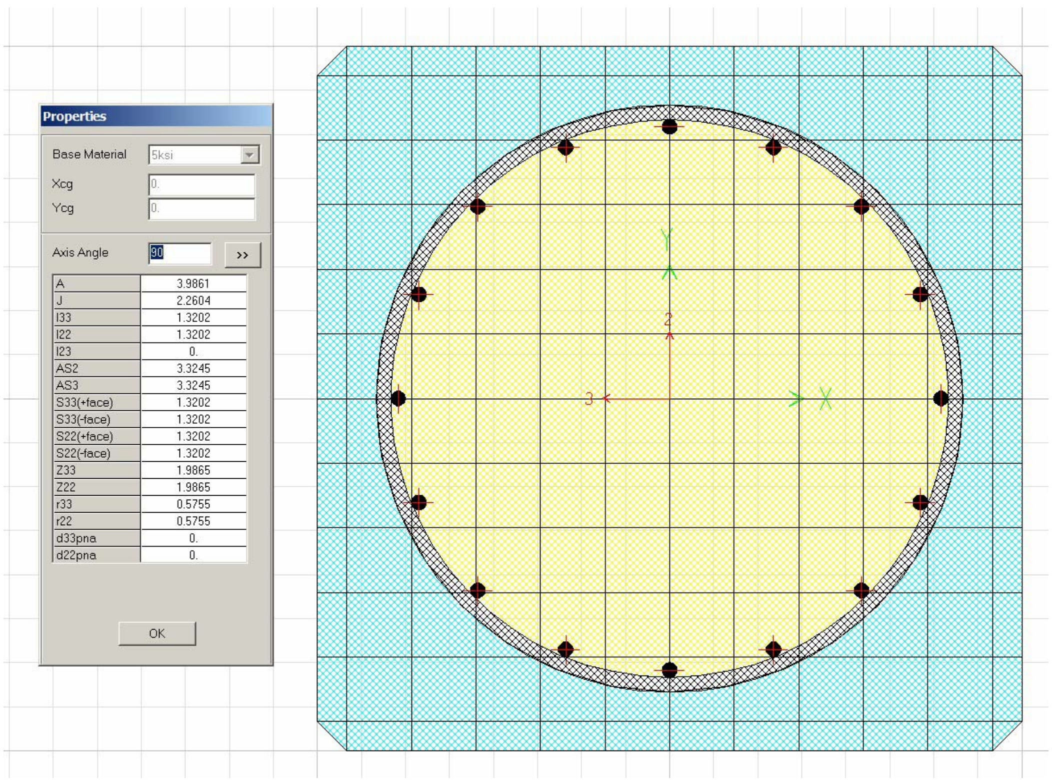


Figure VIII.A.6 Pile Gross Properties (PB)

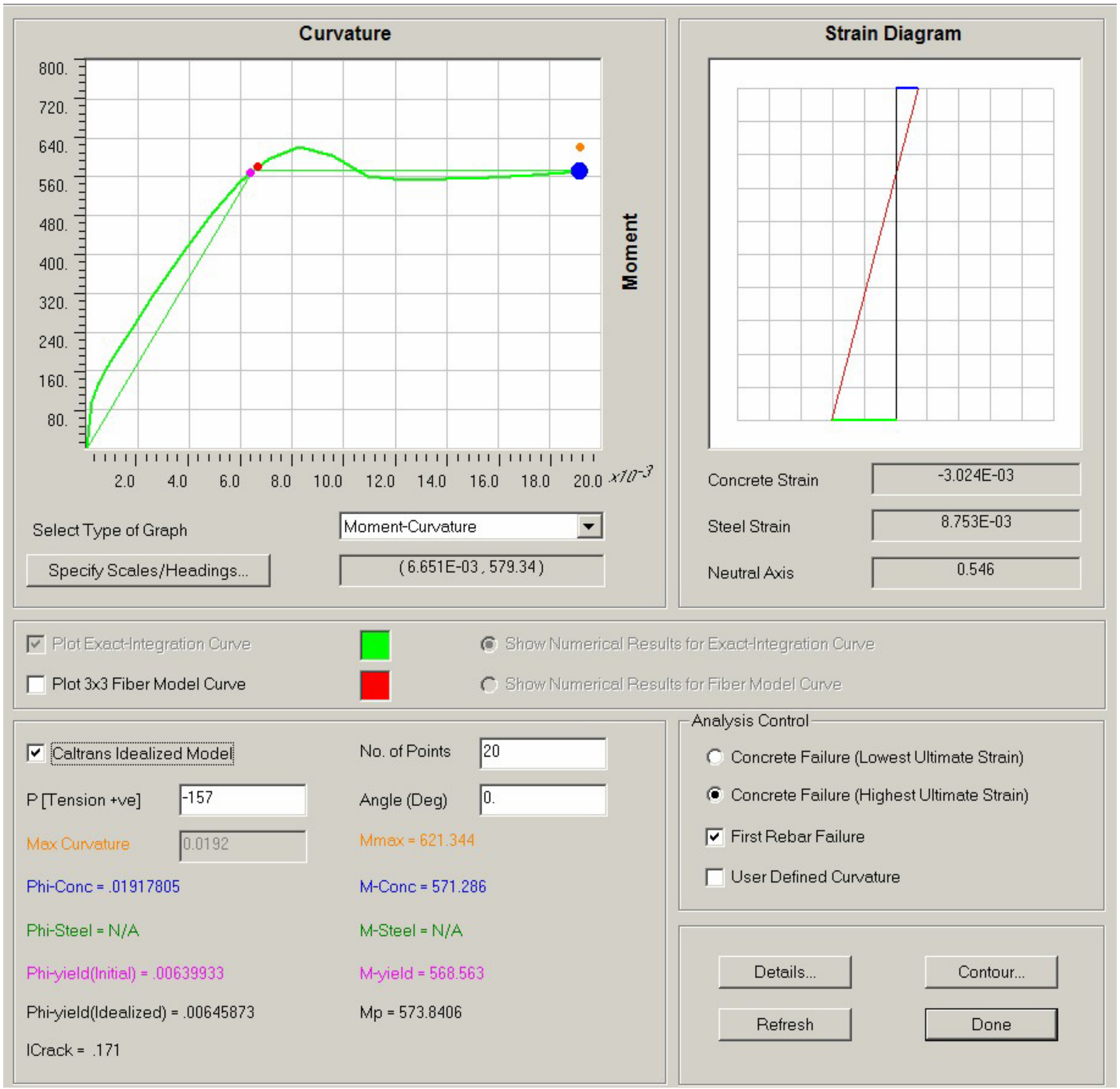


Figure VIII.A.7 Moment Curvature Analysis (PB)

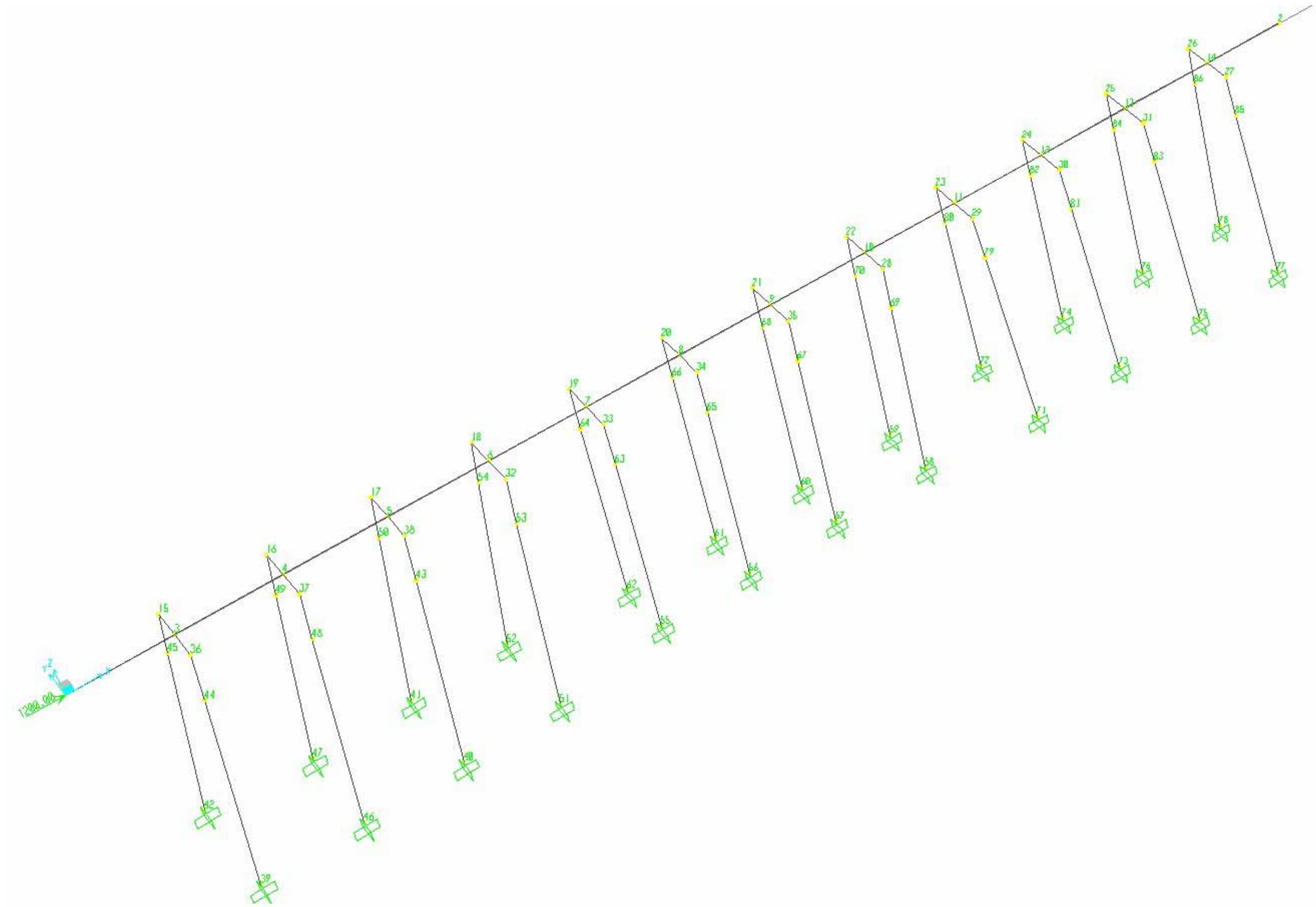


Figure VIII.A.8 CSI-SAP Model Subjected to 1200 Kips Transverse Loading.

