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MFN 06-407
Supplement 2

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Subject: **Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – Structural Analysis - RAI Numbers 3.8-81 S01, 3.8-85 S01, 3.8-86 S01, 3.8-88 S01, 3.8-92 S01, 3.8-93 S01, 3.8-94 S01, 3.8-96 S01 and 3.8-99 S01**

Enclosure 1 contains supplemental responses to the subject NRC RAIs resulting from the December 2006 Structural Follow-up audit. GE's original responses were submitted in the Reference 1 letter.

If you have any questions or require additional information regarding the information provided here, please contact me.

Sincerely,

Kathy Sedney for

James C. Kinsey
Project Manager, ESBWR Licensing

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References:

1. MFN 06-407, Letter from David H. Hinds to U. S. Nuclear Regulatory Commission, *Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – Structural Analysis - RAI Numbers 3.8-17, 3.8-24, 3.8-28, 3.8-32, 3.8-33 through 3.8-38, 3.8-44, 3.8-59, 3.8-62, 3.8-65, 3.8-69, 3.8-73, 3.8-76, 3.8-77, 3.8-79, 3.8-80, 3.8-81, 3.8-84, 3.8-85, 3.8-86, 3.8-88, 3.8-89, 3.8-92, 3.8-93 through 3.8-97, 3.8-99, 3.8-101, 3.8-102 and 3.8-103*, November 8, 2006
2. MFN 06-407, Supplement 1, Letter from James C. Kinsey to U. S. Nuclear Regulatory Commission, *Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – Structural Analysis - RAI Numbers 3.8-17 S01, 3.8-24 S01, 3.8-28 S01, 3.8-44 S01, 3.8-59 S01, 3.8-62 S01, 3.8-65 S01, 3.8-69 S01, 3.8-76 S01, 3.8-77 S01, 3.8-79 S01, 3.8-80 S01, 3.8-84 S01, 3.8-95 S01, 3.8-97 S01, 3.8-101 S01, 3.8-102 S01 and 3.8-103 S01 – Supplement 1*, February 1, 2007

Enclosure:

1. MFN 06-407, Supplement 2 – Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – Structural Analysis - RAI Numbers 3.8-81 S01, 3.8-85 S01, 3.8-86 S01, 3.8-88 S01, 3.8-92 S01, 3.8-93 S01, 3.8-94 S01, 3.8-96 S01 and 3.8-99 S01

cc:	AE Cubbage	USNRC (with enclosures)
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	eDRF	0000-0064-1130/2

ENCLOSURE 1

MFN 06-407, SUPPLEMENT 2

Response to Portion of NRC Request for

Additional Information Letter No. 38

Related to ESBWR Design Certification Application

Structural Analysis

RAI Numbers 3.8-81 S01, 3.8-85 S01, 3.8-86 S01, 3.8-88 S01, 3.8-92 S01, 3.8-93 S01, 3.8-94 S01, 3.8-96 S01 and 3.8-99 S01

Original Response previously submitted under MFN 06-407 without DCD updates is included to provide historical continuity during review.

NRC RAI 3.8-81

The DCD does not discuss testing and inservice inspection requirements for Other Seismic Category I Structures. This information is normally included in Section 3.8.4.7, but has not been provided in the ESBWR DCD. Describe any requirements for testing and inservice inspection of Other Seismic Category I Structures. Explain whether Regulatory Guide 1.160 and 10 CFR 50.65 requirements, related to structures monitoring and maintenance, are applicable to the ESBWR Other Seismic Category I Structures. If not, explain why not.

Include this information in new DCD Section 3.8.4.7.

GE Response

Regulatory Guide 1.160 will be referenced in a new DCD Tier 2 Section 3.8.4.7 for monitoring of the Seismic Category I structures of the ESBWR listed in DCD Tier 2 Table 19.2-4.

A markup of DCD Tier 2 Section 3.8.4.7 was provided in MFN 06-407.

NRC RAI 3.8-81, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

The new DCD Section 3.8.4.7 (Testing and In-Service Inspection Requirements) refers to monitoring of Seismic Category I structures, in accordance with Section 1.5 of RG 1.160, for those structures listed in Table 19.2-4. However, Table 19.2-4, which is referenced for the list of structures to be monitored, cannot be located. Clarify if this is the correct table reference. 10 CFR 50.65 also needs to be referenced, along with reference to RG 1.160. ESBWR Seismic Category II structures also are subject to 10 CFR 50.65 and RG 1.160. This needs to identified and discussed in the DCD.

In addition, DCD Section 3.8.4.7 does not discuss any special post-construction testing and/or inservice surveillance programs for Other Category I Structures (identified for staff review in SRP 3.8.4.I.7). These may include items such as periodic examination of inaccessible areas, monitoring of groundwater chemistry, monitoring for degradation of reinforced concrete/porous concrete/mud mat foundations due to flowing groundwater, and monitoring of settlements and differential displacements. Describe how will these be addressed, or explain why they are not applicable.

During the audit, the issues raised by the first paragraph above, are addressed by the review performed under RAIs 3.8-58. Also, reference to Table 19.2-4 will be deleted. For the issue raised in the second paragraph, GE indicated that condition monitoring and consideration of lessons learned will be defined in the DCD as a COL Action Item.

In addition, GE is to address:

1. *accessibility and periodic inspection of buried tanks, piping and components.*
2. *ensuring the leak-tight integrity of inaccessible, embedded portions of steel liners for concrete containments.*

GE Response

DCD Tier 2 Subsection 3.8.4.7 has been revised to reference NUREG-1801, 10 CFR 50.65 and RG 1.160.

Inaccessible areas are addressed in the response to NRC RAI 3.8-59 S01.

Concrete specified in the ESBWR is watertight, and a crystalline powder admixture waterproofing is used in the foundation. See also the response to NRC RAI 3.8-96, Supplement 1, Item (6).

Settlements are similarly investigated at the start of the COL approval activities. Allowable differential settlements in the ESBWR are addressed in response to NRC RAI 3.8-93 S01.

There are no C-I buried tanks, piping and components that need to be included in special inspection programs in the ESBWR design. Firewater piping is located inside concrete trenches that are easily accessible for maintenance and inspection.

The only liner portion of the RCCV not accessible for visual inspection is the portion under the sacrificial concrete located under the RPV. This liner portion is a small fraction of the entire liner surface area and will be subject to preservice examinations described in DCD Tier 2 Subsections 3.8.1.7.3 and 3.8.1.7.3.12. All other portions of the liner are accessible for visual inspection.

The page (p. 3.8-37) revised in DCD Tier 2 Revision 3 for this response is attached.

DCD Impact

As stated above.

NRC RAI 3.8-85

DCD Section 3.8.4.2 indicates that the design and construction of Other Seismic Category I Structures conform to ACI 349-01 and Regulatory Guide 1.142, November 2001, as indicated in Table 3.8-9. RG 1.142, states the staff's position on the use of ACI 349-97. Since the staff has not formally reviewed and endorsed ACI 349-01 at this time, identify all deviations between ACI 349-97/RG 1.142 and ACI 349-01 that affect the ESBWR design. Also provide the technical basis for ensuring that a comparable level of safety is achieved for each such deviation.

GE Response

In the attached table, the differences between ACI 349-01 and ACI 349-97/RG 1.142 (with NRC-accepted supplemental requirements) that affect the ESBWR design are compared and summarized. As shown in the table, the following items are the most important ones that affect the design of ESBWR structures:

1. Design load combinations shown in DCD Tier 2 Table 3.8-15 satisfy requirements of ACI 349 (including exceptions of RG 1.142) and SRP 3.8.4.
2. Two kinds of dynamic fluid effects are considered in the design of the containment and buildings. One is hydrodynamic load in the suppression pool due to LOCA/SRV discharge, and the other is sloshing loads due to earthquakes.
3. DCD Tier 2 Section 3.8 does not postulate loads that are due to malevolent vehicle assault, aircraft impact, and accidental explosion.

No DCD change was made in response to this RAI.

Table 3.8-85(1) Comparison of ACI 349-97/RG 1.142 and ACI 349-01

RG 1.142, Regulatory Positions on the use of ACI 349-97		ACI 349-97	ACI 349-01	ESBWR
1	Structures required to withstand pressures and to maintain a certain degree of leaktightness during operating and accident conditions will be reviewed in accordance with the provisions of Section 3.8.3 of the Standard Review Plan, NUREG-0800. To include these structures under ACI 349-97, the following additional provisions should be added to ACI 349-97. a. Provision for crack control under service loads, including test pressure load; b. Provisions to deal with the transition from the concrete portion of the drywell to the steel portion of the drywell; and c. Provisions for preoperational testing and inservice inspections.	No equivalent provision is given in ACI 349.	No equivalent provision is given in ACI 349.	RG 1.142 is applicable.
2	When concrete structures are used to provide radiation shielding, provisions of ANSI/ANS 6.4-1997 (Appendix A) are applicable to the extent that they enhance the radiation shielding function of these structures. Reduction in shielding effectiveness from embedments, penetrations, and openings should be fully evaluated.	The provisions noted for radiation shielding in ACI 349 are as follows. The other description regarding shielding function was not found. 3.3.1 Concrete aggregates shall conform to one of the following specifications: a) "Specification for Concrete Aggregates" (ASTM C 33). b) "Specification for Aggregates for Radiation-Shielding Concrete" (ASTM C 637).	Same as ACI 349-97 adding: Exception: Aggregates failing to meet ASTM C 33 but which have been shown by special test or actual service to produce concrete of adequate strength and durability shall be permitted to be used for normal-weight concrete where authorized by the engineer.	RG 1.142 is applicable.
3	Where structural components, normally defined as walls, slabs, and foundations, actually exhibit a structural response	Chapter 10—Flexure and Axial Loads Chapter 10 is identical to that of ACI 318 except as described below for Sections	CHAPTER R10—Flexure and Axial Loads Chapter 10 is identical to that of ACI 318	Basically, walls, slabs, and foundations are designed using section

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
consistent with the response of structural frames, such 1.142-7 components should conform to the requirements of Chapters 10, 11, and 21 of ACI 349-97, in addition to Chapters 13, 14, and 15 as appropriate. The response of structural components should be considered as consistent with the response of structural frames when the flexural moment from seismic loads exceeds two-thirds of the design flexural capacity of the section in the absence of axial forces.	10.6 . Chapter 11—Shear and Torsion. The commentary in ACI 318 is applicable to this chapter except as noted herein. Chapter 21—Special Provisions for Seismic Design	except as described below for Sections 10.6. CHAPTER R11— Shear and Torsion The commentary in ACI 318 is applicable to this chapter except as noted herein. Chapter 21—Special Provisions for Seismic Design.	forces and moments obtained from FE analyses, and "Flexure and Axial Loads" (ACI 349 Chapter 10) and "Shear and Torsion" (Chapter 11) are considered in their design.
4 In addition to meeting the standards of Section 1.3.1 of ACI 349-97, the concrete QA inspectors should have sufficient experience in reinforced and prestressed concrete practice as applied to the construction of nuclear power plants.	1.3.1 The Owner is responsible for the inspection of concrete construction throughout all work stages. The Owner shall require compliance with design drawings and specifications and keep records required for quality assurance of construction, fabrication, manufacture or installation, and for traceability.	1.3.1 The Owner is responsible for the inspection of concrete construction throughout all work stages. The Owner shall require compliance with design drawings and specifications. The Owner shall also keep records required for quality assurance and traceability of construction, fabrication, material procurement, manufacture, or installation.	RG 1.142 is applicable.
5 In lieu of the frequency of compressive strength testing specified by Section 5.6.1.1 of ACI 349-97 or that specified by ASME/NQA-2, the following is acceptable: Samples for strength tests of concrete should be taken at least once per day for each class of concrete placed or at least once for each 100 cu yd of concrete placed. When the standard deviation for 30 consecutive tests of a given class is less than 600 psi, the amount of concrete placed between tests may be increased by 50 cu yd for each 100 psi the standard deviation is below 600 psi, except that the	5.6.1.1 Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 150 yd3 of concrete, nor less than once for each 5000 ft2 of surface area for slabs or walls.	Same as ACI 349-97.	RG 1.142 is applicable.

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
minimum testing rate should not be less than one test for each shift when concrete is placed on more than one shift per day or not less than one test for each 200 cu yd of concrete placed. The test frequency should revert to once for each 100 cu yd placed if the data for any 30 consecutive tests indicate a higher standard deviation than the value controlling the decreased test frequency.			
6 The load factors used in Section 9.2.1 of ACI 349-97 are acceptable to the staff except for the following: 6.1 In load combinations 9, 10, and 11, 1.2 To should be used in place of 1.05 To. 6.2 In load combination 6, 1.4Pa should be used in place of 1.25Pa.	9.2.1 The required strength U shall be at least equal to the greatest of the following: 1. U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7Ro 2. U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7Eo + 1.7Ro 3. U = 1.4D + 1.4F + 1.7L + 1.7H + 1.7W + 1.7Ro 4. U = D + F + L + H + To + Ro + Ess 5. U = D + F + L + H + To + Ro + Wt 6. U = D + F + L + H + Ta + Ra + 1.25Pa 7. U = D + F + L + H + Ta + Ra + 1.15Pa + 1.0(Yr + Yj + Ym) + 1.15Eo 8. U = D + F + L + H + Ta + Ra + 1.0Pa + 1.0(Yr + Yj + Ym) + 1.0Ess 9. U = 1.05D + 1.05F + 1.3L + 1.3H + 1.05To + 1.3Ro 10. U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3Eo + 1.05To + 1.3Ro	Sama as ACI 349-97.	Design load combinations shown in DCD Table 3.8-15 satisfy requirements of RG 1.142 as follows: 2. 1.05D + 1.05F + 1.3L + 1.3H + <u>1.3To</u> + 1.3Ro 4. 1.05D + 1.05F + 1.3L + 1.3H + 1.3W + <u>1.3To</u> + 1.3Ro 8. D + F + L + H + Ta + Ra + <u>1.5Pa</u>

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
	11. U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3W + 1.05To + 1.3Ro		
7 Loads that are due to malevolent vehicle assault, aircraft impact, and accidental explosion should be taken as Wt in load combination 5.	There is no provision about vehicle assault, aircraft impact, and accidental explosion in ACI 349-97.	Same as ACI 349-97.	DCD Section 3.8 does not postulate loads that are due to malevolent vehicle assault, aircraft impact, and accidental explosion.
8 Hydrodynamic loads associated with seismic loads (i.e., the impulsive and sloshing loads for fluids in tanks) are to be considered as Ess in load cases 4 and 8, and EO in load cases 2, 7, and 10. All other hydrodynamic loads should be taken as Yj, in load combinations 7 and 8.	There is no provision noted hydrodynamic loads.	Same as ACI 349-97.	Hydrodynamic loads of pool water due to earthquakes are included in Seismic loads in the design of the RBFB and RCCV, refer to the response to NRC RAI 3.8-15 and 16.
9 The consideration of loads that are due to pool dynamics for the concrete structures in pressure-suppression containments will be evaluated on a case-by-case basis.	R9.1—General The discharge of safety relief valves into a suppression pool generates loads which are unique to BWR power plant structures. Specific classification of these loads is not given by the Code at this time due to ongoing efforts by the industry to quantify them.	Same as ACI 349-97.	Loads that are due to pool dynamics for the concrete structures are considered in the RCCV design. Refer to the response to NRC RAI 3.8-15 and 16.
10 The local exceedance of section strengths in accordance with Appendix C of ACI 349-97 is acceptable in analyses for impactive or impulsive effects of Yr, Yj, and Ym in load combinations 7 and 8, load combinations of tornado-generated missiles, and loads described in Regulatory Position 7 in load combination 5 except for the following: 10.1 The deformation and degradation of	In ACI 349 following provisions are provided. 9.0—Notation Yj = jet impingement load, or related internal moments and forces, on the structure generated by a postulated pipe break Ym = missile impact load, or related internal moments and forces, on the	Same as ACI 349-97.	Design based on ductility ratios is not included in DCD Section 3.8. ESBWR complies with ACI 349-01 and RG 1.142.

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
the structure resulting from such an analysis must not cause loss of function of any safety-related structures, systems, or components. 10.2 The section strengths should be adequate to satisfy these load combinations without the impactive or impulsive effects.	structure generated by a postulated pipe break, such as pipe whip Yr = loads, or related internal moments and forces, on the structure generated by the reaction of the broken pipe during a postulated break		
10.3 In Section C.3.5 of ACI 349-97, the maximum permissible ductility ratios ( ) when a concrete structure is subjected to a pressure pulse caused by compartment pressurization or external explosion (blast) loading should be as follows. 10.3.1 For the structure as a whole =1.0 except as noted in 10.5. 10.3.2 For a localized area in the structure = 3.0.	C.3.5 The permissible ductility ratio in flexure shall not exceed 3.0 for loads such as blast and compartment pressurization which could affect the integrity of the structure as a whole.	Same as ACI 349-97.	
10.4 In Section C.3.7 of ACI 349-97, where shear controls the design, the maximum permissible ductility ratios should be as follows. 10.4.1 When shear is carried by concrete alone, = 1.0. 10.4.2 When shear is carried by a combination of concrete and stirrups or bent bars, = 1.3.	C.3.7 For beams, walls, and slabs where shear controls design, the permissible ductility ratio shall be taken as: a) For shear carried by concrete alone, the permissible ductility ratio shall be 1.3. b) For shear carried by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6, or c) For shear carried completely by stirrups, the permissible ductility ratio shall be 3.0.	Same as ACI 349-97.	

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
10.5 In Section C.3.8 of ACI 349-97, the maximum permissible ductility ratio in flexure should be as follows: 10.5.1 When the compressive load is greater than 0.1f'c Ag or one-third of that which would produce balanced conditions, whichever is smaller, the maximum permissible ductility ratio should be 1.0. 10.5.2 When the compression load is less than 0.1f'c Ag or one-third of that which would produce balanced conditions, whichever is smaller, the permissible ductility ratio should be as given in C.3.3 or C.3.4 of ACI 349-97. 10.5.3 The permissible ductility ratio should vary linearly from 1.0 to that given in C.3.3 or C.3.4 of ACI 349-97 for conditions between those specified in 10.5.1 and 10.5.2.	C.3.8 For beam-columns, walls, and slabs carrying axial compression loads and subject to impulsive or impactive loads producing flexure, the permissible ductility ratio in flexure shall be as follows: a) When compression controls the design, as defined by an interaction diagram, the permissible ductility ratio shall be 1.3. b) When the compression load does not exceed 0.1f'cAg or one-third of that which would produce balanced conditions, whichever is smaller, the permissible ductility ratio shall be as given in C.3.3 or C.3.4. c) The permissible ductility ratio shall vary linearly from 1.3 to that given in C.3.3 or C.3.4 for conditions between those specified in (a) and (b).	Same as ACI 349-97.	
10.6 In Section C.2.1 of ACI 349-97, the dynamic increase factor is to be considered as 1.0 for all materials when the dynamic load factor associated with the impactive or impulsive loading is less than 1.2.	C.2.1 Dynamic increase factors (DIF) appropriate for the strain rates involved may be applied to static material strengths of steel and concrete for purposes of determining section strength but shall not exceed the following: Material DIF Reinforcing steel fy = 40 ksi .... 1.20 fy = 50 ksi .... 1.15 fy = 60 ksi 1.10	Same as ACI 349-97.	

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
	Prestressing steel .... 1.00 Concrete Axial and flexural compression.. 1.25 Shear.......................................1.10		
11 The local exceedance of section strengths in accordance with Appendix C of ACI 349-97 is acceptable under the impactive and impulsive loadings associated with malevolent vehicle assault, aircraft impact, turbine missiles, and a localized pressure transient during an explosion, subject to the applicable exceptions of Regulatory Position 10.	ACI 349 provides following provisions: C.1.4 Impactive loads are time-dependent loads due to collision of masses which are associated with finite amounts of kinetic energy. Impactive loading may be defined in terms of time-dependent force or pressure. Impactive loads to be considered shall include, but not be limited to, the following types of loading: (a) tornado-generated missiles; (b) whipping pipes; (c) aircraft missiles; (d) fuel cask drop; and (e) other internal and external missiles. C.1.5 Impulsive loads are time-dependent loads which are not associated with collision of solid masses. Impulsive loads to be considered shall include, but not be limited to, the following types of loading: (a) jet impingement; (b) blast pressure; (c) compartment pressurization; and (d) pipe-whip restraint reactions.	Same as ACI 349-97.	DCD Section 3.8 does not postulate loads that are due to malevolent vehicle assault, aircraft impact, and accidental explosion.

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
12 The generic criteria of Appendix A, "Thermal Consideration," of ACI 349-97 are acceptable for the analysis of structures under loads To and Ta.	A.1.1 Nuclear safety related reinforced concrete structures shall conform to the minimum provisions of this Code and to the special provisions of this Appendix for structural members subjected to time-dependent and position-dependent temperature variations. A.1.2 The provisions of this Appendix apply to concrete structures which are subjected to normal operating conditions as well as thermal accident conditions and which have restraint such that thermal strains would result in thermal stresses. A.1.3 The design provisions of this Appendix are based on the strength design method. The assumptions, principles, and requirements specified in 10.1 and 10.2 are applicable for both normal operating and accident conditions.	Same as ACI 349-97	Analyses of structures under loads To and Ta are performed in compliance with Appendix A of ACI 349.
13 The design of composite members used in modular construction should conform to the intent of Code provisions of Chapter 10.14 and Chapter 17 of ACI 349 (i.e., the same rules used in computing the strength of regular reinforced concrete should apply). Until ACI 349-97 contains more specific requirements for modular construction, future designs will be evaluated on a case-by-case basis.	10.14—Composite compression members Chapter 17—Composite Concrete Flexural Members	Same as ACI 349-97 except following descriptions and provision. 1.1.7.2 This Code does not govern the design of structural concrete slabs cast on stay-in-place, composite steel form deck. Concrete used in the construction of such slabs shall be governed by Parts 1, 2, and 3 of this Code, where applicable. 10.16—Composite compression members	Same as ACI 349-01. RB floor slabs that are composite structures are designed using the design methods for regular reinforced concrete, i.e., steel plates are regarded as equivalent reinforcing bars in design calculations.

RG 1.142, Regulatory Positions on the use of ACI 349-97	ACI 349-97	ACI 349-01	ESBWR
14 Slabs and walls that frame into concrete containments and will participate in resisting accident and seismic loads should meet the standards of ACI 349-97 or ACI 359 as appropriate.	No equivalent description in ACI 349.	Same as ACI 349-97.	As stated in DCD Section 3.8.1.1.3, structural components that are integral with the containment structure are treated the same as the containment as far as loads and loading combinations are concerned in the design. See also response to NRC RAI 3.8-4.
15 Members that are subject to torsion and combined shear and torsion should be evaluated to the standards of Section 11.6 of ACI 318-99 instead of the requirements of Section 11.6 of ACI 349-97.	11.6—Combined shear and torsion strength for nonprestressed members with rectangular or flanged sections 11.6.1 Torsion effects shall be included with shear and flexure where factored torsional moment Tu exceeds f(0.5 Sx2y). Otherwise, torsion effects may be neglected.	CHAPTER 11—SHEAR AND TORSION 11.6.1 It shall be permitted to neglect torsion effects when the factored torsional moment Tu is less than: (a) for nonprestressed members: $\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)$ (b) for prestressed members: $\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{f_{pc}}{4\sqrt{f'_c}}}$	Section 11.6 of ACI 318-99 has been included in Section 11.6 of ACI 349-01.

NRC RAI 3.8-85, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

The Table provided in GE's response compares the 15 NRC Regulatory Positions presented in RG 1.142 against ACI 349-97, ACI 349-01, and ESBWR. The RAI requested that GE make the comparison between the staff's current position (ACI 349-97, supplemented by RG 1.142) against the unreviewed ACI 349-01, with consideration of the qualifications identified in RG 1.142 (what the DCD states). To minimize the effort, it would be acceptable to make this comparison only for those provisions in the two sets of codes that are being used. As was done in RAI 3.8-5 for the ASME Code, it would be useful to separate those items which are more stringent in ACI 349-01 from those that are less stringent in ACI 349-01. For those provisions which are less stringent (i.e., less conservative), provide the technical basis for their acceptance, which can be shown by demonstrating that an equivalent level of safety will be achieved.

During the audit, GE indicated that they will review the above and will determine the best way to address this issue.

GE Response

There are two main changes in the ACI 349-01 Edition versus the ACI 349-97 Edition.

First, ACI 349-01 is based on ACI 318-95, (except for Chapter 12 which is based on ACI 318-99) while ACI 349-97 is based on ACI 318-89 (Revised 1992, except for Chapter 12 which is based on ACI 318-95). It should be noted that ACI 349 is a dependent code to ACI 318, and most of the changes in ACI 318 are incorporated in subsequent editions of ACI 349. ACI 318 contains a Commentary for the changes introduced. In general, these changes do not reduce margins and are updates to clarify the code language based on questions received from practitioners.

The second major change affects Appendix B, *Anchoring to Concrete*, and involves going from the concrete-failure cone method to the Concrete Capacity Design (CCD) method. This is also addressed in later versions of ACI 318, Appendix D. The NRC under RG 1.199 (Nov. 2003) accepted ACI 349-01 Appendix B with some exceptions. RG 1.199 is met in the ESBWR design.

In the attached table, the differences between ACI 349-01 and ACI 349-97 for the provisions applicable to the ESBWR design are summarized. It can be concluded that ACI 349-01 is more stringent than ACI 349-97.

DCD Impact

No DCD change was required in response to this RAI Supplement.

Table 3.8-85 (2) Comparison of ACI 349-97 and ACI 349-01

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
CHAPTER 7-DETAILS OF REINFORCEMENT 7.13 - Requirements for structural integrity	7.13.1 through 7.13.4 Detailing of reinforcement and connection to improve integrity of the overall structure	This section is newly added in 2001 Edition.	Requirement is increased in 2001 Edition.
CHAPTER 9 - STRENGTH AND SERVICEABILITY REQUIREMENTS 9.2 - Required strength	9.2.2 Structural effects to be included with the dead load	"expansion of shrinkage–compensating concrete" is added.	Requirement is increased in 2001 Edition.
9.3 - Design strength	9.3.4 Shear strength reduction factor for brittle members	This section is newly added in 2001 Edition. Shear strength reduction factor for a member whose nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength is reduced to 0.6.	Since the provision is more stringent, requirement is increased in 2001 Edition.
9.5 - Control of deflections	9.5.5.2 Deflection of shored composite construction	Replace the phrase "deflection occurring after the member becomes composite need not be computed" to "it is not required to compute deflection occurring after the member becomes composite". Replace "should" with "shall".	Editorial changes to be consistent with ACI 318-95 language. No change in requirements.
CHAPTER 10-FLEXURE AND AXIAL LOADS 10.3 - General principles and requirements	10.3.4 Use of compression reinforcement in conjunction with additional tension reinforcement	Replace the phrase "Compression reinforcement in conjunction with additional tension reinforcement may be used to increase ..." to "Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase ...".	Editorial changes to be consistent with ACI 318-95 language. No change in requirements.
10.5 - Minimum reinforcement of flexural members	10.5.1 Minimum reinforcement of general flexural members	An equation to define the required amount of reinforcement is added.	Since the provision is more stringent, requirement is increased in 2001 Edition.

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
	10.5.2 Minimum reinforcement of T-section with flange in tension	Equations to define the required amount of reinforcement are changed.	Since the provision is more stringent, requirement is increased in 2001 Edition.
	10.5.4 Requirement for slab of uniform thickness (10.5.3 in 1997 Edition)	Requirement for the maximum spacing of reinforcement is changed from "18 in." to "the lesser of three times the thickness and 18 in."	Since the provision is more stringent, requirement is increased in 2001 Edition.
10.10 - Slenderness effects in compression members	10.10.1 Requirements for slenderness effect analysis	The use of a refined nonlinear second-order analysis is permitted.	Provisions in 2001 Edition are identical to those in ACI 318-99 which is recommended to be used by NRC in RG-1.142.
10.11 - Magnified moments: General	General requirements to evaluate magnified moments for slenderness effect	Section 10.11 "Approximate evaluation of slenderness effect" in 1997 Edition was divided into these three sections in 2001 Edition. Major changes are: • Arrangement of provisions dealing with non-sway and sway frames is improved. • It is specifically required to consider the influence of axial loads, the presence of cracking, effects of duration of the loads in an elastic first-order analysis. • Design method of sway frames is revised.	Provisions in 2001 Edition are identical to those in ACI 318-99 which is recommended to be used by NRC in RG-1.142.
10.12 - Magnified moments: Non-sway frames	Requirements to evaluate magnified moments for non-sway frames		
10.13 - Magnified moments: Sway frames	Requirements to evaluate magnified moments for sway frames		
CHAPTER 11 - SHEAR AND TORSION 11.3 - Shear strength provided by concrete for nonprestressed members		Provision of concrete shear strength at sections where factored torsional shear exceeds the limitation (Section 11.3.1.4 in 1997 Edition) is eliminated.	The eliminated provision is not included in ACI 318-99 which is recommended to be used by NRC in RG-1.142.
11.5 - Shear strength provided by shear reinforcement	11.5.5.3 Amount of minimum shear reinforcement	Equation to specify the negligible torsional moment is eliminated. Instead, reference to Section 11.6 "Design for torsion" is added.	The provision in 2001 Edition are identical to that in ACI 318-99 which is recommended to be used by NRC in RG-1.142.

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
11.6 - Design for torsion		Design for torsion is fully changed in 2001 Edition. It is based on a thin walled tube, space truss analogy in accordance with ACI 318-95.	Provisions in 2001 Edition are identical to that in ACI 318-99 except for the requirement on the minimum area of stirrups in Section 11.6.5.2. Since the resulting minimum areas of stirrups are almost same between ACI 349-02 and ACI 318-99 for 5000 psi concrete, the deviation is acceptable.
CHAPTER 13- TWO-WAY SLAB SYSTEMS 13.3- Slab reinforcement (13.4 in 1997 edition)	13.3.6.4 Special reinforcement in slab	Requirement for special reinforcement is revised. It shall be placed in a band parallel to the diagonal in the top of the slab and a band perpendicular to the diagonal in the bottom of the slab instead of the direction of the moment. In the second sentence, replace in the last line "either" and "or" with "both" and "and."	Requirement is made clearer and Level of Safety is improved.
	13.3.8.5 Details of reinforcement in slabs without beams	"All" bottom bars or wires within the column strip, in each direction, shall be continuous instead of "at least two".	Level of Safety is improved in requirement.
13.5- Design Procedure (13.3 in 1997 edition)	13.5.3.2 unbalanced moment given by $\gamma f Mu$ 13.5.3.3 limit to use γf	Formula for the definition of γf (= fraction of unbalanced moment transferred by flexure at slab-column connections) is changed. 13.5.3.3 is newly added in 2001 Edition.	Provisions in 2001 Edition are identical to those in ACI 318-99 which is recommended to be used by NRC in RG-1.142.
13.7—Equivalent frame method	13.7.5.2 torsional stiffness definition	13.7.5.2 in 1997 Edition describes the definition of Stiffness Kt. 13.7.5.4 in 1997 Edition has similar description.	Requirement is made clearer.
CHAPTER 14—WALLS 14.2—General	14.2.7 Waives of Quantity of reinforcement and limits of thickness required by 14.3 and 14.5	Provision "14.3" is added.	Level of Safety is improved in requirement.
14.4—Walls designed as compression members	Subjected walls should be designed in accordance with provisions	Provisions of 10.13, 10.14, and 10.17 are added.	Requirement is made clearer.

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
CHAPTER 15—FOOTINGS 15.6—Development of reinforcement in footings	15.6.2 development on each side of section	Mechanical device is included in development of reinforcement.	Level of Safety is not changed in requirement.
15.8—Transfer of force at base of column, wall, or reinforced pedestal	15.8.3.1 requirement of connection between precast columns or pedestals and supporting member	Description is changed to refer to the 16.5.1.3(a). 16.5.1.3(a) shows new requirement; for columns with a larger cross section than required by consideration of loading, a reduced effective area Ag.	Not applicable for ESBWR
	15.8.3.2 requirement of connection between precast wall and supporting member	Description is changed to refer to the 16.5.1.3(b) and (c). 16.5.1.3(b) shows new requirement; precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 10,000 lb per tie.	Not applicable for ESBWR
CHAPTER 16—PRECAST CONCRETE	Provisions for precast concrete structural members	Composition is fully changed in 2001 Edition.	Not applicable for ESBWR
CHAPTER 17—COMPOSITE CONCRETE FLEXURAL MEMBERS 17.5—Horizontal shear strength	17.5.3 Determination of horizontal shear	It is noted clearly that Section 17.5.3 is treated as an alternative to 17.5.2.	Requirement is made clearer.
CHAPTER 18—PRESTRESSED CONCRETE 18.12—Slab systems	18.12.6 detail of lift slabs	This section is newly added to refer 13.3.8.6.	Requirement is made clearer.

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
CHAPTER 20—STRENGTH EVALUATION OF EXISTING STRUCTURES 20.1—Strength evaluation: General	20.1.2 requirement of analytical evaluation against effect of the strength deficiency 20.1.3 requirement of load test against effect of the strength deficiency 20.1.4 permission to remain in service for a specified time period	This section is newly added in 2001 Edition.	Requirement is made clearer. Not applicable for ESBWR
CHAPTER 21—SPECIAL PROVISIONS FOR SEISMIC DESIGN 21.1—Definitions	Crosstie Hoop Seismic hook	Definition of the seismic hook is added and is referred to the explanation of Crosstie and Hoop.	Definition is made clearer.
21.2—General requirements	21.2.1.4 requirement to the reinforced concrete structural members	This section is newly added in 2001 Edition as all members shall satisfy 21.2 through 21.7 of Chapter 21.	Requirement is made clearer.
21.3—Flexural members of frames	21.3.2—Longitudinal reinforcement 21.3.2.1 minimum reinforcement of flexural member	Exception is added, which is 10.5.3 and minimum requirement of amount of reinforcement not be less than that given by Eq. (10-3).	Provisions in 2001 Edition are identical to those in ACI 318-99 which is recommended to be used by NRC in RG-1.142.
	21.3.3—Transverse reinforcement 21.3.3.4 minimum distance of hoops where not be required	Stirrup is redefined, which has seismic hooks at both ends.	Level of Safety is improved in requirement.
	21.3.4.2—Transverse reinforcement	Subjected length for transverse reinforcement is identified in 21.3.3.1.	Requirement is made clearer.
21.4—Frame members subjected to bending and axial load	21.4.5—Shear strength requirements 21.4.5.2 Transverse reinforcement	Subjected length l_0 for transverse reinforcement is identified in 21.4.4.4.	Requirement is made clearer.
21.5—Joints of frames	21.5.4—Development length of bars in tension 21.5.4.4 epoxy-coated reinforcement	This section is newly added in 2001 Edition. Development lengths shall be multiplied by the applicable factor specified in 12.2.4 or 12.3.5.5.	Level of Safety is improved in requirement.

Chapters	Provisions in 2001 Edition	Changes from 1997 Edition	Comments
21.6—Structural walls, diaphragms, and *trusses*	21.6.4—Diaphragms	This section is newly added in 2001 Edition. Requirement for concrete diaphragms and composite topping slabs serving as diaphragms is added.	Level of Safety is improved in requirement.
21.7 Frame members not proportioned to resist forces induced by earthquake motions	21.7.2 Requirement for the case of induced moment and shear combined factored gravity moment and shear do not exceed the design moment and shear strength	Re-arranged the description based on the induced moment and/or shear. In addition, stirrups are required to be spaced at not more than d/2. The maximum tie spacing is limited to six diameter s of the smallest longitudinal bar or 6 in. And they have to provide for the full column height instead of l_0.	Level of Safety is improved in requirement.
	21.7.3 Requirement for the case of induced moment or shear exceed the design moment or shear strength	Requirements for material and splices are added. In addition, stirrups are required to be spaced at not more than d/2.	Requirement is made clearer.
APPENDIX B—Anchoring to Concrete		1997 Edition has a title "Steel Embedments". This section is fully changed in 2001 Edition, concrete-failure cone method to the CCD method.	This is also addressed in the later versions of ACI 318, Appendix D. ACI-349-01 Appendix B was accepted by the NRC under RG 1.199 Nov. 2003 with some exceptions. RG 1.199 is met in the ESBWR design.
APPENDIX C—Special Provisions for Impulsive and Impactive Effects		Inserted an additional paragraph to guard against unintentional increase in yield strength. Development length requirements of Chapter 12 are implicit.	Requirement is made clearer.

NRC RAI 3.8-86

General Design Criterion 53, in part, requires that the reactor containment be designed to permit appropriate periodic inspection of all important areas. RAI 3.8-1 requests that the applicant address this for the concrete and steel elements of the ESBWR containment structure. A stated industry design criterion for advanced reactors is to accommodate inservice inspection of critical areas. The staff considers that monitoring and maintaining the condition of Other Category I Structures is essential for plant safety. DCD Section 3.8.4 does not address any special design provisions (e.g., providing sufficient physical access, providing alternative means for identification of conditions in inaccessible areas that can lead to degradation, remote visual monitoring of high radiation areas) to accommodate inservice inspection of Other Category I Structures. Please include a description of any special design provisions for other Category I Structures in new DCD Section 3.8.4.7. If none have been incorporated in the ESBWR design, please provide the technical basis for concluding that they are not necessary.

GE Response

(1) In support of the monitoring of the Seismic Category I structures of the ESBWR listed in DCD Tier 2 Table 19.2-4, as described in the new DCD Tier 2 Section 3.8.4.7 (added per NRC RAI 3.8-81), access will be considered in the detailed design process.

Space Control is exercised in the ESBWR by means of a 3D model. It is the means by which interference checking and space control is accomplished. It includes all safety and non-safety related SSC's. Items are added to the model as it is being developed by stages depending on criticality to the plant and construction sequence of the item. Accessibility to equipment, valves, instrumentation, welds, supports, etc. for operation, inspection or removal is characterized by sufficient space to allow unobstructed access and reach of site personnel. Therefore, aisles, platforms, ladders, handrails, etc. are reviewed as the components are laid out. Interferences with access ways, doorways, walkways, truck ways, lifting wells, etc, are constantly monitored.

(2) As indicated in item (1) above, accessibility is constantly monitored, maintained and documented during the plant layout process. Remote tooling would only be included if for some layout reasons the required inspection could not be carried out otherwise.

No DCD change was made in response to this RAI.

NRC RAI 3.8-86, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

Similar to RAI 3.8-59. See staff assessment of response to RAI 3.8-59. Also, see resolution of issue associated with Table 19.2-4, which is addressed under RAI 3.8-81.

GE Response

See response to NRC RAI 3.8-59 S01 and NRC RAI 3.8-81 S01.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 3.8-88

DCD Sections 3.8.5.4 indicates that a main objective of the design of the foundation is to ensure that there is adequate frictional and passive resistance to prevent sliding of the structure when subjected to lateral loads. However, the DCD does not indicate how the analysis is to be performed and how lift-off effects, if appropriate, are to be captured in this analysis. The DCD also indicates that the capability of the foundation to transfer shear is evaluated when waterproofing is used beneath the basemat. The DCD needs to indicate the procedures employed to assess such effects for a potential range of site conditions varying from soil sites with shear wave velocities of the order of 1,000 fps to hard rock sites.

GE Response

See response to NRC RAI 3.8-96. This evaluation was done for the softest soil conditions in the range examined.

No DCD change was made in response to this RAI.

NRC RAI 3.8-88, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE refers to response to RAI 3.8-96. See staff assessment of response to RAI 3.8-96. During the audit, it was agreed to address this issue under RAI 3.8-96.

GE Response

See response to NRC RAI 3.8-96.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 3.8-92

DCD Section 3.8.5.4 indicates that the standard design is developed using a range of soil conditions as detailed in Appendix 3A. Appendix 3A describes the range in shear wave velocities considered in SSI analyses, and only focuses on assumed uniform site conditions. Section 3.8.5.4 also states that total and differential settlements of the foundations must be considered, but refers to Section 3.8.6.2 for COL information. Section 3.8.5.4 does not indicate if any potential effects of static or dynamic differential settlement effects have been incorporated into the design of the standard plant nor the magnitude of settlement that was considered. Also, the effect off settlement on construction procedures is not addressed. DCD Section 3.8.5.4 needs to clarify how settlement issues are incorporated in to the generic design of the standard plant, and identify limitations on the magnitude of settlements.

a) *Explain how the potential for settlement was considered in the ESBWR standard plant design.*

b) *What is the allowable settlement that can be accommodated by the ESBWR foundations/structures?*

Include this information in DCD Section 3.8.5.4. In addition, (1) identify the applicable detailed report/calculation (number, title, revision and date, and brief description of content) that will be available for audit by the staff, and (2) reference this report/calculation in the DCD.

GE Response

Three types of soil conditions are considered in the DCD, which are soft, medium and hard as uniform subgrades. See response to NRC RAI 3.8-93 for clarification on settlement issues.