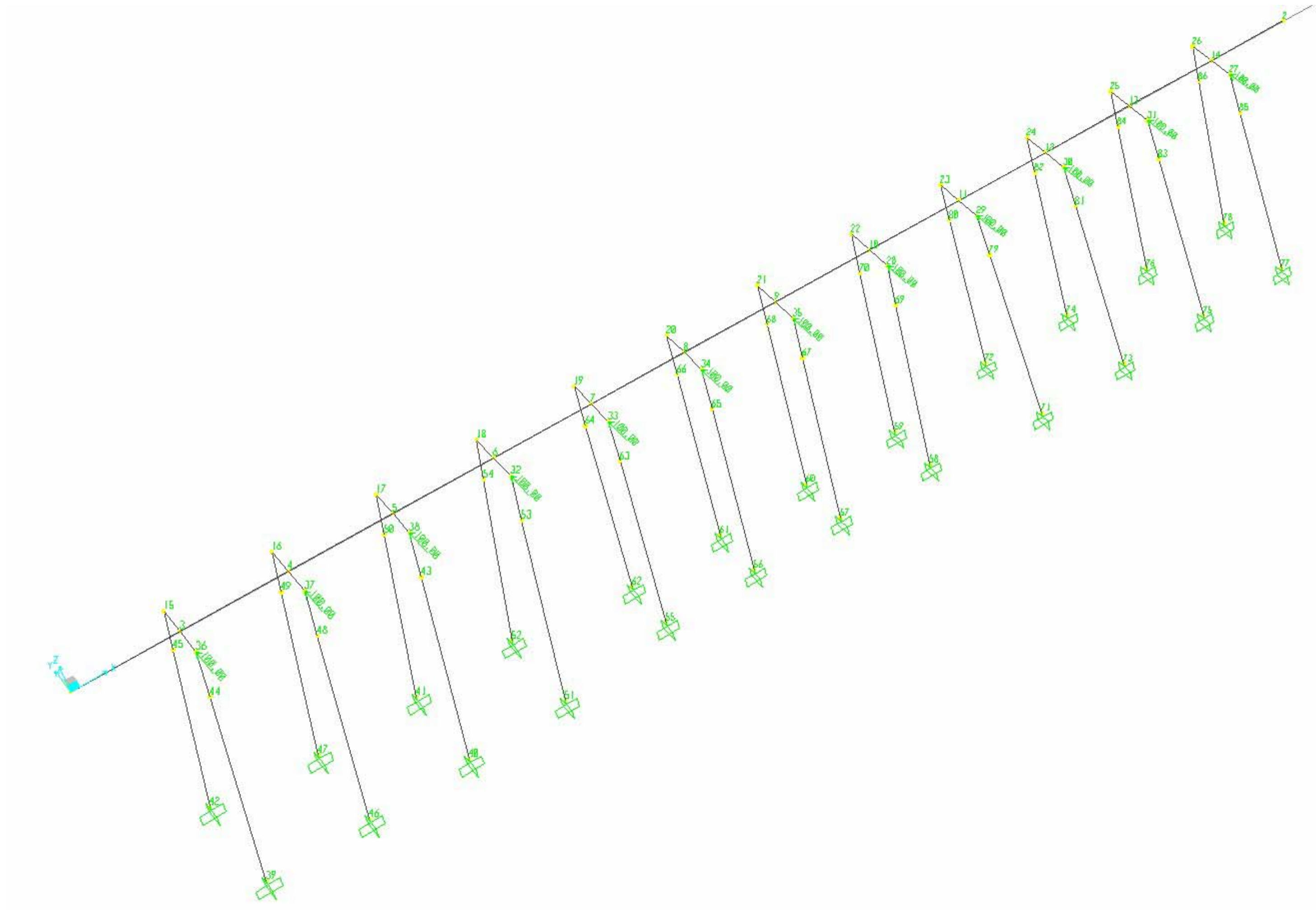


Figure VIII.A.9 CSI-SAP Model Subjected to 1200 Kips Longitudinal Loading

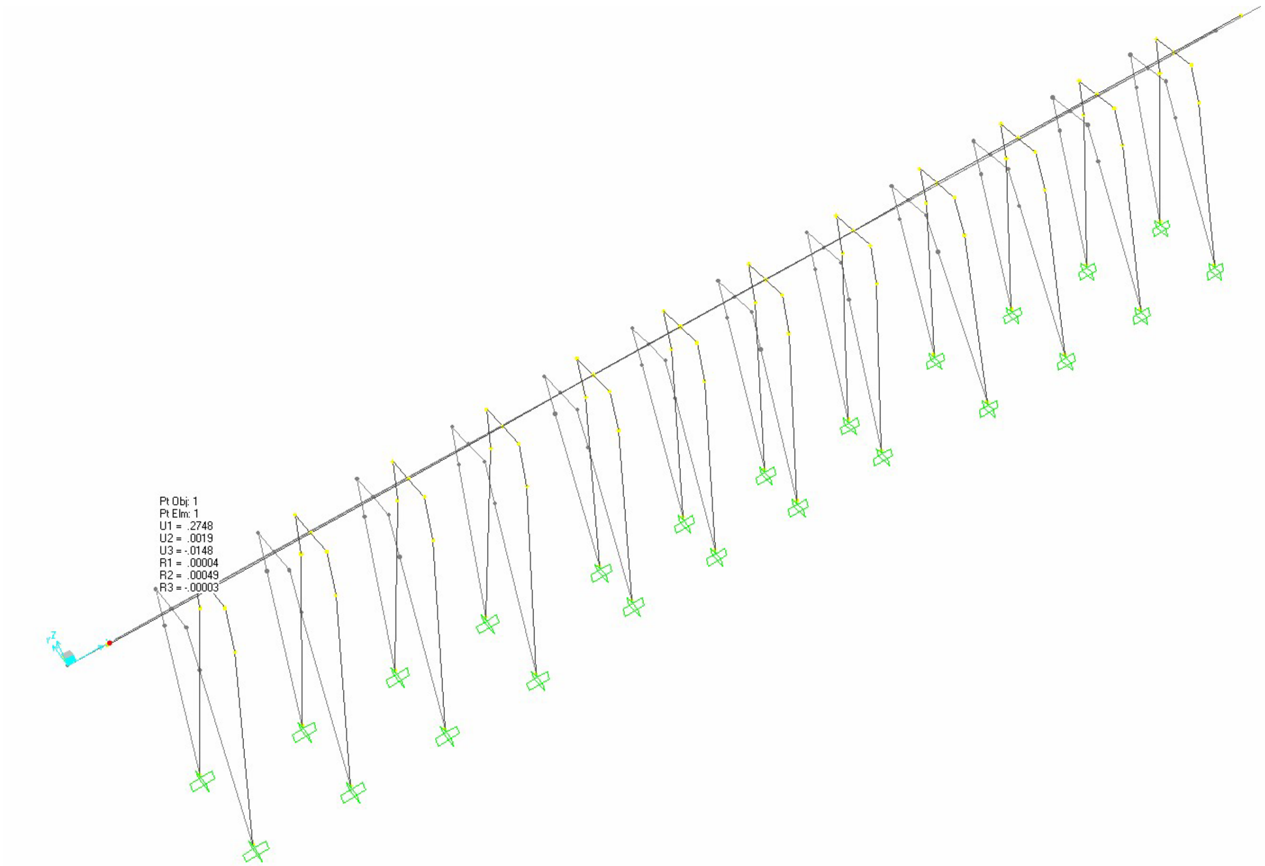


Figure VIII.A.10 Pile Bent Transverse Displacement Subject to 1200 Kips Loading

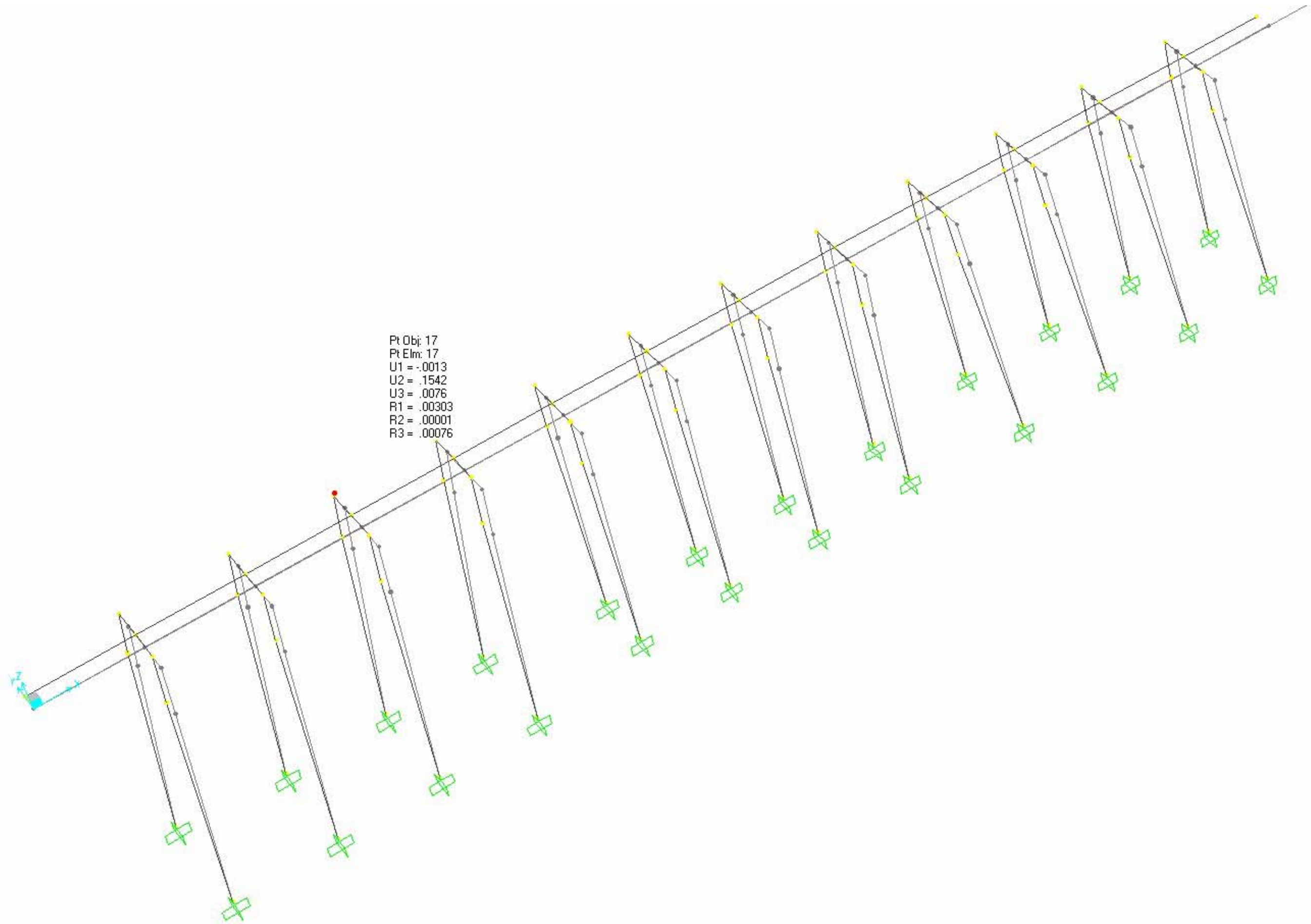


Figure VIII.A.11 Pile Bent Longitudinal Dispalcement Subject to 1200 Kips Loading

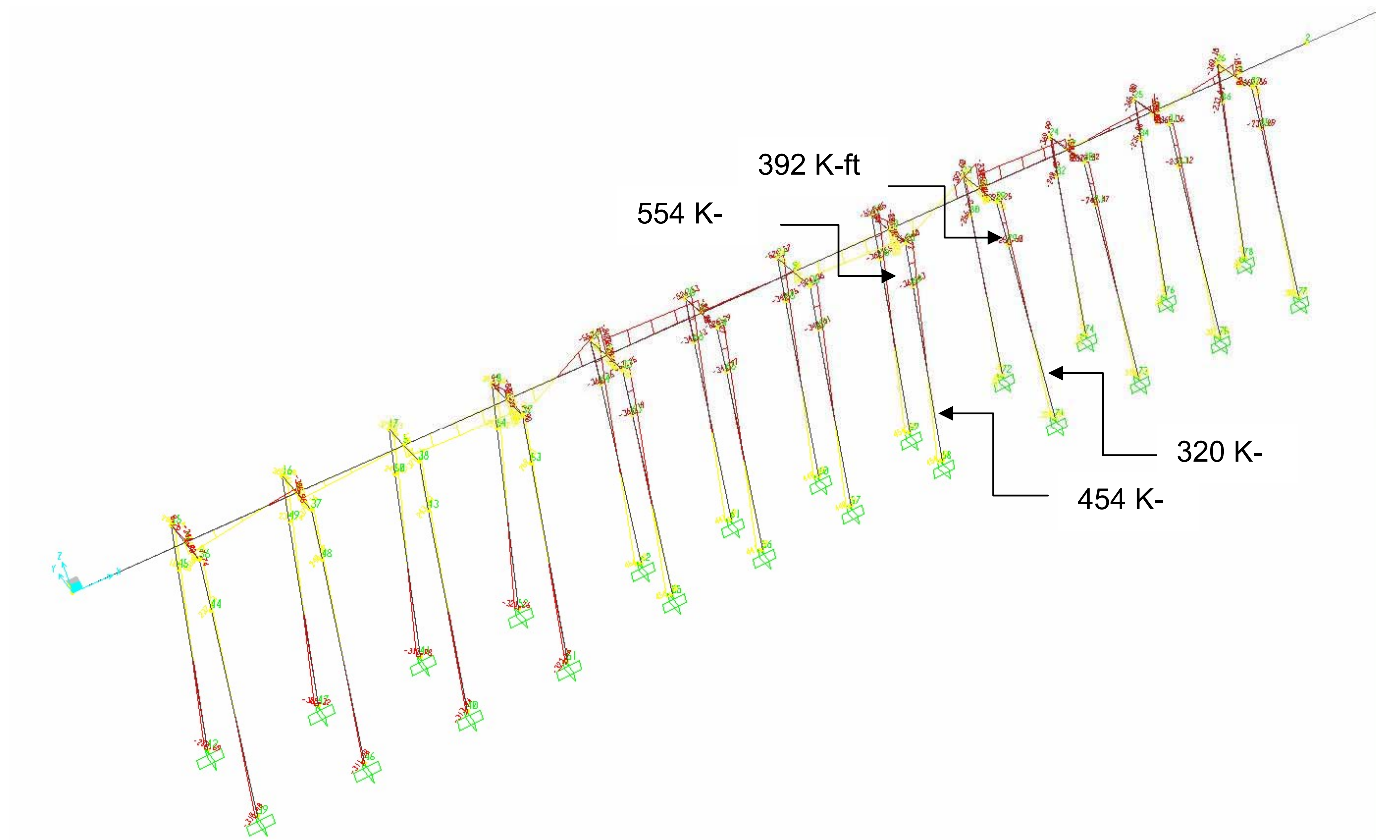


Figure VIII.A.12 Moment Demand Subject to 1200 Kips of Transverse Loading

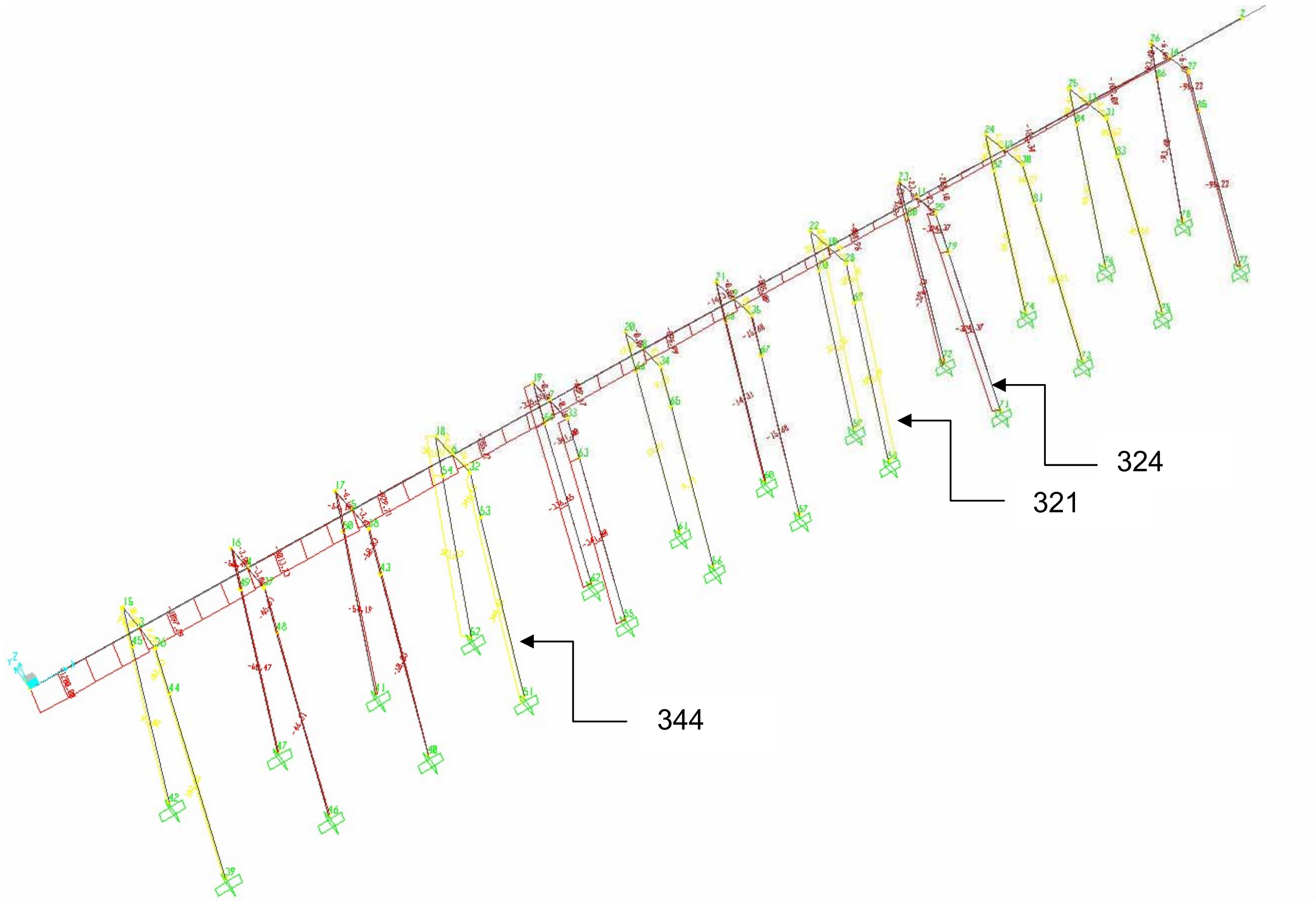


Figure VIII.A.13 Axial Force Demand Subject to 1200 Kips of Transverse Loading

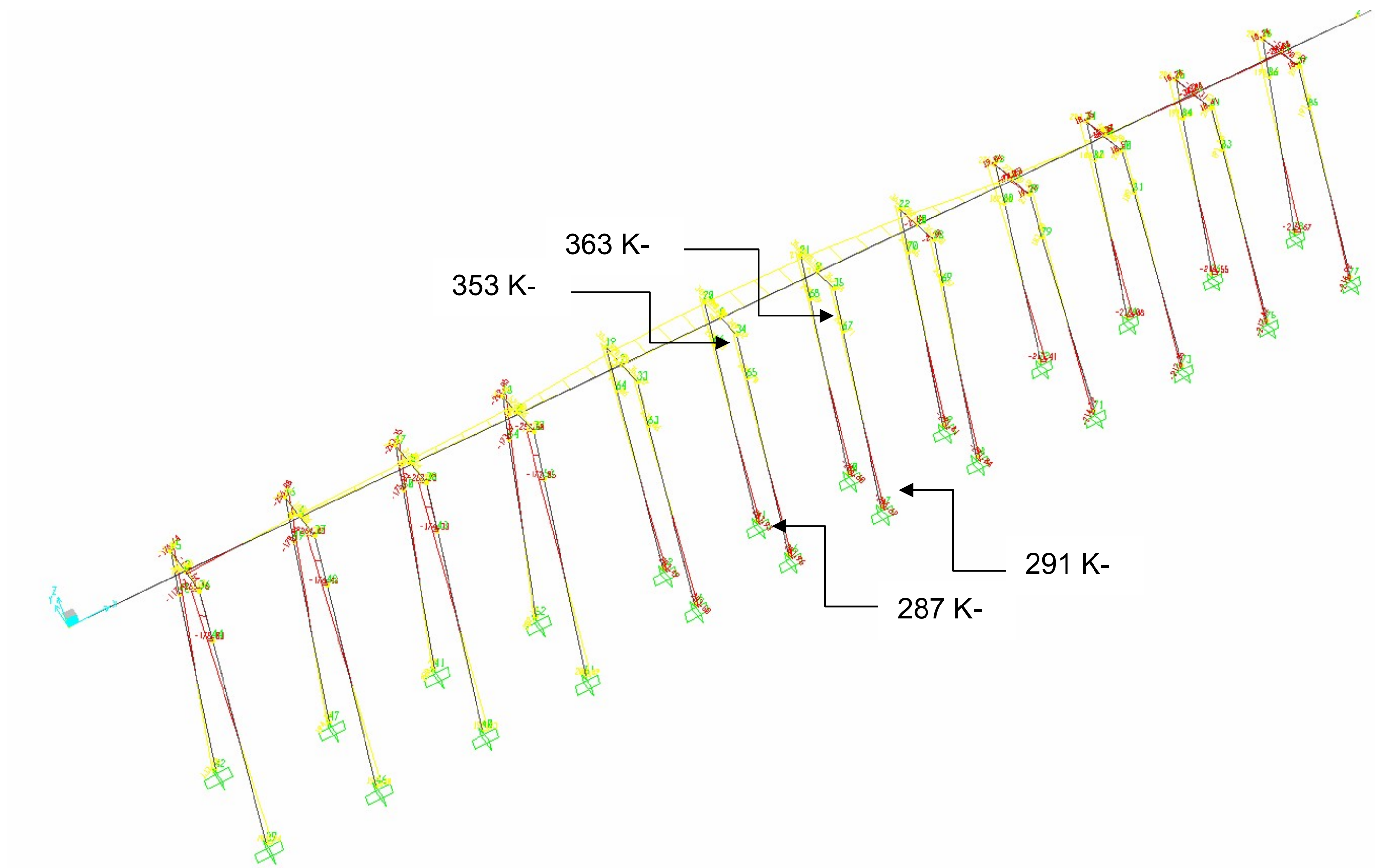


Figure VIII.A.14 Moment Demand Subject to 1200 Kips of Longitudinal Loading

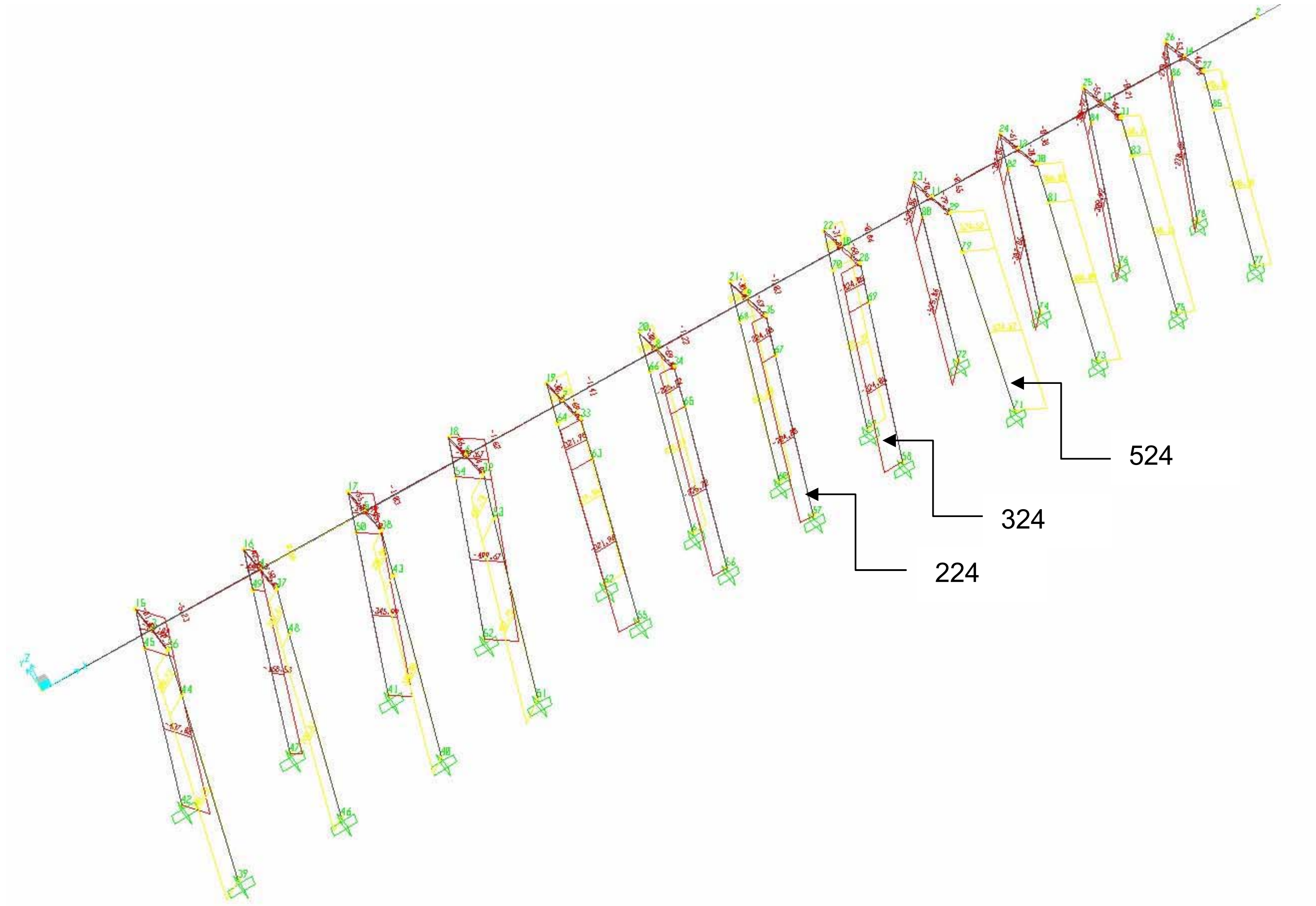


Figure VIII.A.15 Axial Force Demand Subject to 1200 Kips of Longitudinal Loading

VIII.B: Superstructure Details

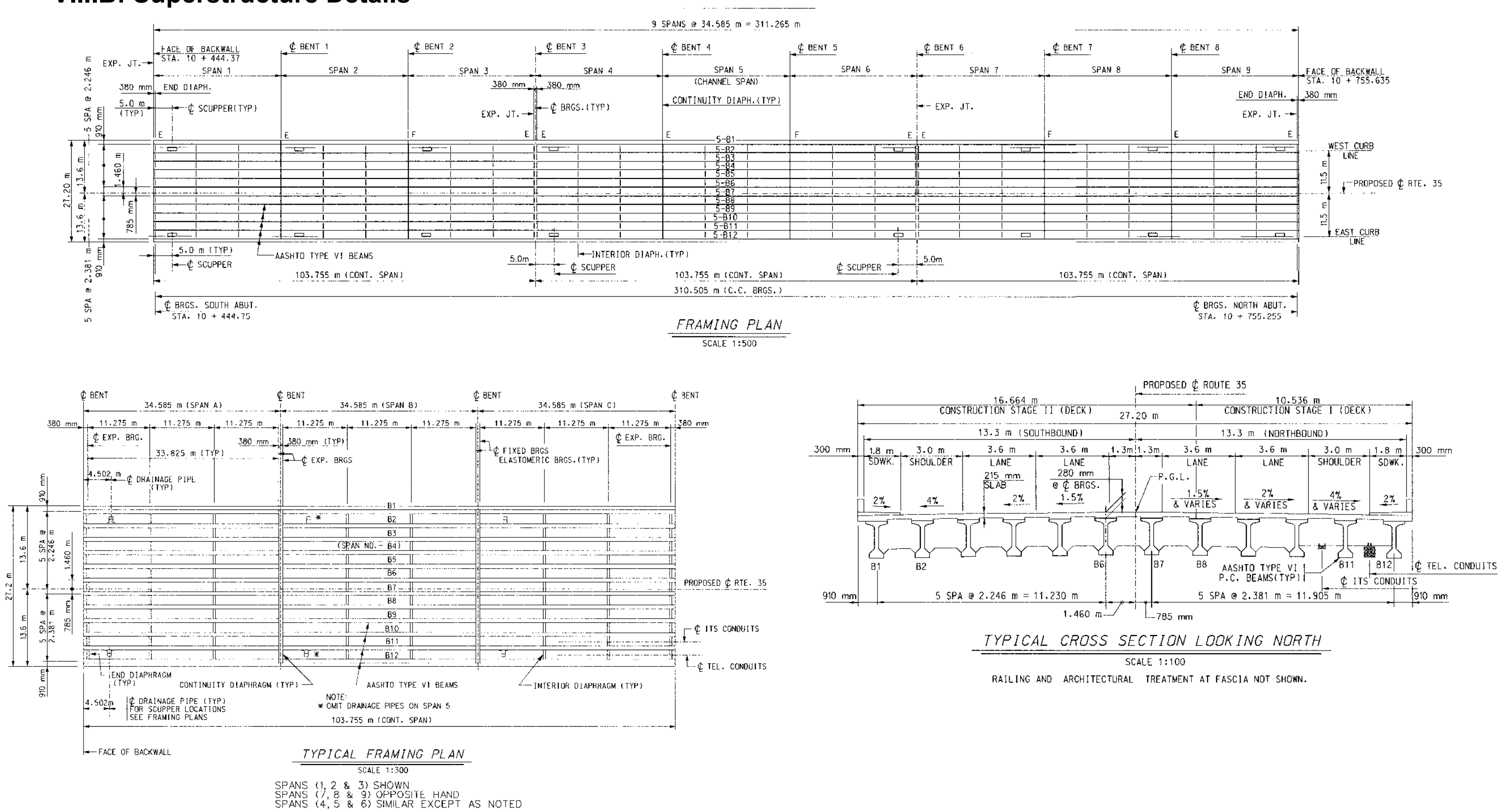


Figure VIII.B.1 Framing Plan and Cross Sections

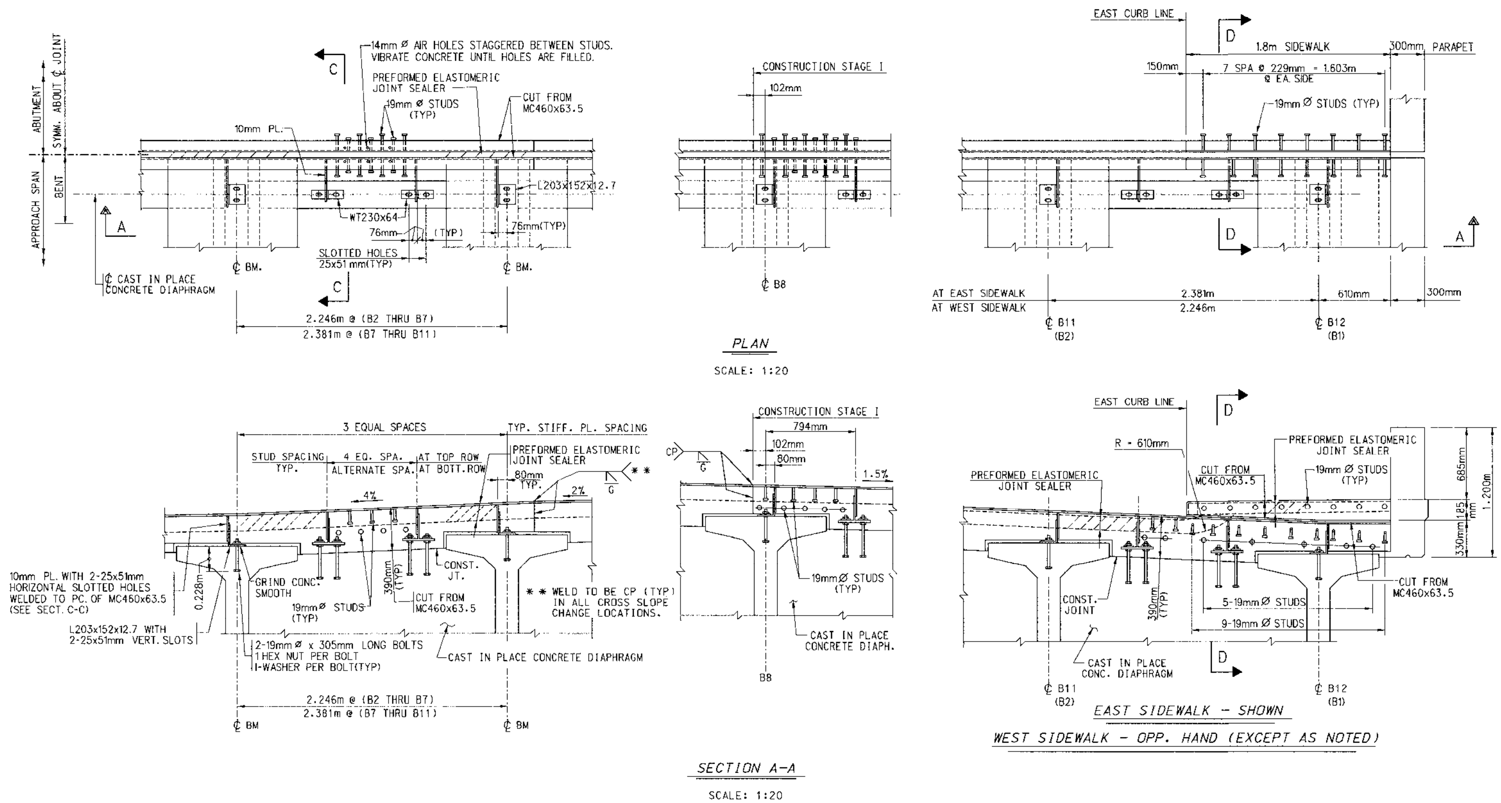


Figure VIII.B.2 Expansion Joint Plan and Sections