No DCD change was made in response to this RAI.

NRC RAI 3.8-92, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE refers to response to RAI 3.8-93. See staff assessment of response to RAI 3.8-93. During the audit, it was agreed to address this issue under RAIs 3.8-93.

GE Response

See response to NRC RAI 3.8-93.

DCD Impact

No DCD change was required in response to this RAI Supplement.

NRC RAI 3.8-93

Section 3.8.5.4 states that total and differential settlements of the foundations must be considered, but refers to Section 3.8.6.2 for COL information. The DCD needs to clarify how settlement issues are incorporated into the generic design of the standard plant, and identify limitations on the magnitude of settlements, so that the COL applicant can ensure compliance with the standard design. Define the COL applicant actions required to confirm that the predicted site-specific settlement meets the standard plant design assumptions.

Include this information in the DCD. In addition, (1) identify the applicable detailed report/calculation (number, title, revision and date, and brief description of content) that will be available for audit by the staff, and (2) reference this report/calculation in the DCD.

GE Response

This response is similar to NRC RAI 3.8-92. The stipulated settlements will be incorporated into the total plant design as a requirement. The following evaluation, *Settlement Effect on Basemat Design*, clarifies the settlement issues. The COL holder will have to demonstrate that differential settlements at the site do not exceed this value by instituting a settlement monitoring program or justify in the COL why it would not be necessary.

The confirmation for settlement effect on basemat design is provided by parametric analysis considering a variety of soil conditions and construction sequences as shown in the following evaluation, *Settlement Effect on Basemat Design.* As a result, the basemat stresses reported in the DCD are not affected by horizontal variations in spring stiffness. Also basemat stresses during construction are much smaller than DCD design stresses.

Settlement Effect on Basemat Design

1. Scope

Additional topics discussed at audit have been stated in response to NRC RAIs 3.8-13 and 92 regarding the ESBWR basemat design. Additionally NRC recommended that to refer DCD of AP600 including the discussion during construction period. The main purpose of these requests is to estimate the differential settlement effect on basemat design during Operation and Construction Phase respectively. The discussion was stated in response to NRC RAI 3.8-93.

This section provides the result of the estimation concerning following items with the FEM model.

- Non-uniform soil condition under basemat during normal operation
- Settlement effect on basemat during construction period

2. Normal Operation Phase

Analyses are performed under the variety of soil condition (non-uniform condition), and then they are compared to the DCD design. The analytical conditions are as follows:

- FEM Model : Global FEM Model (DCD Design Model)
- Soil Condition : "Hard Spot", stiff spring under the Pedestal area.
  Three types of soil conditions are considered as "Hard Spot" as shown in Figure 3.8-93(1).
- Load : Dead Load

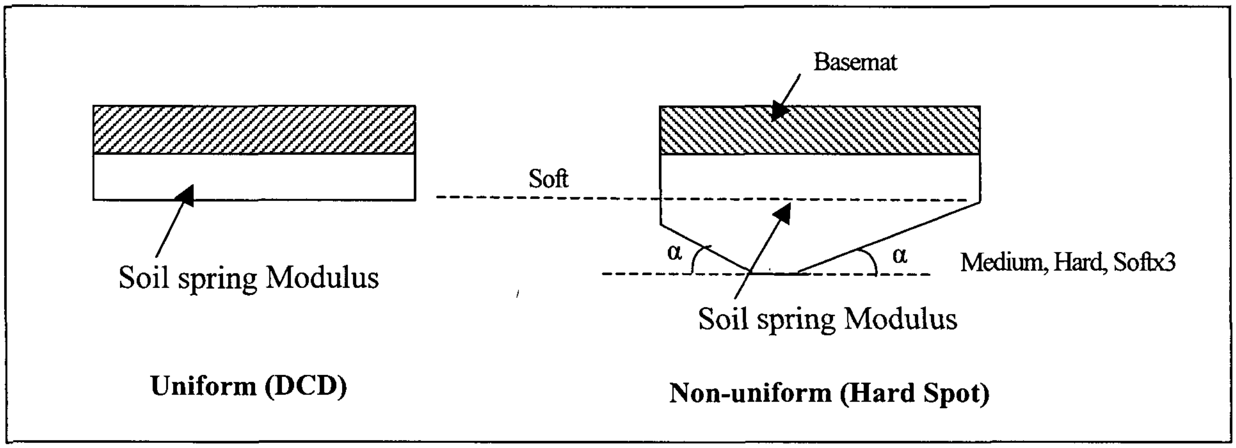


Figure 3.8-93(1) Soil conditions

Figure 3.8-93 (2) and (3) show the basemat deformation and bending moment of basemat comparing with those of DCD design (uniform soil condition). These indicate the bending moment of DCD design is larger or similar to Hard Spot condition since the soil springs are stiffer than DCD condition (uniform soft soil). Therefore there is no concern about basemat design if the building is settled on Hard Spot soil conditions.

3. Construction Phase

After the completion of basemat several part of building will be constructed based on planned construction sequence. This analytical study is provided to confirm the stress of basemat in construction period. The assumed sequence is as follows (but this is imaginary since these portions are constructed in short time periods):

Case A: sequentially outward construction

1. Pedestal poured up to 5m (below the floor EL-6400, approximately)
2. Apply loads by RCCV and B3F structure
3. Add exterior walls in RB
4. Add walls in FB area

Case B: Sequentially inward construction

Constructed into inverse direction of Case A.

1. FB area poured (below the floor EL-6400, approximately)
2. Add exterior walls in RB

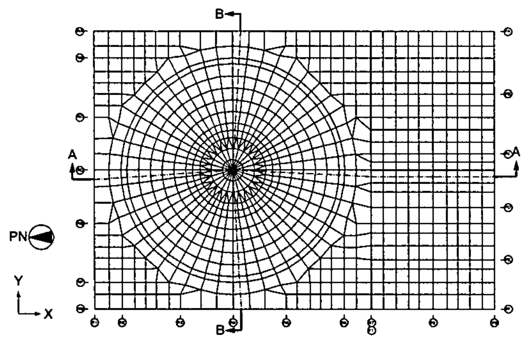
3. Add RCCV and B3F structure
4. Add pedestal

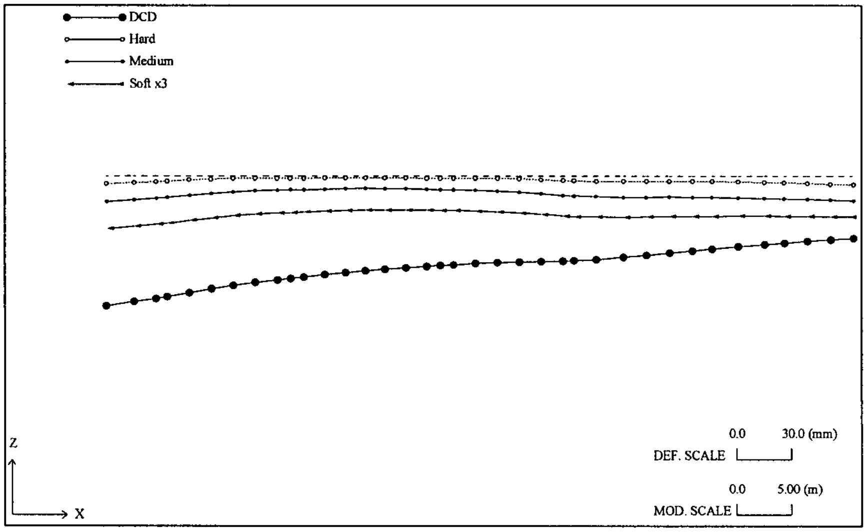
The analytical model has been extracted from global FEM model used for DCD design. Some modification has been applied to this as shown in Figure 3.8-93(4). This model is similar to the "Modified Truncated Model" provided to the NRC for confirmatory analysis. The height of structural members is limited to 5m in every top portion of the model considering construction plan. Figure 3.8-93(4) shows sequence of Case A.

The dead loads are considered per element thickness and density of concrete which in the model. The analytical conditions are as follows:

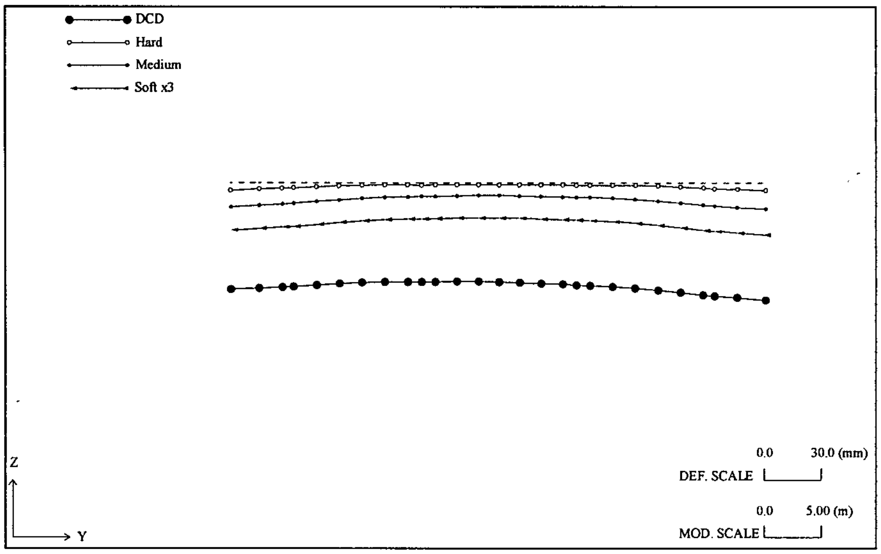
- FEM Model : Based on the "Modified Truncated Model (a part of Global FEM Model).
- Soil Condition : "Uniform", soft soil spring under the basemat
- Load : Dead Load

Figures 3.8-93 (5) and (6) show deformation of basemat. The maximum settlement is 15 mm and the maximum differential settlement is 8 mm. Figures 3.8-93 (7) and (8) show bending moment of basemat comparing with those of DCD design (normal operation). These indicate the bending moment of DCD design is larger than the bending moment during the construction period.



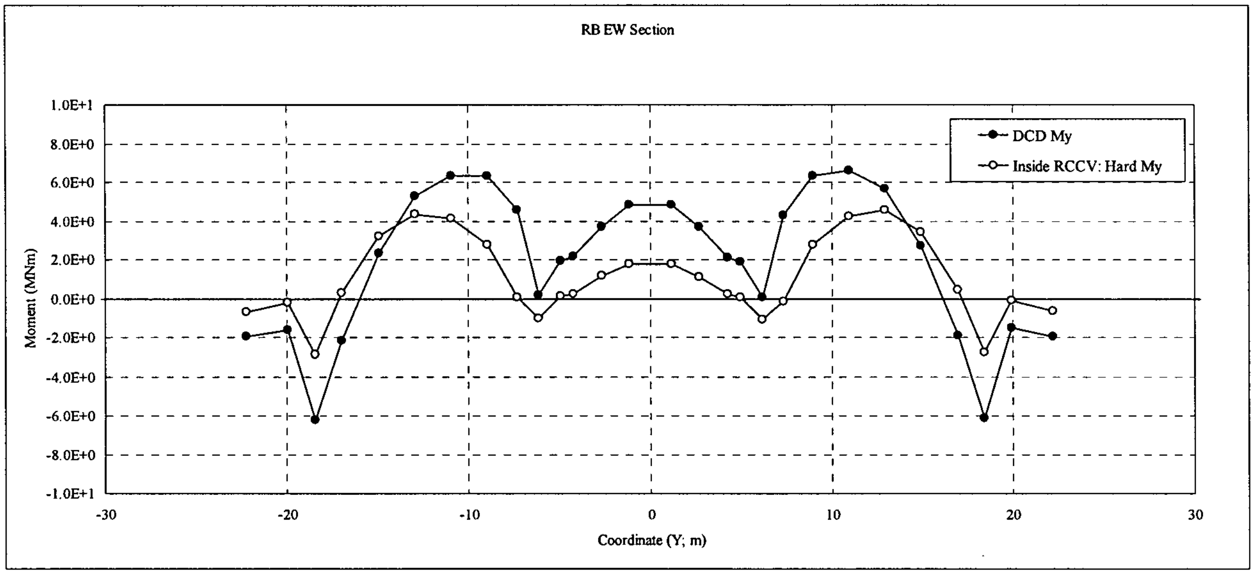


a) A-A Section

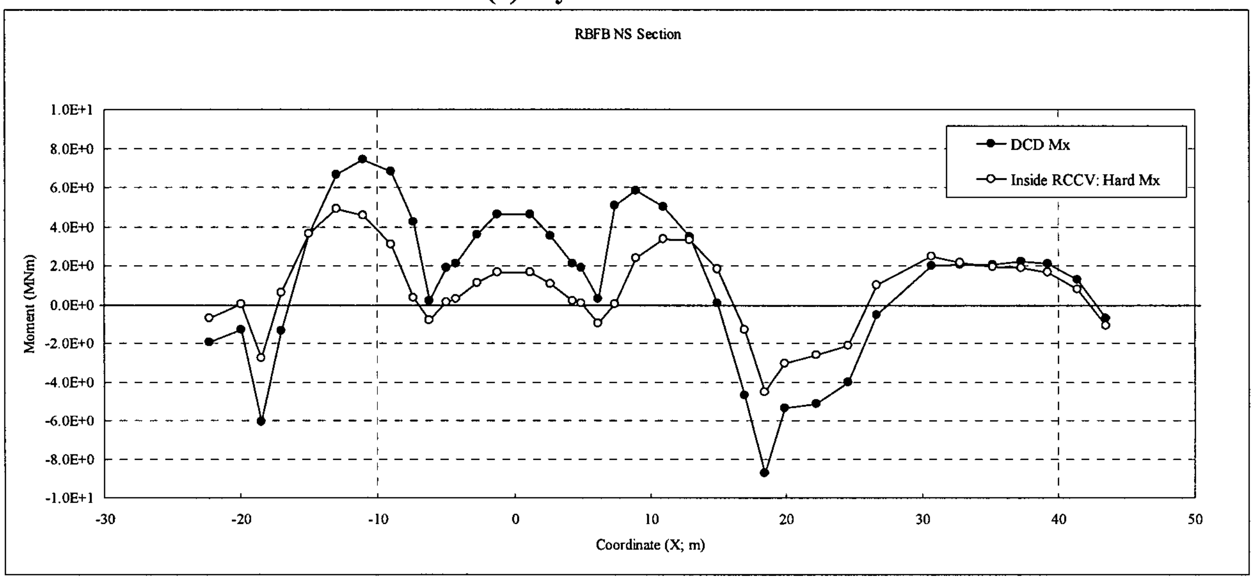


b) B-B Section

Figure 3.8-93(2) Comparison of Basemat Deformation



(a) My in B-B Section



(b) Mx in A-A Section

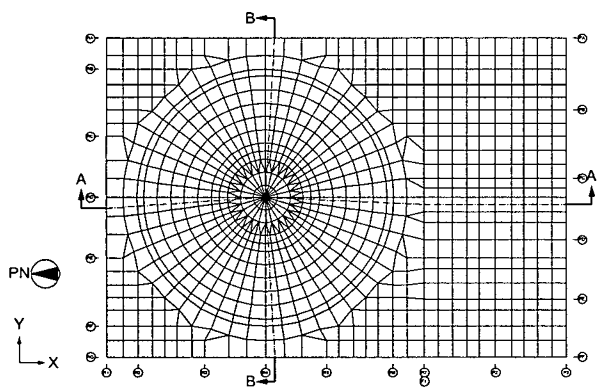
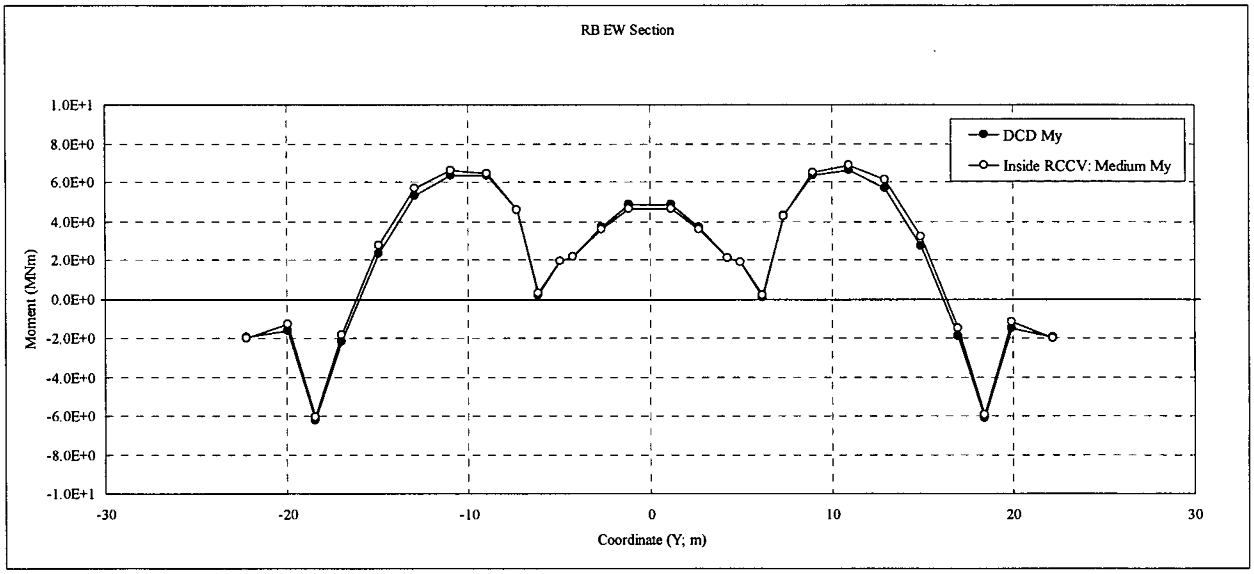
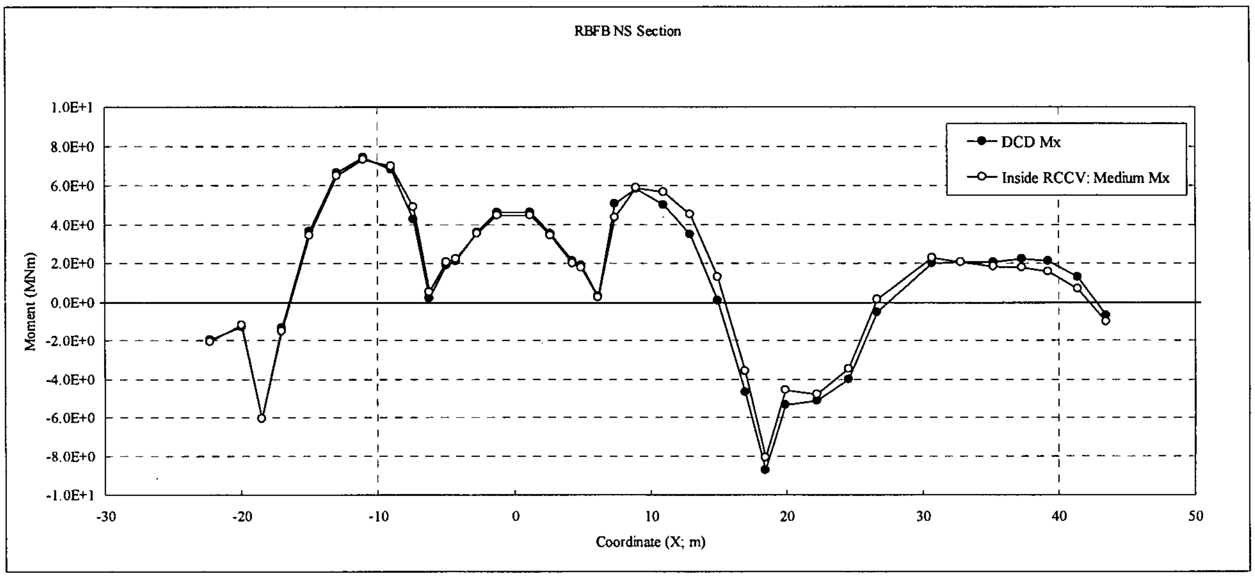


Figure 3.8-93(3)-a Comparison of Basemat Sectional Moments (Hard)



(a) My in B-B Section



(b) Mx in A-A Section

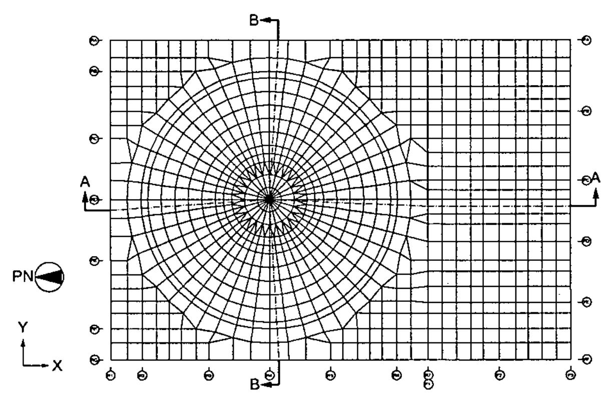
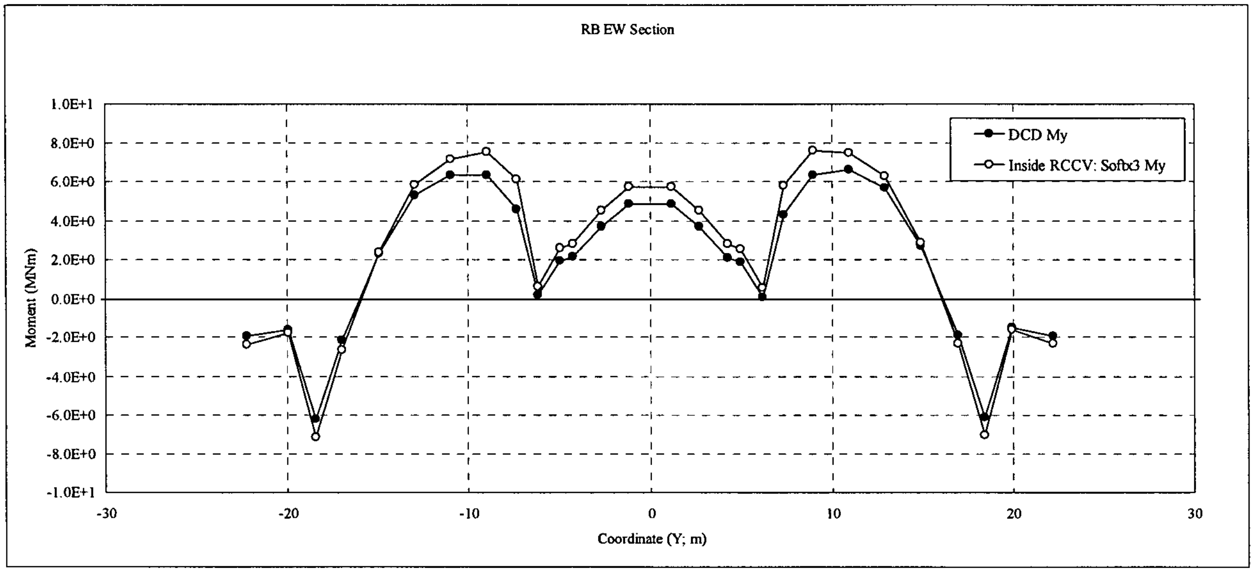
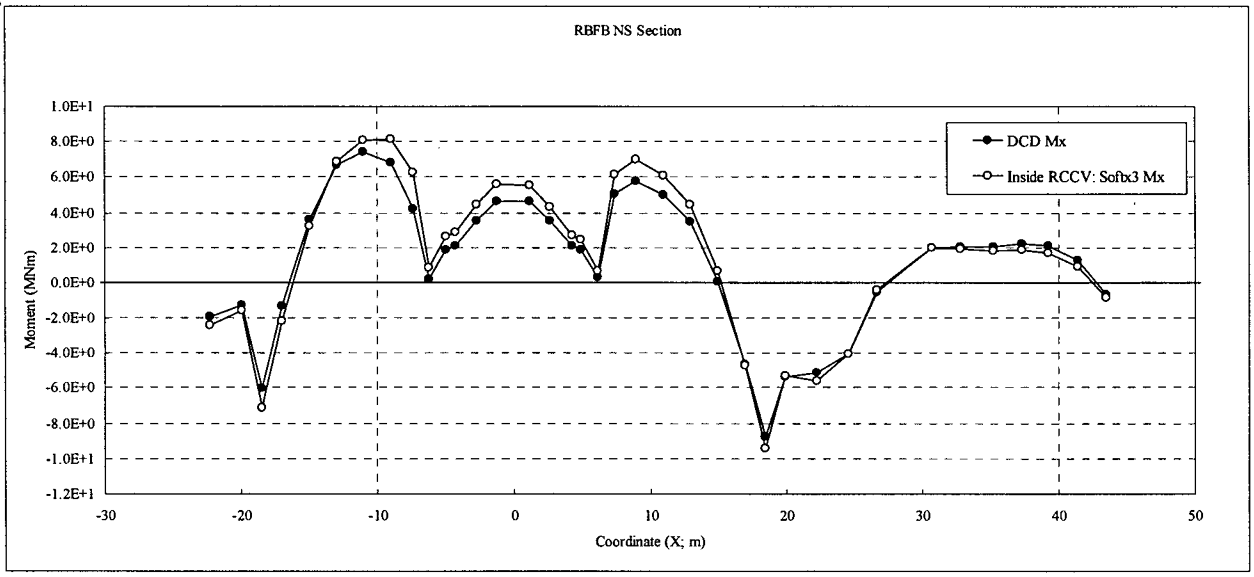


Figure 3.8-93(3)-b Comparison of Basemat Sectional Moments (Medium)



(a) My in B-B Section



(b) Mx in A-A Section

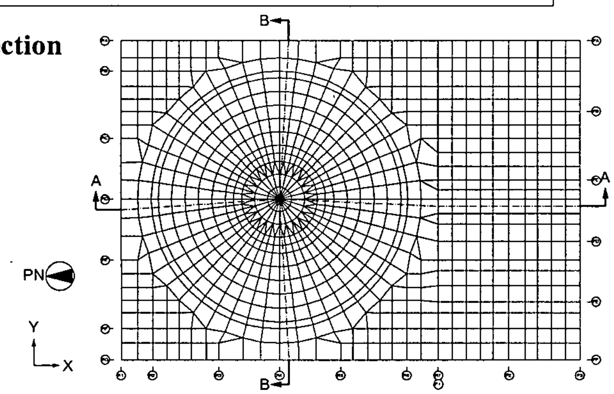


Figure 3.8-93(3)-c Comparison of Basemat Sectional Moments (soft x3)

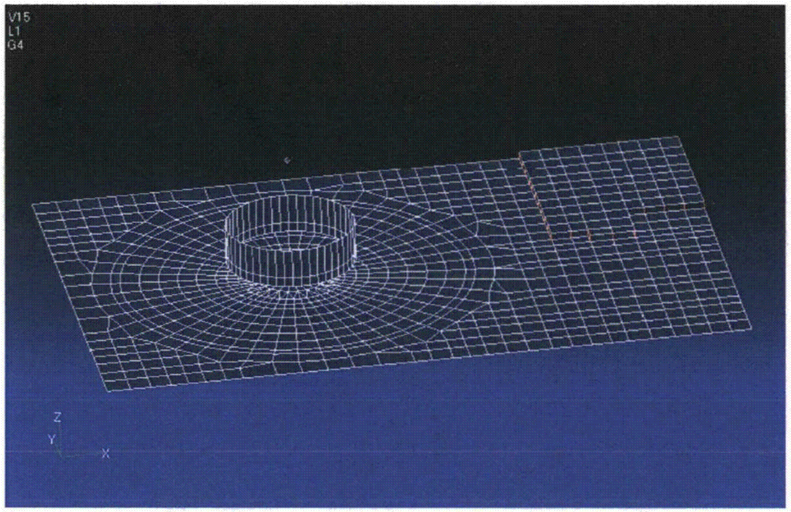


Figure 3.8-93(4)-a Case A (Step 1)

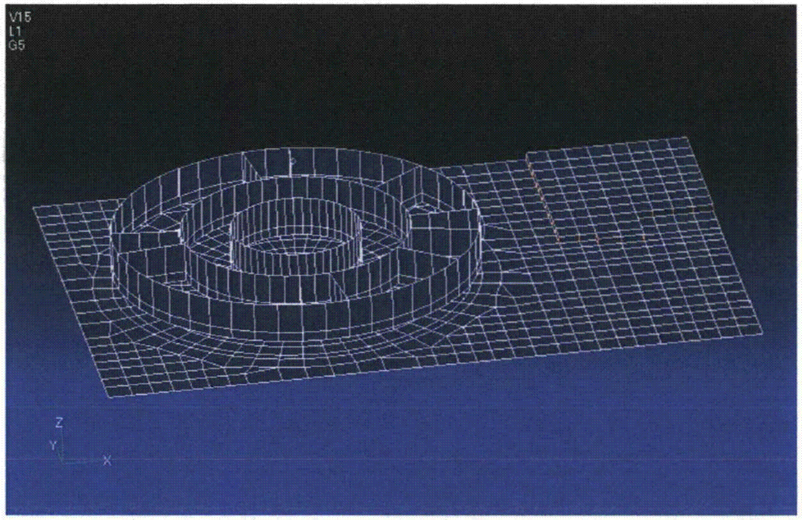


Figure 3.8-93(4)-b Case A (Step 2)

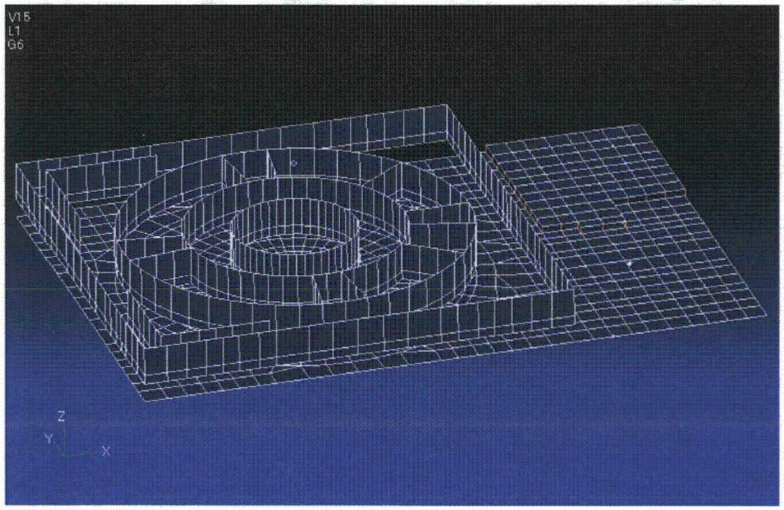


Figure 3.8-93(4)-c Case A (Step 3)

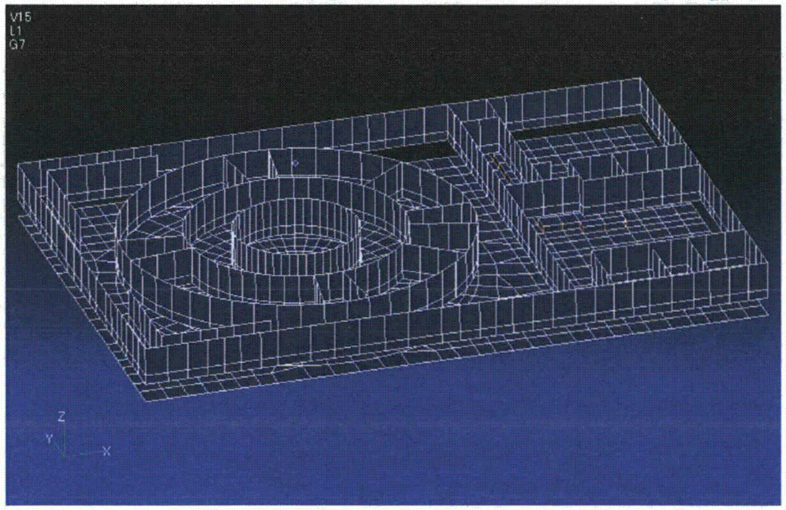


Figure 3.8-93(4)-d Case A (Step 4)

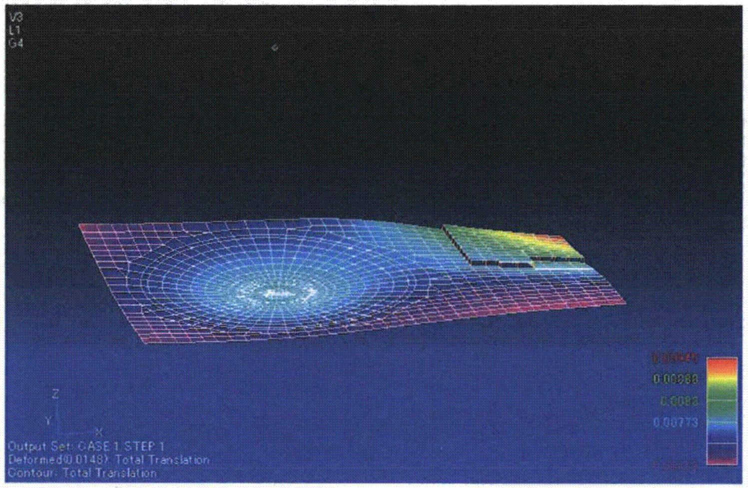


Figure 3.8-93(5) -a Deformation of basemat (CaseA - Step 1)

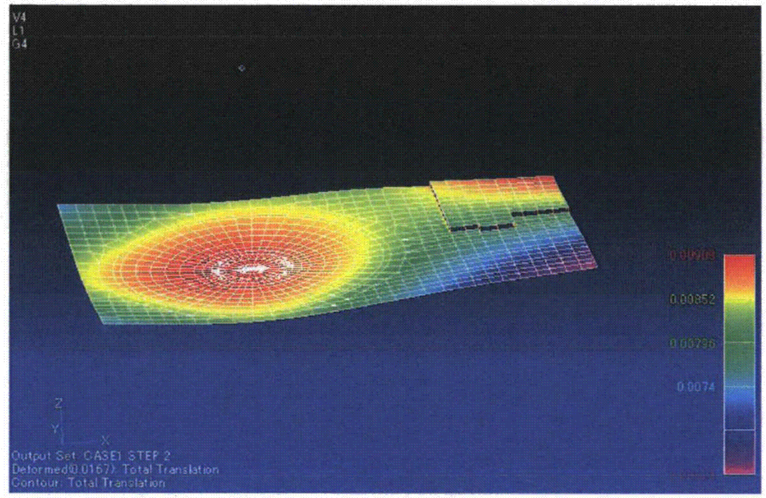


Figure 3.8-93(5) -b Deformation of basemat (CaseA - Step 2)

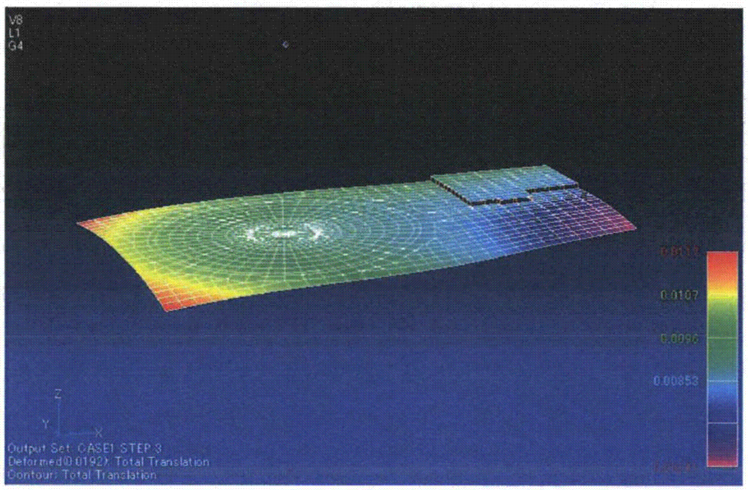


Figure 3.8-93(5) -c Deformation of basemat (CaseA - Step 3)

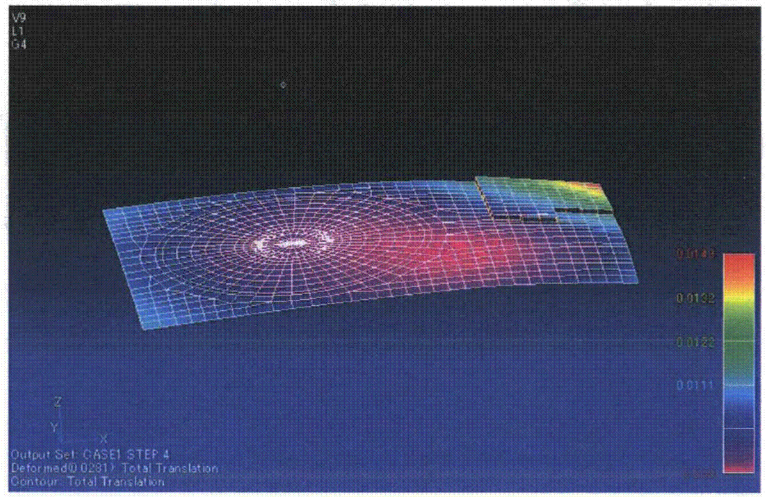


Figure 3.8-93(5) -d Deformation of basemat (CaseA - Step 4)

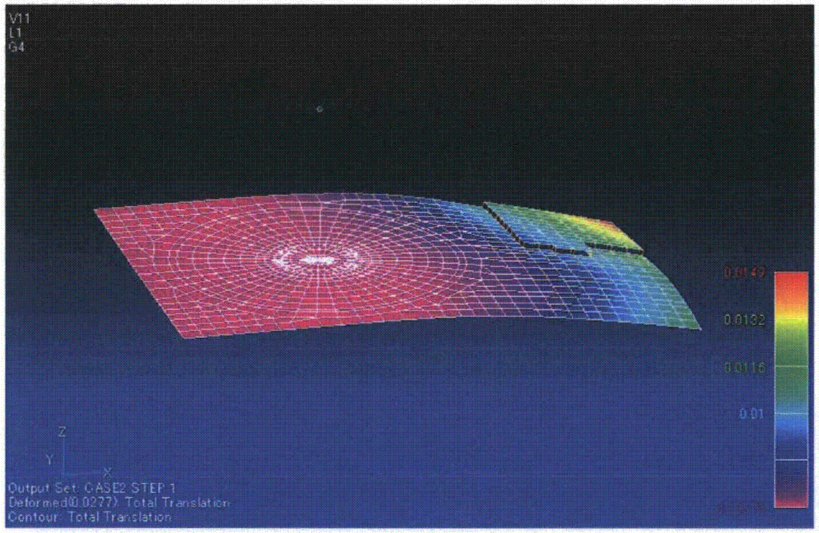


Figure 3.8-93(6) -a Deformation of basemat (CaseB - Step 1)

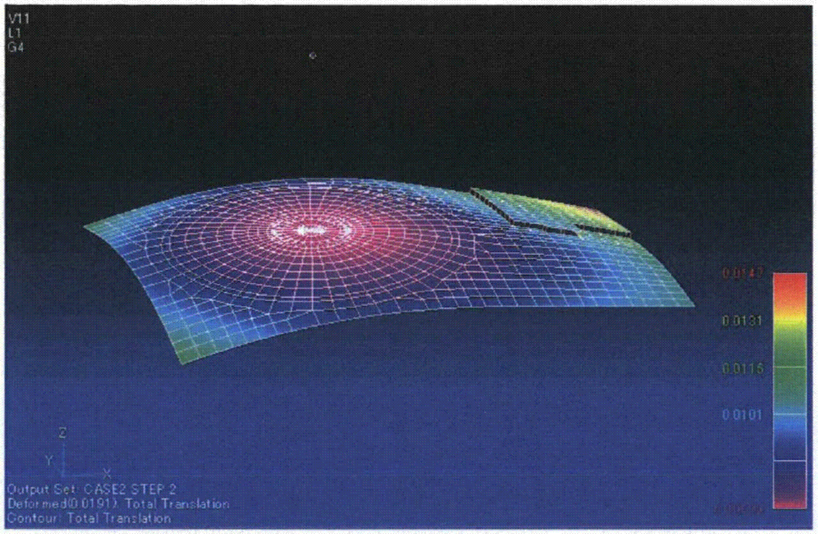


Figure 3.8-93(6) -b Deformation of basemat (CaseB - Step 2)