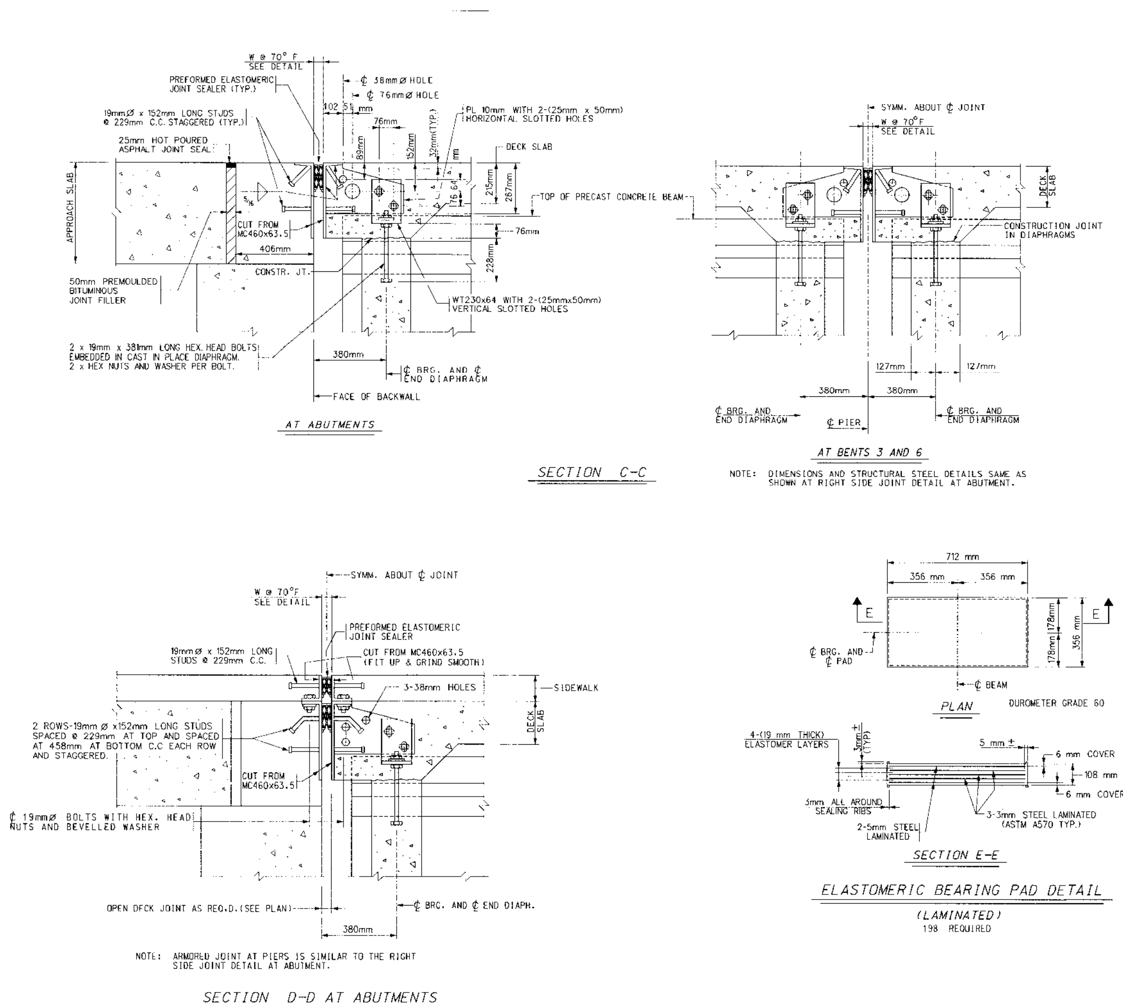
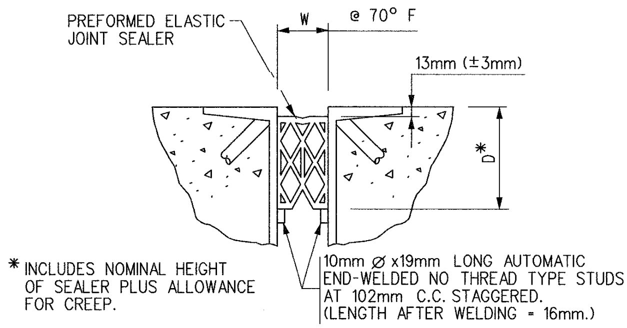
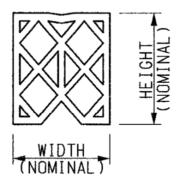


Figure VIII.B.3 Expansion Joints and Bearing Details



JOINT SEALER SEAT DETAIL

SCALE: NONE



PREFORMED ELASTOMERIC

JOINT SEALER DETAIL

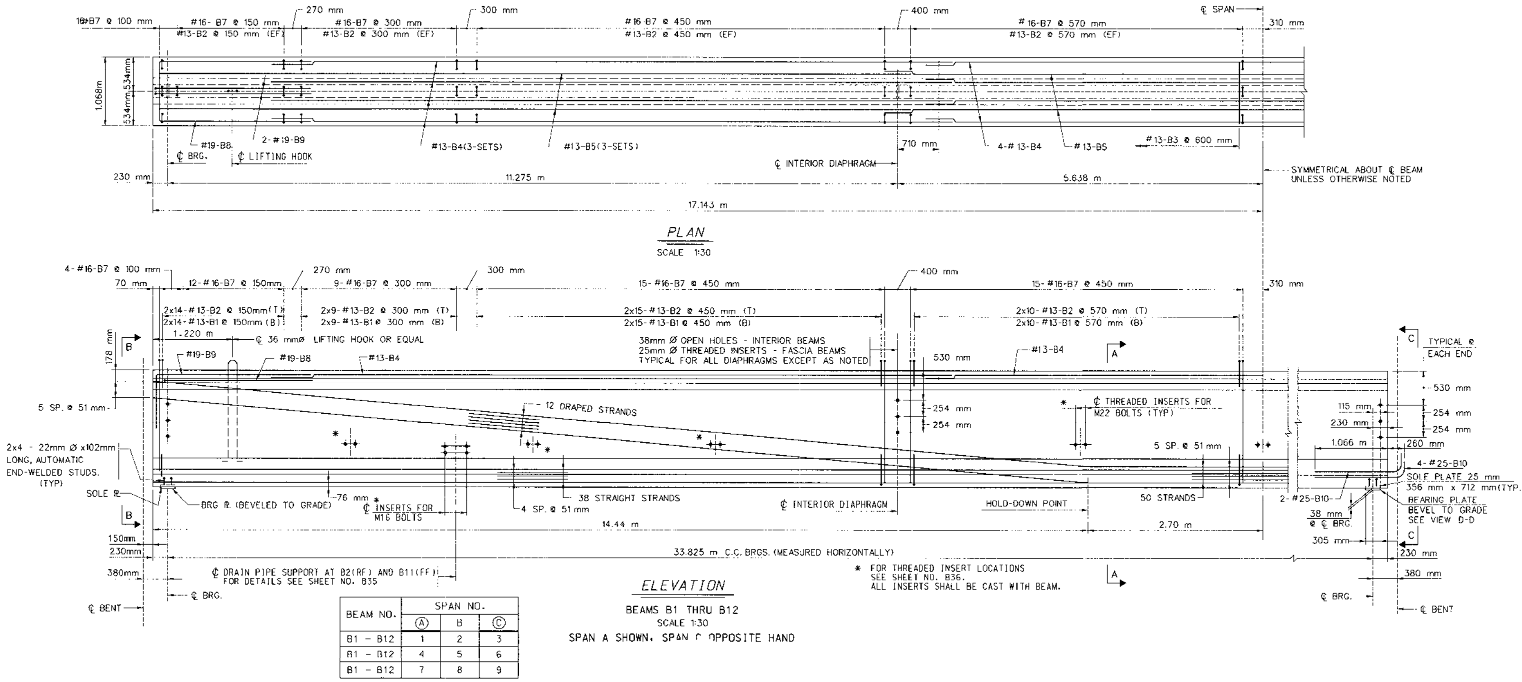
SCALE: NONE

LOCATION	NOMINAL SEALER SIZE		W @ 70° F	"D"
	WIDTH	HEIGHT		
W. ABUT. E. ABUT.	102mm	102mm	67mm	127mm
BENT 6	102mm	102mm	67mm	127mm
BENT 3	127mm	133mm	70mm	152mm

NOMINAL HEIGHT MAY VARY BASED ON MANUFACTURER'S SPECIFICATIONS.

"D" DIMENSION IS EQUAL TO COMPRESSED SEAL HEIGHT PER MANUFACTURER'S SPECIFICATION PLUS 13 mm (3mm ±).
"D" SHALL BE SET IN THE SHOP BY THE FABRICATOR.

Figure VIII.B.4 Joint Sealer Details



BEAM NO.	SPAN NO. Ⓐ	B	Ⓒ
B1 - B12	1	2	3
B1 - B12	4	5	6
B1 - B12	7	8	9

Figure VIII.B.5 Beams B1 thru B12 Plan and Elevation (1/2)

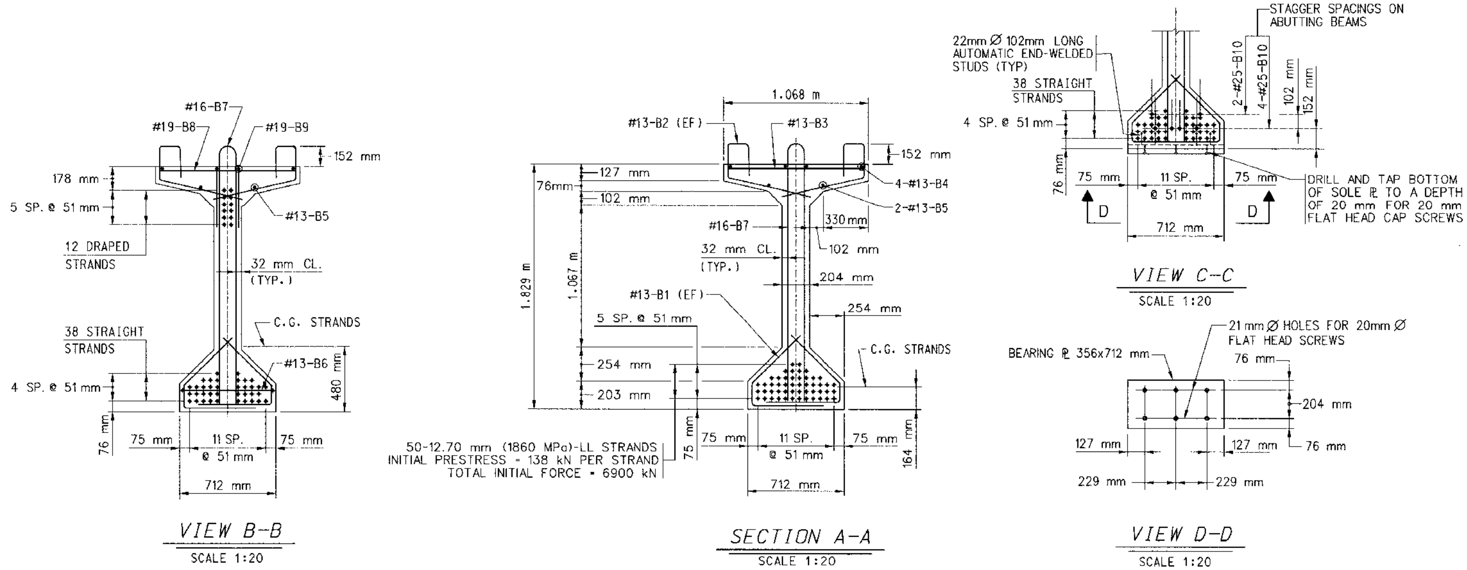
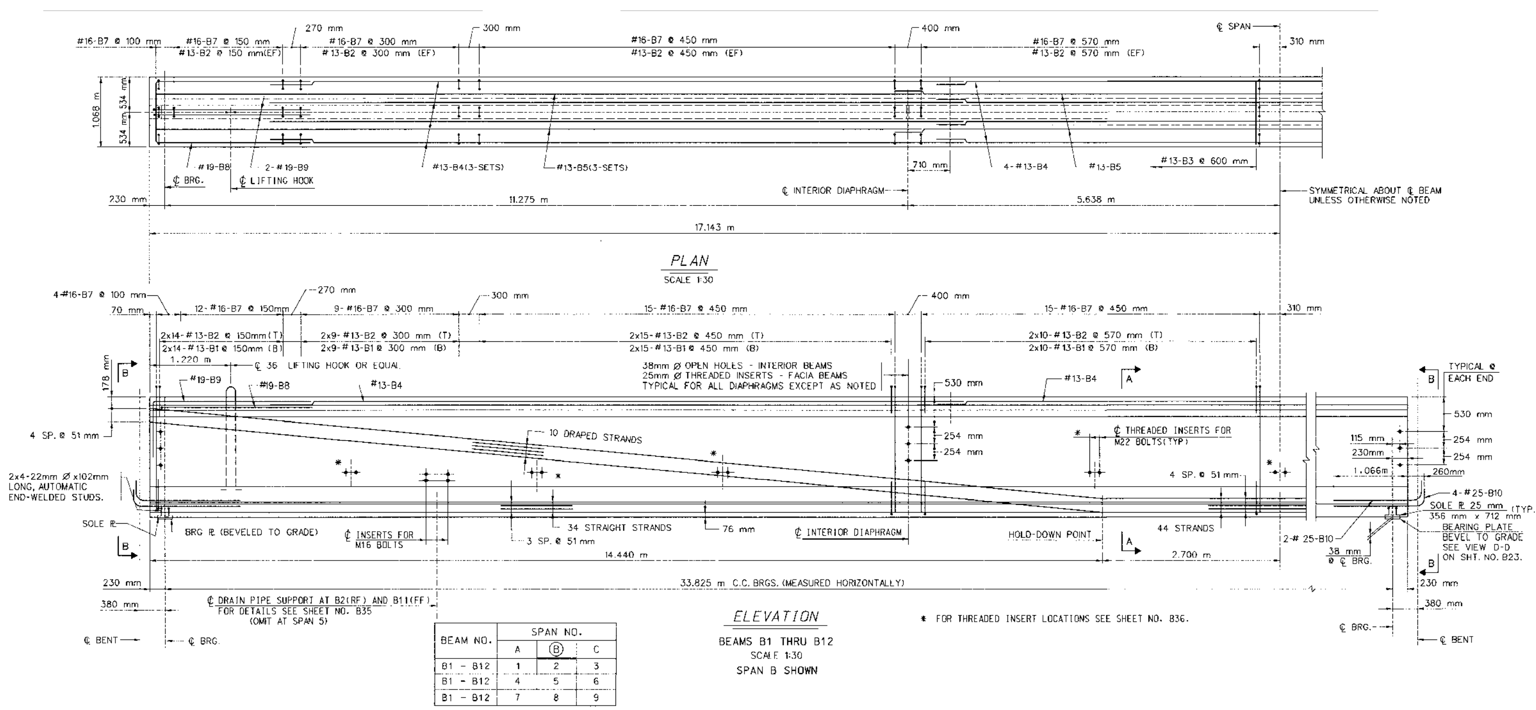


Figure VIII.B.6 Beams B1 thru B12 Sections and Views (1/2)



BEAM NO.	SPAN NO.		
	A	(B)	C
B1 - B12	1	2	3
B1 - B12	4	5	6
B1 - B12	7	8	9

Figure VIII.B.7 Beams B1 thru B12 Plan and Elevation (2/2)

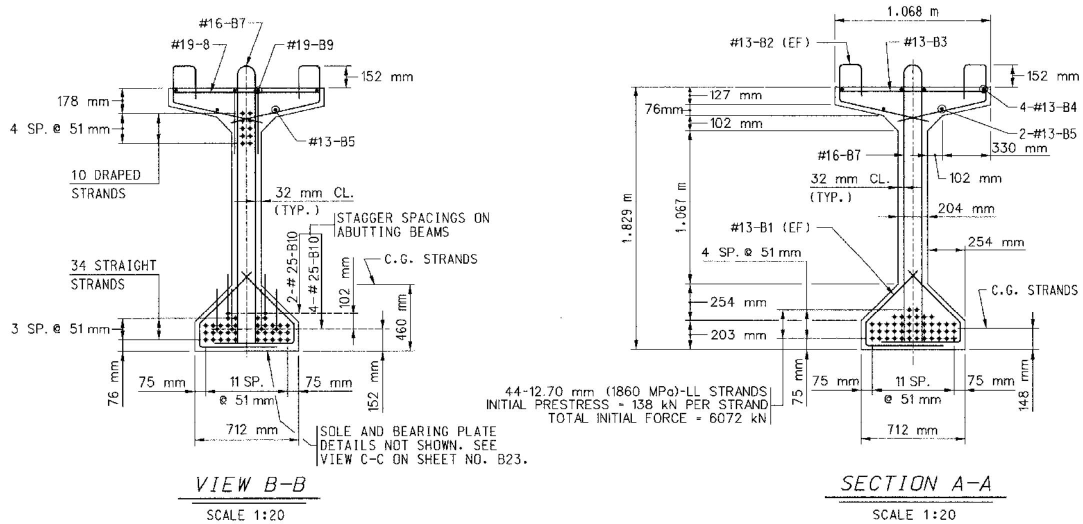


Figure VIII.B.8 Beams B1 thru B12 Sections and Views (2/2)

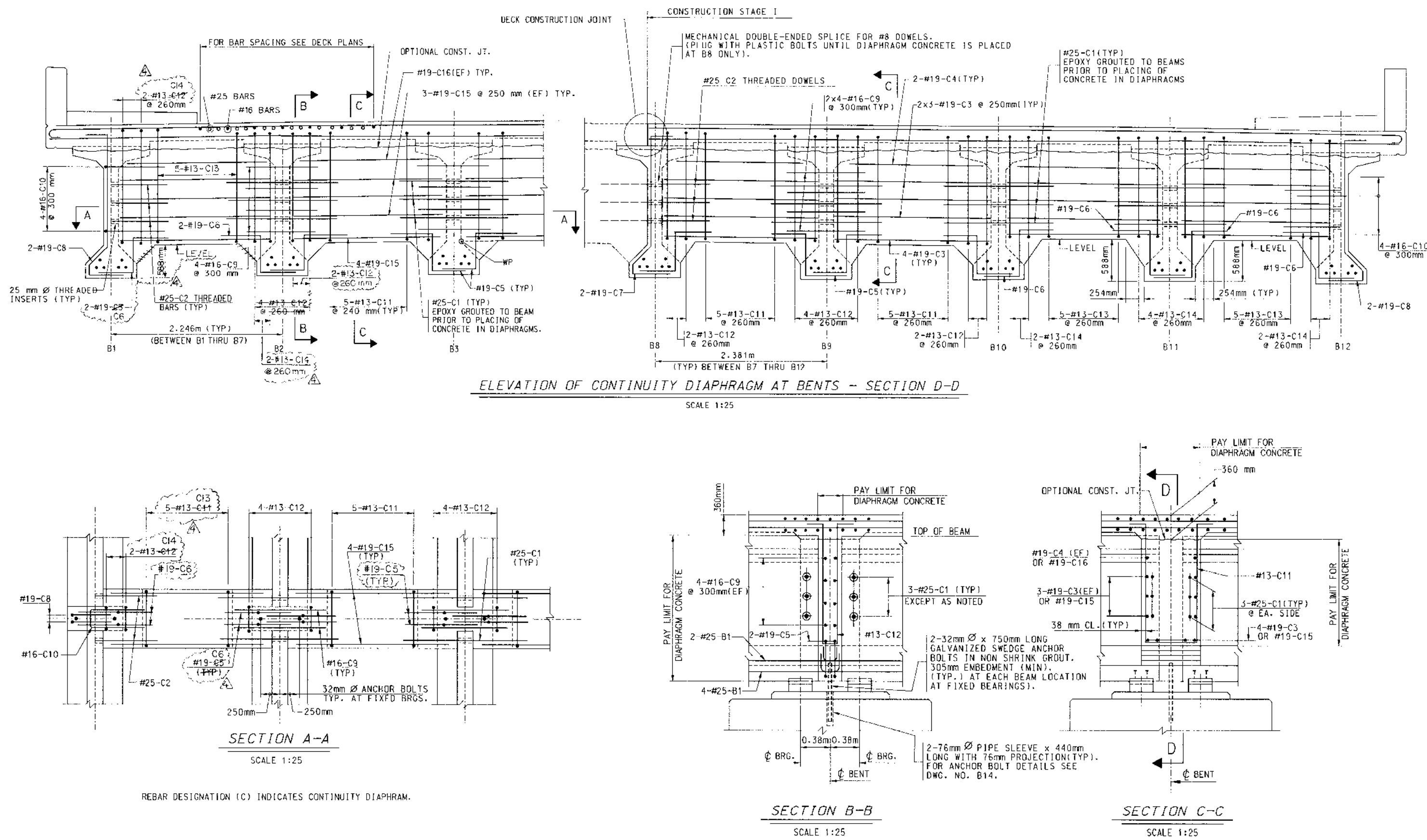


Figure VIII.B.9 Continuity Diaphragm at Bents

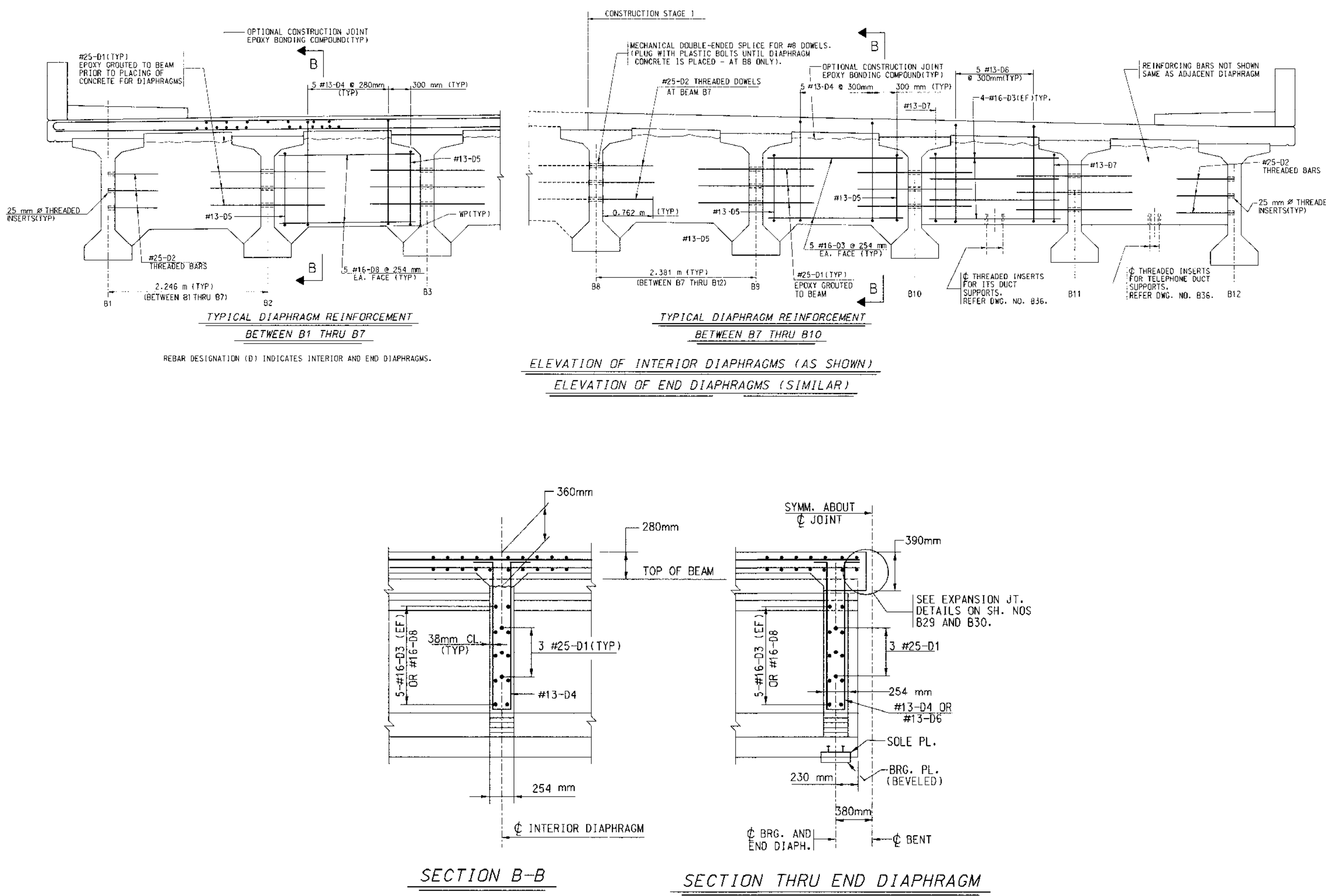


Figure VIII.B.10 Interior and End Diaphragm

TOP OF PAD ELEVATION AT ℄ BEARINGS IN METERS (m)												
BEAM	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10	B11	B12
ELEVATION	2.959	3.049	3.133	3.178	3.217	3.251	3.261	3.225	3.186	3.138	3.054	2.959