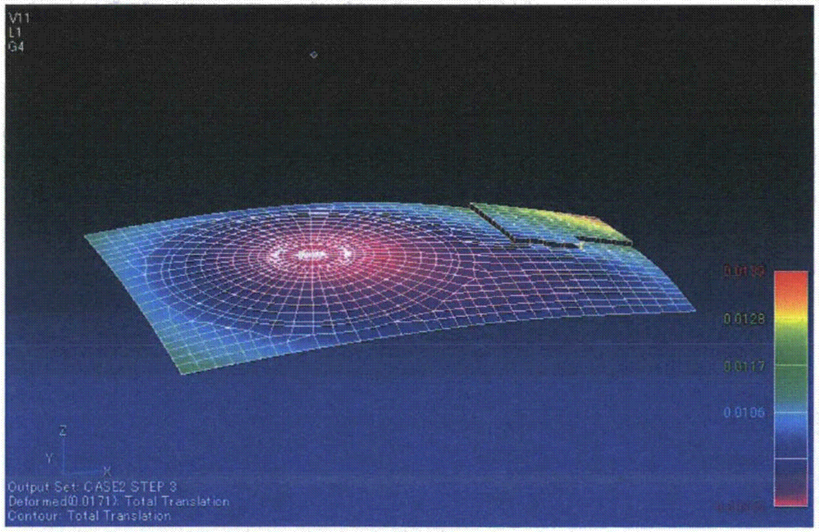


Figure 3.8-93(6) -c Deformation of basemat (CaseB - Step 3)

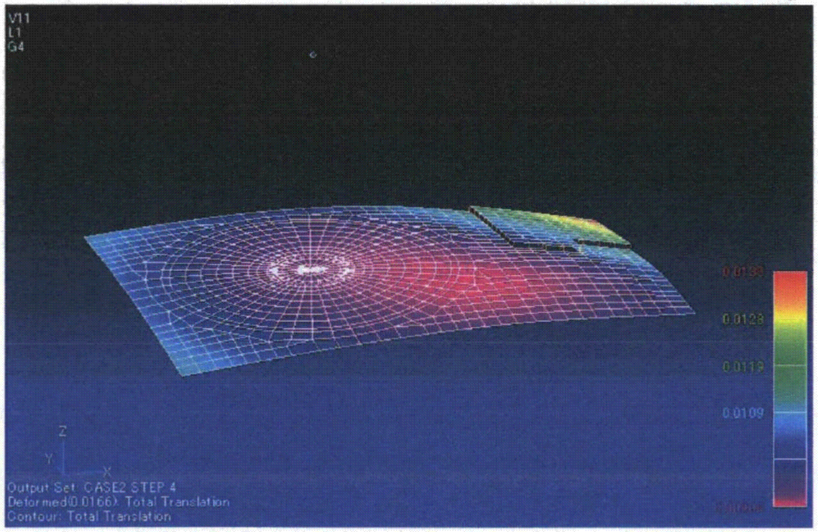
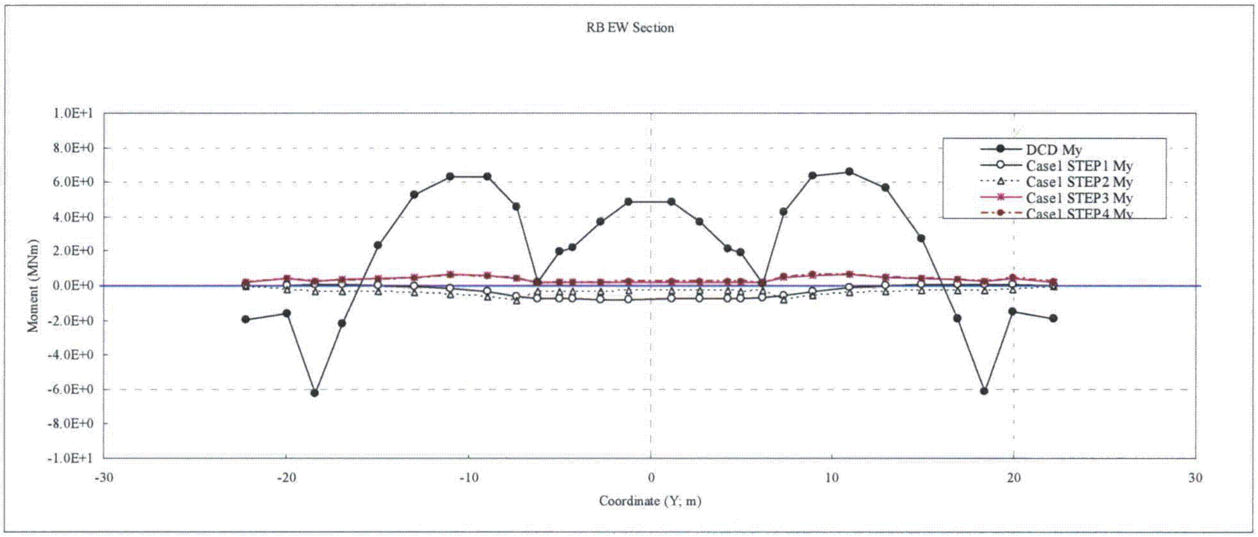
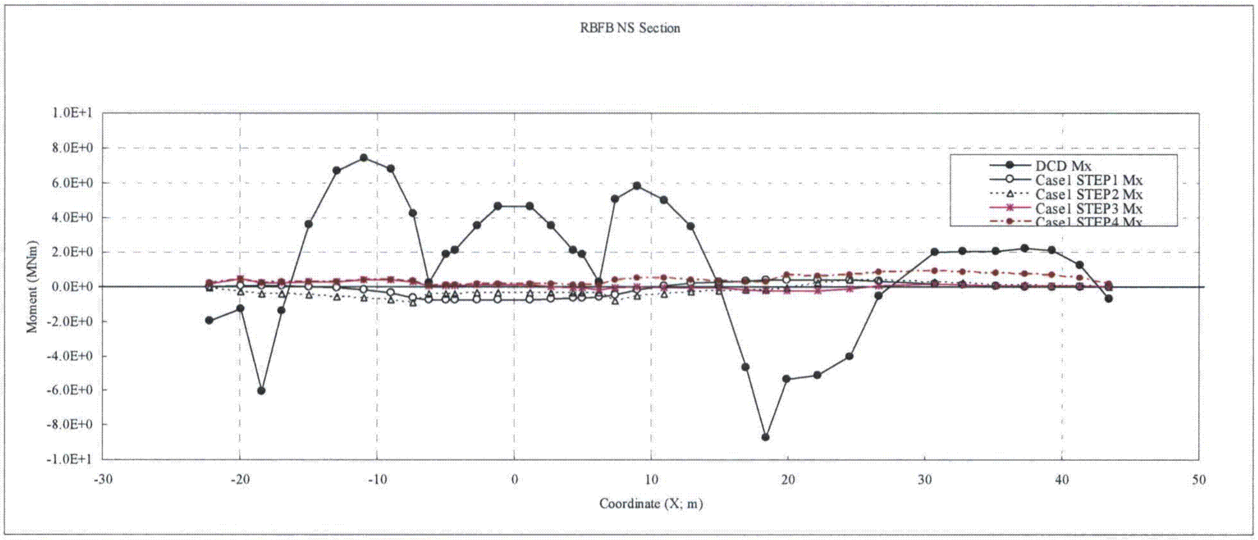


Figure 3.8-93(6) -d Deformation of basemat (CaseB - Step 4)





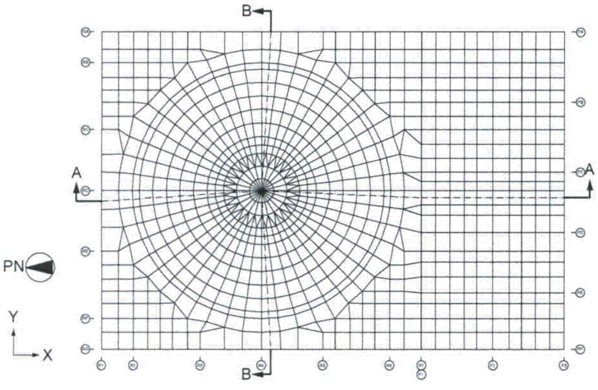
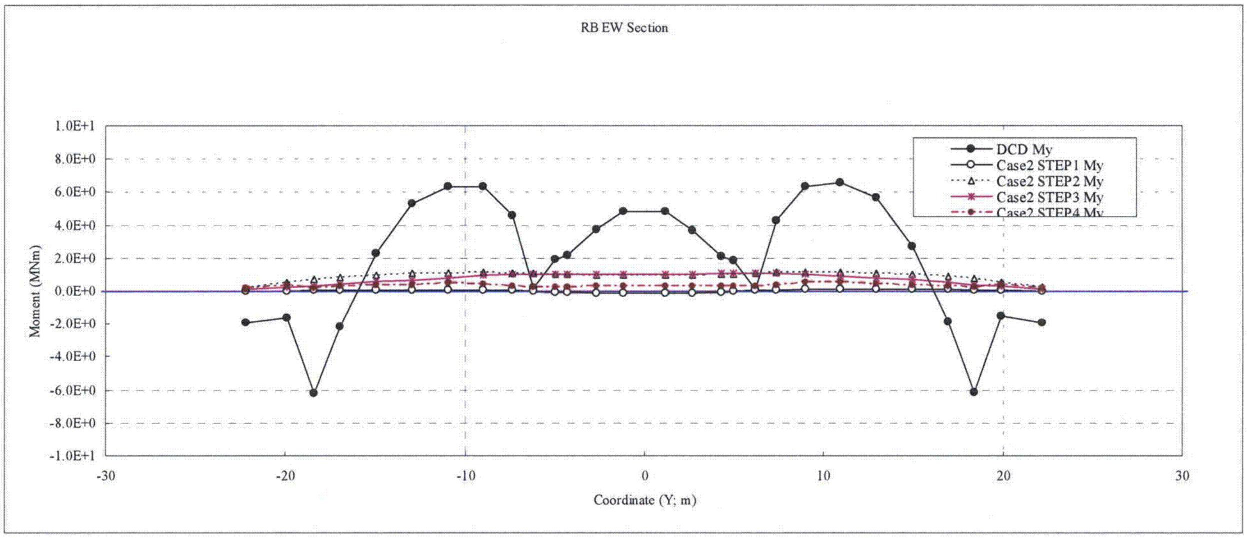
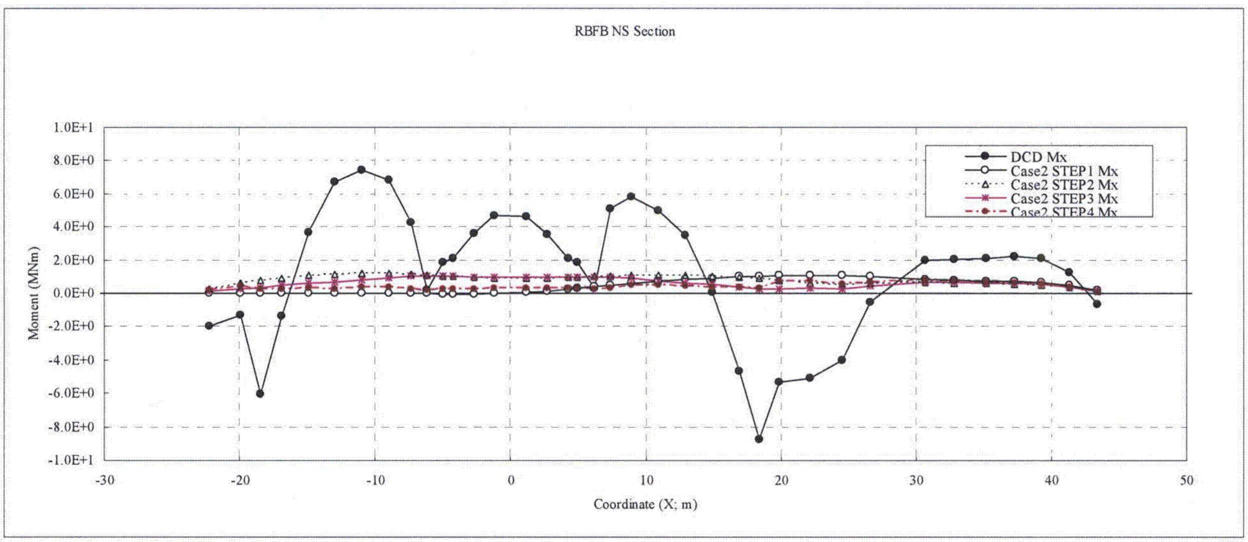


Figure 3.8-93(7) Comparison of Basemat Sectional Moments (Case A)





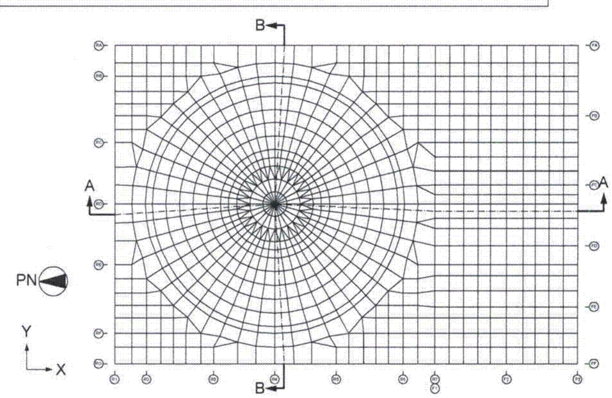


Figure 3.8-93(8) Comparison of Basemat Sectional Moments (Case B)

No DCD change was made in response to this RAI.

NRC RAI 3.8-93, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE needs to explain why DCD Section 3.8.6.2 has been deleted and state where it will be documented in the DCD that "The COL holder will have to demonstrate that differential settlements at the site do not exceed this value by instituting a settlement monitoring program or justify in the COL why it would not be necessary." GE also needs to: (1) clarify "this value" in the previous sentence, (2) why only dead load is considered in the evaluation and clarify what loads are included in the dead load, (3) why is the pedestal area the only area considered to have a potential "hard spot," (4) explain the sentence "Assumed sequence is as follows, but this is imaginary since these portions are constructed in short time periods," (5) clarify if the two construction sequences (Case A and Case B) are a COL requirement, and if not, why not, and (6) why aren't hard spots considered in the construction phase. (7) In the evaluation for variation of horizontal soil springs, were the walls also reviewed in addition to the mat? (8) Regarding Fig. 3.8-93(3)-c, GE needs to explain why the soft X 3 case exceeds the base case.

During the audit, GE provided a draft supplemental response to address the above.

(1) For the settlement and differential settlement criteria, GE agreed to revise DCD Section 2.5 to specify, for all SC I structures, the allowable values that must be met by the COL applicant during the life of the plant for the particular site. The SC I structures are evaluated for these displacement limits and shown to meet design requirements. Also, DCD Section 3.8.5 will identify the need to satisfy these requirements and reference DCD Section 2.5. GE indicated that the precise values given in the RAI response will be reevaluated and will probably be increased since there is more margin.

(2) GE clarified in the draft supplemental response and during the audit why only dead load is considered.

(3) GE will address the concern that there are other horizontal variations of the soil springs (e.g., stiffer springs around the periphery, ...) to consider.

(4) GE clarified in the draft supplemental response and during the audit why the assumed sequence is considered to be "imaginary." That is because a conservative assumption was considered in the analysis.

(5) Regarding the need to specify general construction sequences in the DCD, which were the basis of the design, GE indicated that they will perform additional calculations to consider the effects of the construction sequence of the concrete mat pour and the effects on design. These evaluations will include consideration of the governing soil properties. GE expects that these bounding type calculations will show that the resulting forces and deformations are small. If so, GE will revise the DCD to indicate that the requirements for construction of the mat (based on these evaluations) will be specified in the construction specifications. If not, then a more detailed description of the construction requirements will be provided in the revised DCD.

(6) GE indicated that the hard spots were not considered for the construction phase analyses because the deformations and resulting loads were small and also, these construction related conditions are short term. The staff needs to review this position.

(7) GE indicated that they will review the results for the walls as well and provide their evaluation.

(8) GE indicated that they will review the results and provide their explanation.

GE Response

DCD Tier 2 Subsection 3.8.6.2 is deleted since an analysis of the settlement issue is performed using generic soil parameters, which are subject to confirmation by the COL applicant in DCD Tier 2 Chapter 2. Therefore, no additional COL requirements need to be stated in DCD Tier 2 Subsection 3.8.6.

(1) The allowable total and differential settlements within SC I buildings will be quantified in the next DCD Tier 2 revision. The following evaluation, *Assessment of Building Settlement*, discusses and sets limits for building settlement. The total settlement is defined as the maximum vertical displacement in the building basemat, and the differential settlement is defined as the maximum relative vertical displacement between two opposite corners along the longest dimension of the building basemat. The allowable differential settlement between the RBFB and CB is the relative displacement evaluated using the total settlements of two buildings.

(2) Only DL is considered because during construction of the mat, it imposes the worst loading condition. It consists of all permanent dead loads considered in the design for the "Normal Operation Phase" and the weights of the structures in accordance with the sequence considered in the "Construction Phase". Construction live loads on the order of 100 psf (4.8 kN/m^2) are ignored since the magnitude is only a small fraction (about 5%) of the basemat weight.

(3) Analyses for the inverted soil spring variation, i.e., stiffer springs around the peripheral area of the RPV Pedestal, were performed. The results are described in the following evaluation, *Basemat Design Considering Horizontal Variation of Soil Springs*. Based on the results, the DCD Tier 2 design envelopes the result of horizontal variation of soil spring under the condition that the ratio of the largest to the smallest shear wave velocity over the mat foundation width at foundation level does not exceed 1.7. This will be a COL item in DCD Tier 2 Chapter 2.

(4) Settlement is time dependent. Stiffening walls will be constructed within a few days after the mat pour. For conservatism in the analysis, it is assumed that the stiffening walls will be built a long time after the mat is poured.

(5) The construction sequence is not considered as a COL item since it is shown that under the worst loading condition, the mat can adequately handle the resulting stresses. Basemat construction sequence has no effect on the basemat design. The following evaluation, *Effect of Basemat Construction Sequence*, clarifies the effect of the basemat concrete pour sequence to the basemat stress.

(6) The original response to NRC RAI 3.8-93 in MFN 06-407 shows that the Construction Phase is not as severe as the Normal Operation Phase for the uniform soil condition. Additional evaluation is performed to address the effect of the horizontal variation of soil springs on the

basemat design during the Construction Phase. The following evaluation, *Basemat Design for Construction Phase Considering Horizontal Variation of Soil Springs*, shows the results of the hard spot condition, which confirms that the basemat stress during construction is smaller than the design stress.

(7) The evaluation, *Basemat Design Considering Horizontal Variation of Soil Springs*, mentioned in Item 3 above, also includes the resulting wall bending moments due to the horizontal variation the of soil springs. It is found that the "Soft Spot" condition controls the basemat design forces. Per Item 3 above, the COL applicant is to confirm the uniformity of the shear wave velocity at the foundation level for a given site.

(8) The bending moment distributions were compared for three cases of horizontal soil stiffness variation under the basemat in the original response to NRC RAI 3.8-93 in MFN 06-407. In the original response, the Softx3 case exceeded the base case. The evaluation mentioned in Item 3 above, *Basemat Design Considering Horizontal Variation of Soil Springs*, clarifies the result of horizontal variation of soil springs. Figure 3.8-93(24) shows the relative displacement normalized to the basemat displacement at the centerline position of the RB. It is found that the "Hard Spot" condition results in a different pattern of relative displacements when compared against the DCD Tier 2 analysis results. As a result, a limitation for the maximum variation of horizontal soil stiffness in terms of shear wave velocity is imposed as a COL item stated in Item 3 above.

Evaluation for NRC RAI 3.8-93 (1)

Assessment of Building Settlement

1. Scope

The uniform settlement and differential settlement criteria for the C-I buildings will be specified in DCD for the COL applicant.

This section provides the results of the study regarding the allowable values of the settlement and differential settlement using the global FE model. The magnitudes of settlement are related to soil conditions. Soft soil is considered since it has the most settlement potential. To bound the possible horizontal variation in soil stiffness, two distributions are assumed which are uniform and linear gradient as shown in Figure 3.8-93 (9).

2. Settlement for Uniform Soil Conditions

Evaluations for uniform soil conditions are performed using soil springs corresponding to the soft soil whose shear wave velocity is Vs = 300 m/sec, i.e., soil springs used in the DCD design, since the softer soil generates the larger settlement.
Table 3.8-93(1) shows the maximum and average settlements obtained by FE analyses. The average settlement is the average of vertical displacements at four corners of a building.

3. Settlement for Gradient Soil Conditions

3-1. Design considerations

The gradient stiffness distribution is to address the settlement effect of non-uniform distribution of the foundation soil on the basemat design. Estimated differential settlements are used as the allowable values for the DCD. The differential settlement is specified over the longest dimension of a basemat.

3-2. Analysis condition

Analyses are performed under linearly varying stiffness of soil condition (gradient condition), and the results are compared to the DCD design. Analytical conditions are as follows:

- Assumed differential settlement:
  RBFB: 3.0 inches, CB: 0.5 inches.
- FEM Model: Global FEM Model (DCD Design Model)
- Soil Condition: A varying soil spring stiffness is assumed as shown in Figure 3.8-93 (9). The soil spring moduli for gradient stiffness distribution are determined to yield the assumed differential settlements along four directions, E-W, W-E, N-S and S-N. The stiffness of soil spring at the center of the basemat is set to be the same as the uniform soil condition mentioned above.
- Load: Dead Load

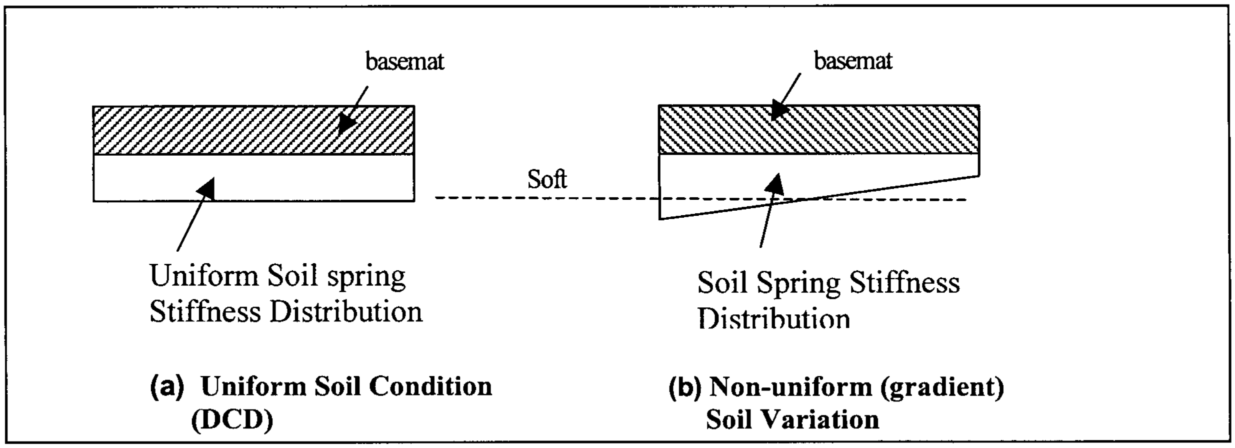


Figure 3.8-93 (9) Soil Conditions

3-3. Analysis result

Table 3.8-93(1) includes the maximum and average settlements obtained by FE analyses for gradient soil conditions. Table 3.8-93(2) summarizes the differential settlements within buildings and between buildings that are determined according to the analysis results. The differential settlement between buildings (RB/FB and CB) shown in Table 3.8-93(2) is the difference between the maximum settlements of the RB/FB and CB which are shown in Table 3.8-93(1). Figures 3.8-93 (10) and (11) show the settlement of buildings. In these figures settlement values are shown at the center of the building, while the maximum settlements indicated in Table 3.8-93(1) occur at any corner of the building. Figures 3.8-93(12) and (13) show the bending moment of the basemat compared to the DCD. It is noted that the effects on the basemat stresses due to the differential settlement are negligible. Therefore, the calculated maximum and differential settlements are used as the allowable values for DCD.

4. Conclusion

The maximum and average settlements under gradient condition shown in Table 3.8-93(1) are used as the allowable maximum settlement within buildings.

The differential settlement obtained in the analyses for the gradient soil conditions shown in Table 3.8-93(2) is used as the allowable differential settlement within a building.

The differential settlement between buildings (RB/FB and CB) shown in Table 3.8-93(2) is used as the allowable value.

Table 3.8-93(1) Calculated Settlement of Buildings

Building		Uniform Soil Condition	Gradient Soil Condition
RB/FB	Maximum	74 mm (2.9 in.)	103 mm (4.0 in.)
	Average*	53 mm (2.1 in.)	65 mm (2.6 in.)
CB	Maximum	11 mm (0.43 in.)	18 mm (0.7 in.)
	Average*	10 mm (0.4 in.)	11 mm (0.43 in.)

* Settlement values of four corner are averaged

Table 3.8-93(2) Calculated Differential Settlement

Building	Differential Settlement within Building	Differential Settlement between Buildings
RB/FB	77 mm (3.0 in.)	85 mm (3.3 in.)
CB	13 mm (0.5 in.)	

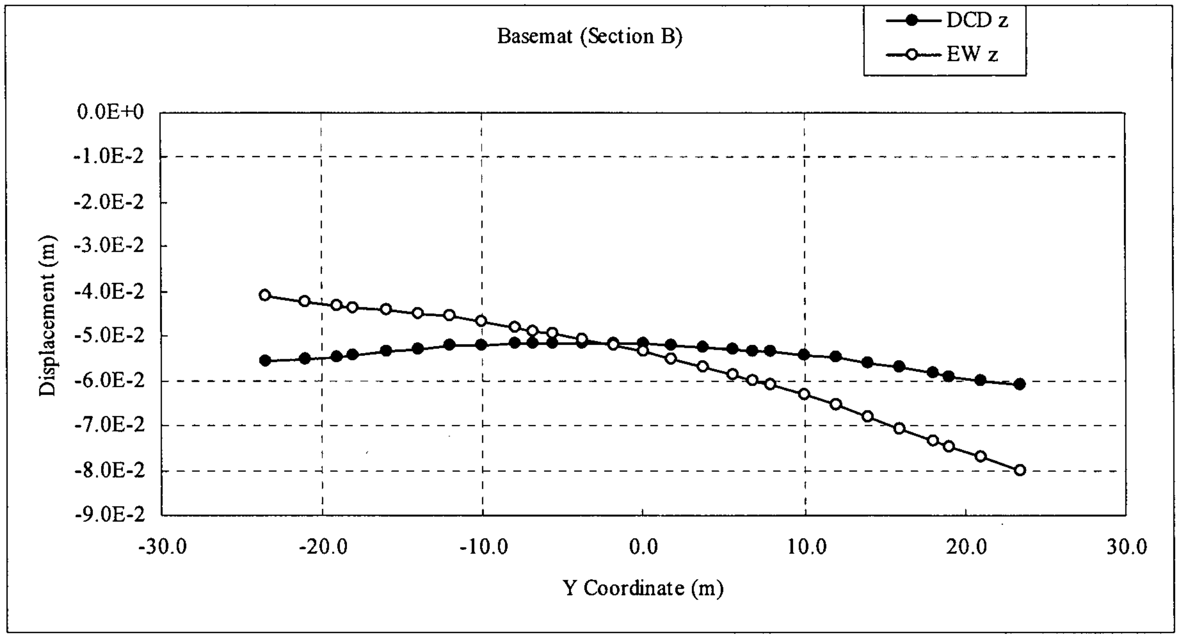


Figure 3.8-93 (10)-a Comparison of Settlement (RBFB, E-W direction)

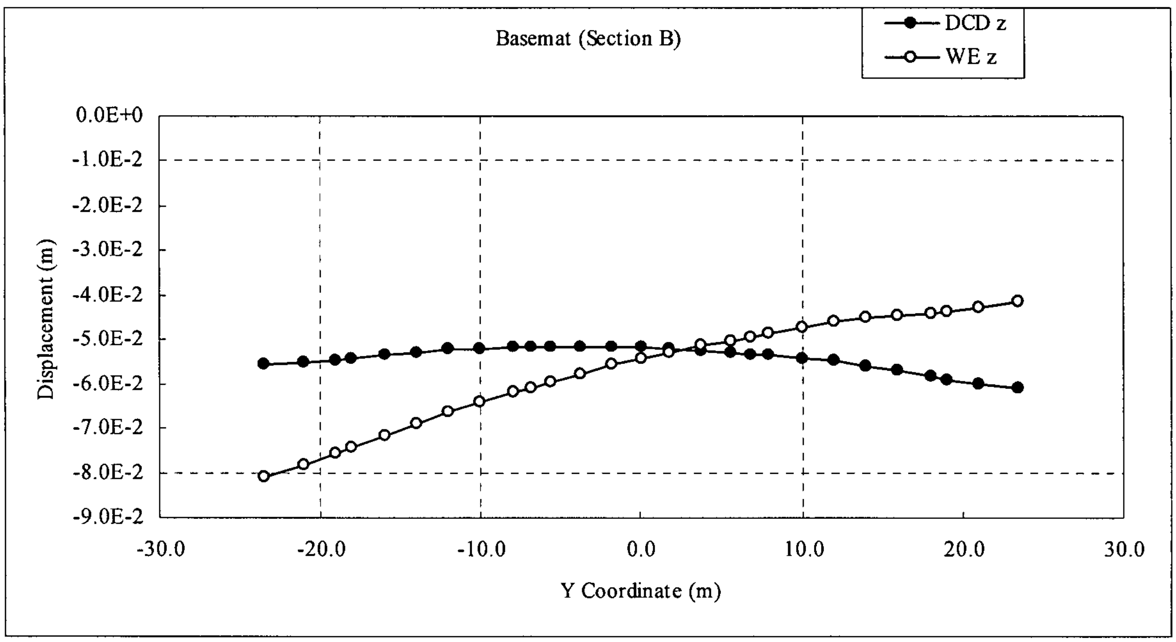
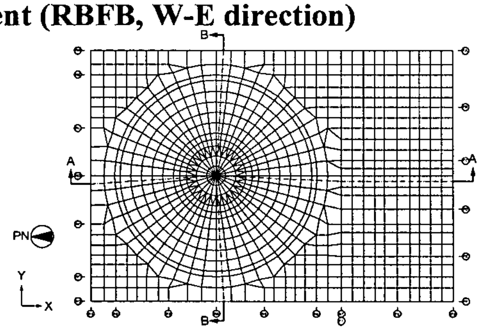


Figure 3.8-93 (10)-b Comparison of Settlement (RBFB, W-E direction)



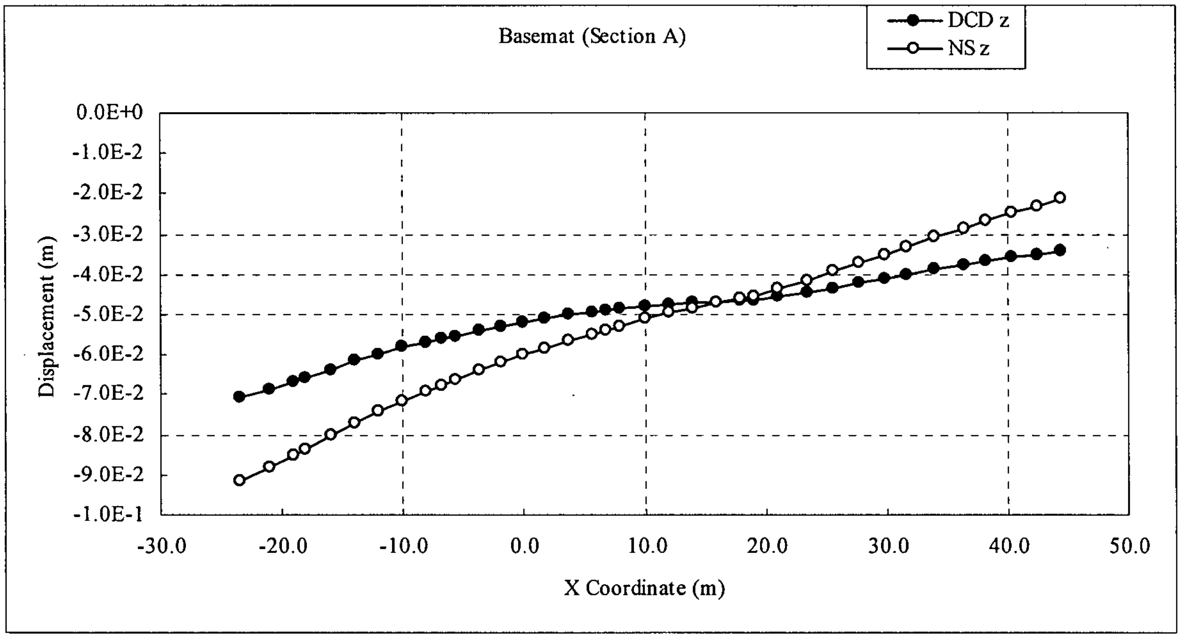


Figure 3.8-93 (10)-c Comparison of Settlement (RBFB, N-S direction)

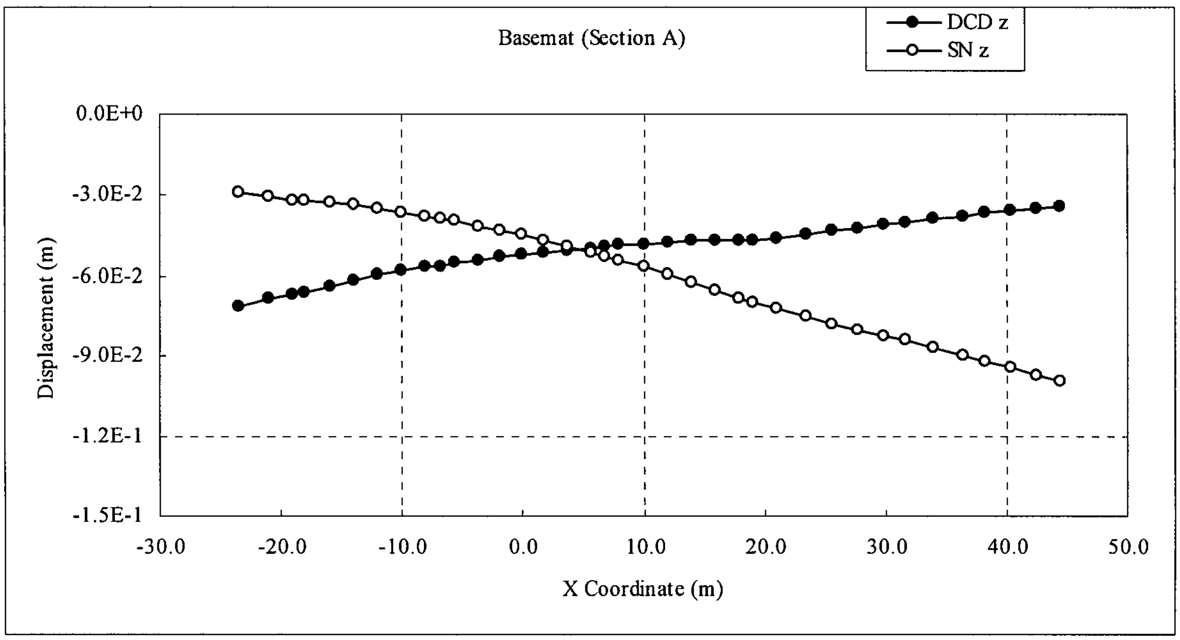
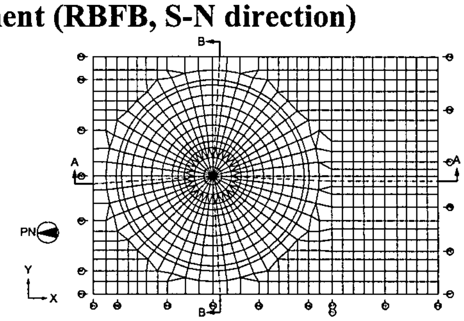


Figure 3.8-93 (10)-d Comparison of Settlement (RBFB, S-N direction)



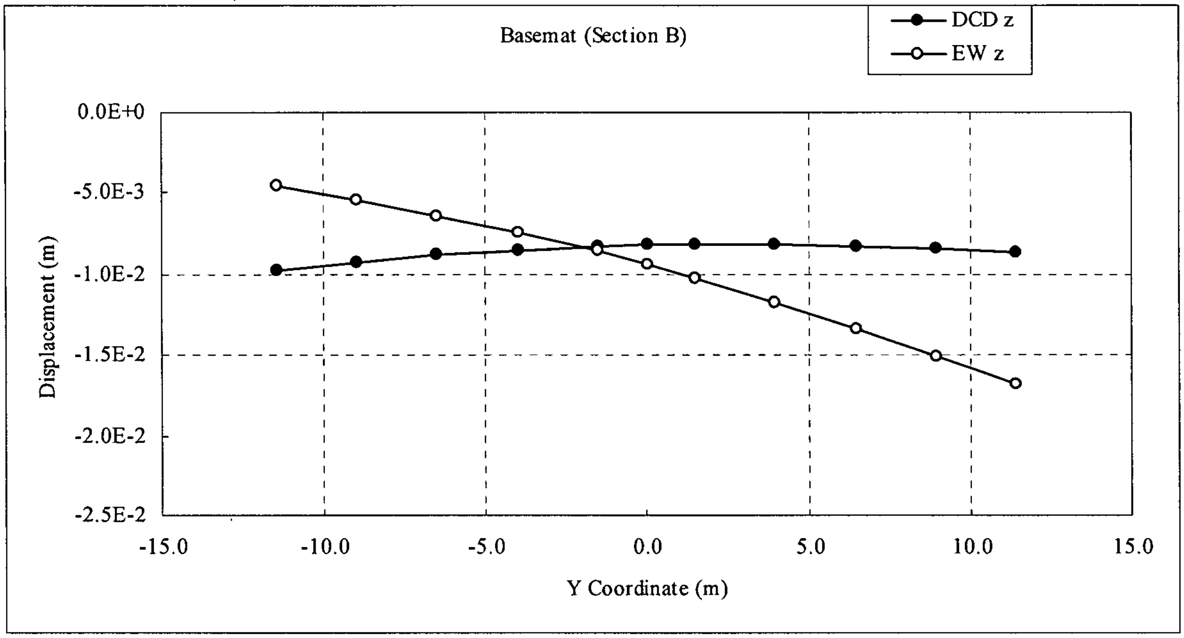


Figure 3.8-93 (11)-a Comparison of Settlement (CB, E-W direction)

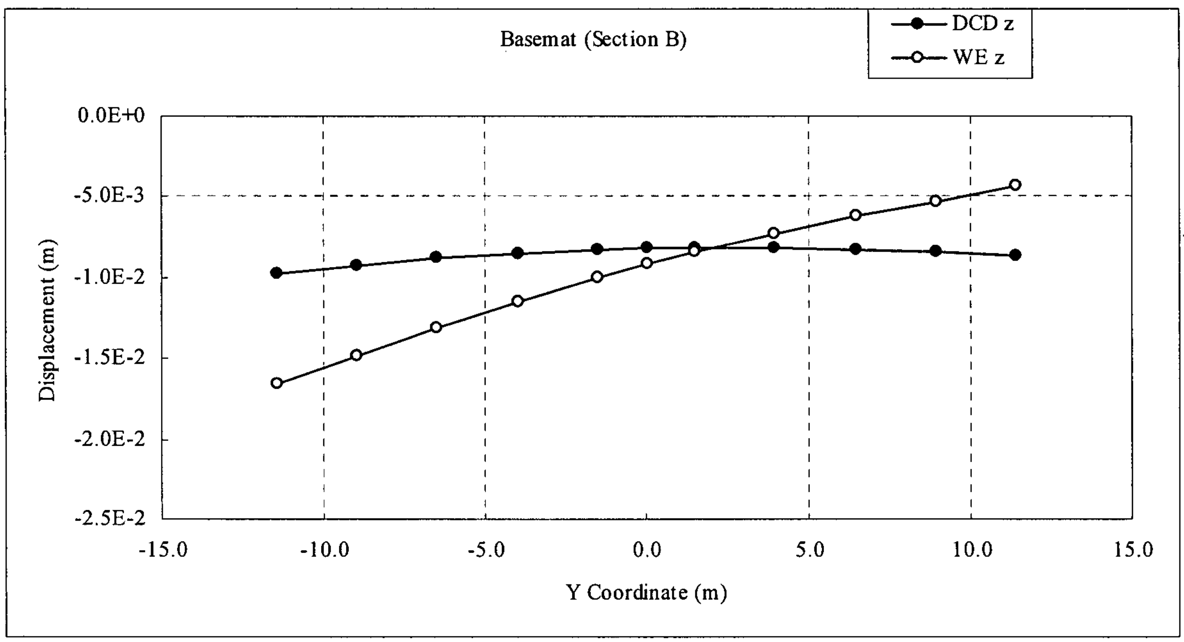
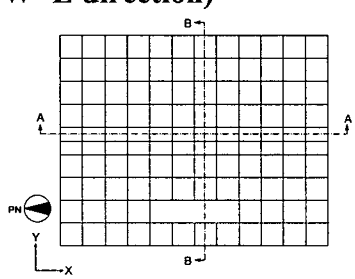


Figure 3.8-93 (11)-b Comparison of Settlement (CB, W -E direction)



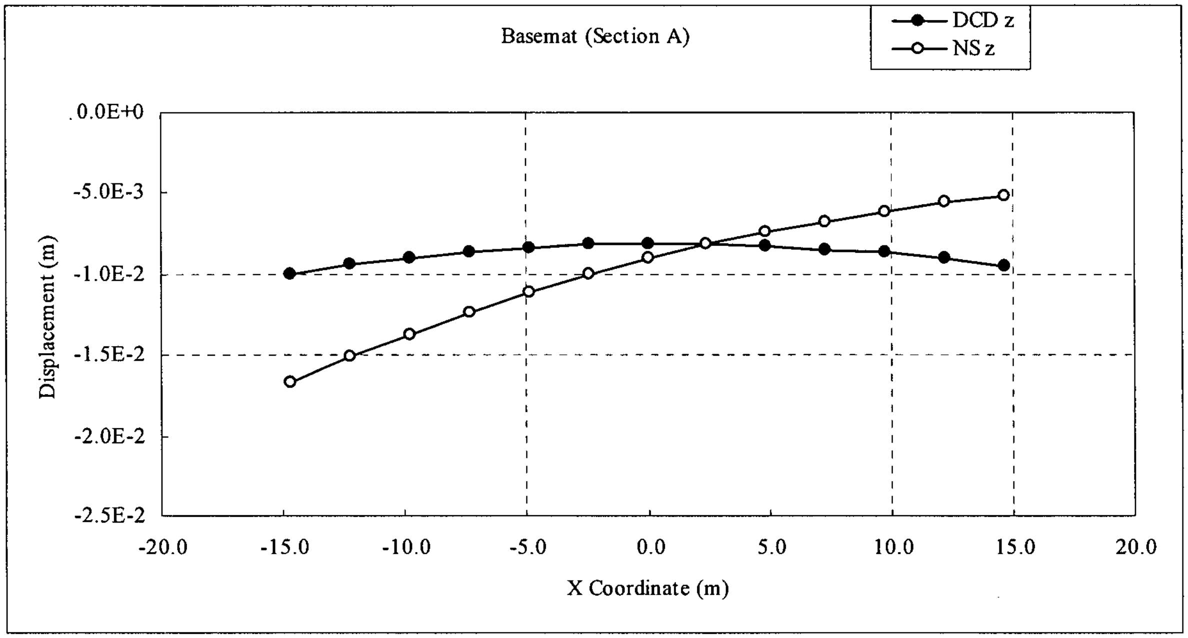


Figure 3.8-93 (11)-c Comparison of Settlement (CB, N-S direction)

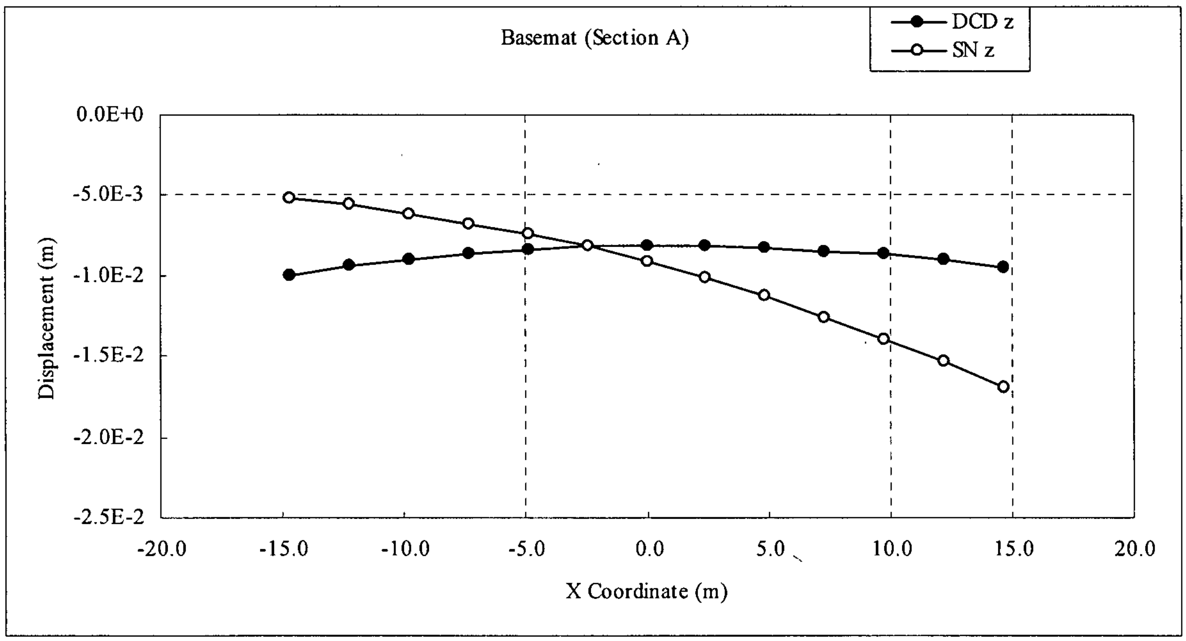
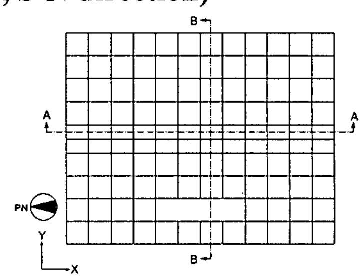
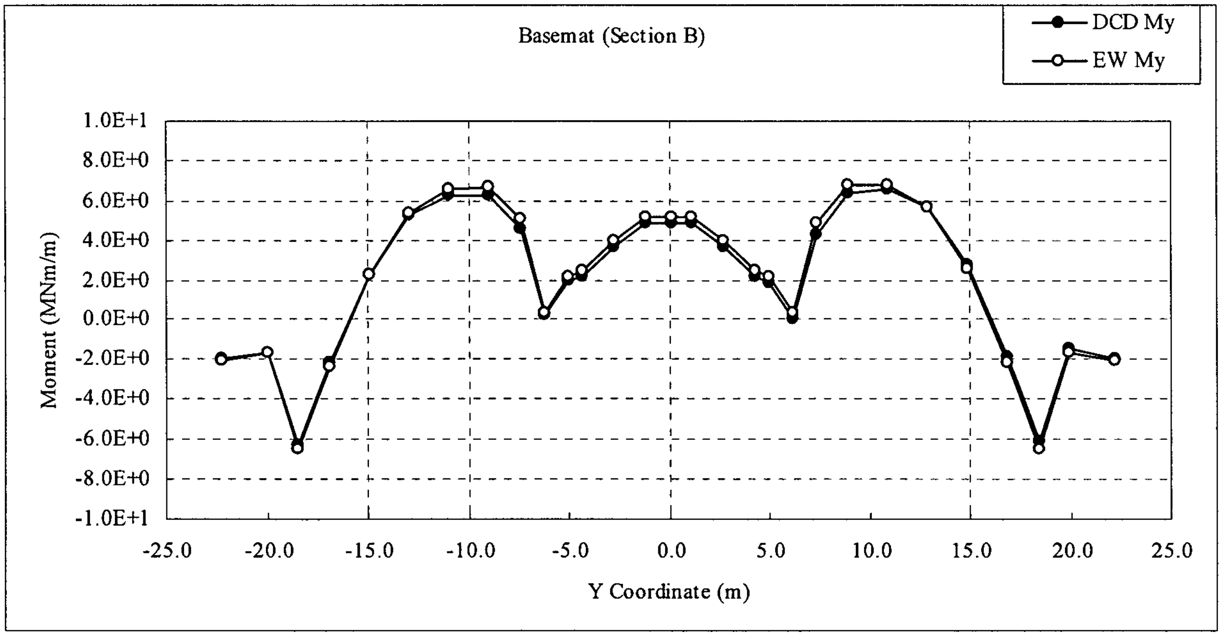
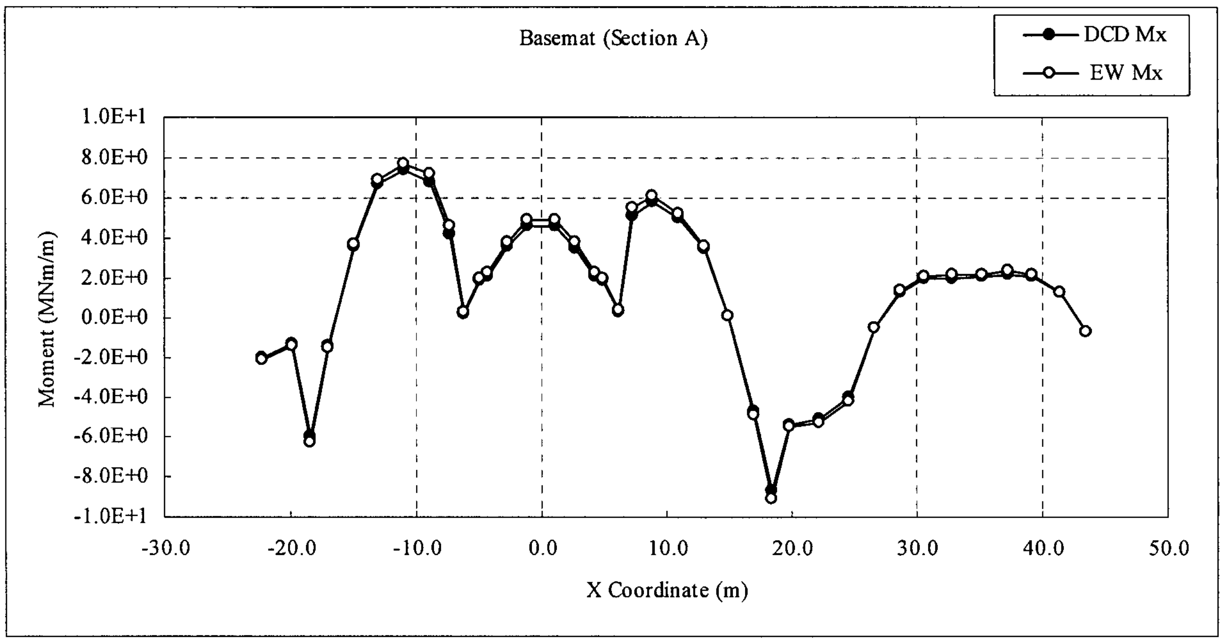


Figure 3.8-93 (11)-d Comparison of Settlement (CB, S-N direction)



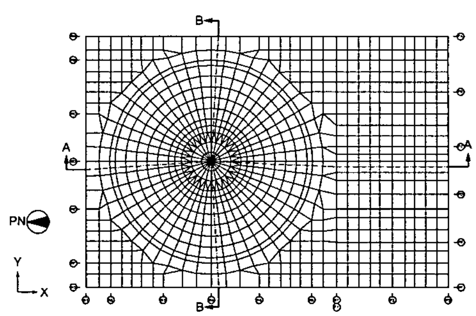


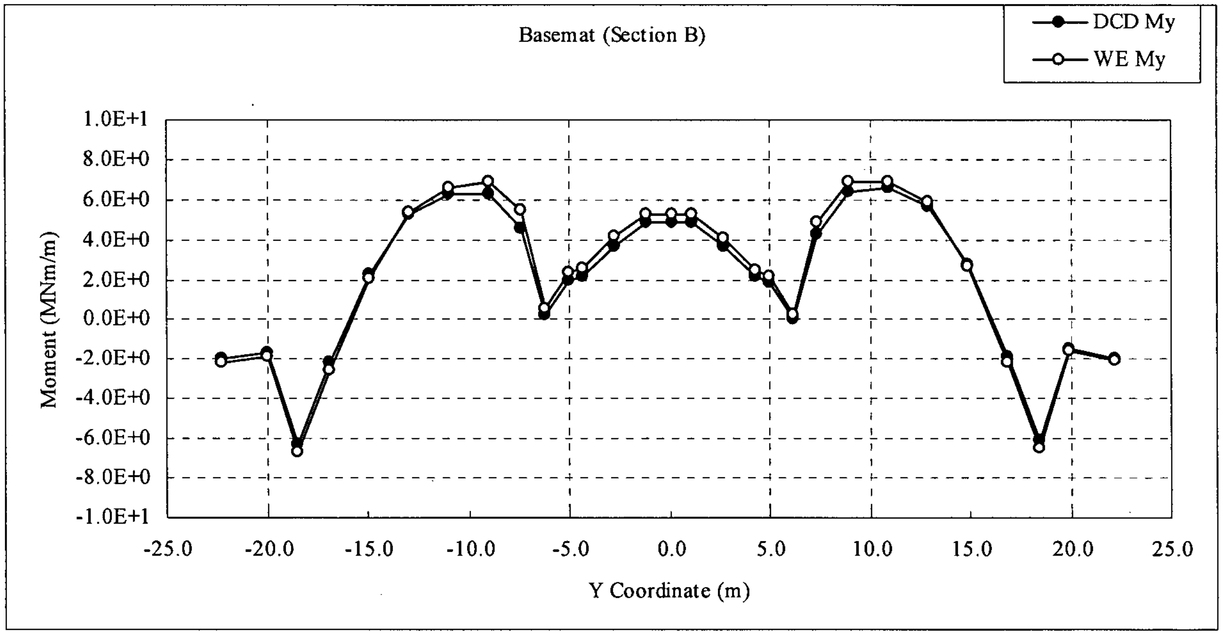
(a) My in E-W Section



(b) Mx in N-S Section

Figure 3.8-93 (12)-a Comparison of Basemat Sectional Moments (RBFB, E-W direction)





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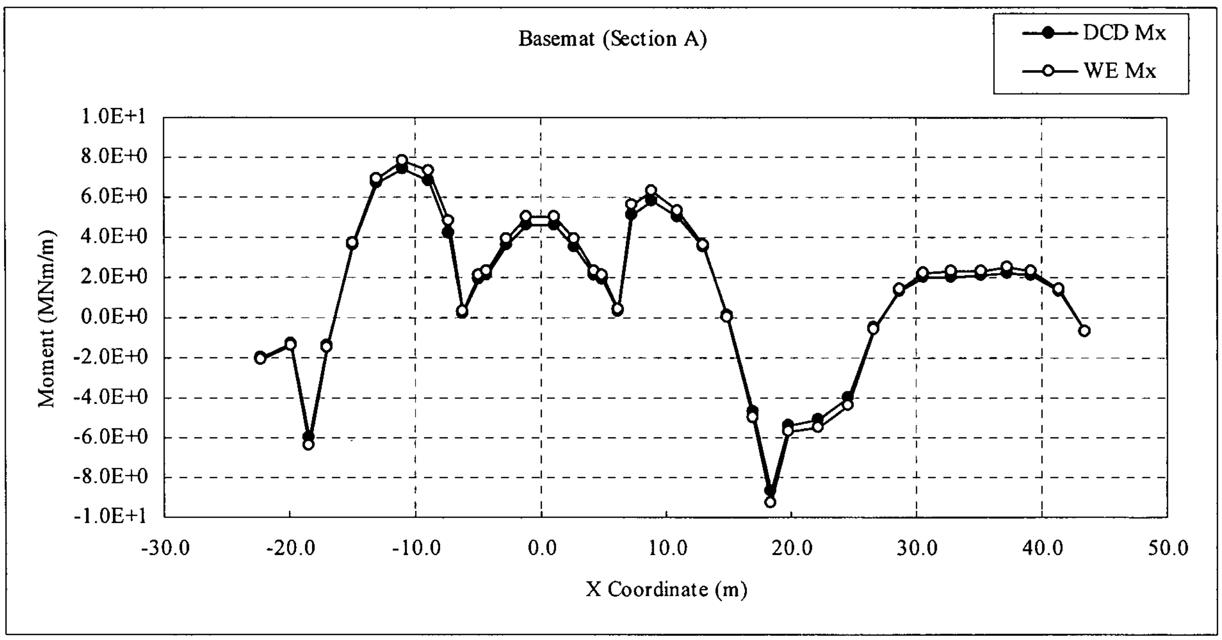
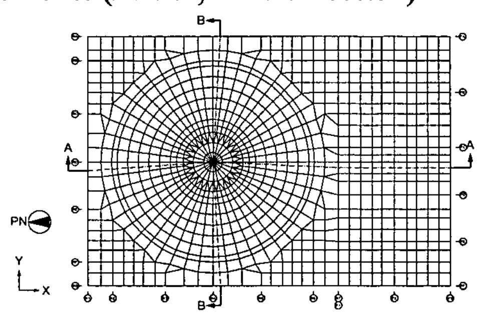


(b) Mx in N-S Section

Figure 3.8-93 (12)-b Comparison of Basemat Sectional Moments (RBFB, W-E direction)



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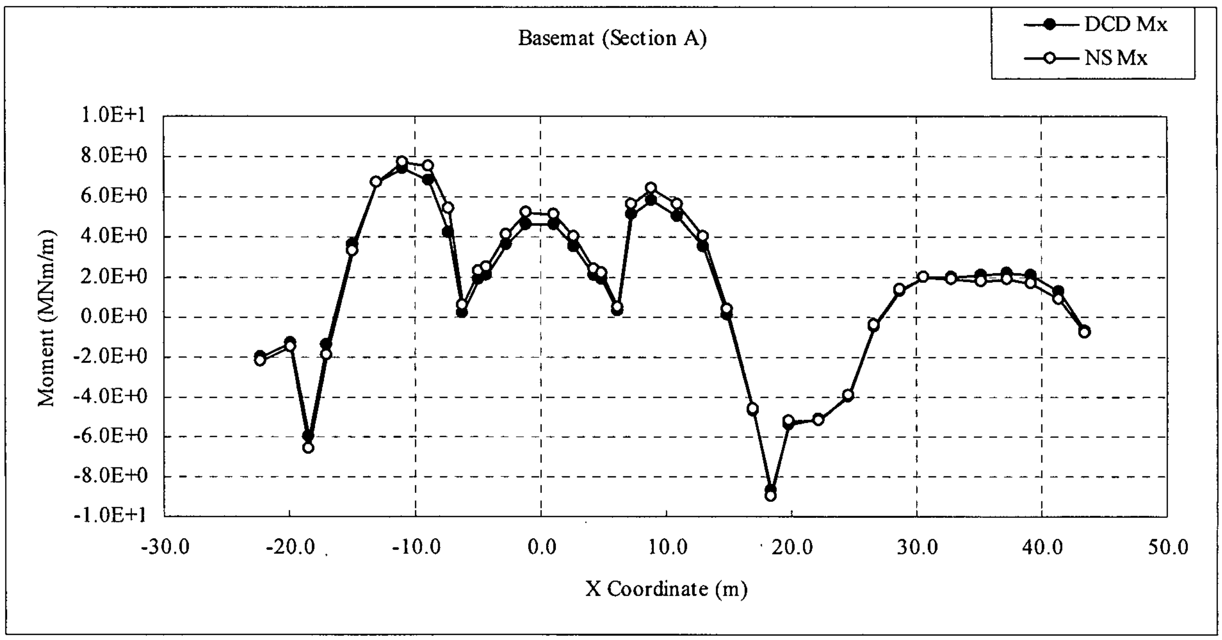
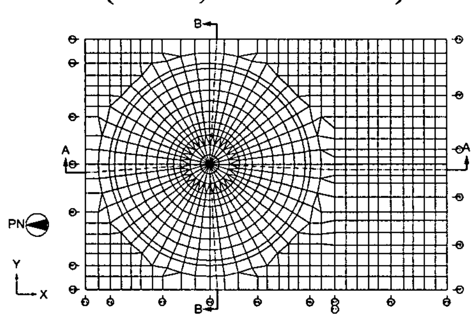
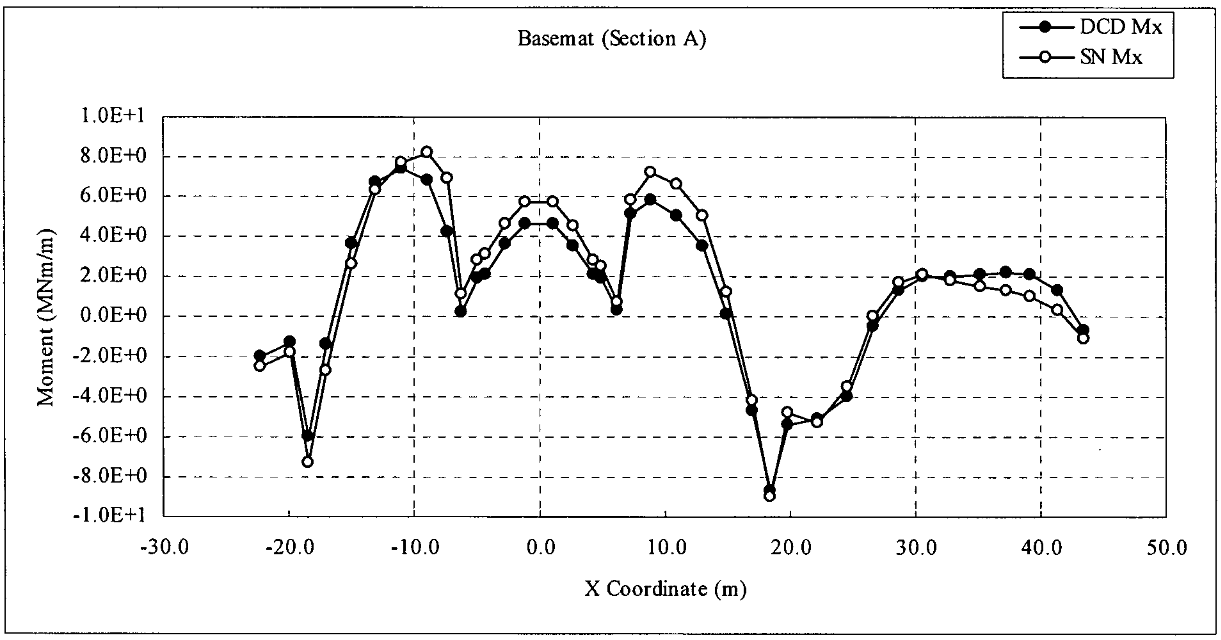


(b) Mx in N-S Section

Figure 3.8-93 (12)-c Comparison of Basemat Sectional Moments (RBFB, N-S direction)

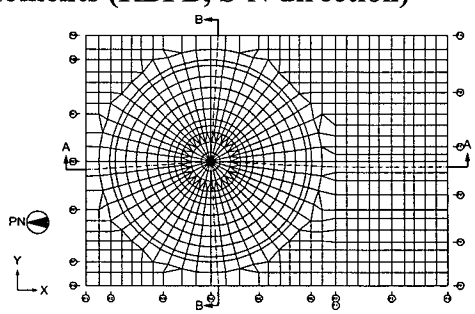


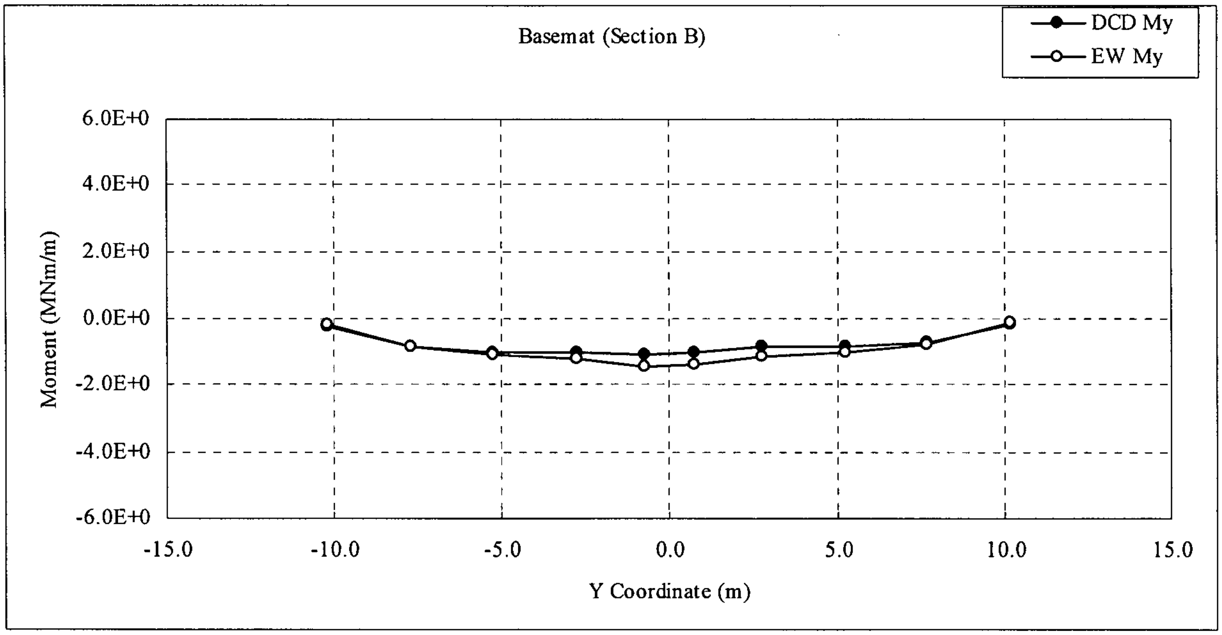
(a) My in E-W Section



(b) Mx in N-S Section

Figure 3.8-93 (12)-d Comparison of Basemat Sectional Moments (RBFB, S-N direction)





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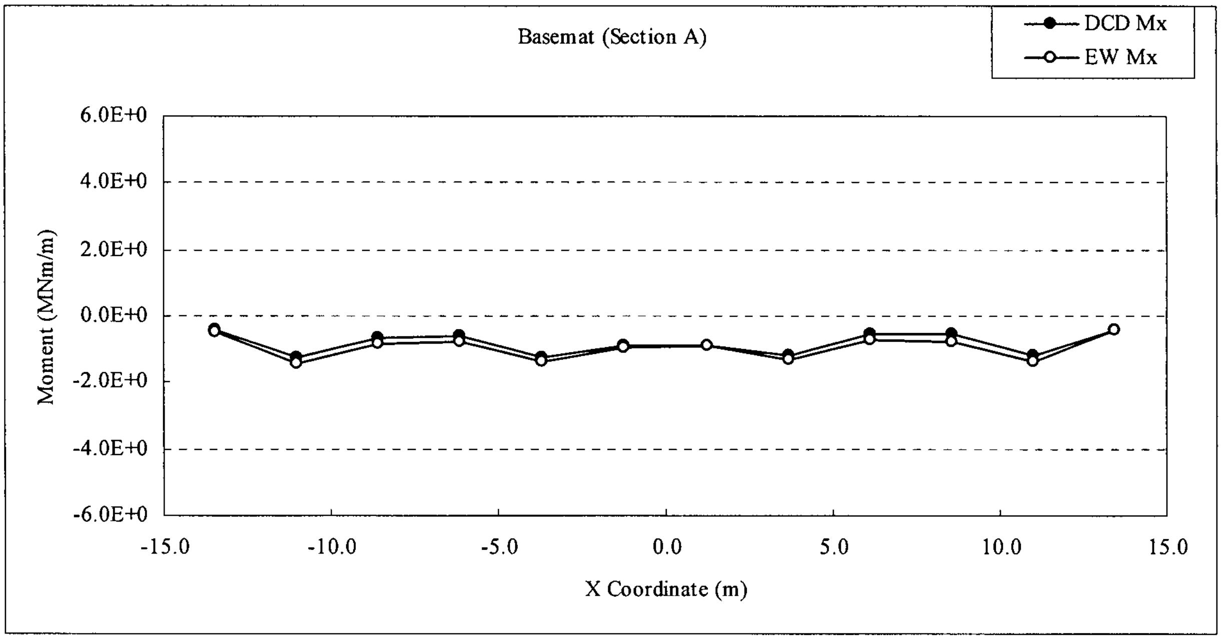
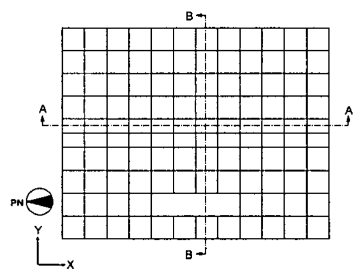
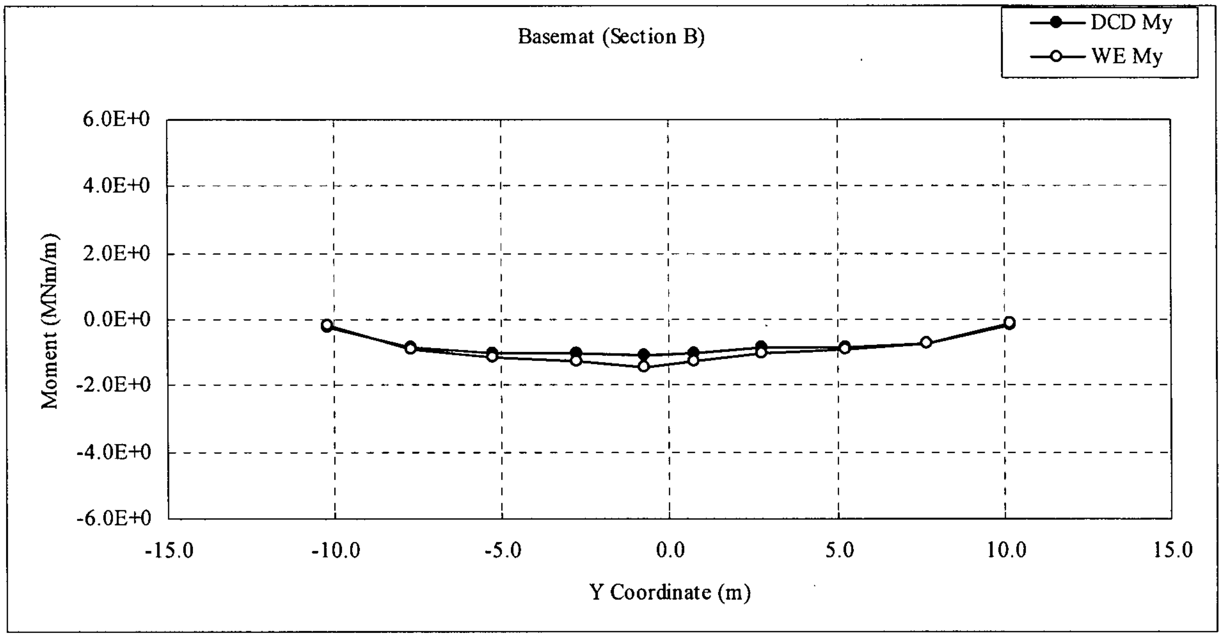


(b) Mx in N-S Section

Figure 3.8-93 (13)-a Comparison of Basemat Sectional Moments (CB, E-W direction)





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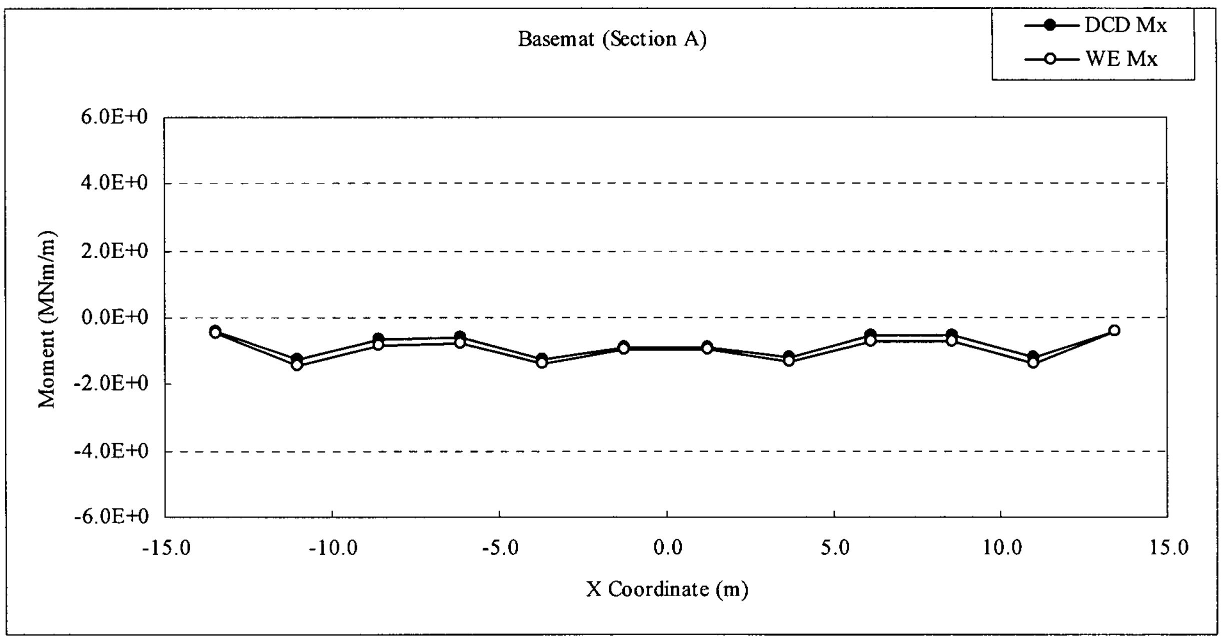
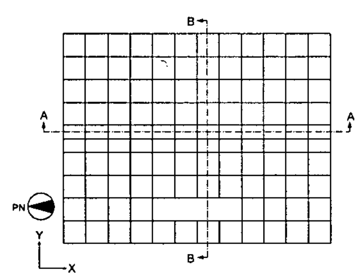
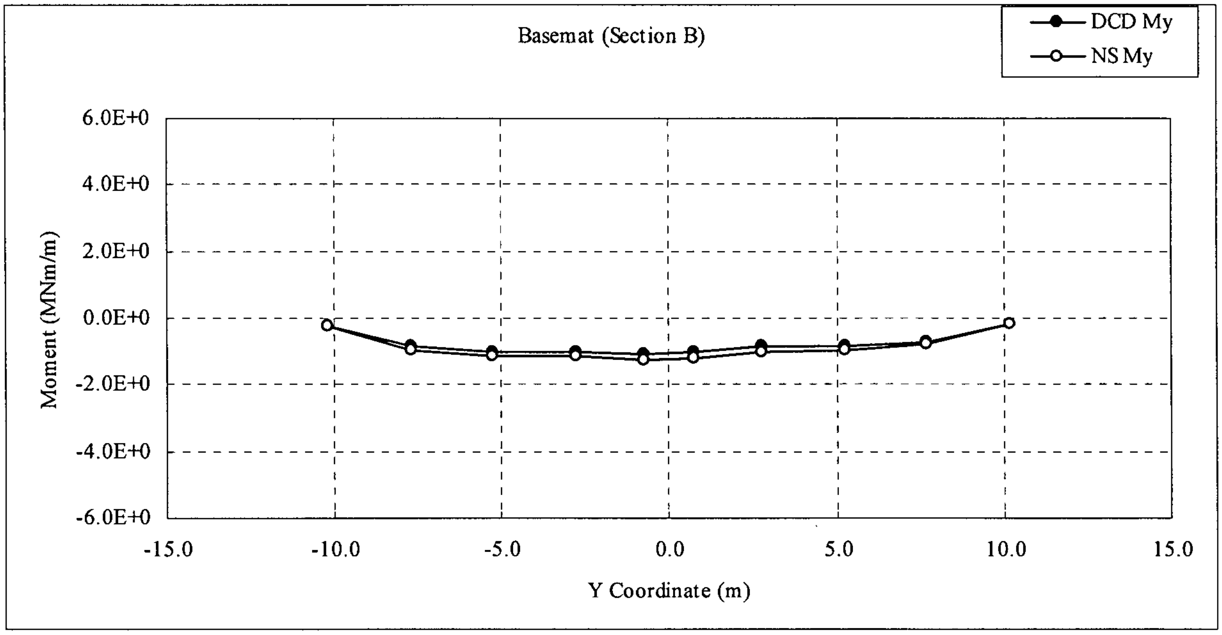


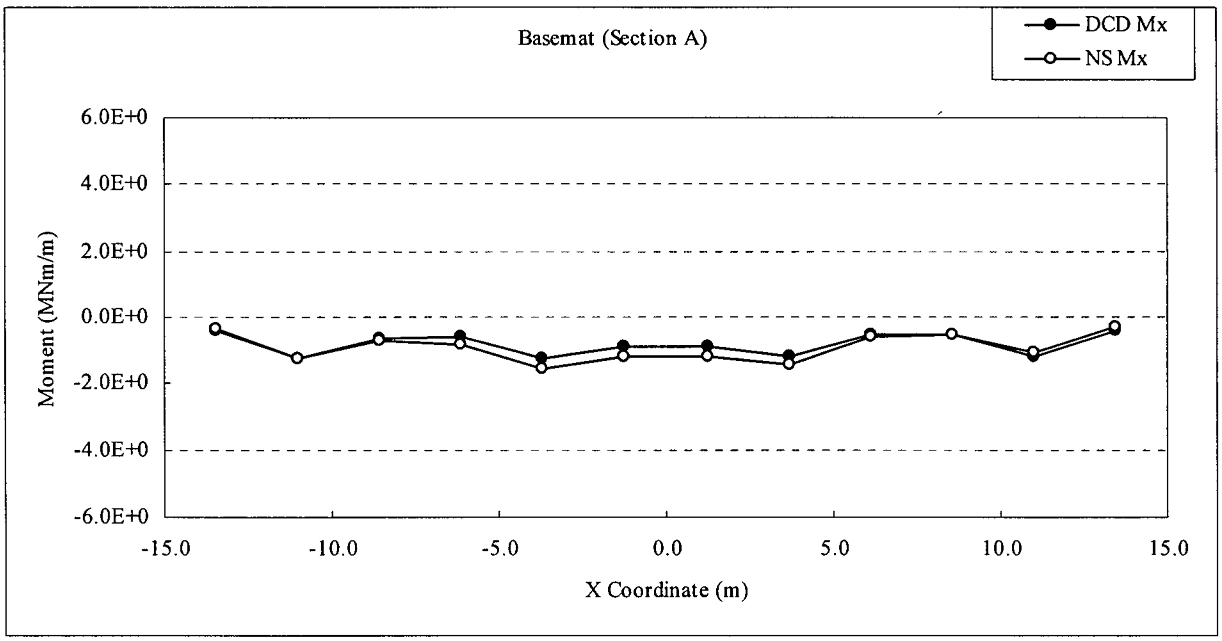
(b) Mx in N-S Section

Figure 3.8-93 (13)-b Comparison of Basemat Sectional Moments (CB, W-E direction)



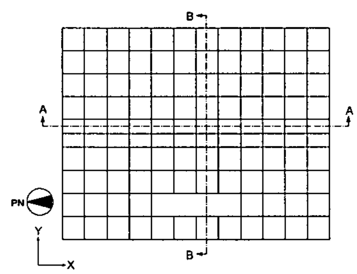


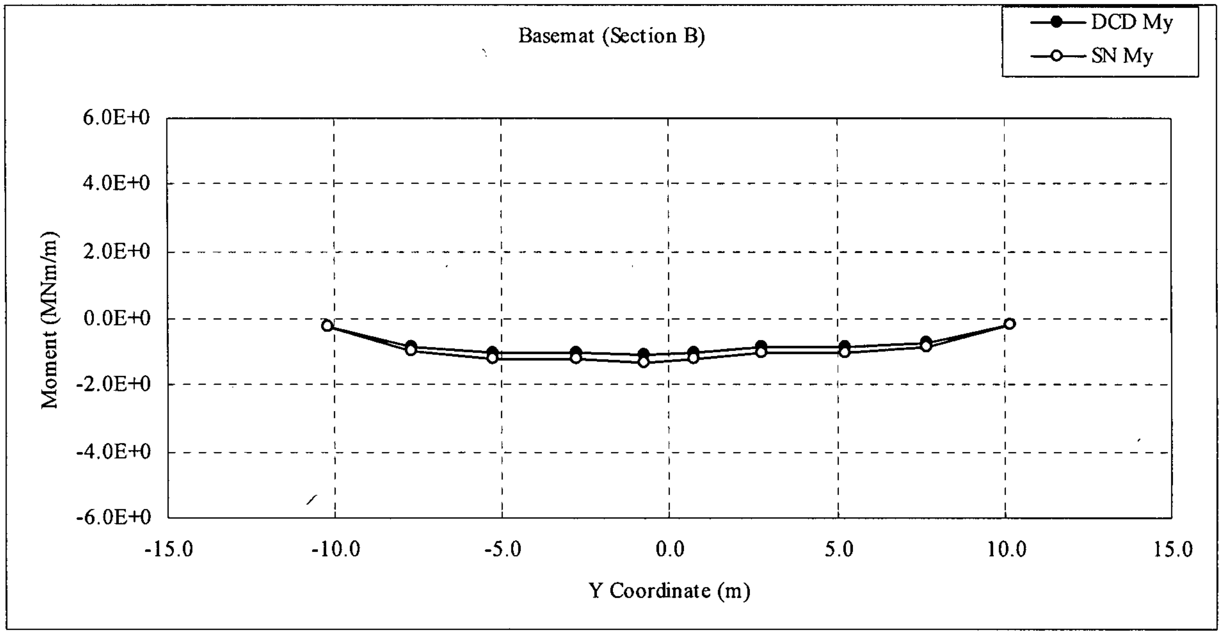
(a) My in E-W Section



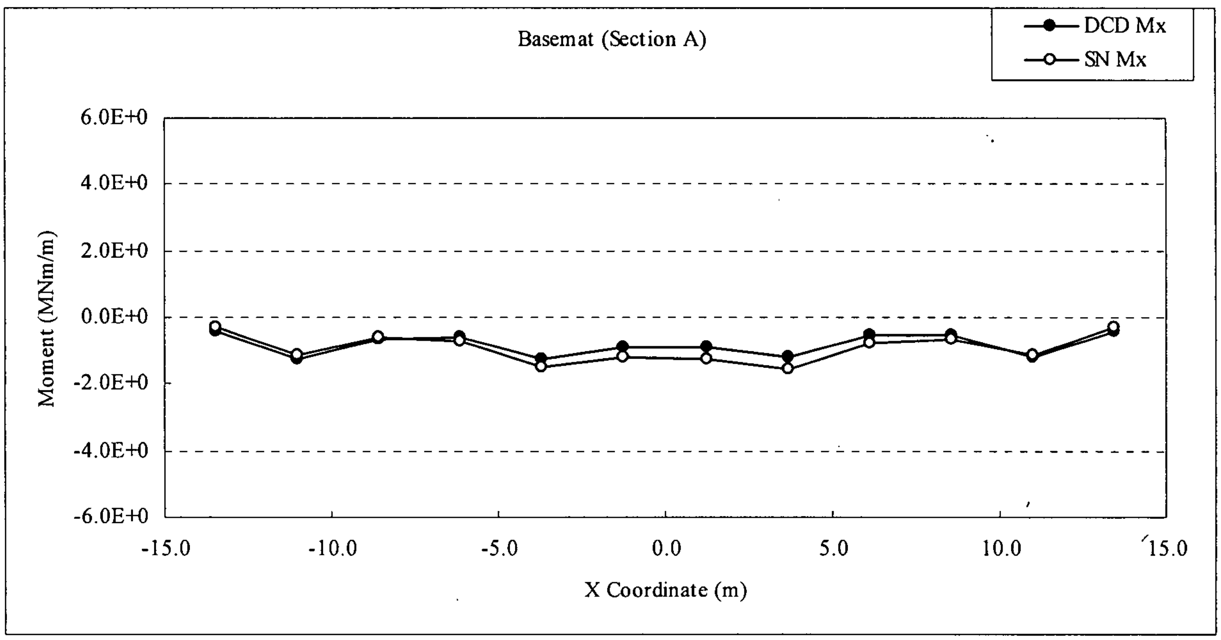
(b) Mx in N-S Section

Figure 3.8-93 (13)-c Comparison of Basemat Sectional Moments (CB, N-S direction)



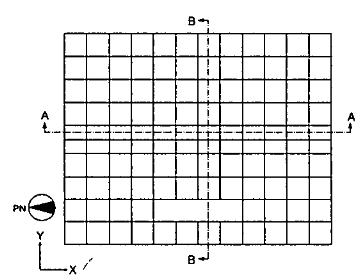


(a) My in E-W Section



(b) Mx in N-S Section

Figure 3.8-93 (13)-d Comparison of Basemat Sectional Moments (CB, S-N direction)



Evaluation for NRC RAI 3.8-93 (3), (7) and (8)

Basemat Design Considering Horizontal Variation of Soil Springs

1. Scope

The original response to NRC RAI 3.8-93 in MFN 06-407 was reviewed by the NRC at the 2nd structural audit in San Jose. It provided the study results based on the "Hard Spot" soil condition under the RPV pedestal area. The NRC requested that other variations such as the inverted type of soil condition (i.e., considering softer soil under the pedestal, "Soft Spot") be added.