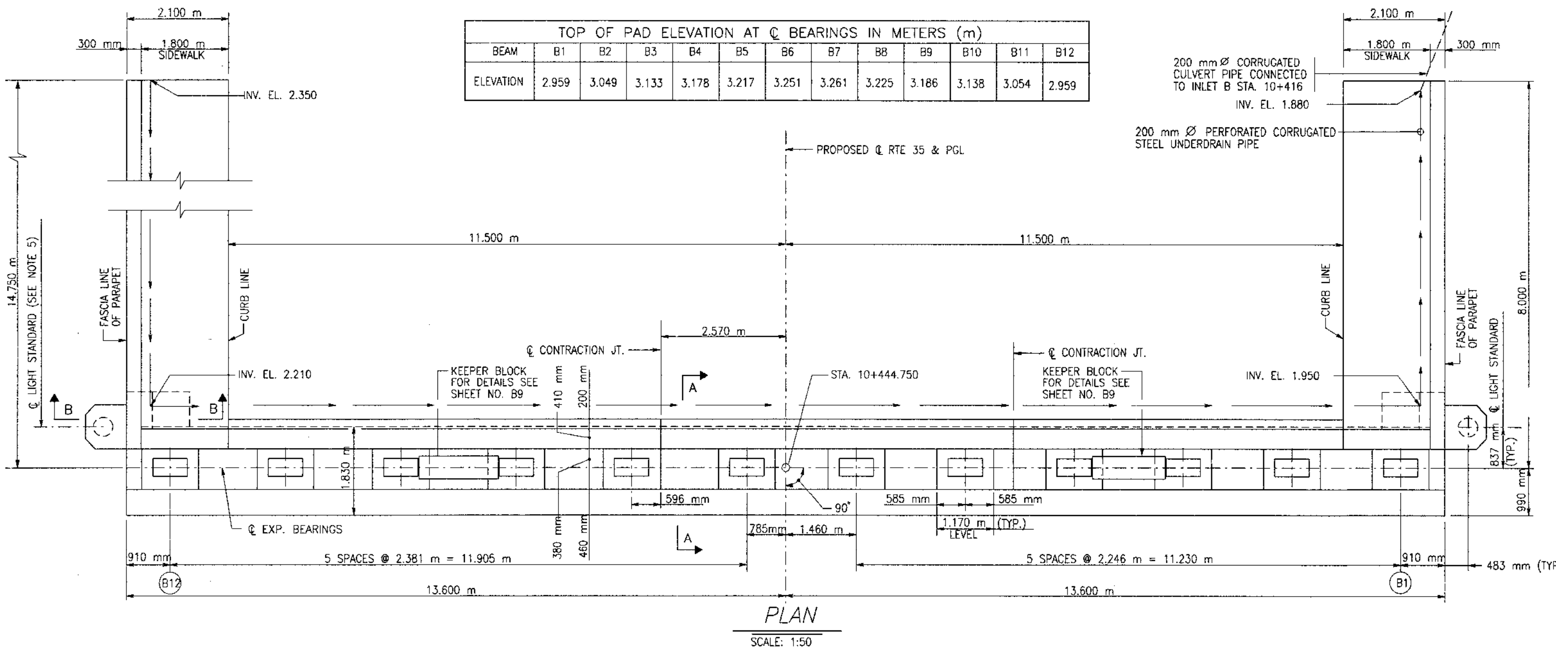


Figure VIII.C.1 South Abutment Plan

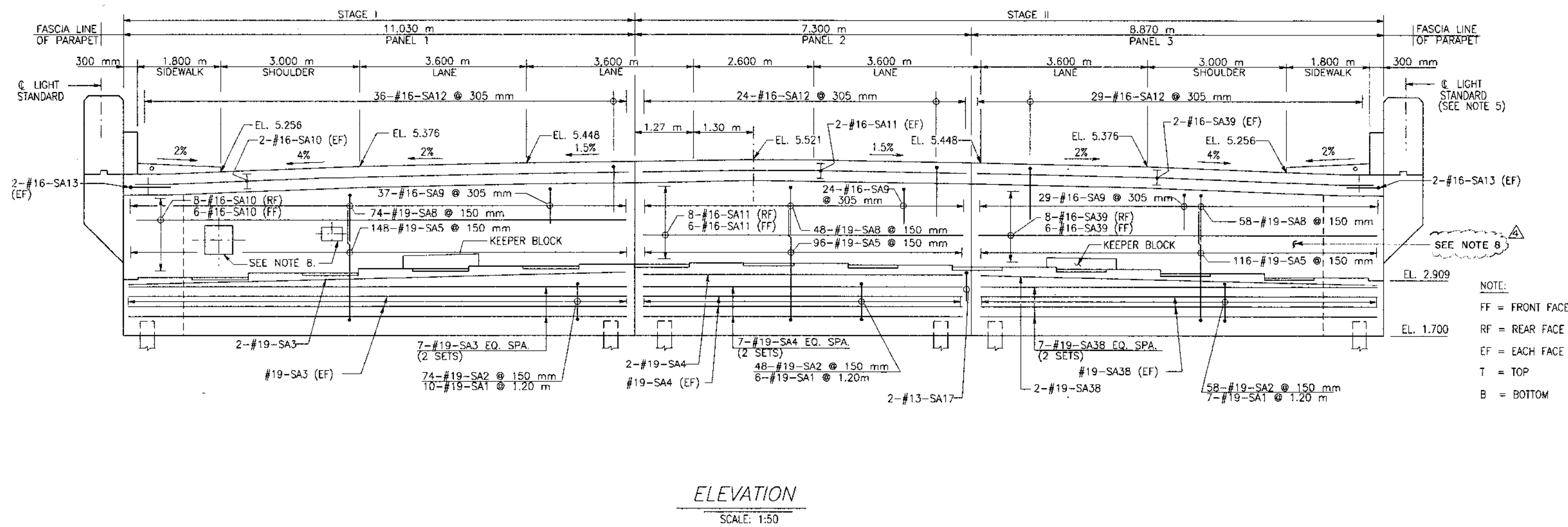


Figure VIII.C.2 South Abutment Elevation

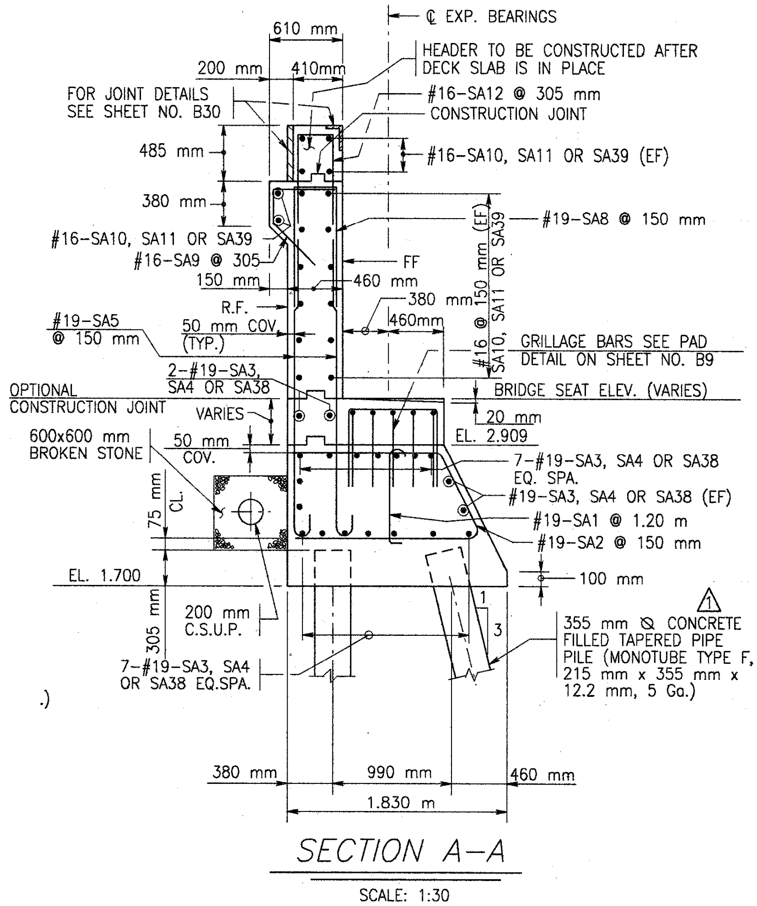


Figure VIII.C.3 South Abutment Section A-A

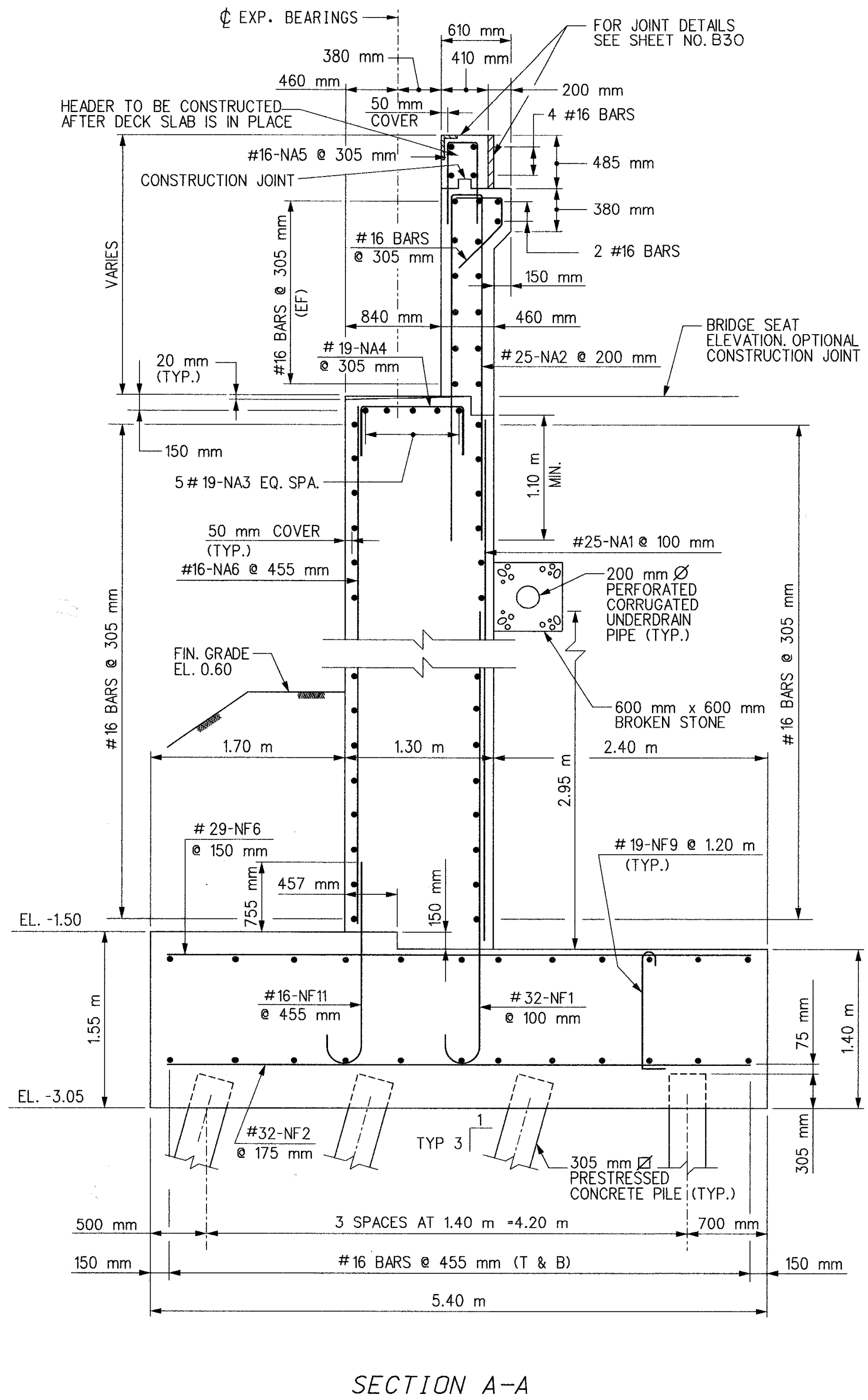
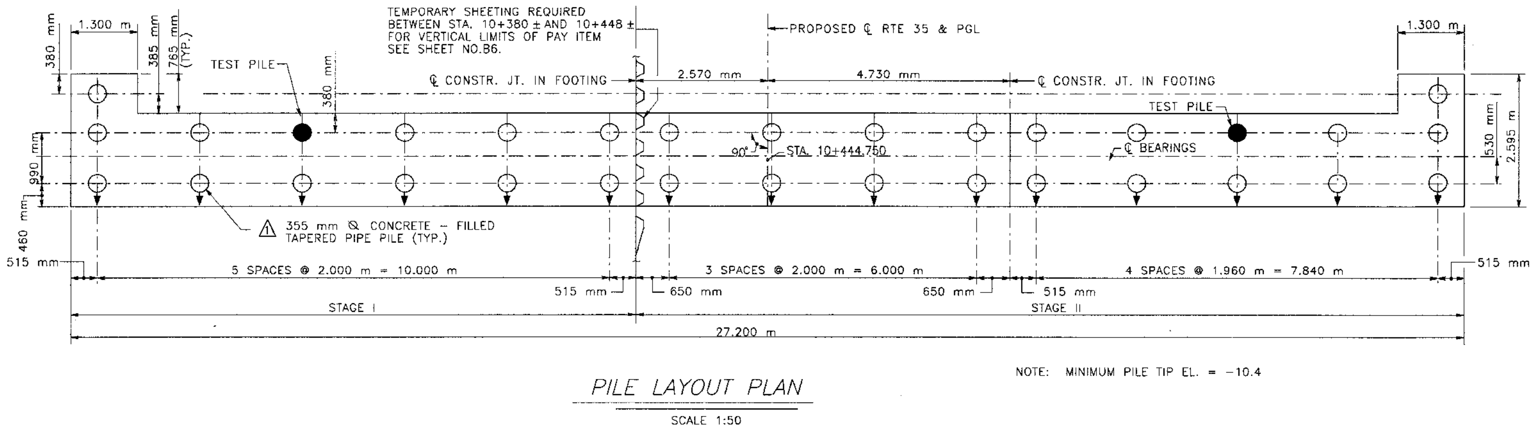