This section provides the results of the global model FE analyses concerning the "Soft Spot" soil condition, in addition to the results for "Hard Spot."

2. Analysis condition

Analyses are performed for different soil conditions (non-uniform condition), which are then compared to the DCD design. Analytical conditions are as follows:

- FEM Model: Global FEM Model (DCD Design Model) [Base Case]
- Soil Condition: "Hard Spot", stiff springs are placed under the Pedestal area.
  "Soft Spot", stiff springs are placed peripheral area of pedestal.
  Four types of soil conditions are considered as shown in Figure 3.8-93 (14) in each Spot condition.
- Load: Dead Load
- Analysis cases: Stiffer area has four types of soil stiffness as shown Table 3.8-93(3) and Figure 3.8-93(14).

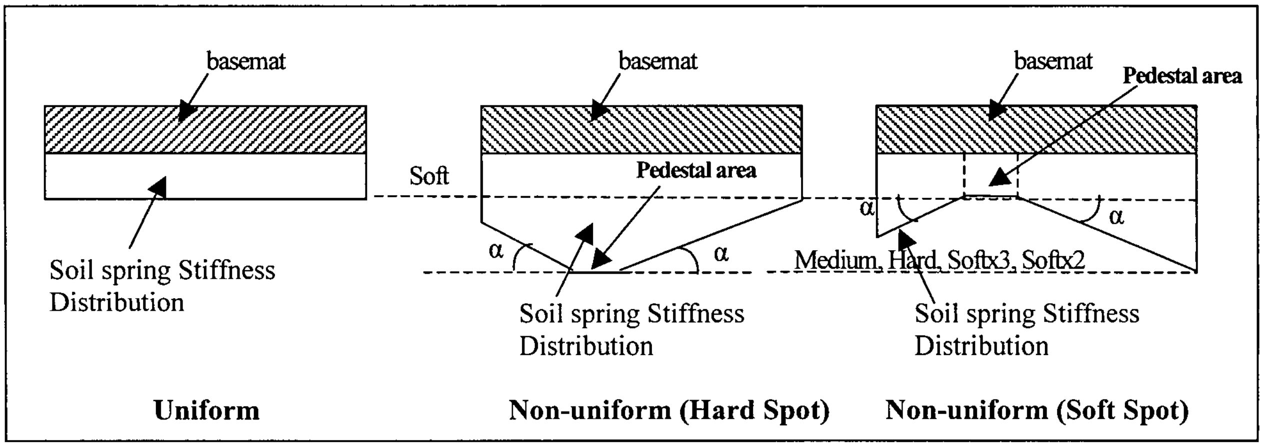


Figure 3.8-93 (14) Soil conditions

Table 3.8-93(3) Analysis Case

Type	No.	Soil Conditions*	
		Center(Inner Pedestal)	Outer(edge of mat)
Hard Spot	H-1	Hard	Soft
	H-2	Medium	
	H-3	Softx3	
	H-4	Softx2	
Soft Spot	S-1	Soft	Hard
	S-2		Medium
	S-3		Softx3
	S-4		Softx2

* See DCD Tier 2 Table 3A.3-1 for site properties. Softx3 and Softx2 have three and two times the soft soil stiffness respectively.

3. Analysis result

Figures 3.8-93(15) and (16) show the basemat deformation and bending moment compared with the DCD design (uniform soil condition), including "Hard Spot" condition. These figures indicate that the bending moment for the Hard Spot condition is smaller or similar to the DCD design since the soil springs are stiffer than the DCD condition (uniform soft soil). However, the "Soft Spot" condition slightly exceeds the DCD condition. Figures 3.8-93(17) through (23) compare the moments at the bottom of the walls. Also in these figures, some of the "Soft Spot" cases show slight differences from the DCD envelope. However, the DCD design envelops the result of the horizontal soil spring variations as long as the ratio of the spring stiffness at the basemat center to that at the basemat edge does not exceed 3. This spring stiffness ratio converts to $\sqrt{3}$ (1.7) for the corresponding shear wave velocity ratio.

4. Conclusion

Figure 3.8-93(24) shows the relative displacement normalized to the basemat displacement at the center position of the RPV. It is found that the "Hard Spot" condition results in a different pattern of relative displacements when compared against the DCD Tier 2 analysis results.

Figure 3.8-93(16)-c indicates that moments for the "Soft x 3" case exceed the base case (DCD) under the "Hard Spot" condition. For the design allowable, slightly less than 3x soft or hard conditions is used. In fact, ACI-336.2R, *Suggested Analysis and Design Procedure for Combined Footings and Mats*, Section 6.9, suggests only using 2x.

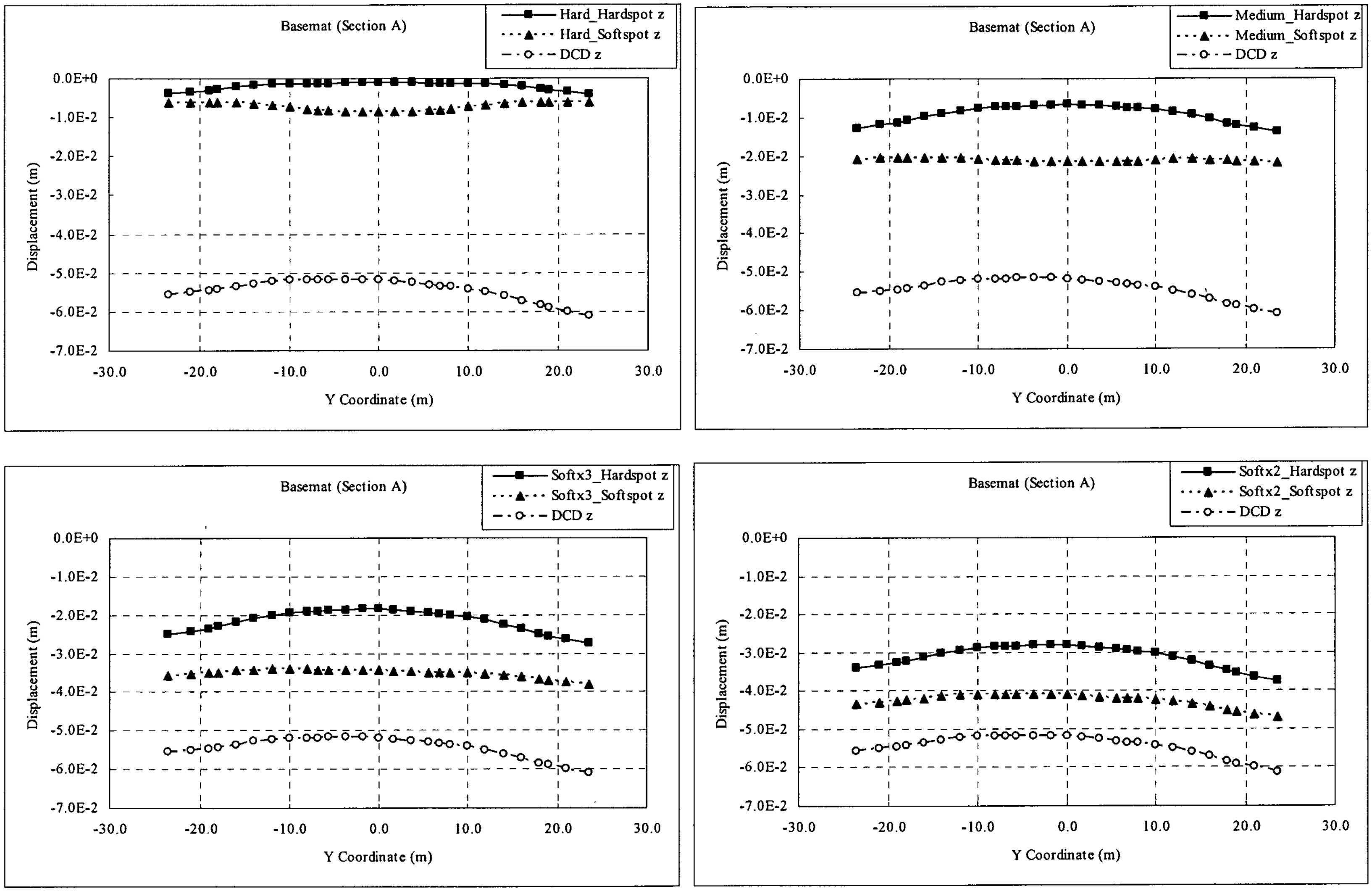


Figure 3.8-93 (15) -a Comparison of Basemat Deformation (E-W Section)

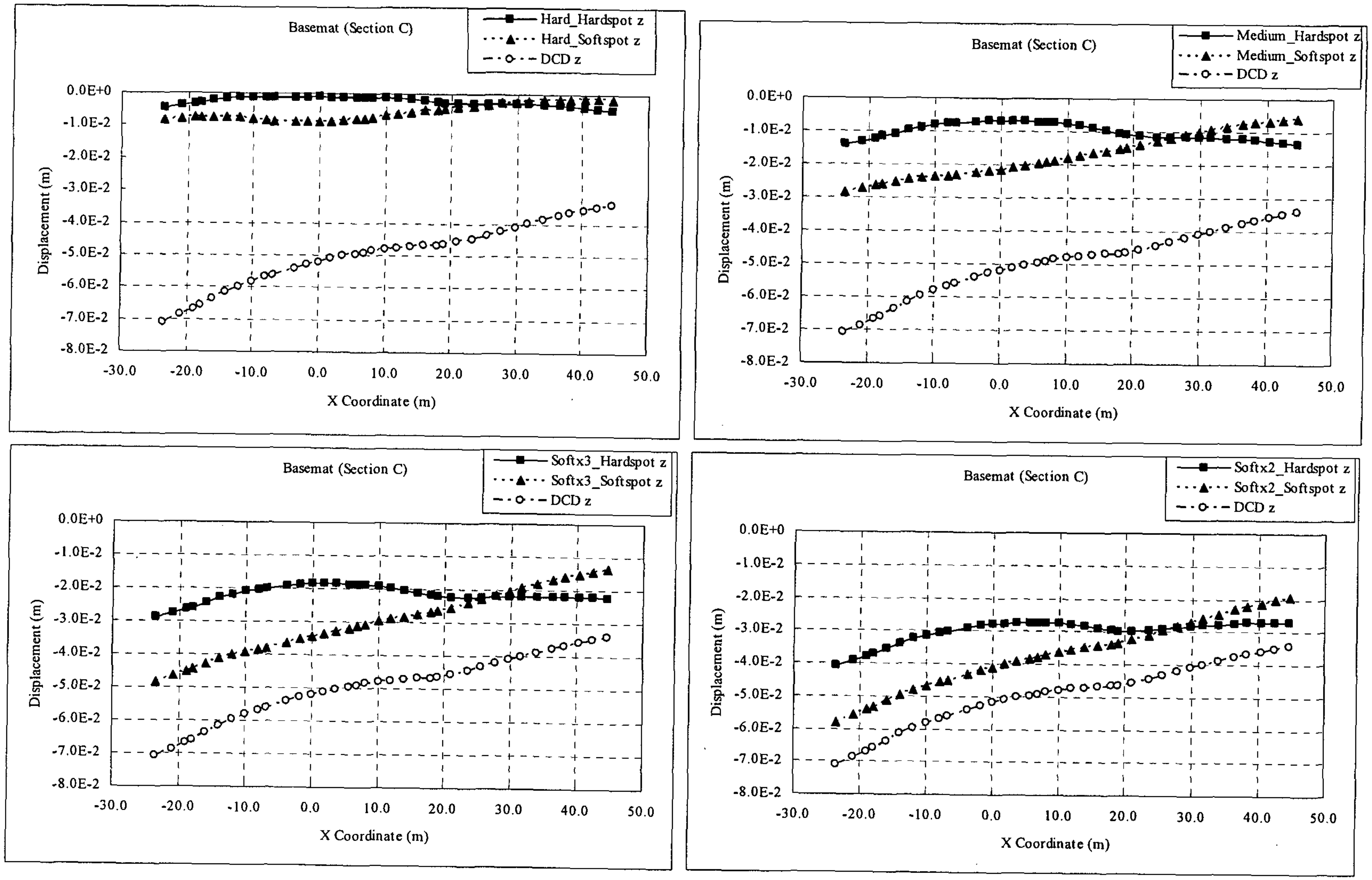
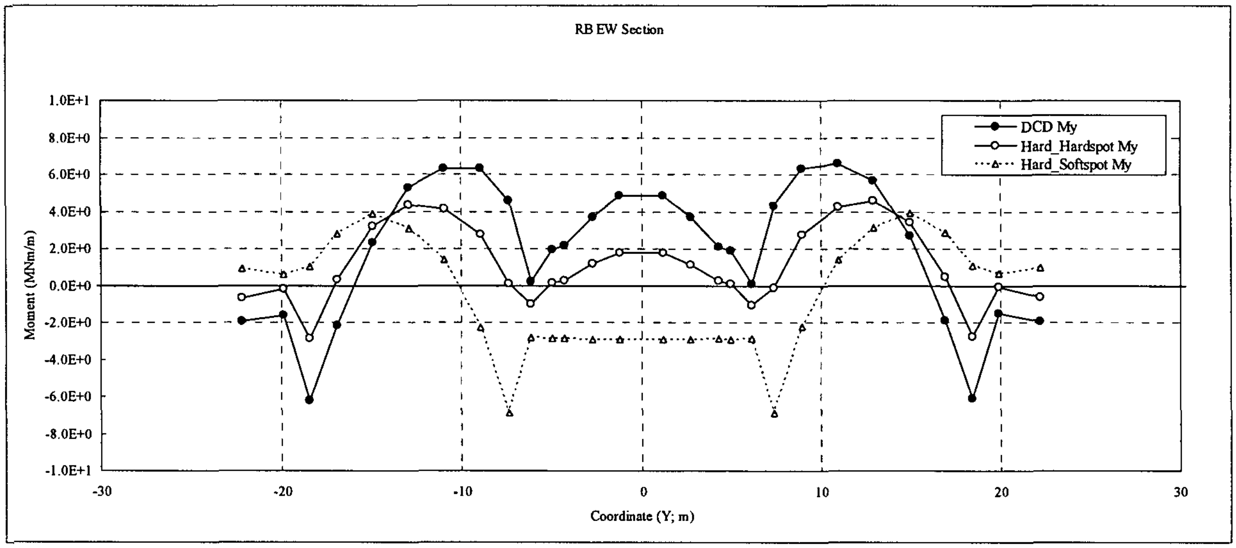
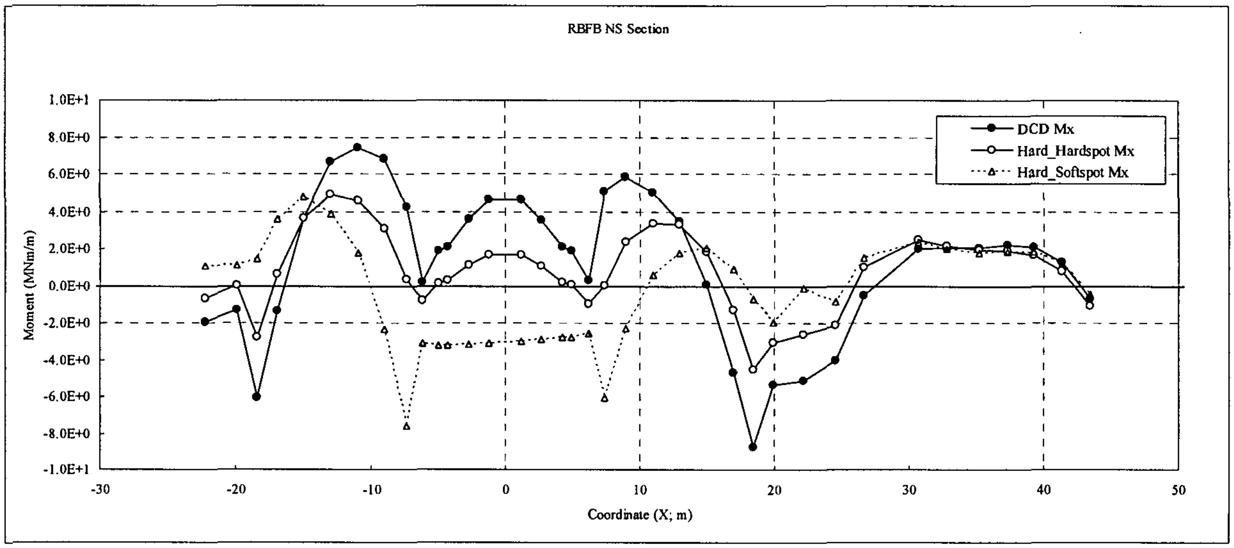


Figure 3.8-93 (15)-b Comparison of Basemat Deformation N-S Section)

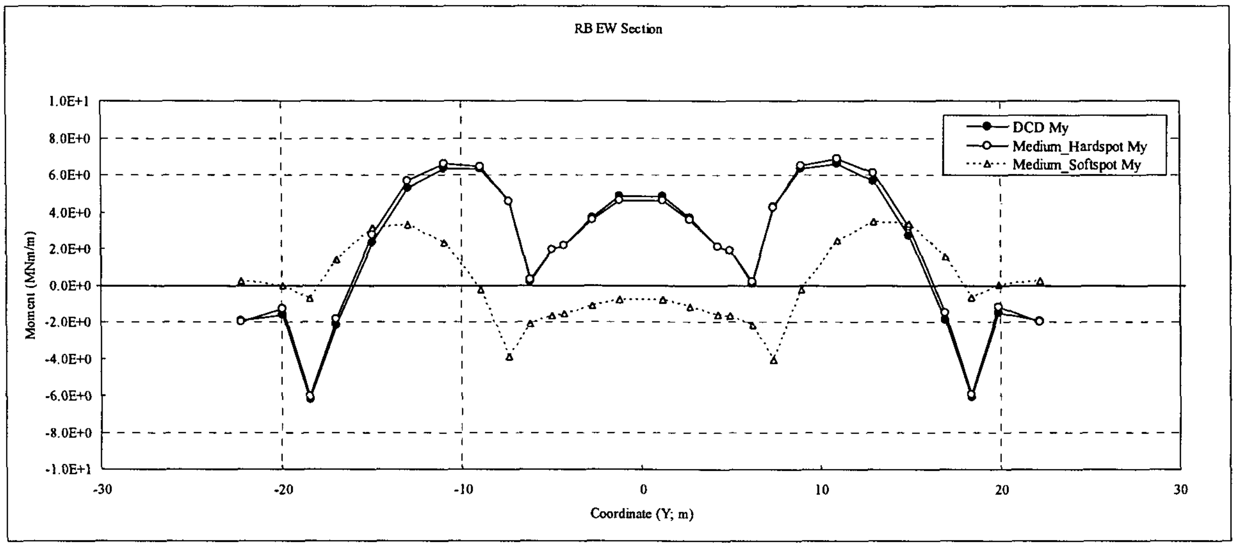


(a) My in E-W Section

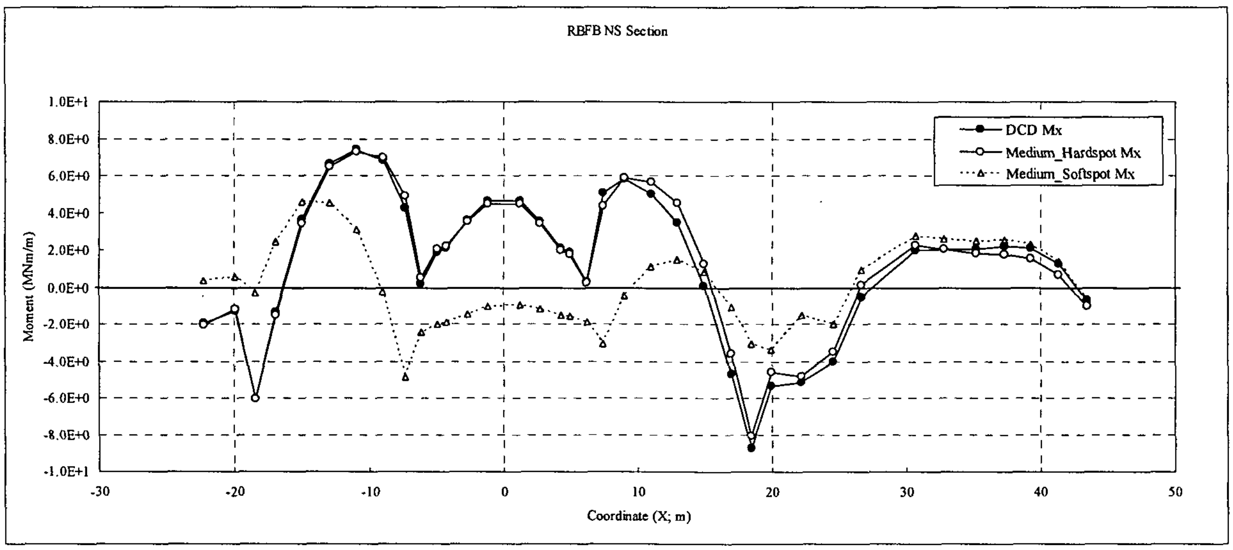


(b) Mx in N-S Section

Figure 3.8-93 (16)-a Comparison of Basemat Sectional Moments (Hard)

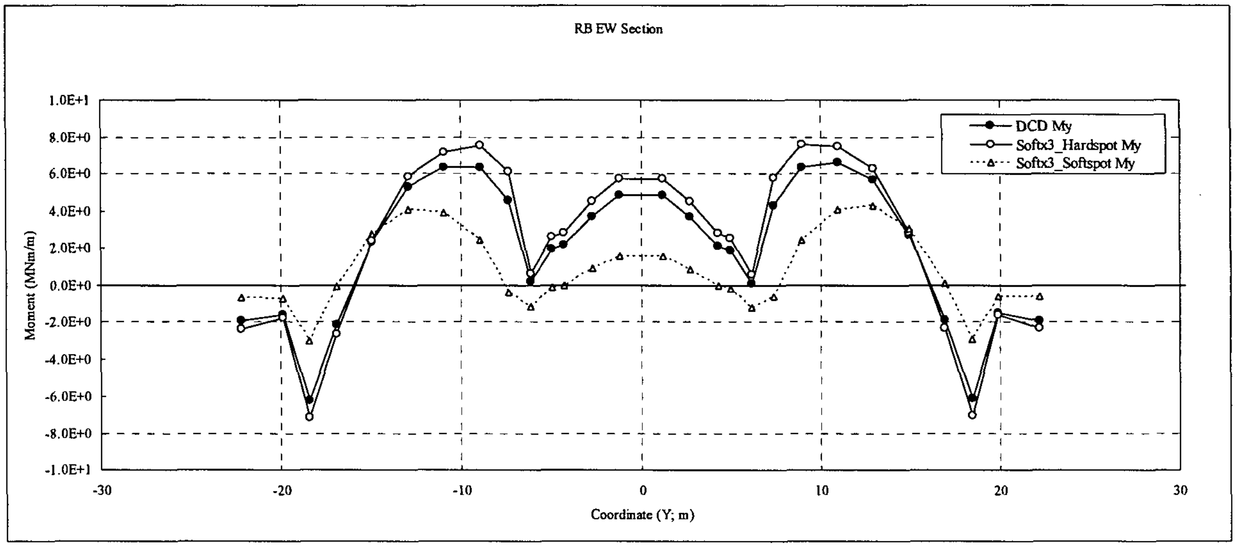


(a) My in E-W Section

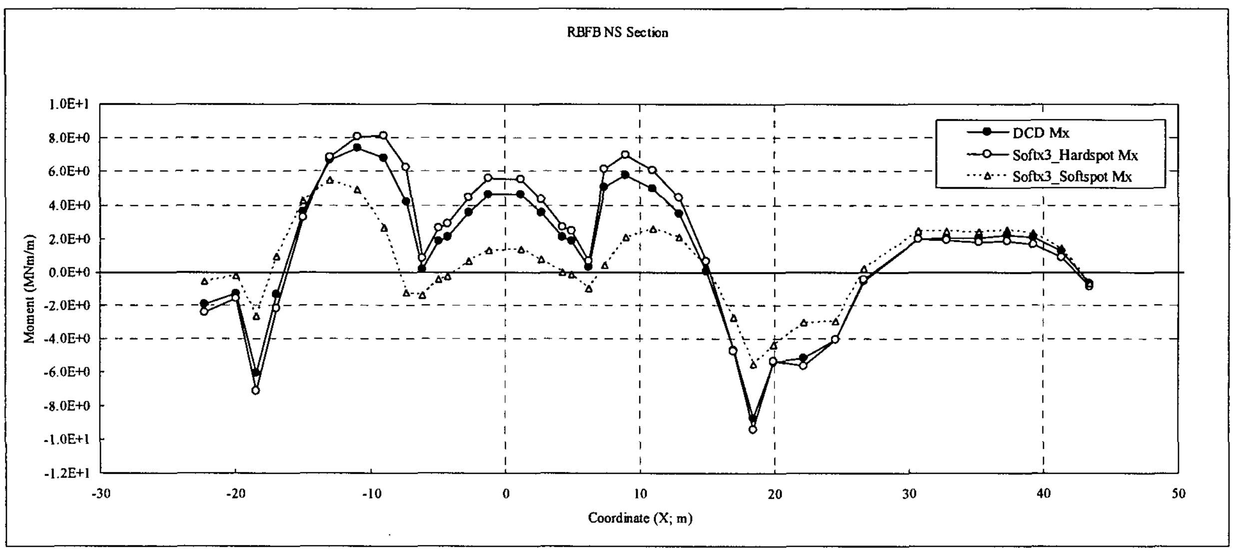


(b) Mx in N-S Section

Figure 3.8-93 (16)-b Comparison of Basemat Sectional Moments (Medium)

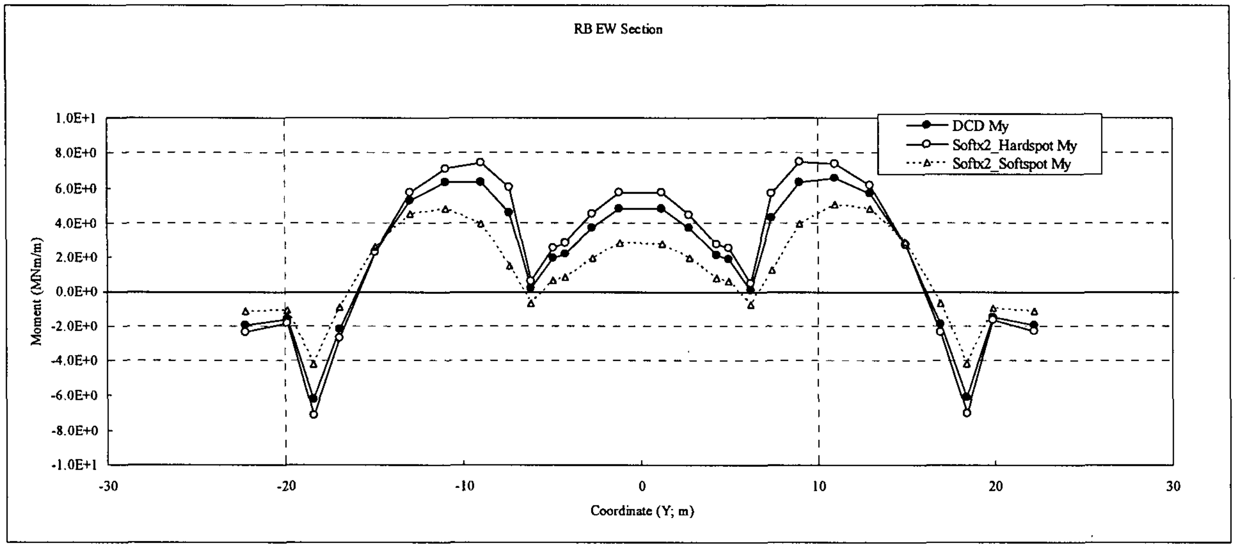


(a) My in E-W Section

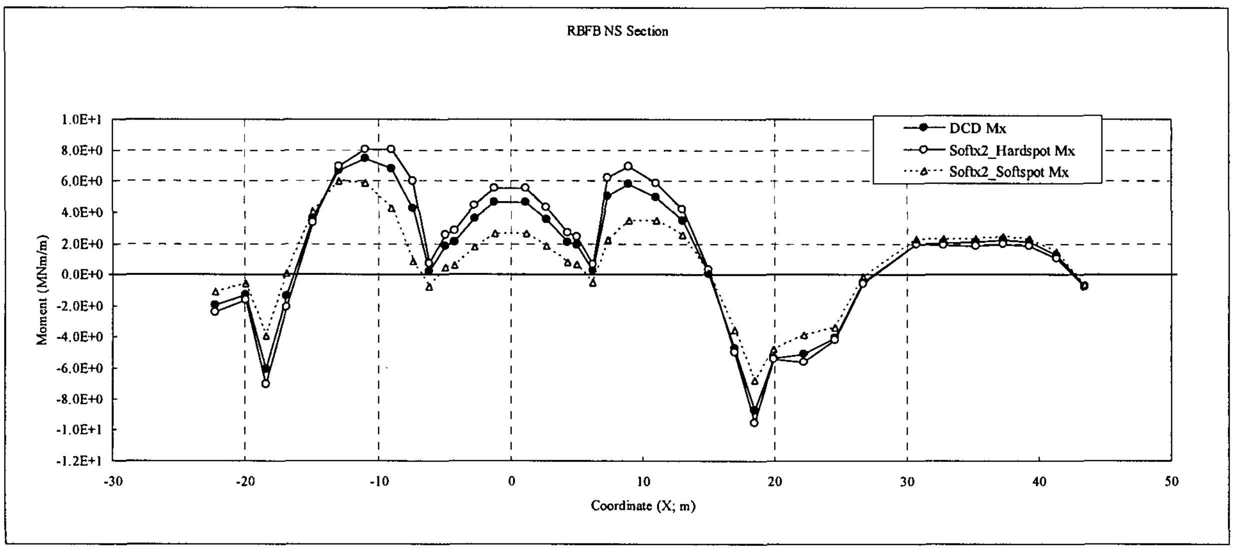


(b) Mx in N-S Section

Figure 3.8-93 (16)-c Comparison of Basemat Sectional Moments (soft x3)



(a) My in E-W Section



(b) Mx in N-S Section

Figure 3.8-93 (16)-d Comparison of Basemat Sectional Moments (soft x2)

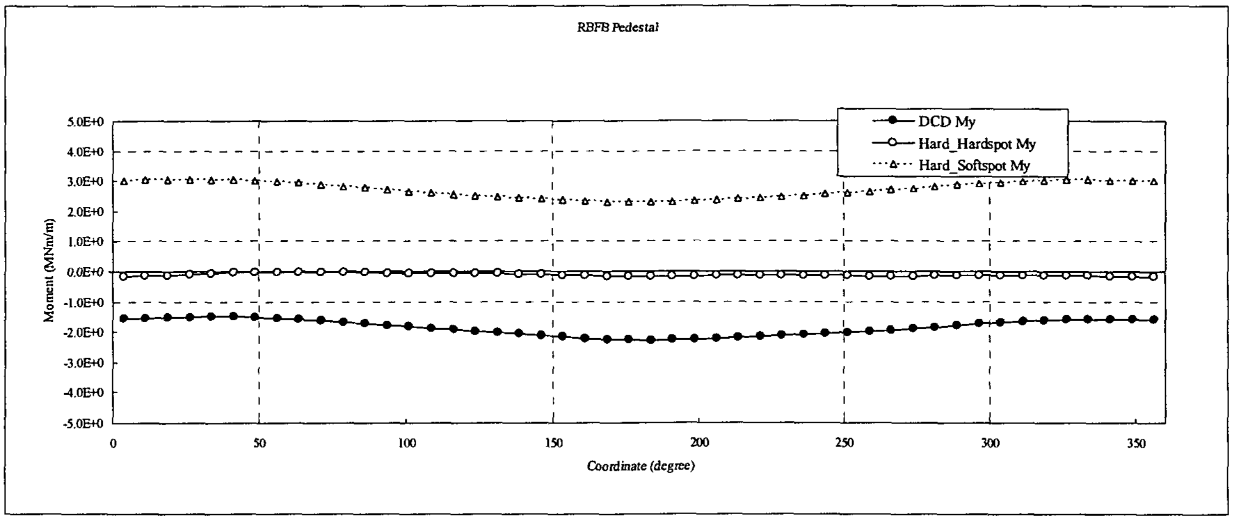


Figure 3.8-93 (17)-a Comparison of Sectional Moments (Pedestal, Hard)

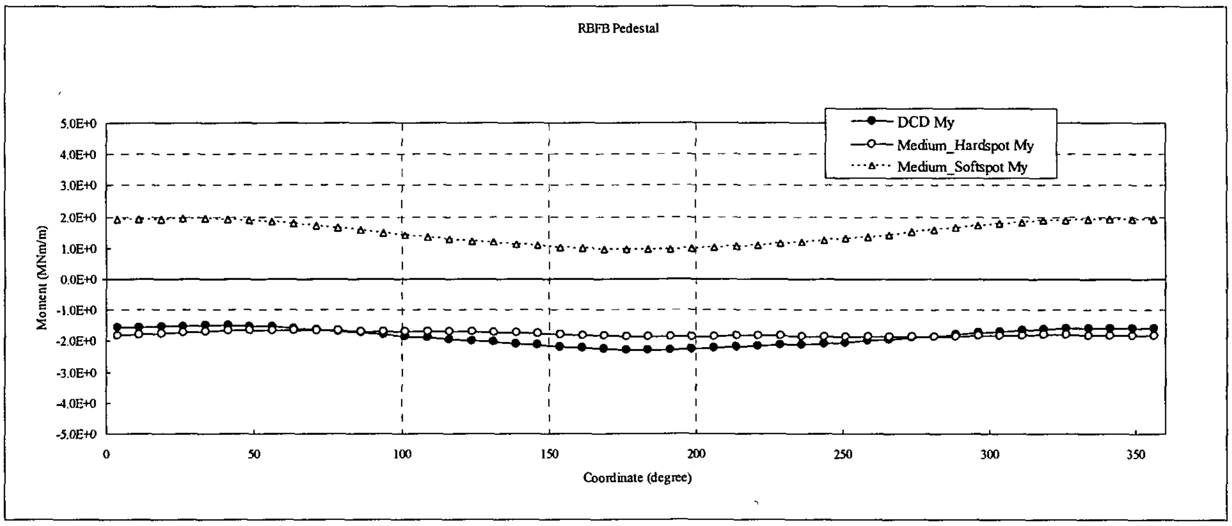


Figure 3.8-93 (17)-b Comparison of Sectional Moments (Pedestal, Medium)

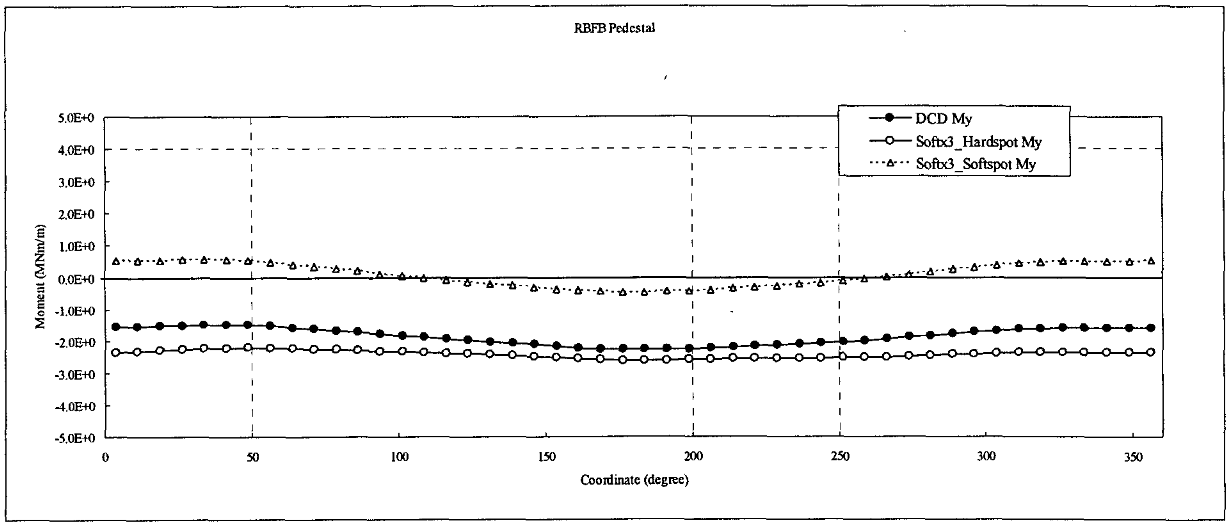


Figure 3.8-93 (17)-c Comparison of Sectional Moments (Pedestal, Soft x3)