Figure VIII.C.4 North Abutment Section A-A



PILE LAYOUT PLAN

SCALE 1:50

Figure VIII.C.5 South Abutment Pile Layout Plan

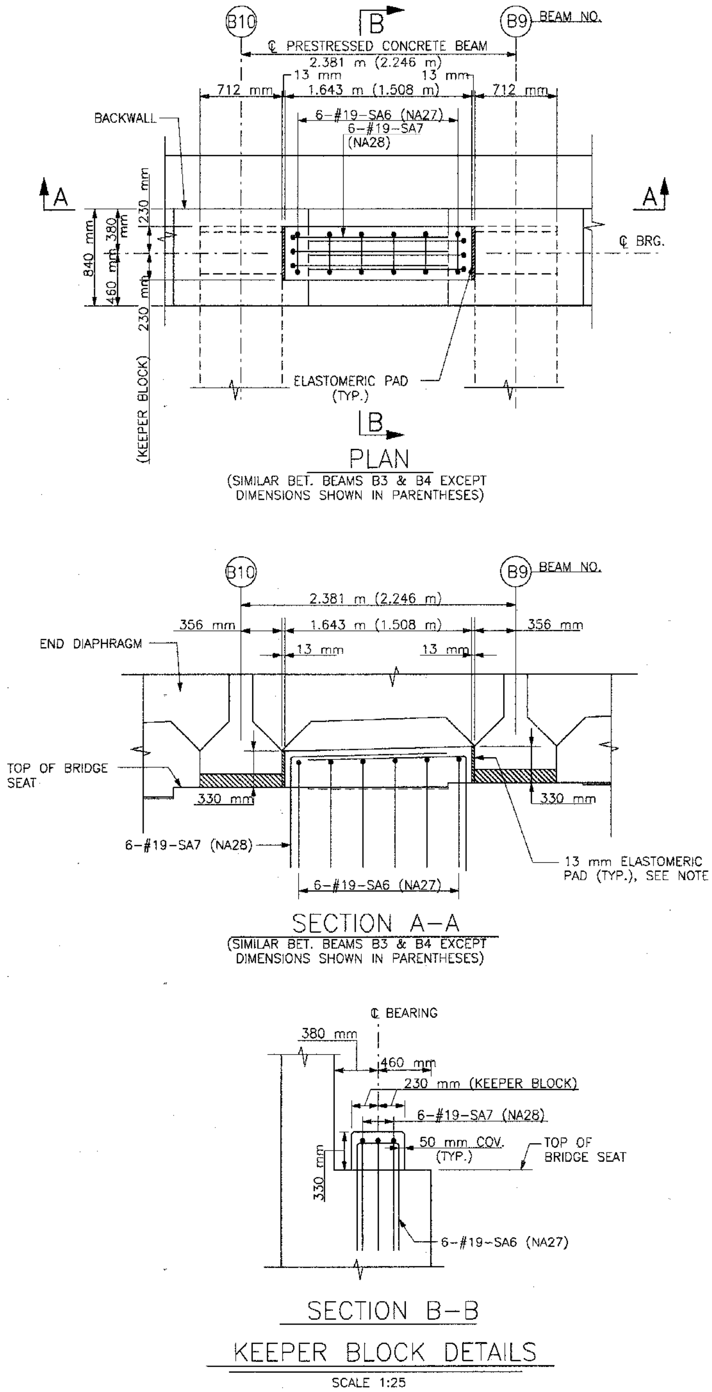
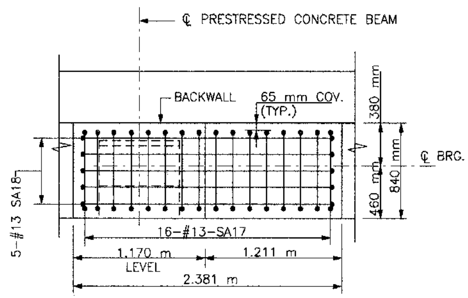
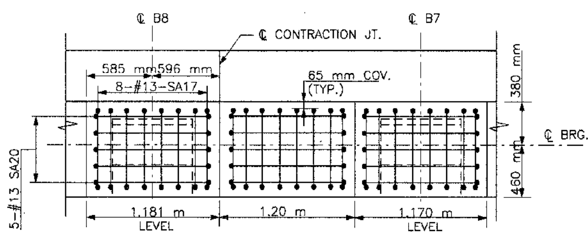


Figure VIII.C.6 South Abutment Keeper Block Details



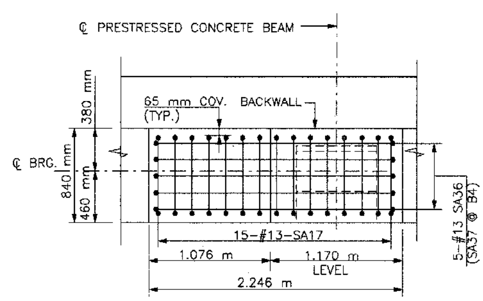
PAD DETAIL AT BEAMS B9, B10 AND B11

SCALE 1:25



PAD DETAIL AT BEAMS B7 AND B8

SCALE 1:25



PAD DETAIL AT BEAMS B2, B3, B4, B5 AND B6

SCALE 1:25

Figure VIII.C.7 South Abutment Pad Details

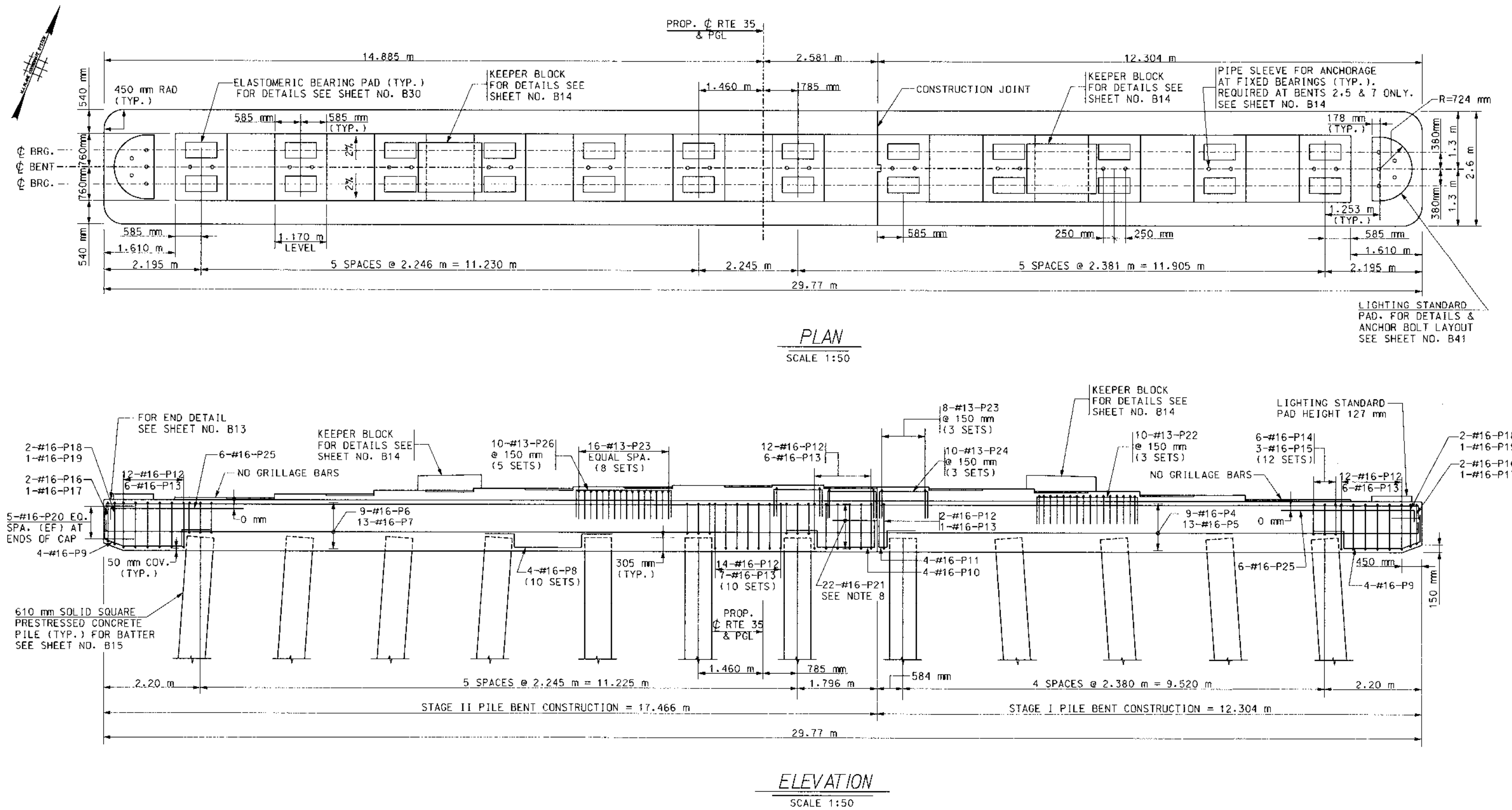
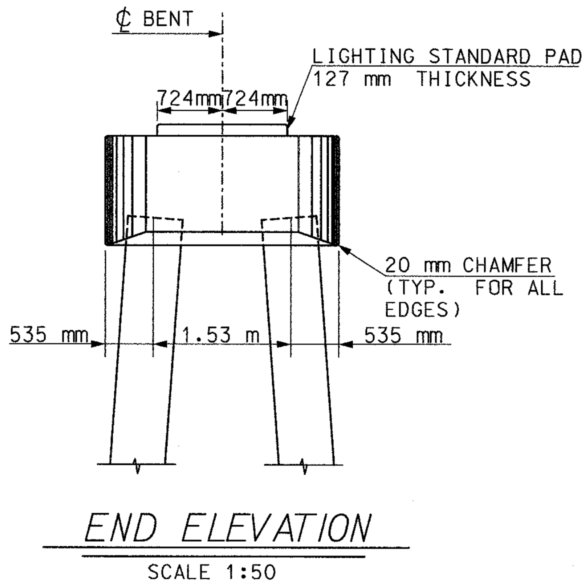