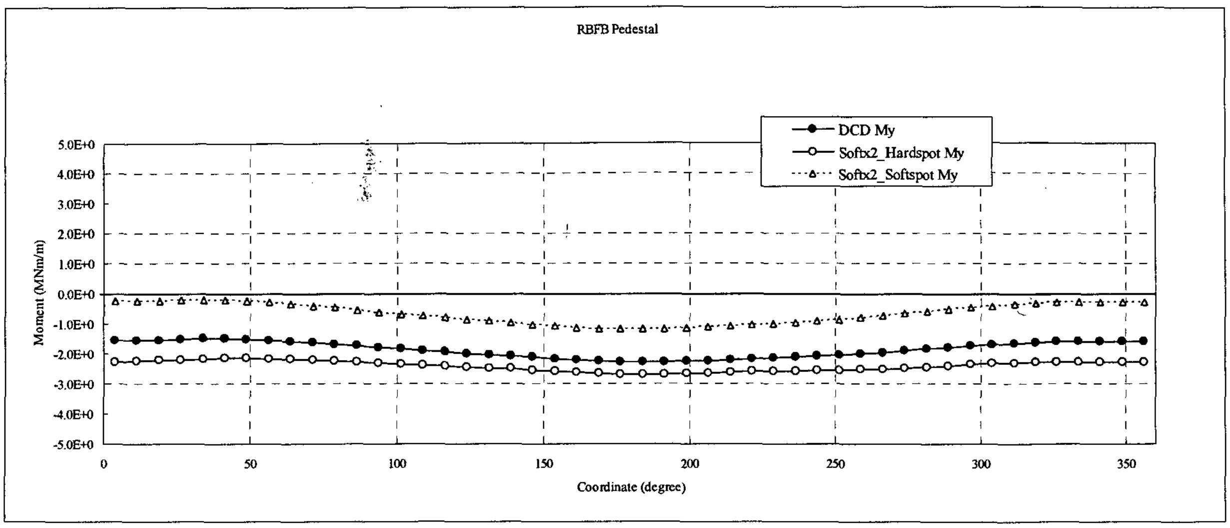


Figure 3.8-93 (17)-d Comparison of Sectional Moments (Pedestal, Soft x2)

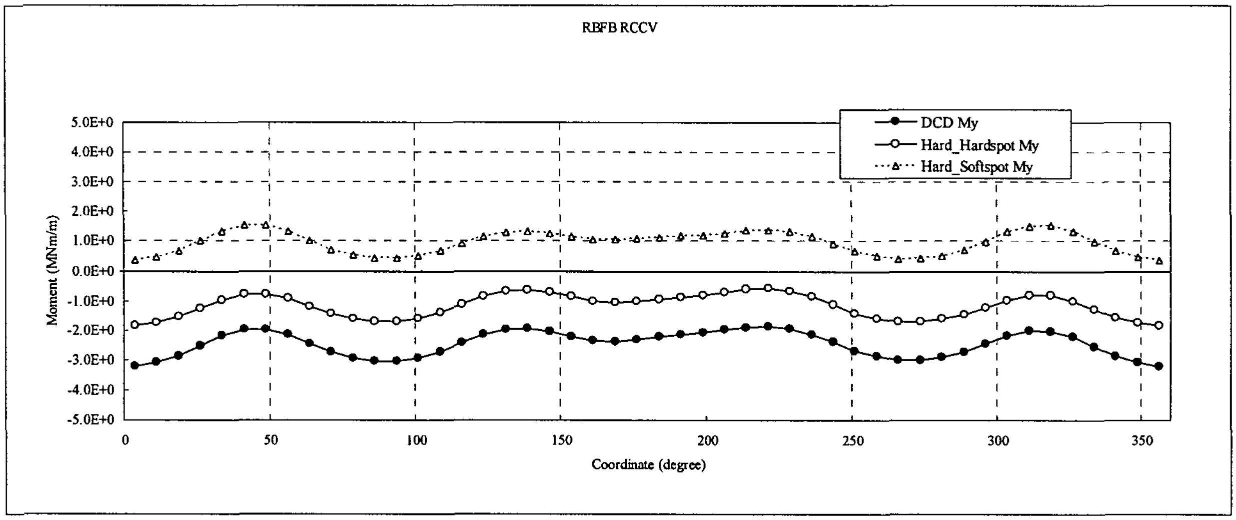


Figure 3.8-93 (18)-a Comparison of Sectional Moments (RCCV bottom, Hard)

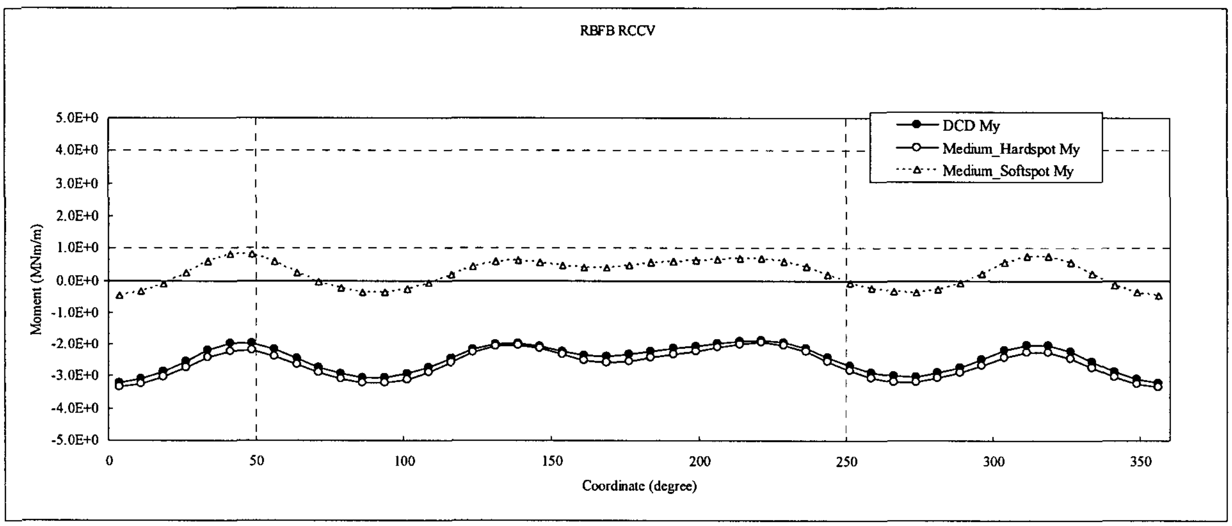


Figure 3.8-93 (18)-b Comparison of Sectional Moments (RCCV bottom, Medium)

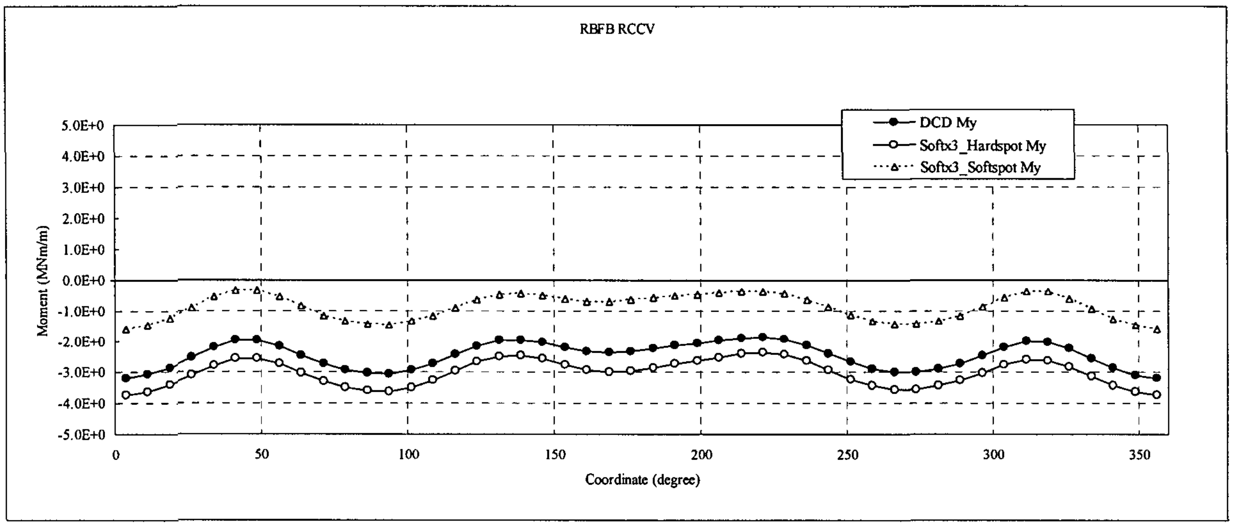


Figure 3.8-93 (18)-c Comparison of Sectional Moments (RCCV bottom, Soft x3)

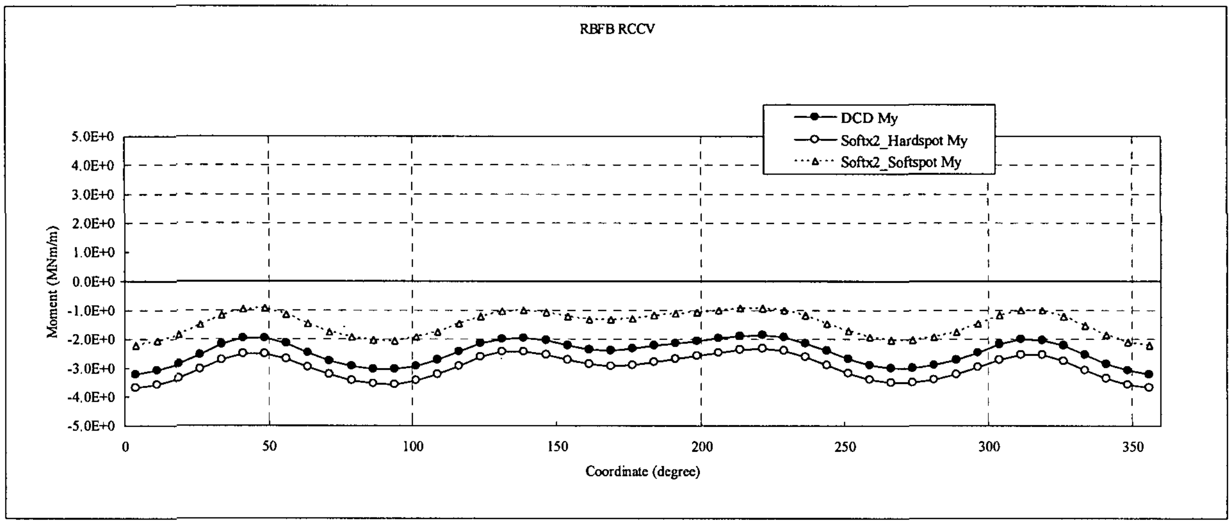


Figure 3.8-93 (18)-d Comparison of Sectional Moments (RCCV bottom, Soft x2)

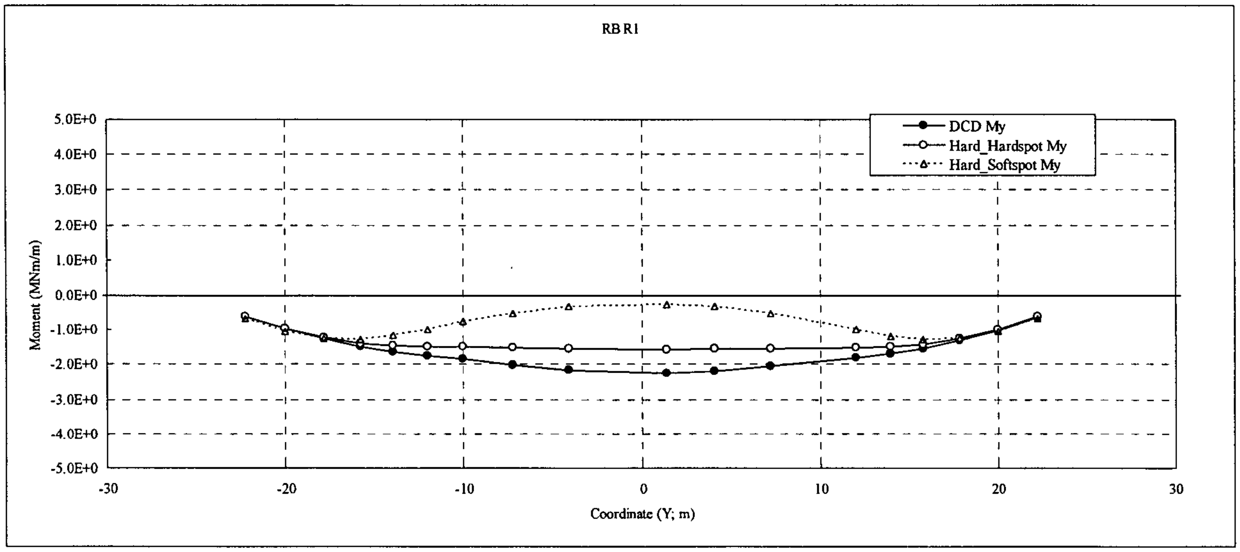


Figure 3.8-93 (19)-a Comparison of Sectional Moments (R1, Hard)

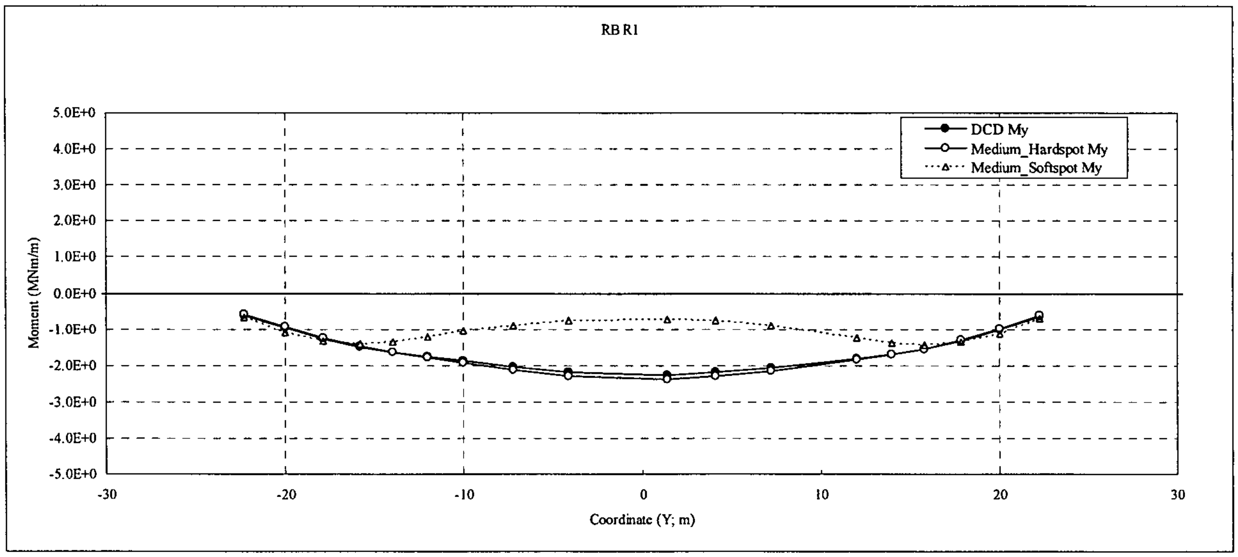


Figure 3.8-93 (19)-b Comparison of Sectional Moments (R1, Medium)

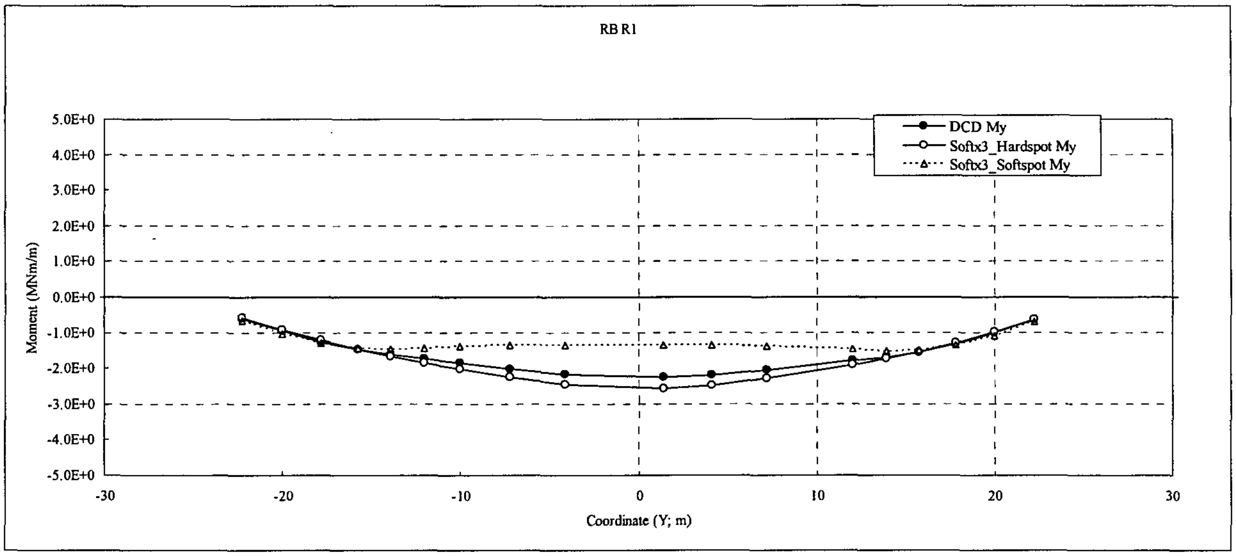


Figure 3.8-93 (19)-c Comparison of Sectional Moments (R1, Soft x3)

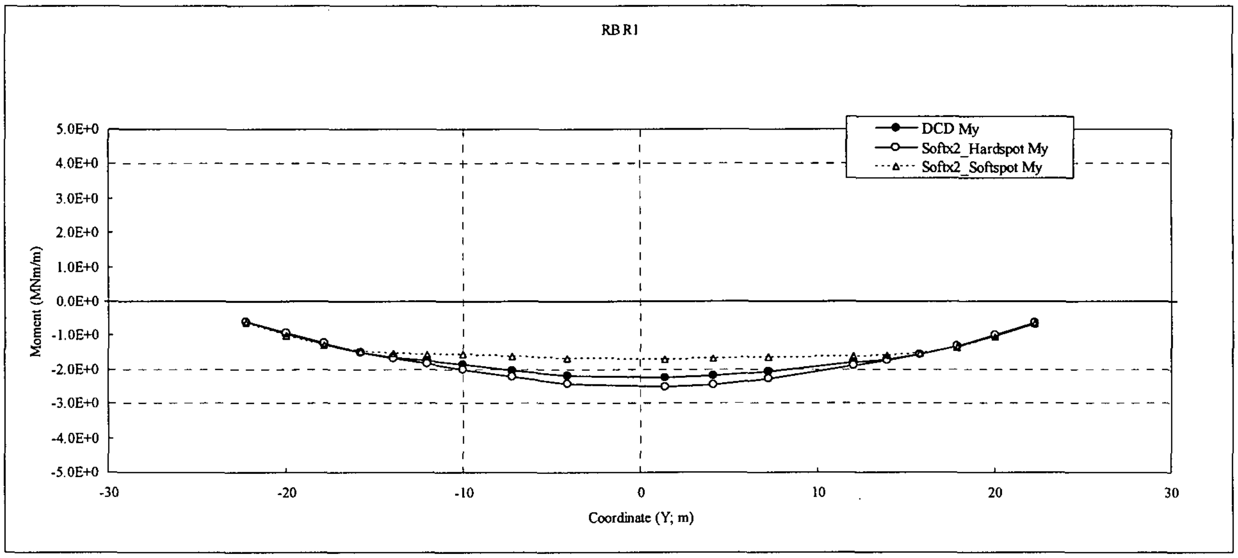


Figure 3.8-93 (19)-d Comparison of Sectional Moments (R1, Soft x2)

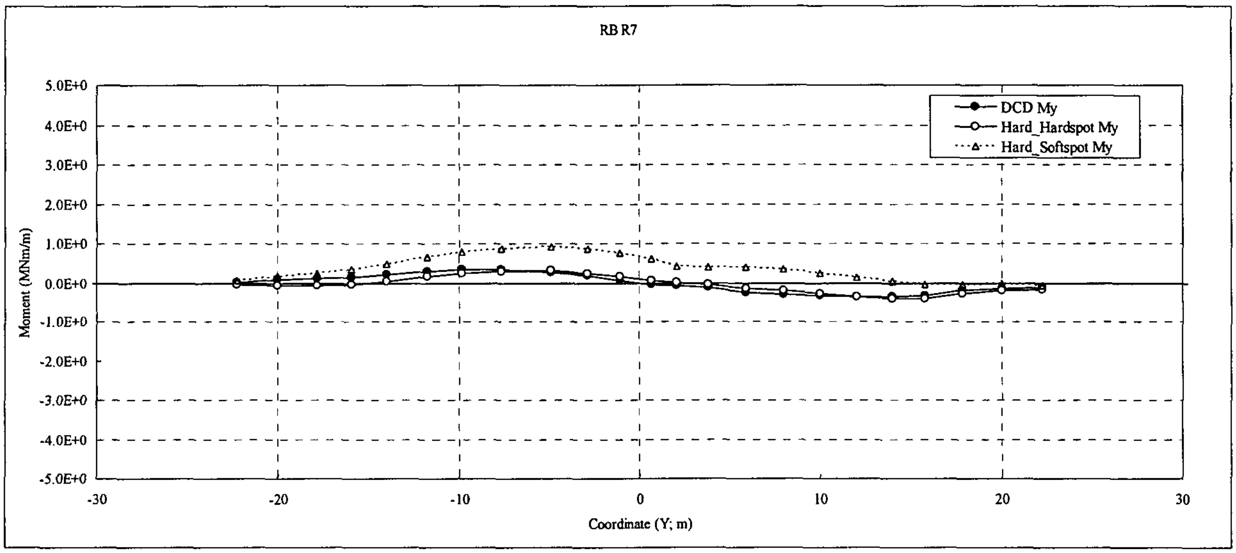


Figure 3.8-93 (20)-a Comparison of Sectional Moments (R7, Hard)

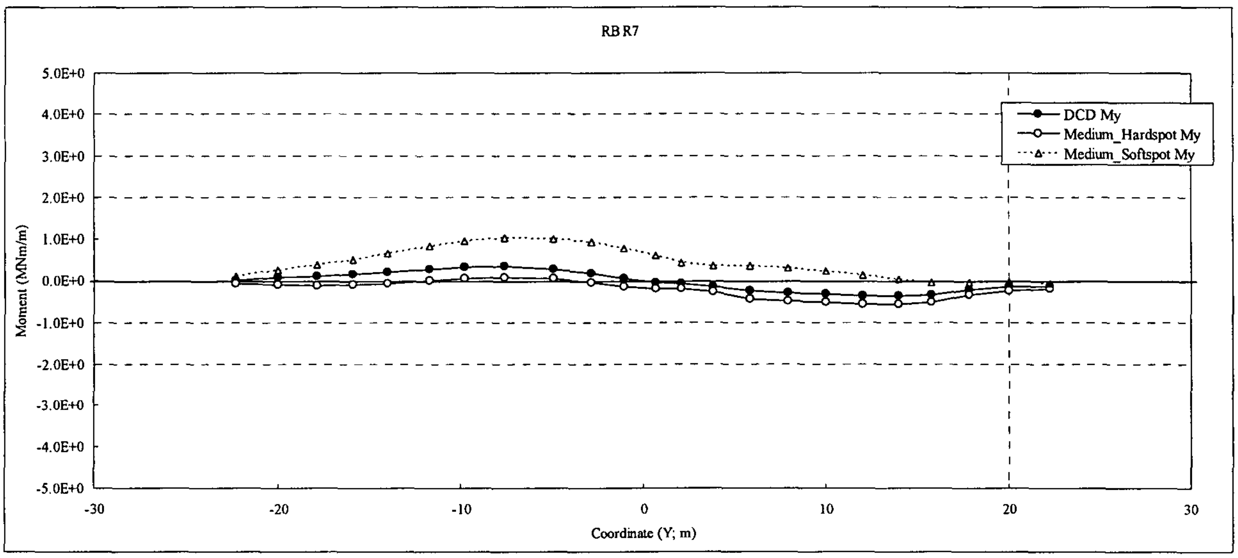


Figure 3.8-93 (20)-b Comparison of Sectional Moments (R7, Medium)

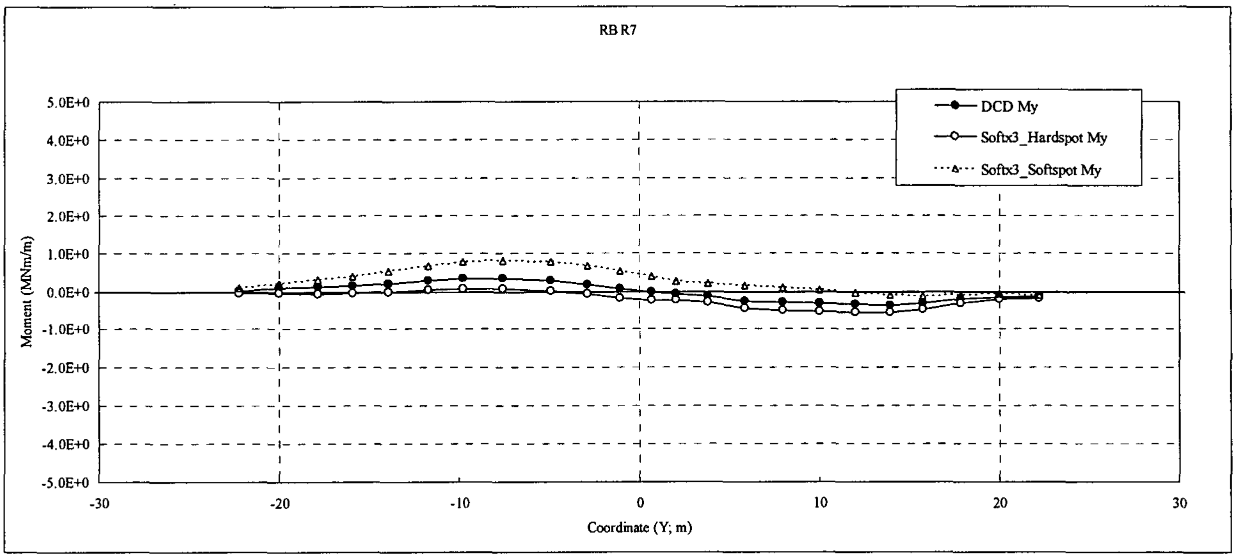


Figure 3.8-93 (20)-c Comparison of Sectional Moments (R7, Soft x3)

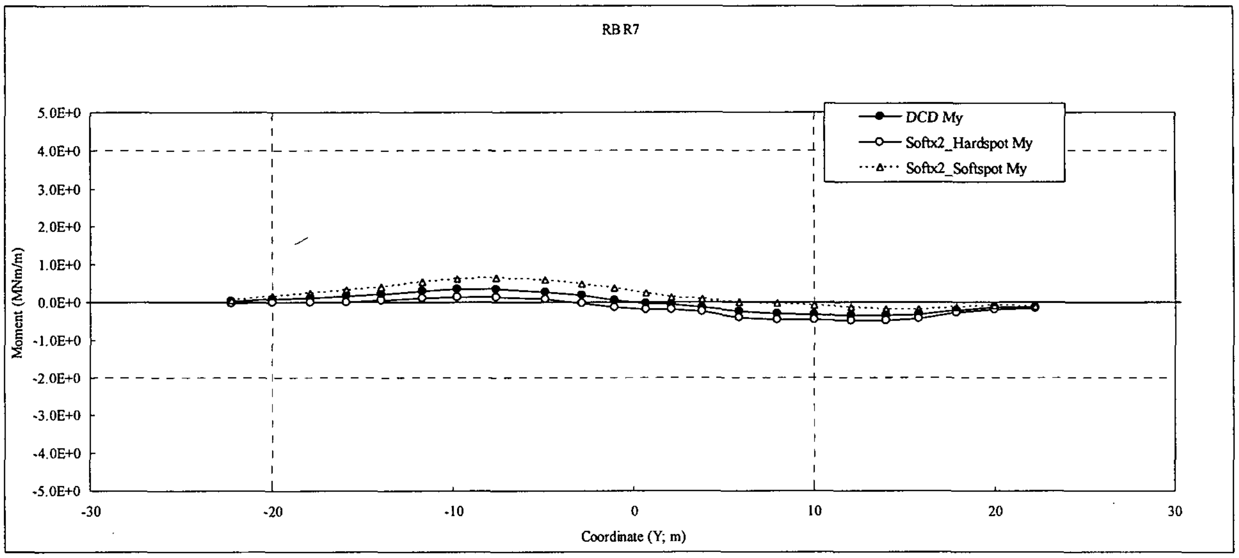


Figure 3.8-93 (20)-d Comparison of Sectional Moments (R7, Soft x2)

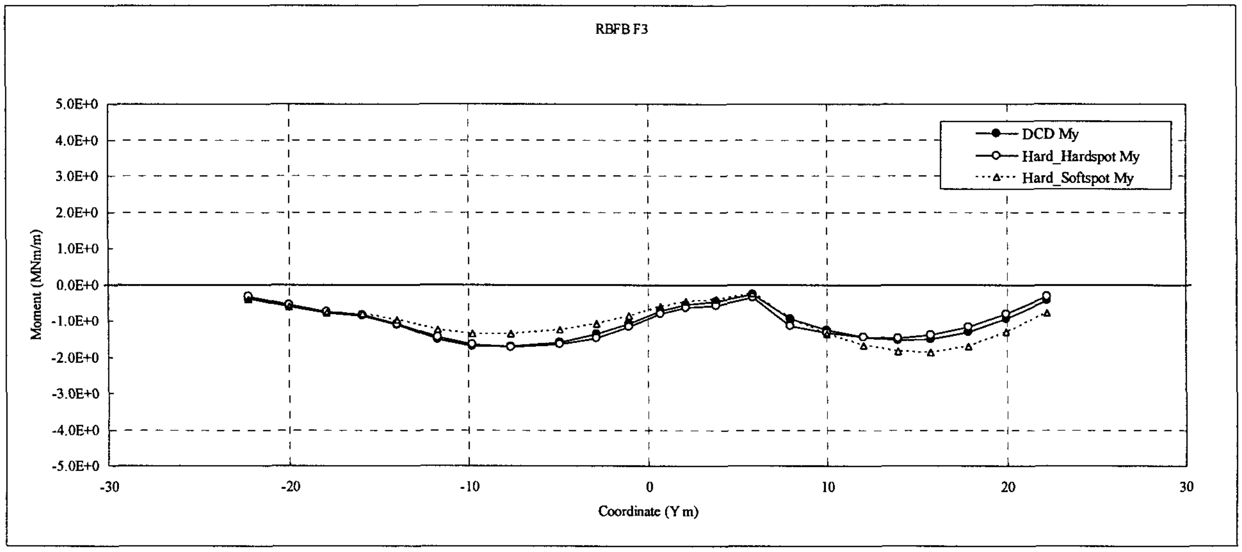


Figure 3.8-93 (21)-a Comparison of Sectional Moments (F3, Hard)

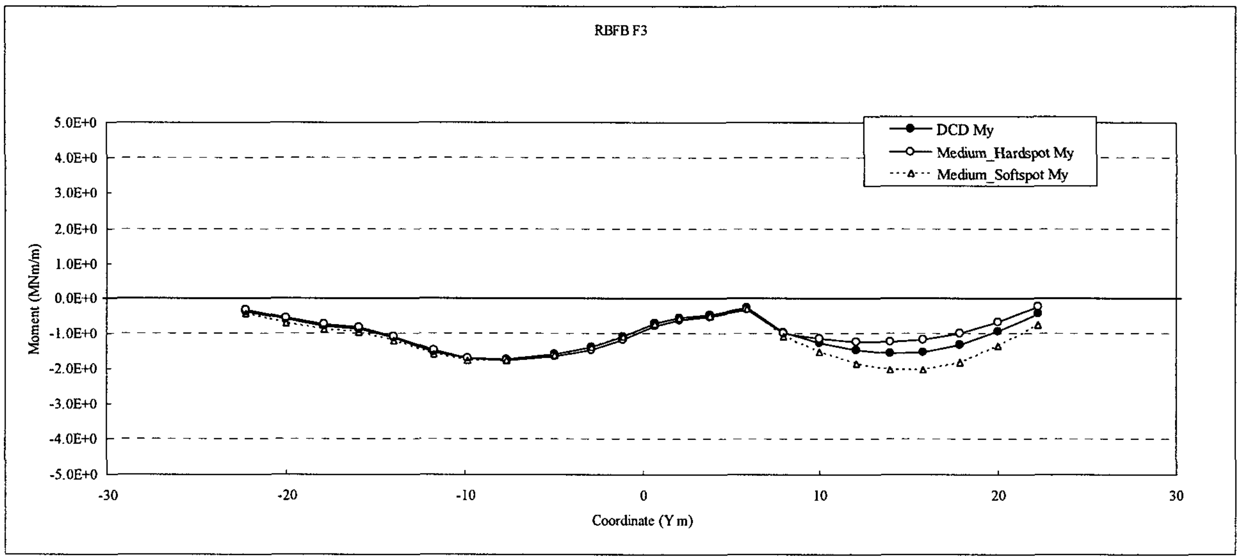


Figure 3.8-93 (21)-b Comparison of Sectional Moments (F3, Medium)

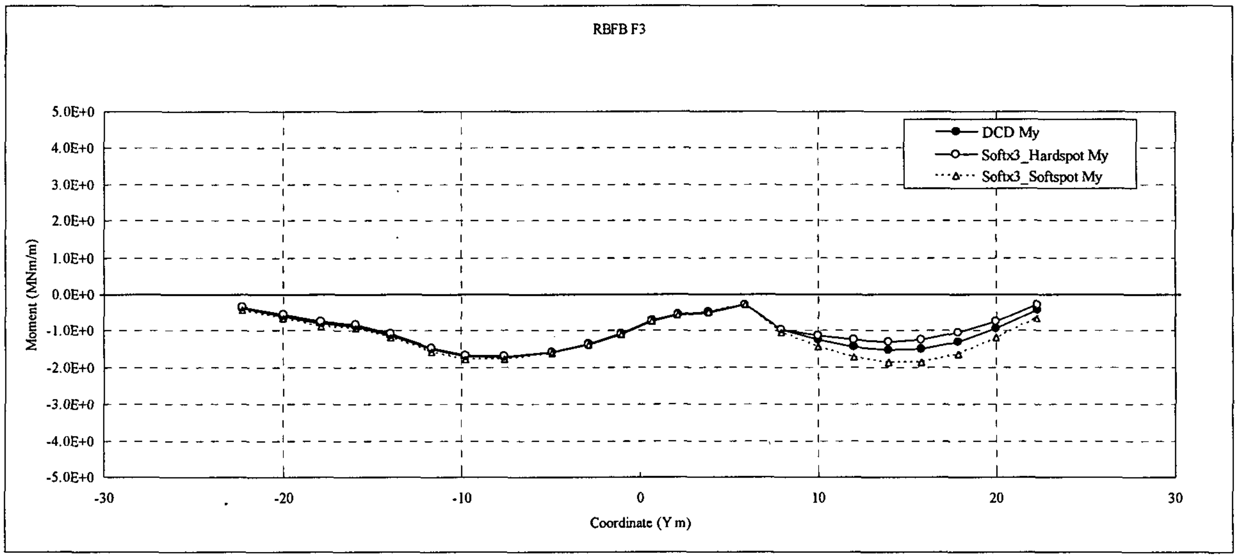


Figure 3.8-93 (21)-c Comparison of Sectional Moments (F3, Soft x3)

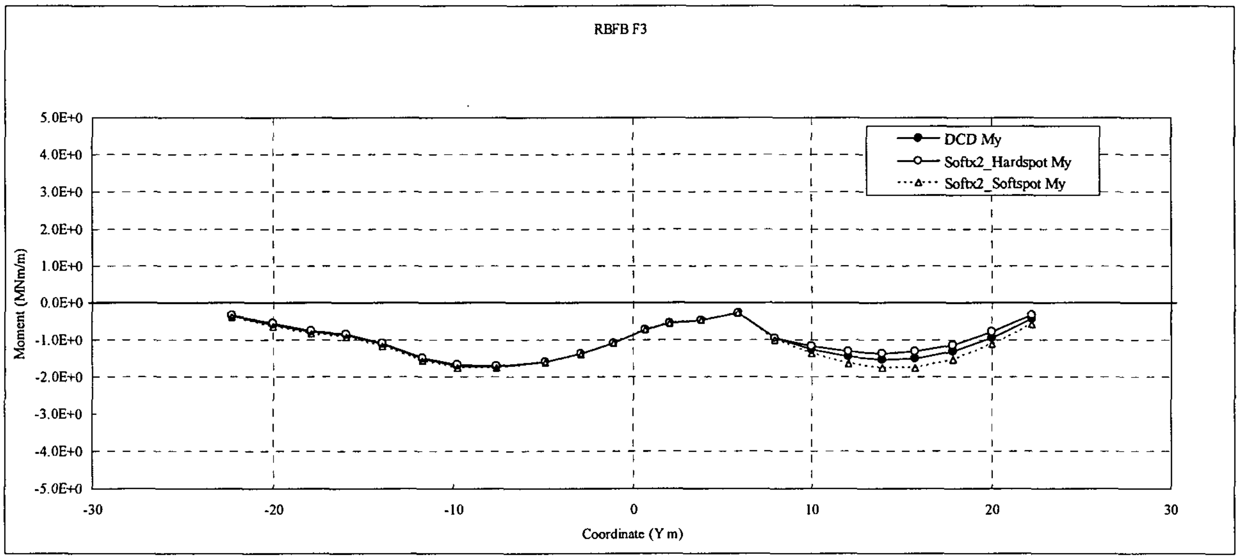


Figure 3.8-93 (21)-d Comparison of Sectional Moments (F3, Soft x2)

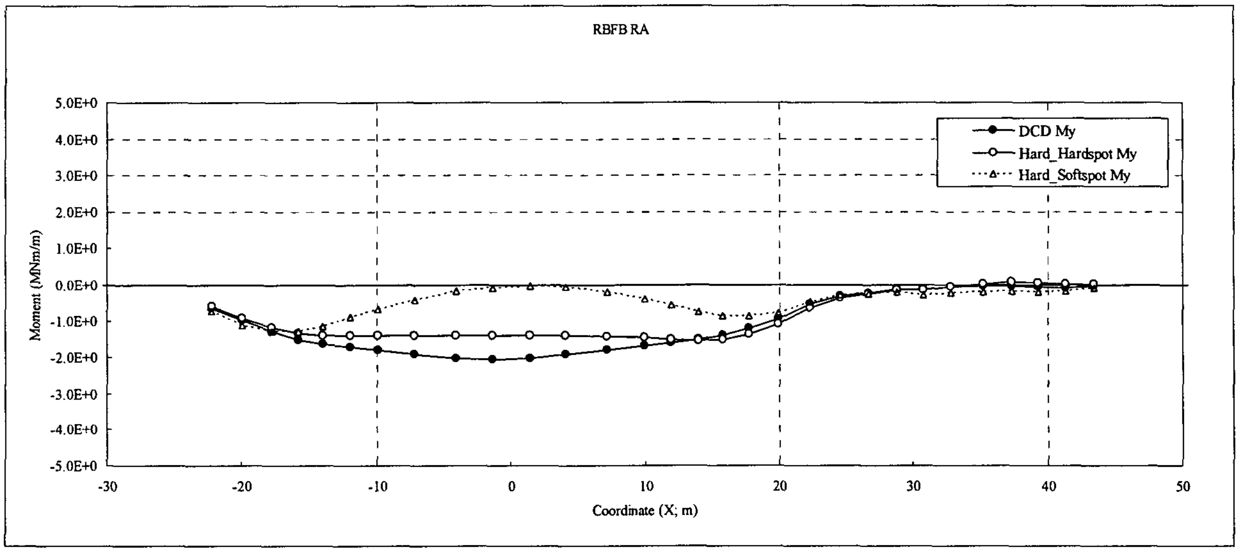


Figure 3.8-93 (22)-a Comparison of Sectional Moments (RA, Hard)

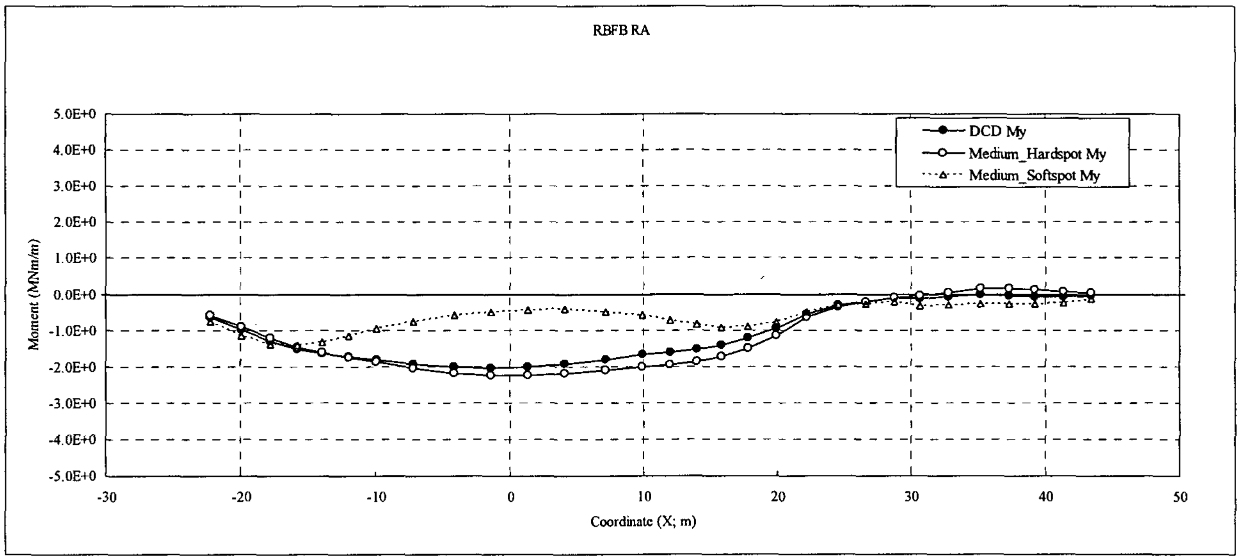


Figure 3.8-93 (22)-b Comparison of Sectional Moments (RA, Medium)

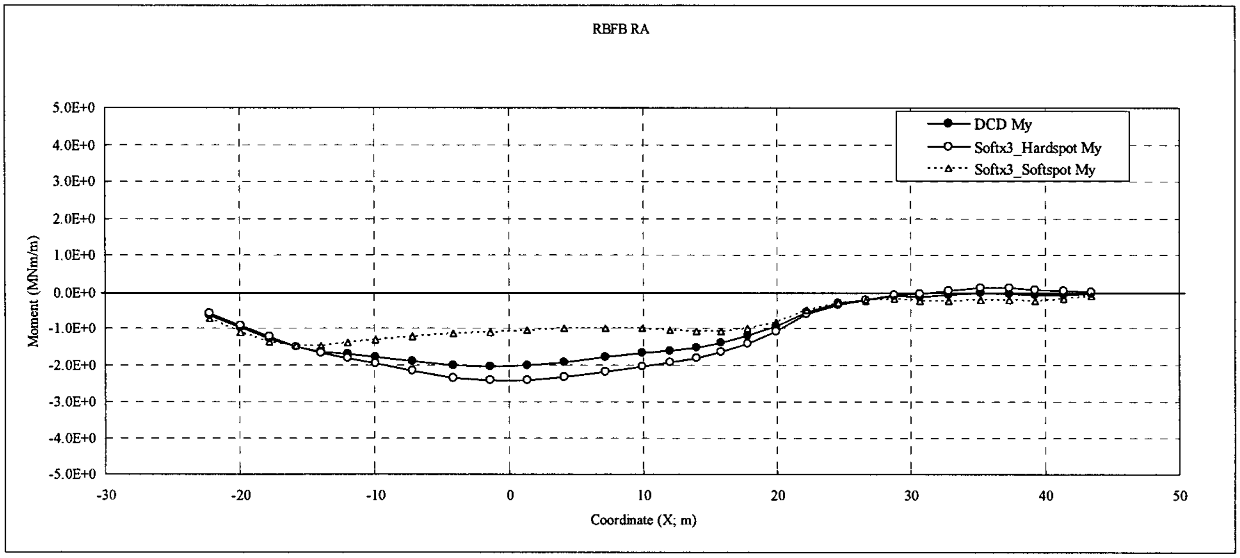


Figure 3.8-93 (22)-c Comparison of Sectional Moments (RA, Soft x3)

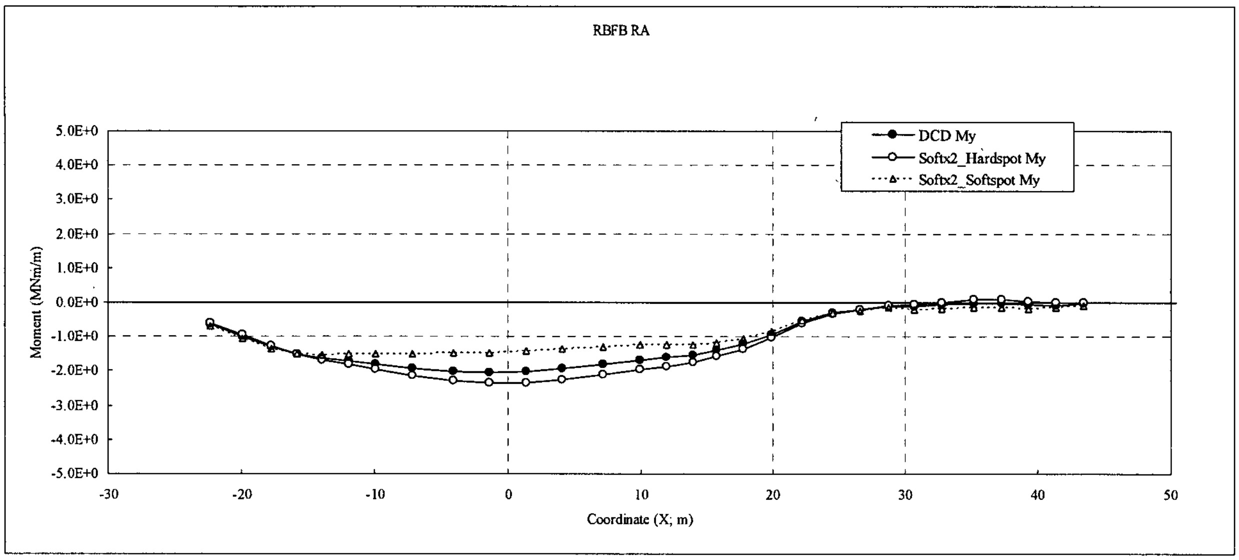


Figure 3.8-93 (22)-d Comparison of Sectional Moments (RA, Soft x2)

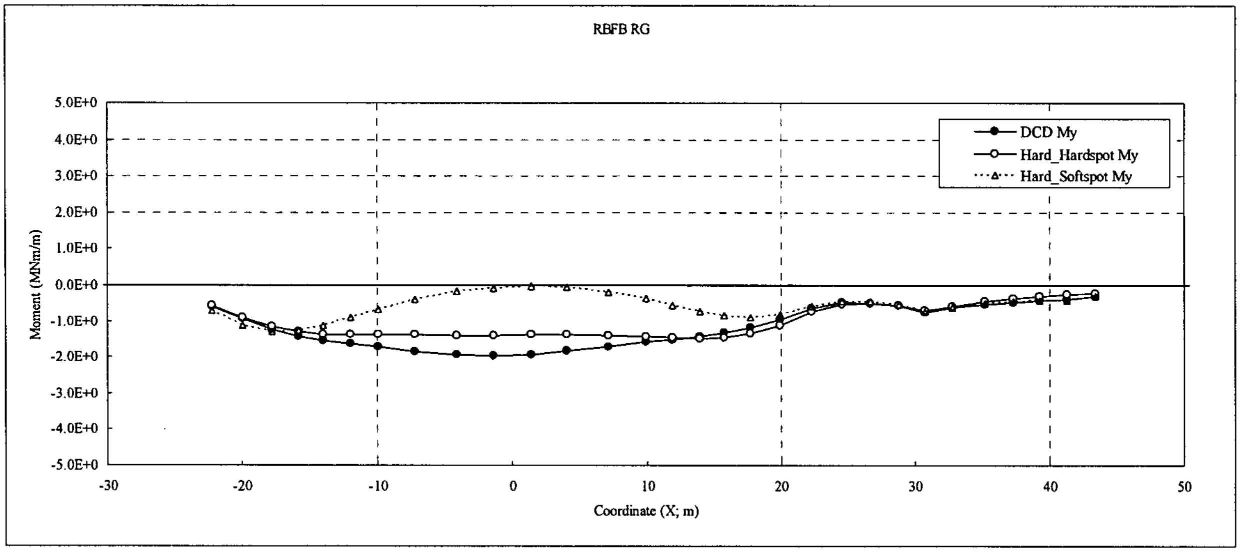


Figure 3.8-93 (23)-a Comparison of Sectional Moments (RG, Hard)

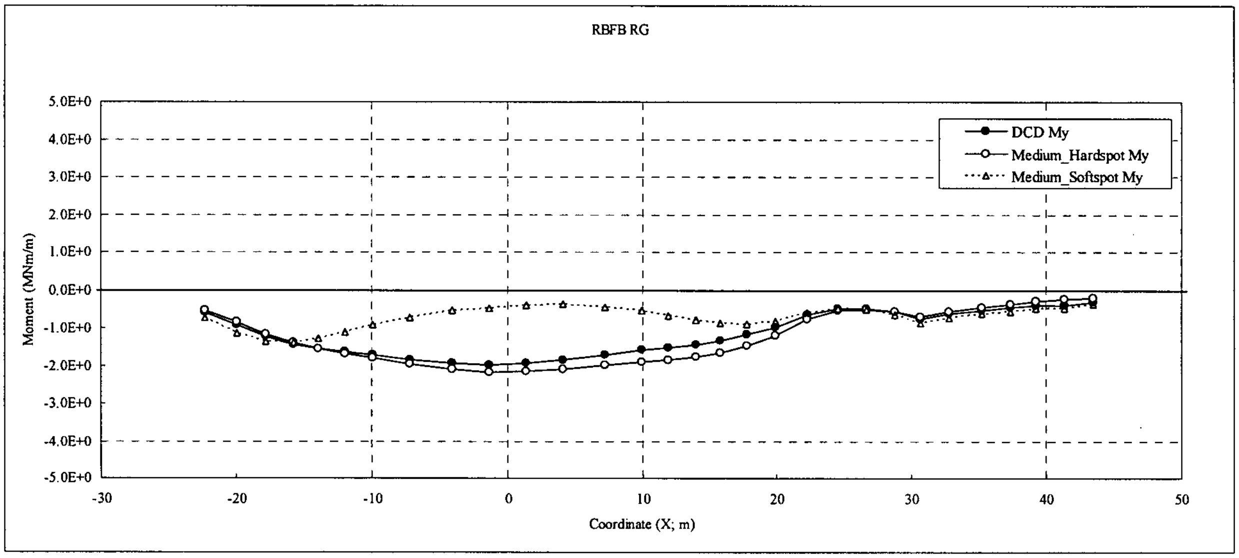


Figure 3.8-93 (23)-b Comparison of Sectional Moments (RG, Medium)

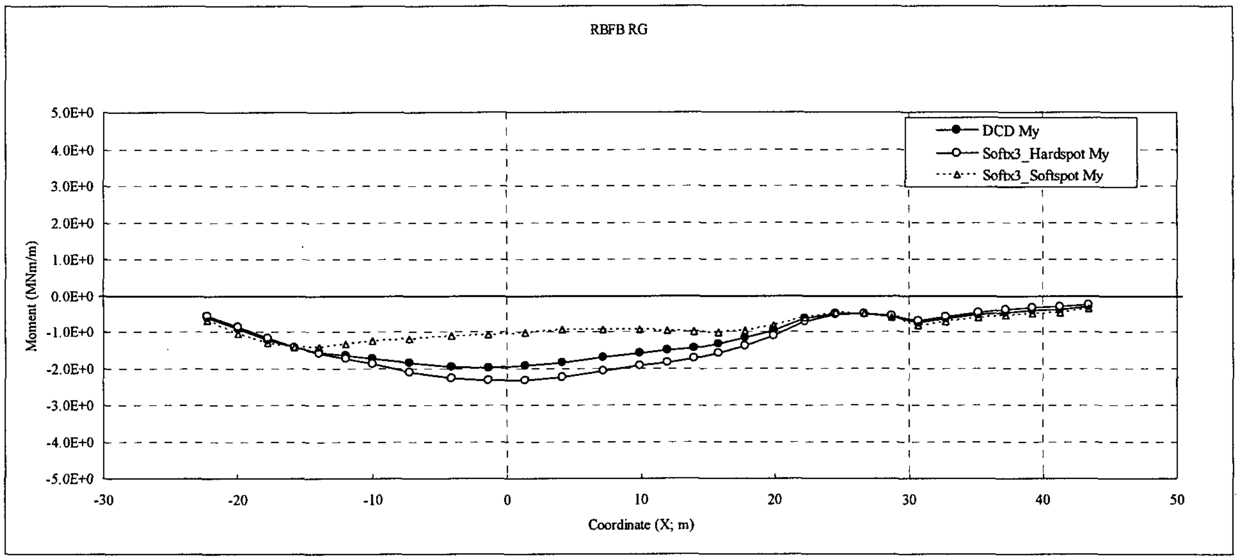


Figure 3.8-93 (23)-c Comparison of Sectional Moments (RG, Soft x3)

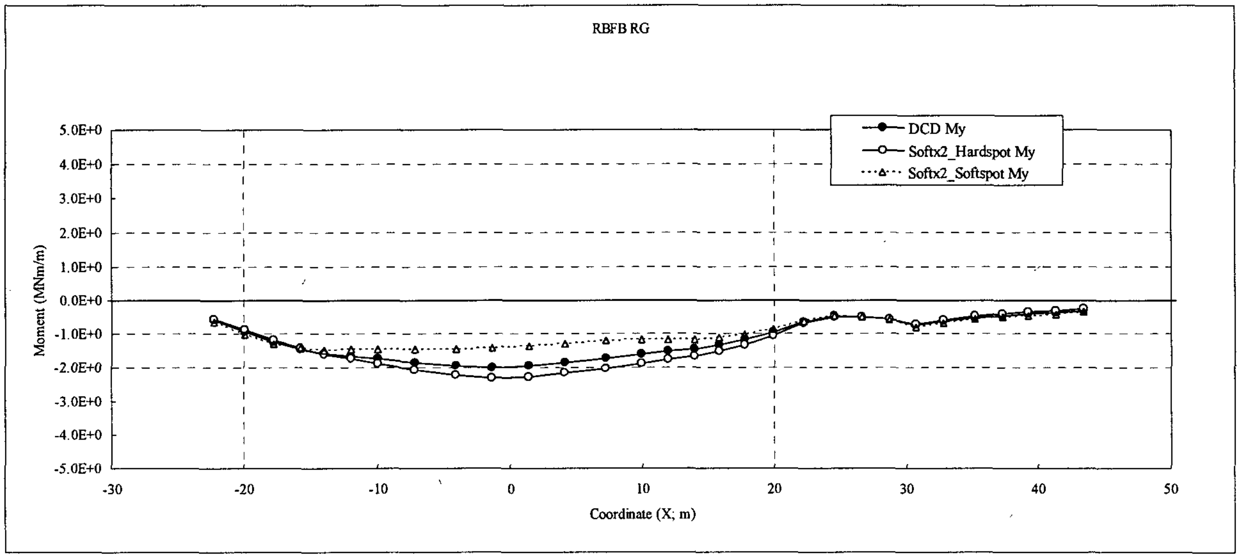


Figure 3.8-93 (23)-d Comparison of Sectional Moments (RG, Soft x2)

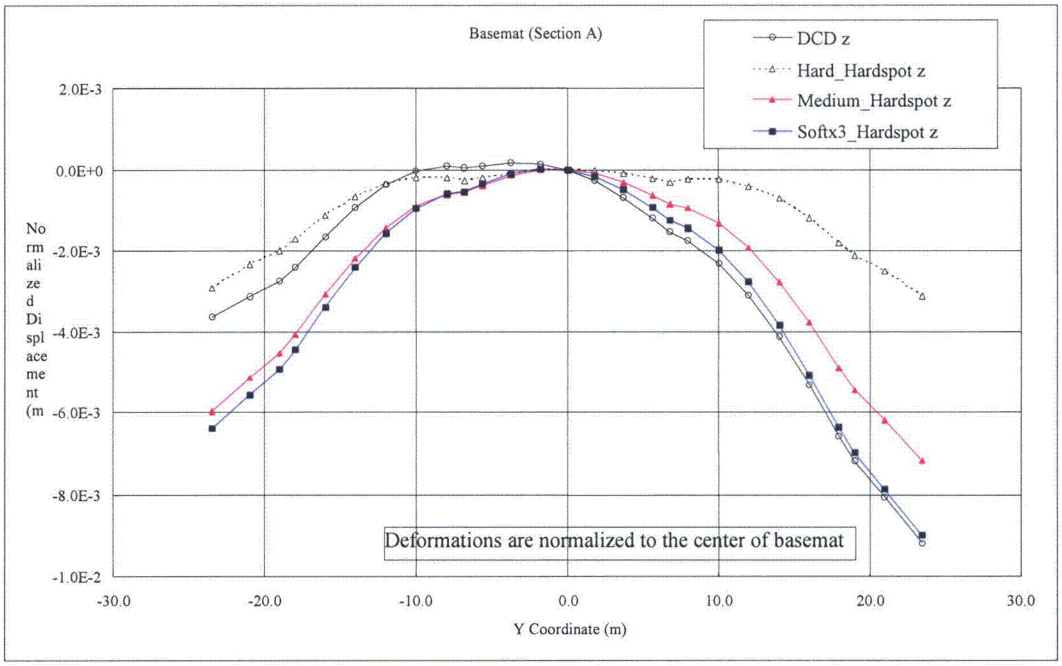


Figure 3.8-93 (24)-a Comparison of Normalized Basemat Displacement (E-W Section)

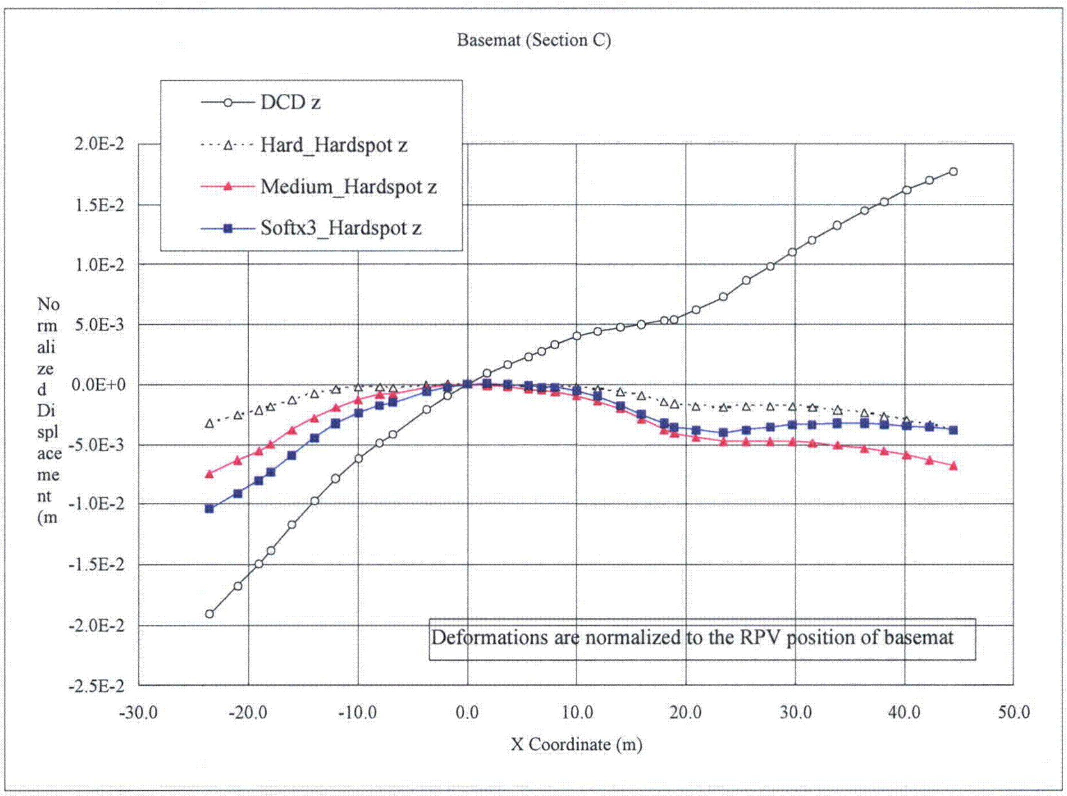


Figure 3.8-93 (24)-b Comparison of Normalized Basemat Displacement (N-S Section)

Evaluation for NRC RAI 3.8-93 (5)

Effect of Basemat Construction Sequence

1. Scope

This study is performed to evaluate the effect of the basemat concrete pour sequence on the basemat stress.

Moments at different locations in the basemat during construction are calculated by FE analyses with variations of construction sequences and soil conditions.

2. Analysis Method

2.1 Construction Sequence

Based on the construction techniques and experience of a recent ABWR plant basemat, the RBFB basemat is divided into 7 concrete pour zones (see Figure 3.8-93(25).)

At the first stage of construction, 0.9 m thick concrete is poured. This thickness is sufficient to cover the basemat bottom layer of rebars.

Then, 3.1 m thick concrete is sequentially poured on the 0.9 m thick concrete. The concrete pour sequences are shown in Table 3.8-93(4). Three cases are considered in this study. Among these cases, Case 3 is selected as an example of concrete poured randomly.

2.2 Soil Condition

The following three conditions are considered:

- Uniform Soft Soil
- Hard Spot: Hard (stiffness is three times the Soft soil stiffness) below the RPV Pedestal region and Soft condition at the edge of basemat
- Soft Spot: Soft condition below the RPV Pedestal region and Hard (stiffness being three times that of the Soft soil) at the edge of basemat

2.3 Analysis Method

Analyses are performed using the basemat FE model, which is extracted from the RBFB global FE model.

Analysis procedure is as follows:

Step 1: The entire basemat is modeled with 0.9 m thick concrete elements

Step 2: Weight of 3.1 m thick concrete and construction live load of 4.8 kN/m^2 (100 psf) are applied to the elements located in the region where concrete is poured as the first stage.

Step 3: The 0.9m thick elements that were poured in Step 1are increased to 4.0 m. Then, 3.1 m thick concrete and live load are applied to the elements located in the other 0.9m region. This is the second stage.

Step 3 is repeated in similar manner until the basemat concrete pour is completed.

3. Analysis Results

Figures 3.8-93(26) through (29) compare the moments at the final stage of concrete pour and at three sections shown in Figure 3.8-93(25). The design bending moment envelope due to the dead load obtained by the RBFB global FE model analysis are also included in the figures for comparison.

As shown in the figures, moment differences between the sequences considered are negligibly small in comparison with the design moments due to dead load during Normal Operation considered in the basemat design. In addition, the effects of varying soil conditions are not controlling.

Figures 3.8-93(30) through (32) show distributions of the principal tensile stresses generated in the 0.9 m thick concrete slab. The maximum principal tensile stress is summarized in Table 3.8-93(5) for each soil condition and construction sequence. The tensile stress generated by the concrete pour is less than the tensile strength of concrete, which is $0.1f_c' = 2.76\,\text{MPa}$.

Therefore, it can be concluded that:

- The basemat construction sequence has no effect on the basemat design, and
- Significant cracking will not be generated in the basemat during construction.

Table 3.8-93(4) Assumed Sequences of Basemat Concrete Pour

Case	Stage	Sequence of Concrete Pour*							Element Thickness (m)						
		Zone							Zone						
		1	2	3	4	5	6	7	1	2	3	4	5	6	7
1	1	P							0.9	0.9	0.9	0.9	0.9	0.9	0.9
	2	H	P						4.0	0.9	0.9	0.9	0.9	0.9	0.9
	3	H	H	P					4.0	4.0	0.9	0.9	0.9	0.9	0.9
	4	H	H	H	P				4.0	4.0	4.0	0.9	0.9	0.9	0.9
	5	H	H	H	H	P			4.0	4.0	4.0	4.0	0.9	0.9	0.9
	6	H	H	H	H	H	P		4.0	4.0	4.0	4.0	4.0	0.9	0.9
	7	H	H	H	H	H	H	P	4.0	4.0	4.0	4.0	4.0	4.0	0.9
2	1							P	0.9	0.9	0.9	0.9	0.9	0.9	0.9
	2						P	H	0.9	0.9	0.9	0.9	0.9	0.9	4.0
	3					P	H	H	0.9	0.9	0.9	0.9	0.9	4.0	4.0
	4				P	H	H	H	0.9	0.9	0.9	0.9	4.0	4.0	4.0
	5			P	H	H	H	H	0.9	0.9	0.9	4.0	4.0	4.0	4.0
	6		P	H	H	H	H	H	0.9	0.9	4.0	4.0	4.0	4.0	4.0
	7	P	H	H	H	H	H	H	0.9	4.0	4.0	4.0	4.0	4.0	4.0
3	1	P							0.9	0.9	0.9	0.9	0.9	0.9	0.9
	2	H	P						4.0	0.9	0.9	0.9	0.9	0.9	0.9
	3	H	H		P				4.0	4.0	0.9	0.9	0.9	0.9	0.9
	4	H	H		H		P		4.0	4.0	0.9	4.0	0.9	0.9	0.9
	5	H	H		H		H	P	4.0	4.0	0.9	4.0	0.9	4.0	0.9
	6	H	H		H	P	H	H	4.0	4.0	0.9	4.0	0.9	4.0	4.0
	7	H	H	P	H	H	H	H	4.0	4.0	0.9	4.0	4.0	4.0	4.0

Note *: "P" indicates the zone where concrete is being poured in the stage.
"H" indicates the zone where concrete has hardened in the previous stage.
For "Zone", see Figure 3.8-93(25).

Example: Case 1, Stage 3
3.1m thick concrete is being placed into Zone 3. Concrete pour for Zones 1 and 2 has been completed in Stages 1 and 2, and concrete in the zones is considered to be hardened. Therefore, Zone 3 is marked as "P", and Zones 1 and 2 are marked as "H".

Table 3.8-93(5) Maximum Principal Tensile Stress in 0.9 m Thick Slab

Case	Soil Condition (MPa)		
	Soft Soil	Hard Spot	Soft Spot
1	1.96	1.76	1.33
2	1.99	1.50	1.83
3	2.12	1.69	1.39

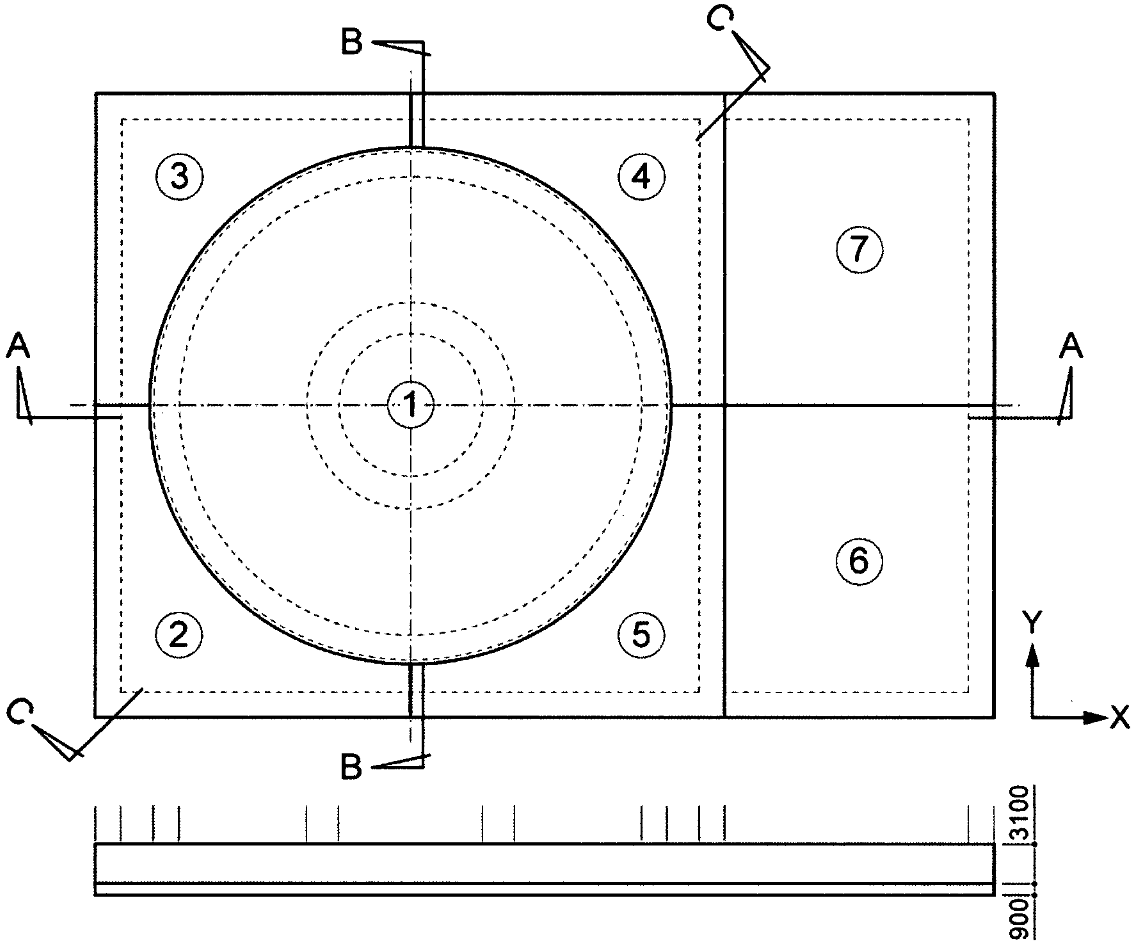
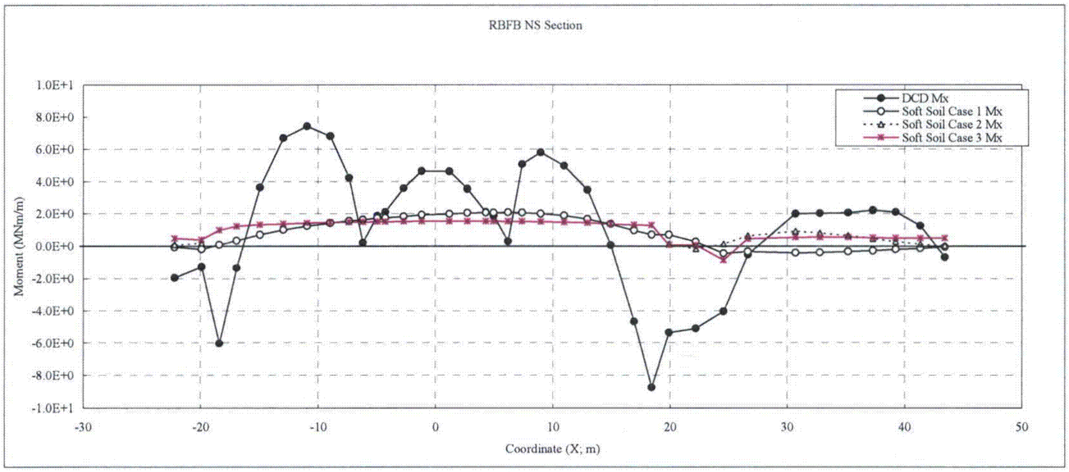
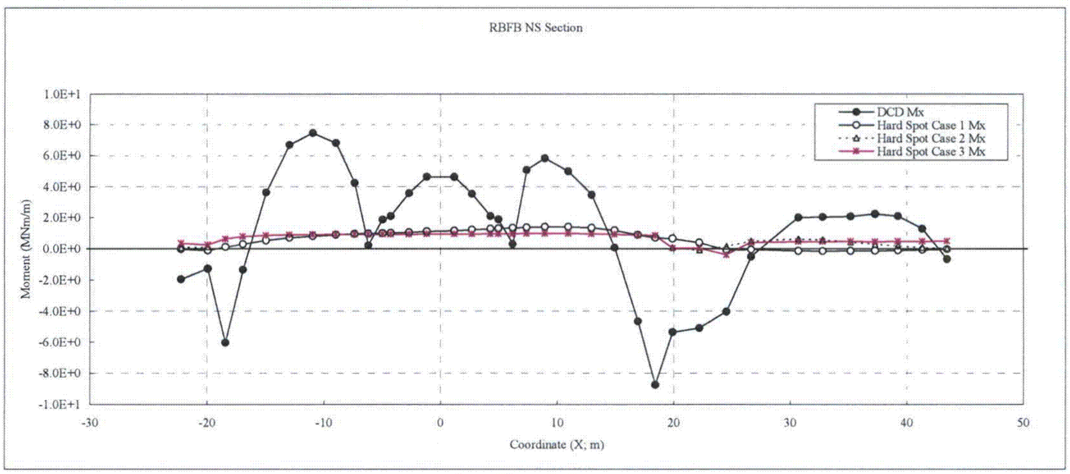


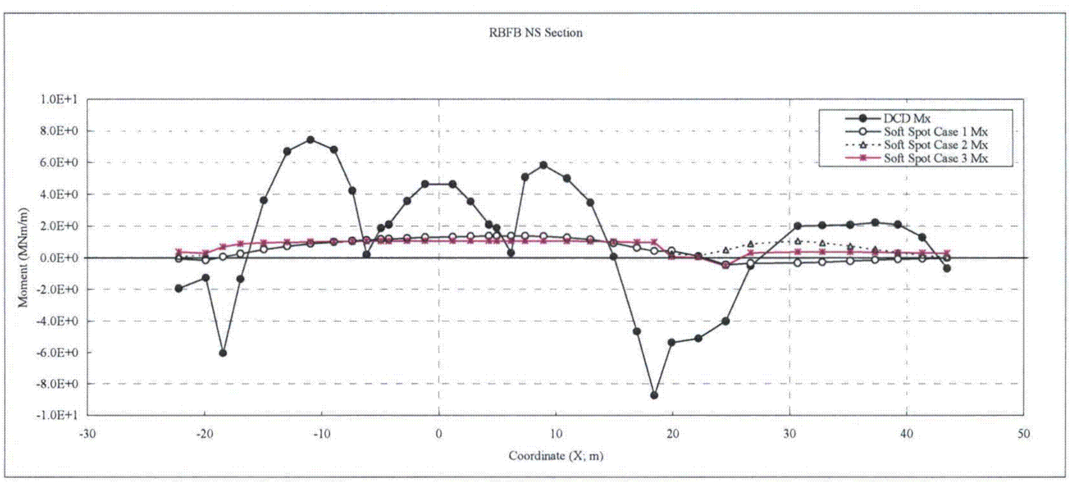
Figure 3.8-93(25) Zone of Basemat for Concrete Pour



(a) Soft Soil

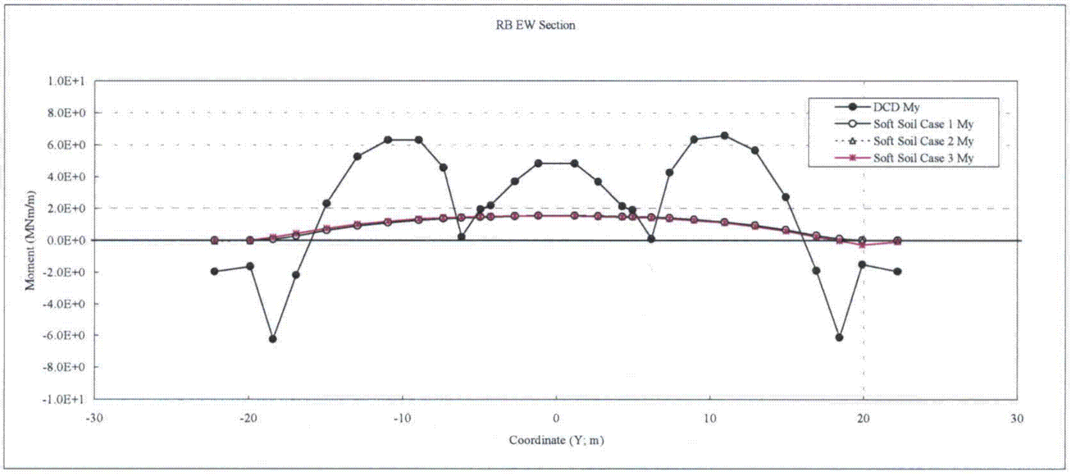


(b) Hard Spot

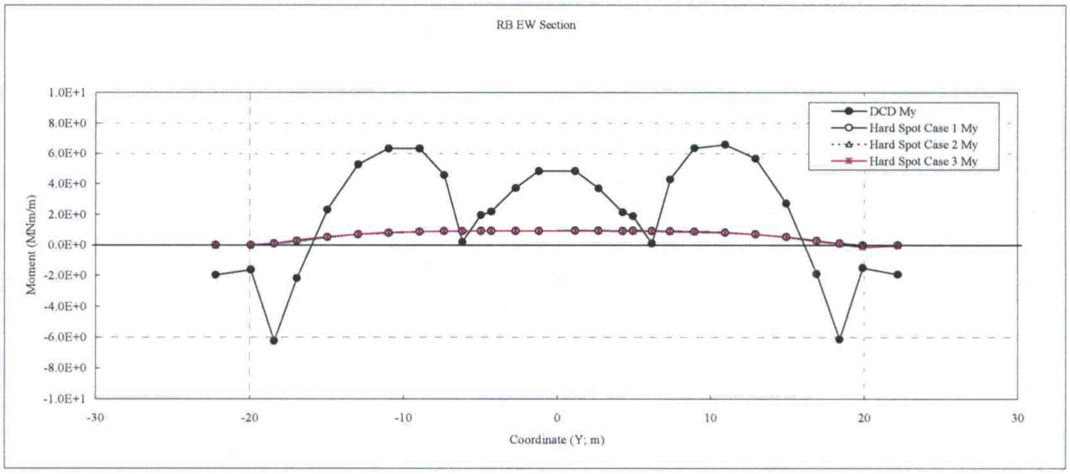


(c) Soft Spot

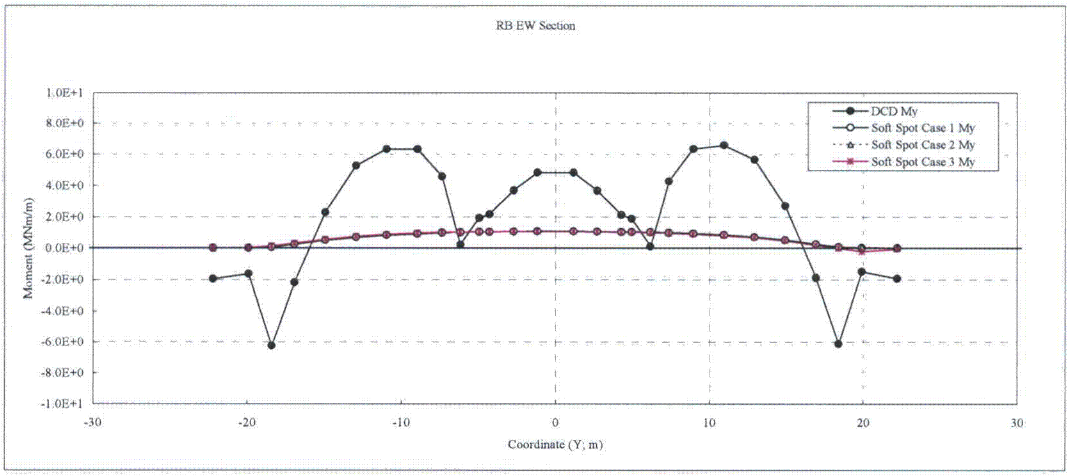
Figure 3.8-93(26) Comparison of Basemat Moment (Section A: Moment in X-dir., Mx)