Figure VIII.C.8 Pile Bent Plan and Elevation



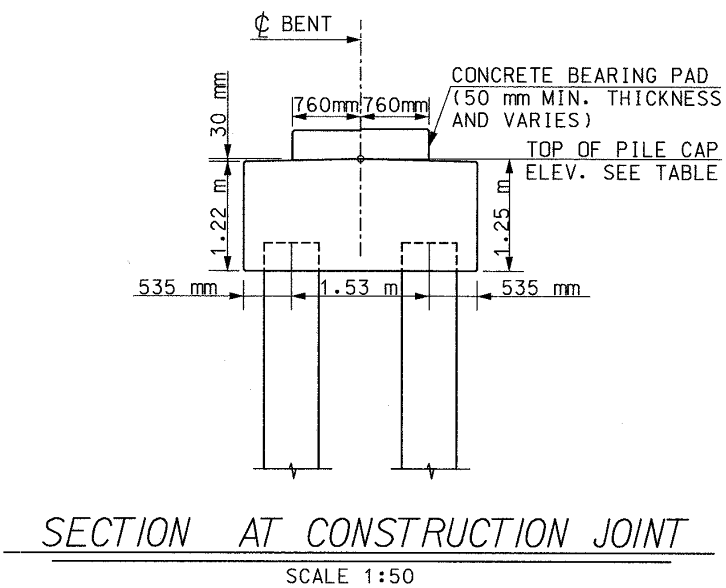


Figure VIII.C.9 Pile Bent Elevations

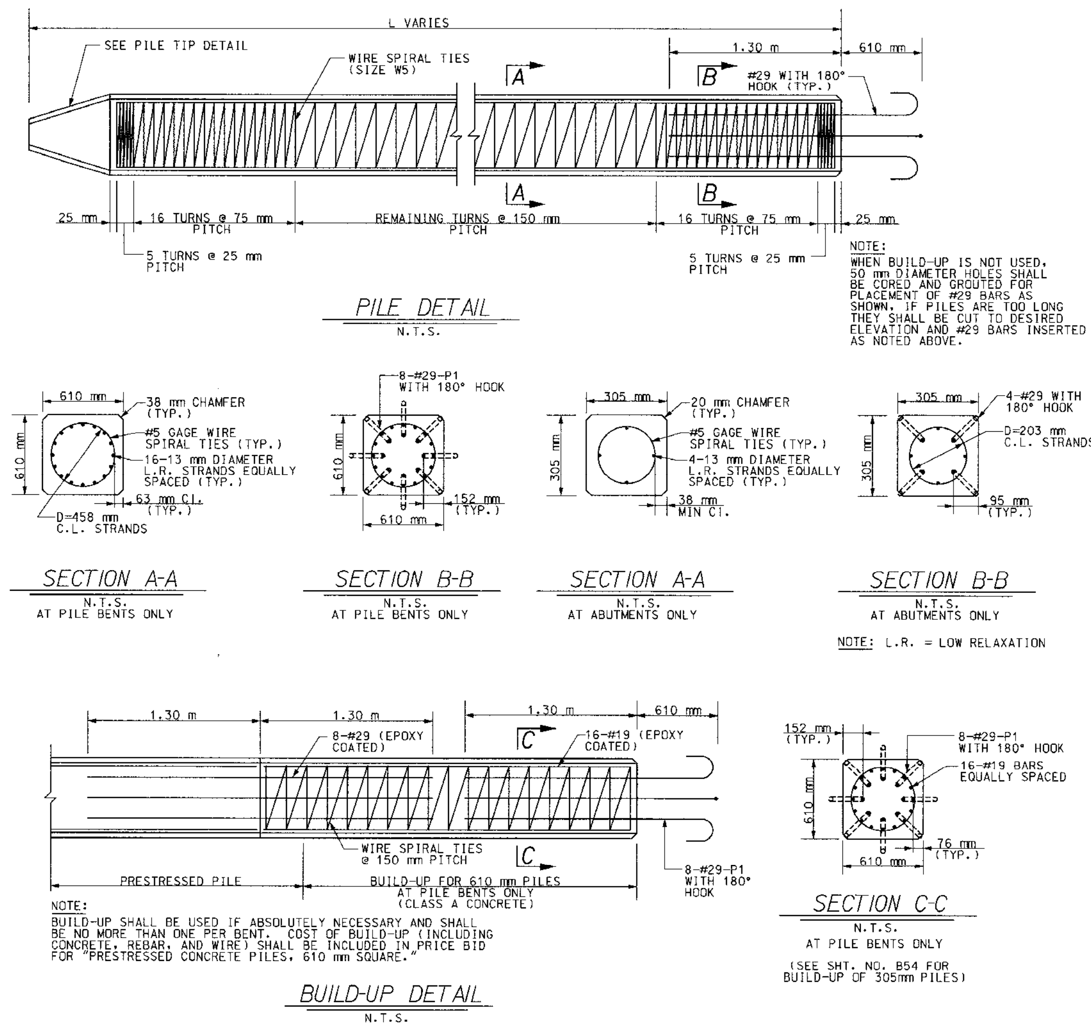


Figure VIII.C.10 Pile Details

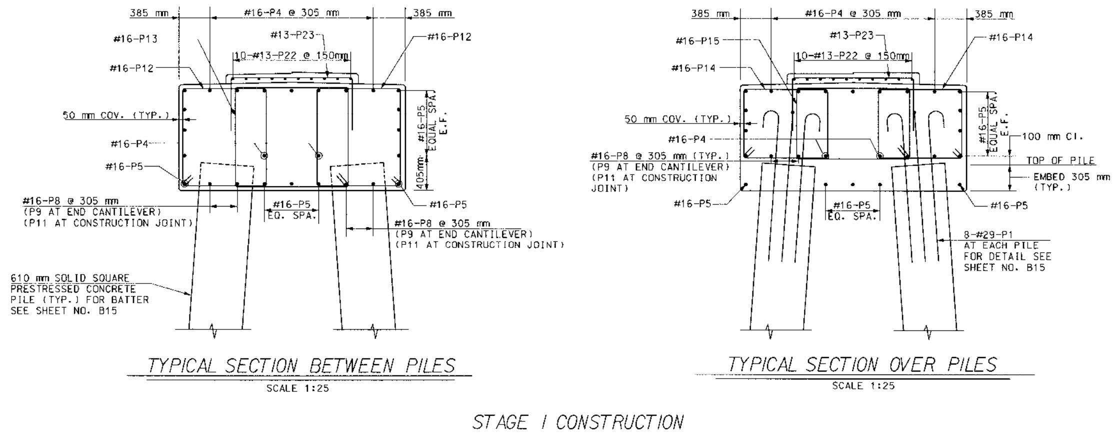


Figure VIII.C.11 Pile Bent Section Between Piles and Over Piles Stage I

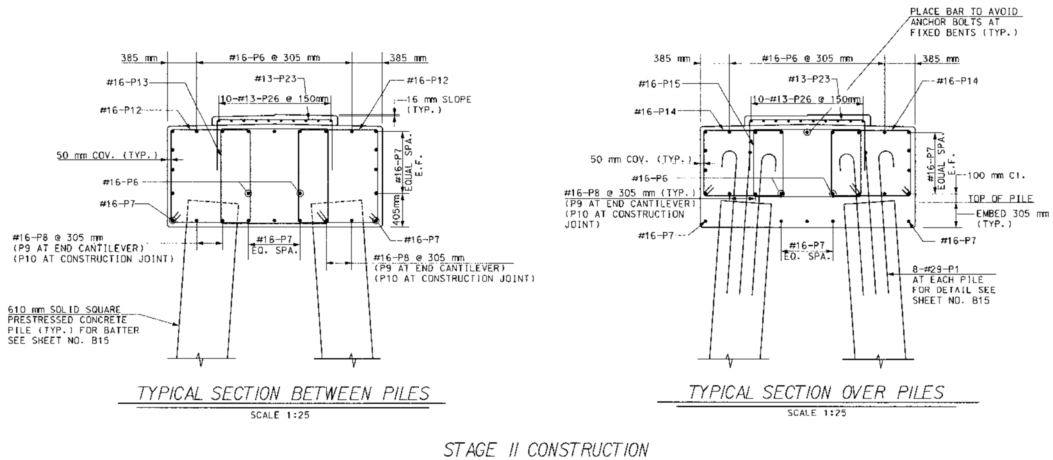


Figure VIII.C.12 Pile Bent Section Between Piles and Over Piles Stage II

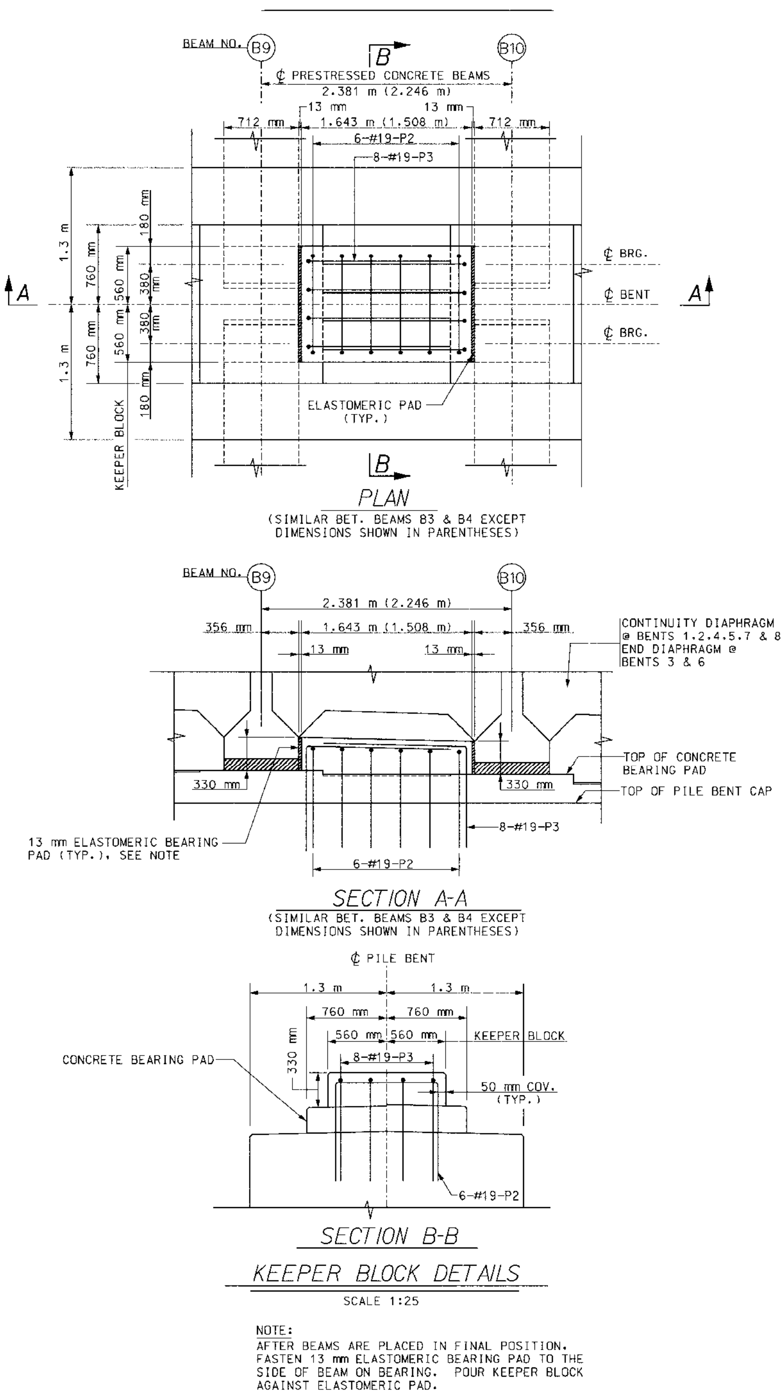


Figure VIII.C.13 Keeper Block Details at Pile Bents

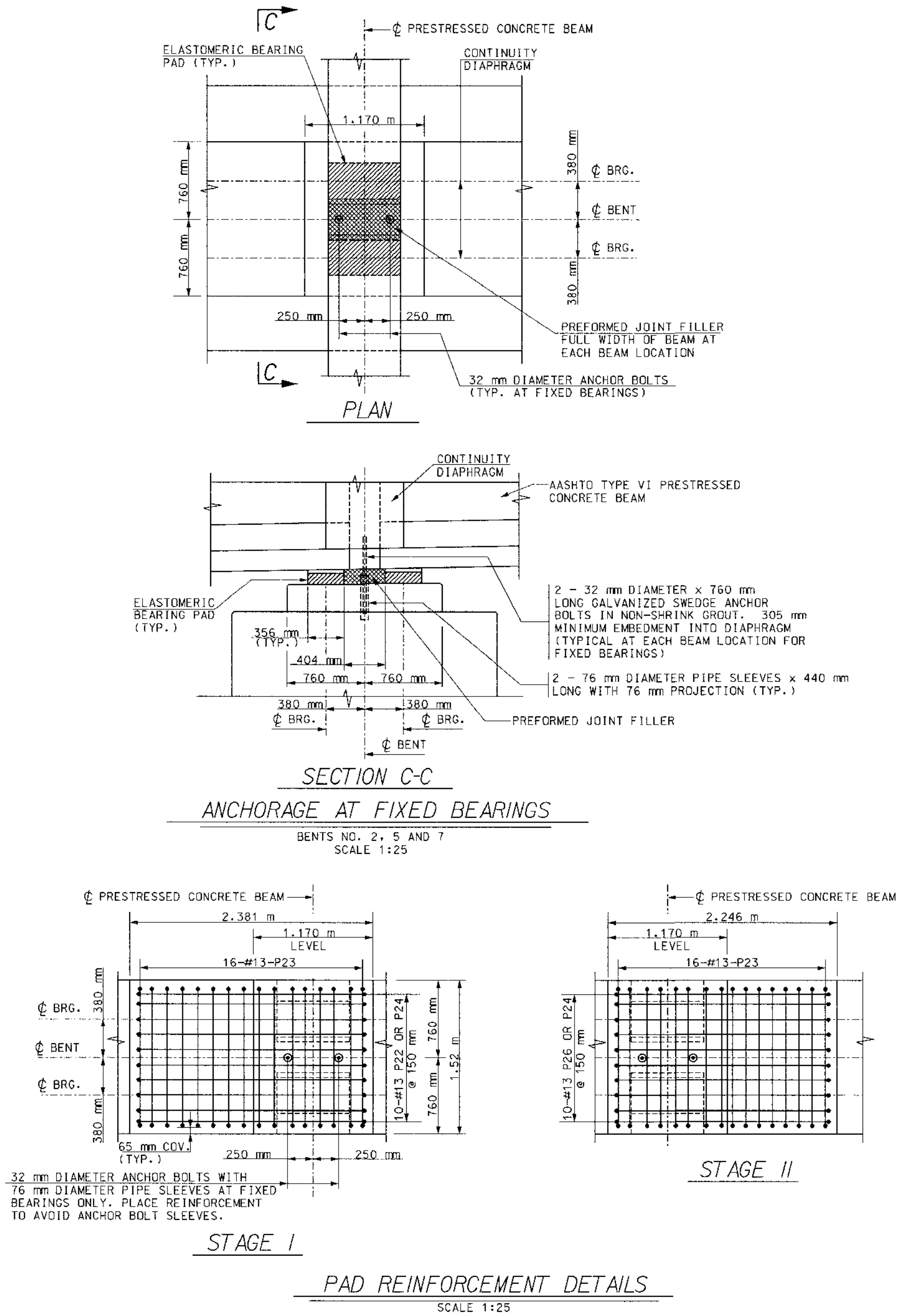


Figure VIII.C.14 Pad Reinforcement Details at Pile Bents