(a) Soft Soil

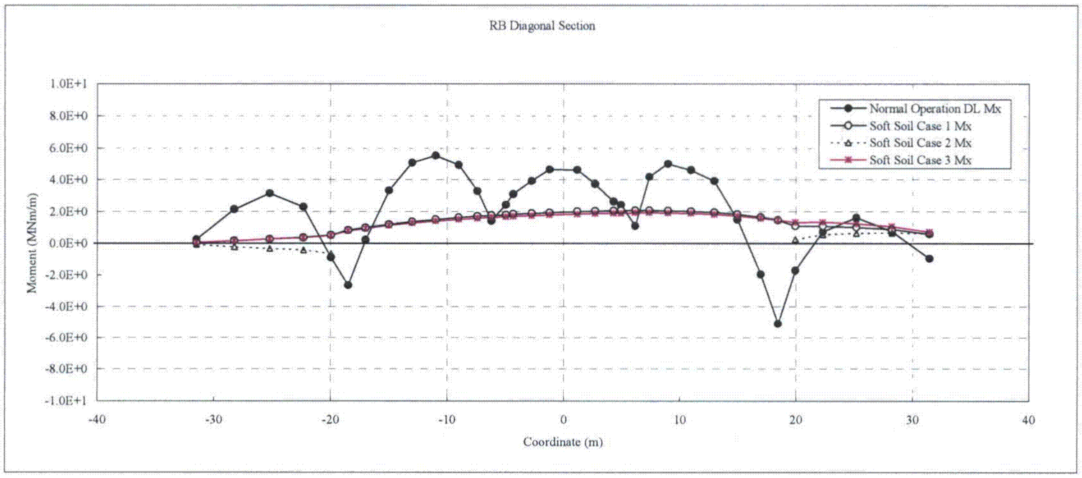


(b) Hard Spot

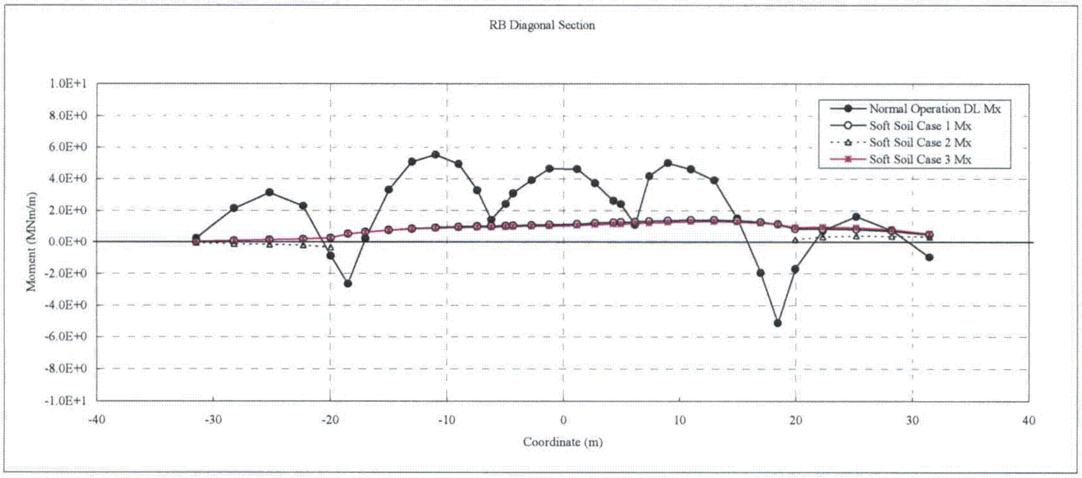


(c) Soft Spot

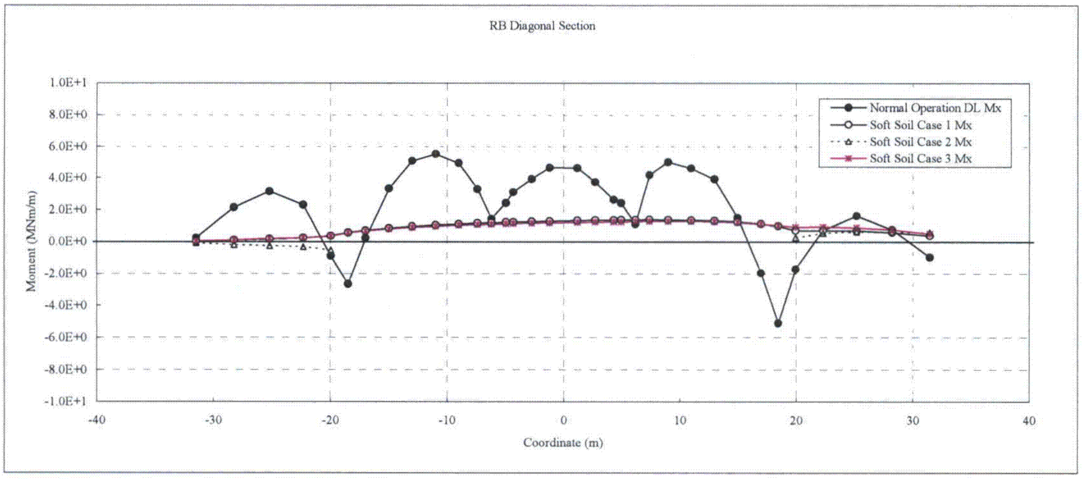
Figure 3.8-93(27) Comparison of Basemat Moment (Section B: Moment in Y-dir., My)



(a) Soft Soil

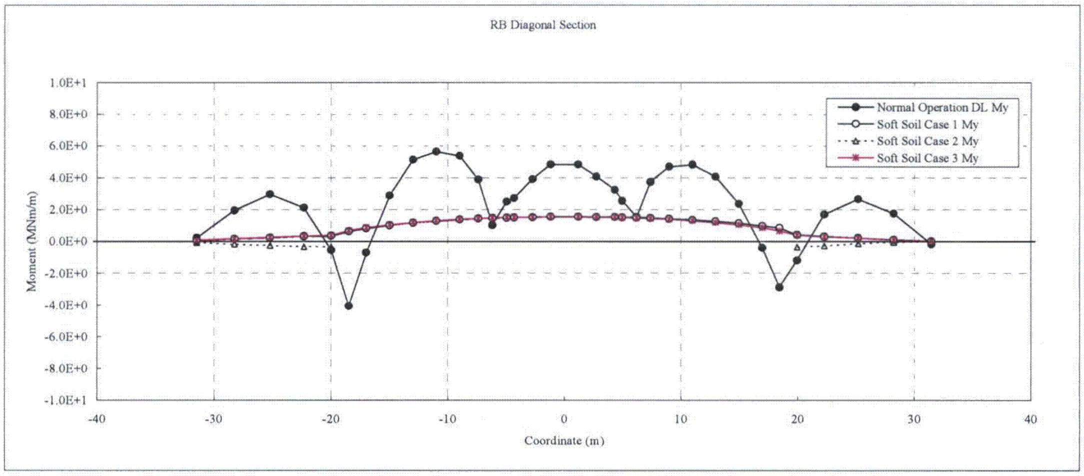


(b) Hard Spot

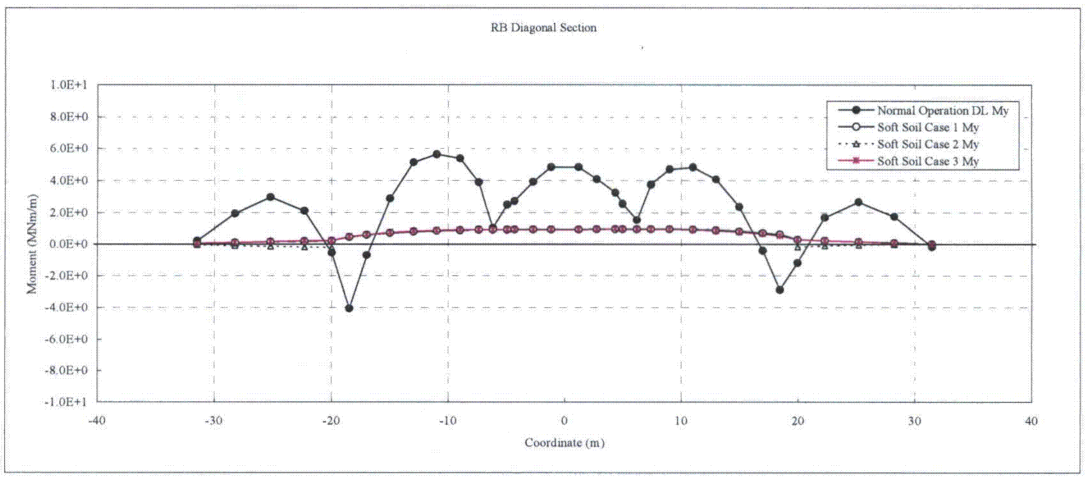


(c) Soft Spot

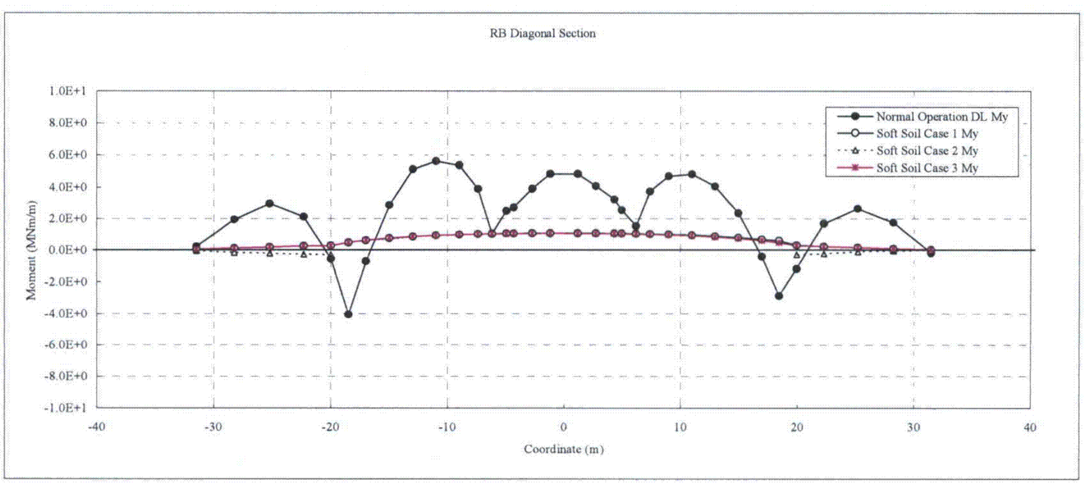
Figure 3.8-93(28) Comparison of Basemat Moment (Section C: Moment in X-dir., Mx)



(a) Soft Soil

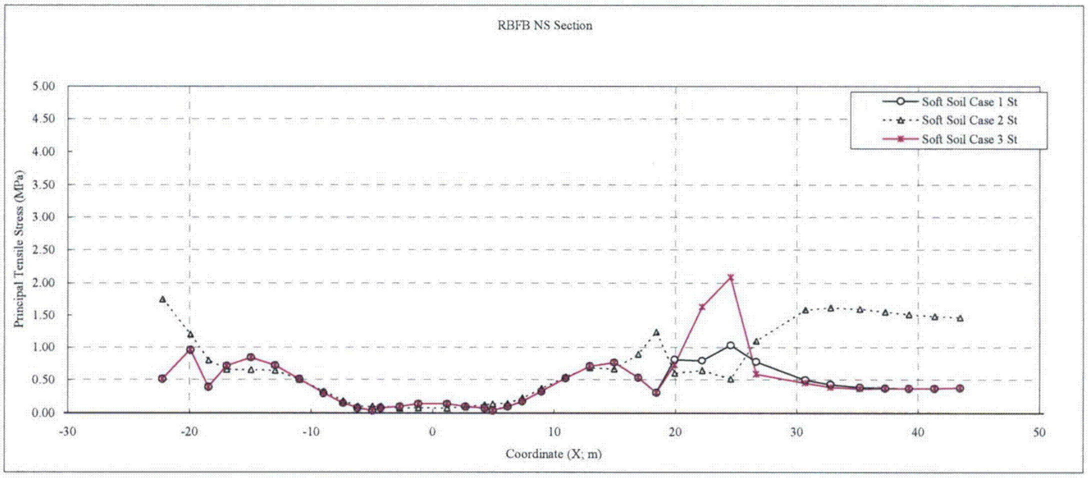


(b) Hard Spot

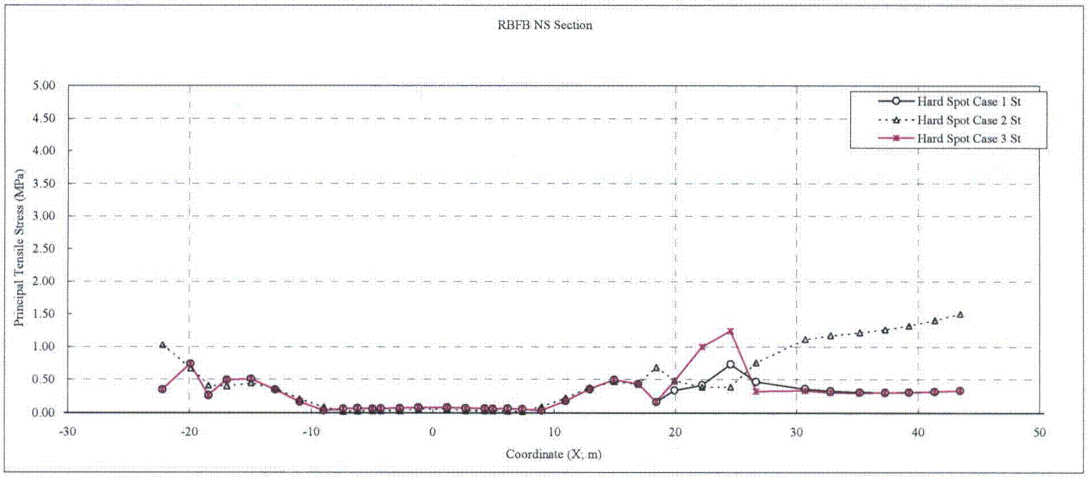


(c) Soft Spot

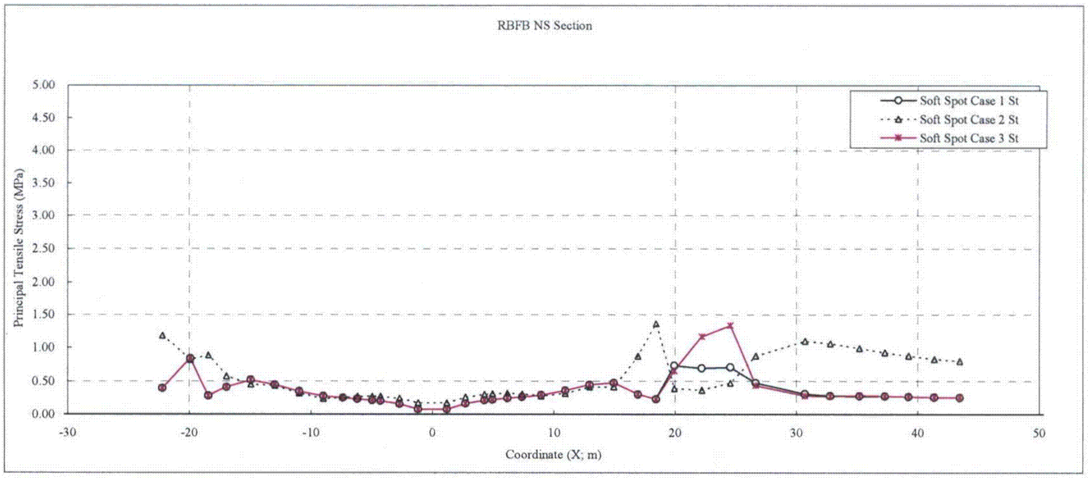
Figure 3.8-93(29) Comparison of Basemat Moment (Section C: Moment in Y-dir., My)



(a) Soft Soil

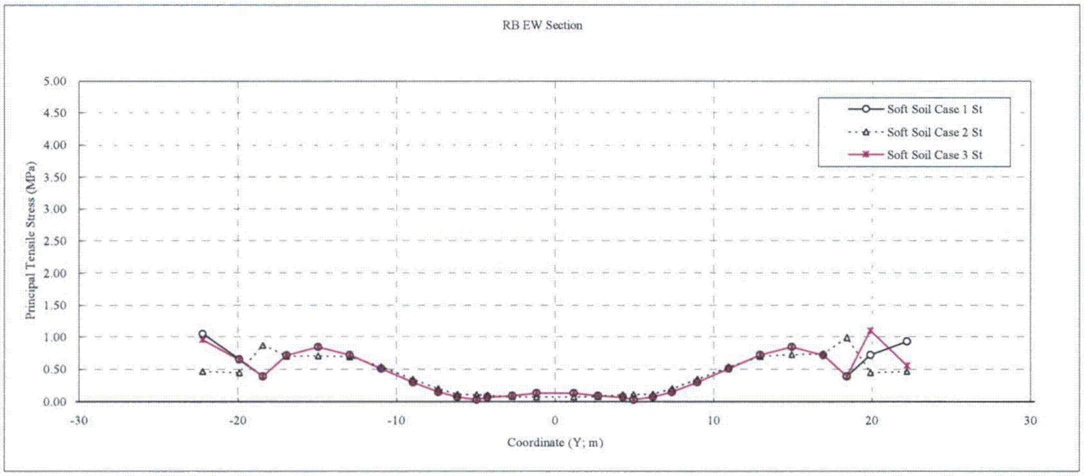


(b) Hard Spot

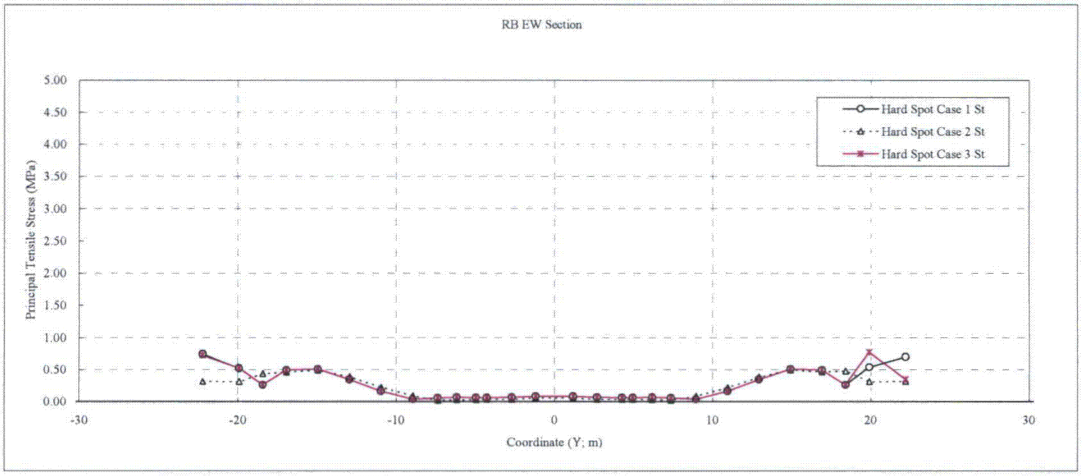


(c) Soft Spot

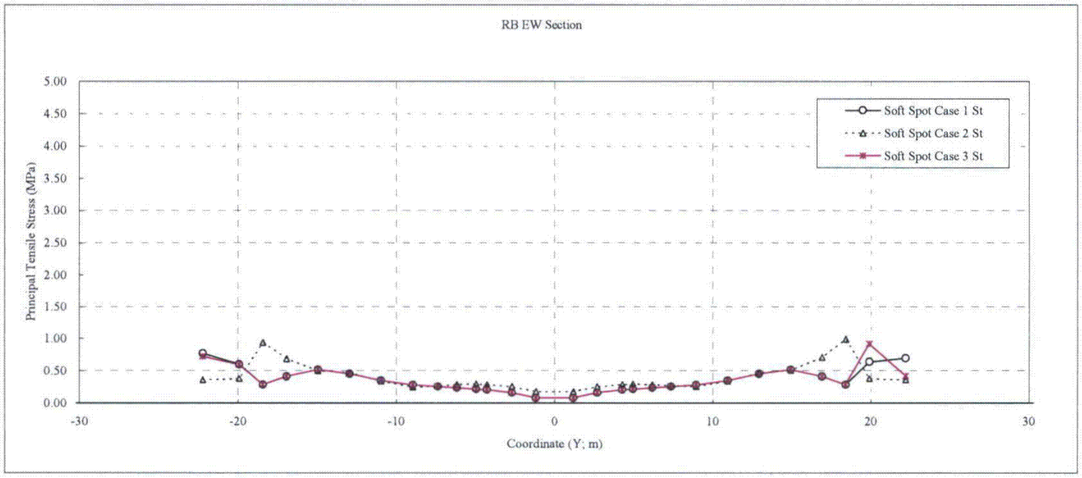
Figure 3.8-93(30) Comparison of Principal Tensile Stress in 0.9 m Thick Slab (Section A)



(a) Soft Soil

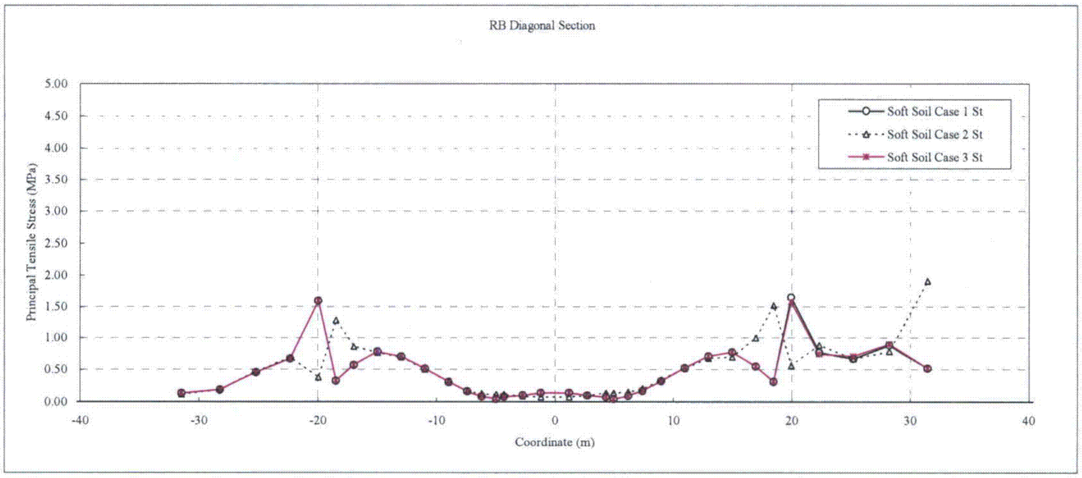


(b) Hard Spot

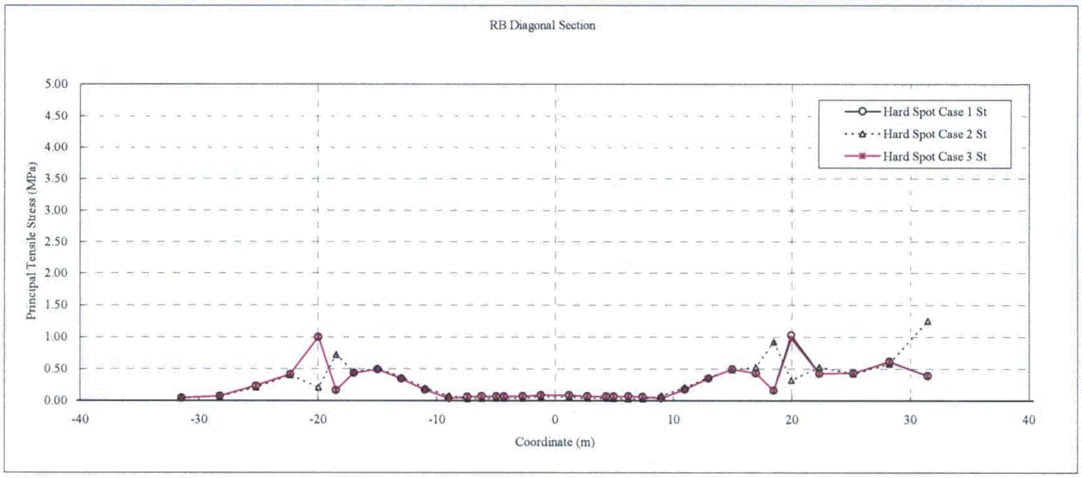


(c) Soft Spot

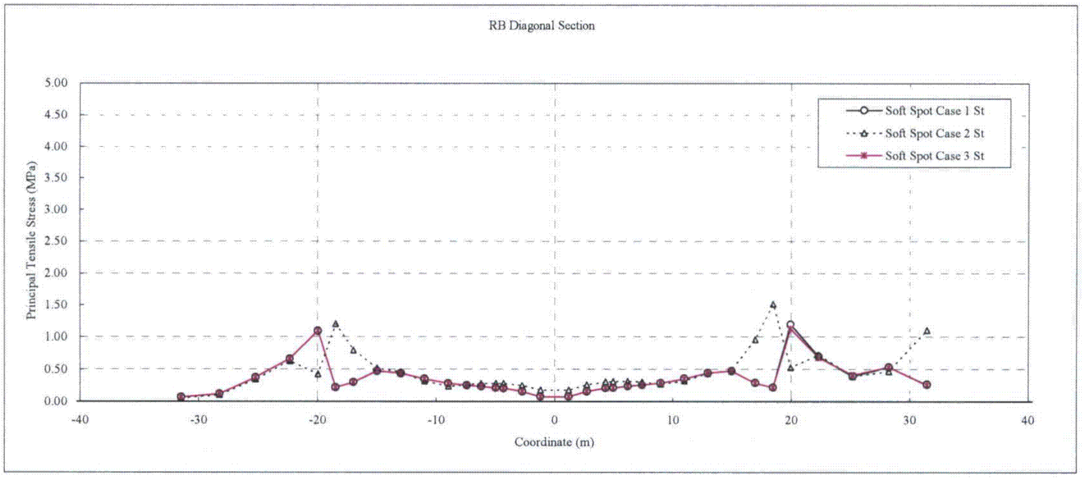
Figure 3.8-93(31) Comparison of Principal Tensile Stress in 0.9 m Thick Slab (Section B)



(a) Soft Soil



(b) Hard Spot



(c) Soft Spot

Figure 3.8-93(32) Comparison of Principal Tensile Stress in 0.9 m Thick Slab (Section C)

Evaluation for NRC RAI 3.8-93 (6)

Basemat Design for Construction Phase Considering Horizontal Variation of Soil Springs

1. Scope

The evaluation provided in the original response to NRC RAI 3.8-93 in MFN 06-407 is expanded herein to confirm the basemat design adequacy during the construction phase, taking into account both "Hard Spot" and "Soft Spot" horizontal variation of soil springs.

2. Analysis Conditions

The analysis condition is the same as in the original response to NRC RAI 3.8-93 plus the varying soil conditions under the basemat. After the completion of the basemat, several parts of the building will be constructed based on a planned construction sequence. This analytical study assesses the stress on the basemat during this construction period.

2-1. Construction Sequence

Per the original response to NRC RAI 3.8-93, the RBFB is assumed to be built sequentially outward as shown in Figure 3.8-93 (4). The assumed sequence is as follows:

Step 1. Pedestal poured up to 5m (below the floor EL-6400, approximately)
Step 2. Apply loads from pour of the RCCV and B3F structure
Step 3. Add pour of exterior walls in RB
Step 4. Add pour of walls in FB area

2-2. Soil Conditions

The following two conditions are considered:

- Hard Spot: Hard (stiffness is three times that of Soft soil; Softx3) below the RPV Pedestal region and Soft condition at the edge of basemat
- Soft Spot: Soft condition below the RPV Pedestal region and Hard (stiffness is three times that of Soft soil; Softx3) at the edge of basemat

2-3. Analysis Method

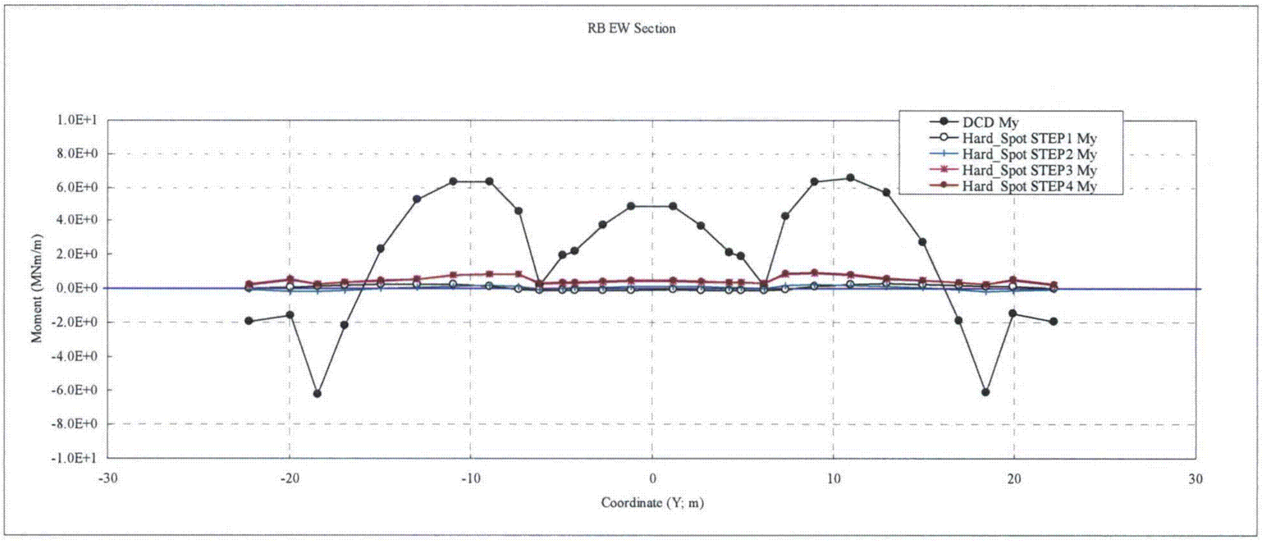
The analytical model has been extracted from the global FEM model used for the DCD design. The dead loads are generated within the program considering element thickness and density of concrete. The analytical conditions are as follows:

- FEM Model: Based on the "Modified Truncated Model (a part of Global FEM Model).

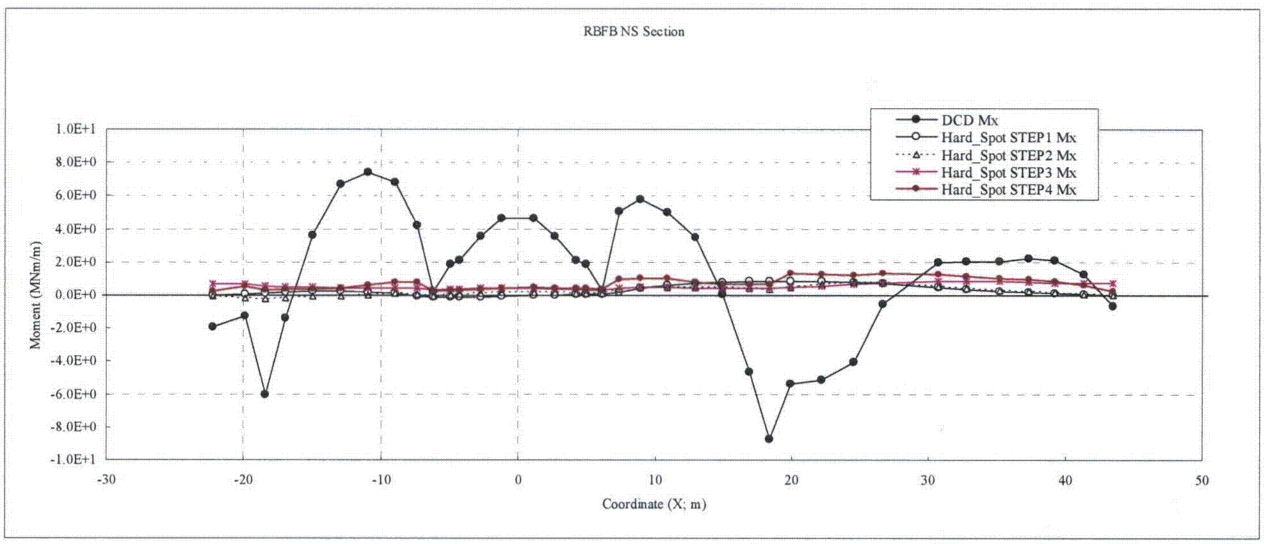
- Soil Condition: "Hard Spot" and "Soft Spot" are shown in Figure 3.8-93 (14). Using the Softx3 is considered as stiffer soil springs under the basemat.
- Load: Dead Load

3. Analysis Results

Figures 3.8-93 (33) and (34) show bending moments of the basemat compared to those of the DCD design (normal operation). The results indicate that the bending moment of the DCD design envelope is larger than the bending moment during the construction phase.



(a) E-W Section



(b) N-S Section

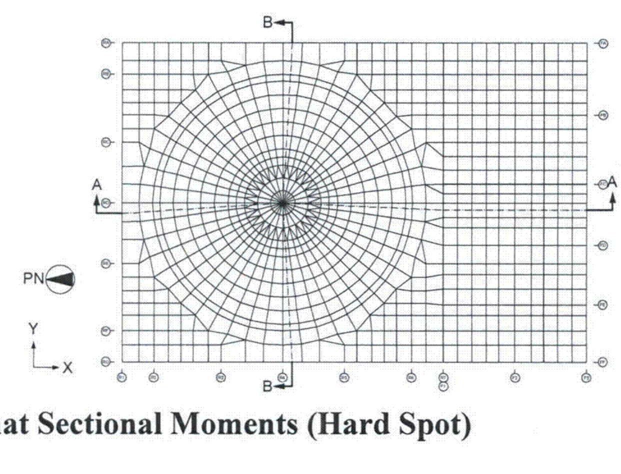
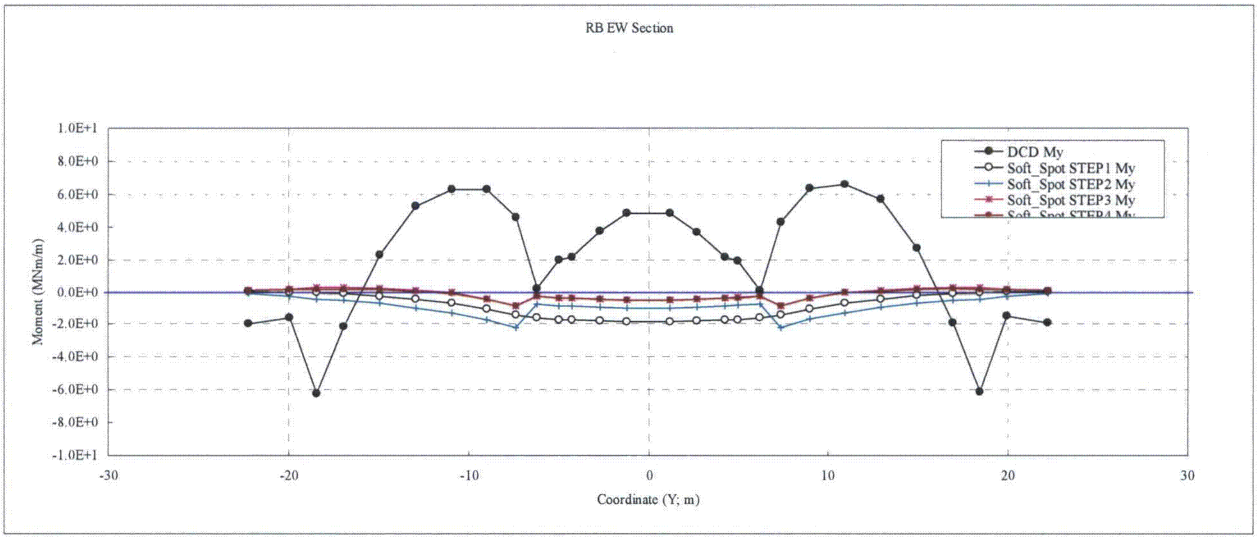
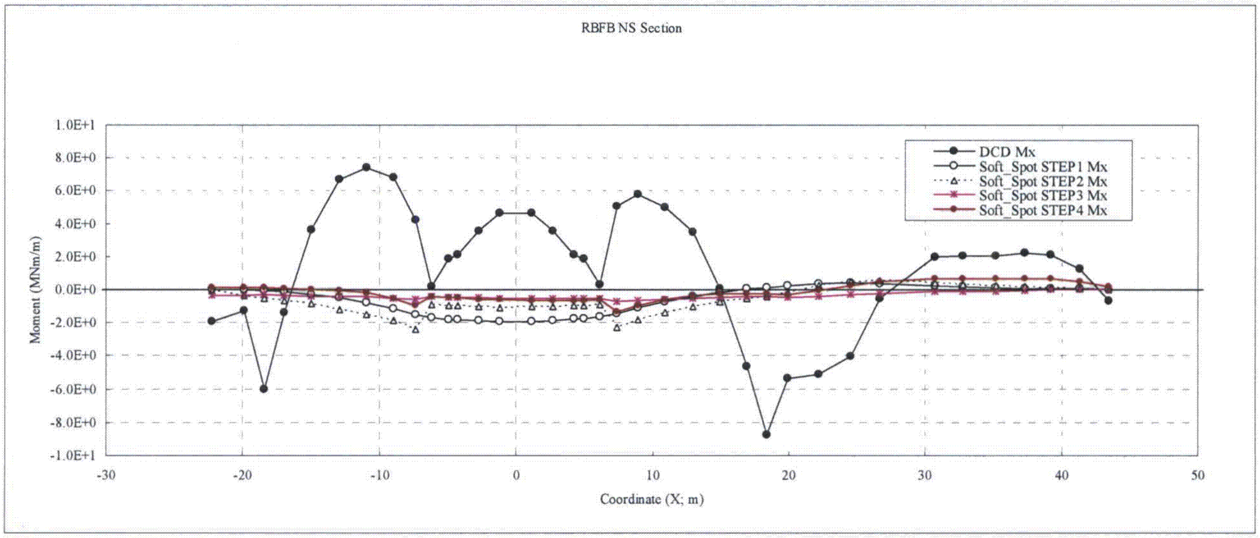


Figure 3.8-93(33) Comparison of Basemat Sectional Moments (Hard Spot)



(a) E-W Section



(b) N-S Section

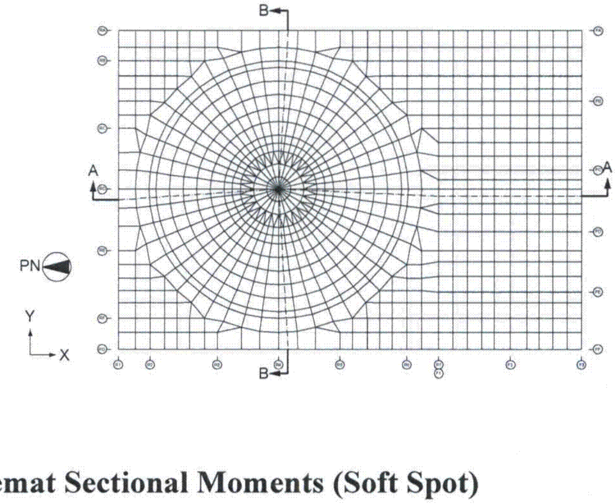


Figure 3.8-93(34) Comparison of Basemat Sectional Moments (Soft Spot)

DCD Tier 2 Table 2.0-1 and Subsection 3G.1.5.5 have been revised. DCD Tier 2 Subsections 3G.1.5.5.2, 3G.1.5.5.3, 3G.1.5.5.4 & 3G.2.5.5.1 have been added. The pages (pp. 2.0-4, 2.0-6, 3G-16, 3G-17 & 3G-194) revised in DCD Tier 2 Revision 3 for this response are attached.

DCD Impact

As stated above.

NRC RAI 3.8-94

DCD Section 3.8.5.4 indicates that the design incorporates an evaluation of the worst loads resulting from the superstructures and loads directly applied to the foundation mat, due to static and dynamic load combinations. However, the DCD does not identify the maximum allowable toe pressure that is acceptable for the basemat design, under the worst-case static and dynamic loads. This information is needed so that evaluations can be made at the COL state for site-specific conditions. Include the maximum toe pressure used in the basemat design in DCD Table 3.8-13.

GE Response

Maximum soil bearing stresses involving SSE are summarized in DCD Tier 2 Table 3G.1-58 for soft, medium and hard site conditions. Maximum soil bearing stress due to dead plus live loads is 699 kPa as shown in DCD Tier 2 Appendix 3G.1.5.5. The site-specific allowable bearing capacities need to be larger than the maximum stress depending on its site condition.

The values indicated in DCD Tier 2 Table 3G.1-58 are evaluated by using the Energy Balance Method, which is described in the Reference cited in response to NRC RAI 3.7-48 Supplement 1. In the evaluations, the basemat is assumed to be rigid, and uplift of the basemat is considered.

The soil pressures obtained from the RB/FB global FE model analyses used for the basemat section design are summarized in Table 3.8-94(1). This table also includes the results of the basemat uplift analyses, which were performed to respond NRC RAI 3.8-13. Seismic loads used for the FE analyses are worst-case loads, i.e., the enveloped values for all site conditions included in DCD Tier 2 Table 3G.1-58. In the FE analyses, the basemat is assumed to be flexible.

As shown in Table 3.8-94(1), the bearing pressures obtained by the FE analyses are less than the worst case maximum bearing pressure in DCD Tier 2 Table 3G.1-58, which is 5.33 MPa for the hard site. Therefore, it can be concluded that the maximum bearing pressures in DCD Tier 2 Table 3G.1-58 are evaluated conservatively.

No DCD change was made in response to this RAI.

Table 3.8-94(1) Maximum Bearing Pressure

Seismic Direction	Case	Max. Pressure (MPa)	Location	Combination
NS	DCD	4.18	Northeast	1.0NS+0.4EW+0.4V
	Uplift*1	4.56	Northeast	1.0NS+0.4EW+0.4V
EW	DCD	4.16	Northeast	0.4NS+1.0EW+0.4V
	Uplift*1	4.49	Northeast	0.4NS+1.0EW+0.4V

Note *1: See response to NRC RAI 3.8-13 Supplement 1.

NRC RAI 3.8-94, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE's response refers to Table 3G.1-58 which provides the maximum soil bearing stress involving SSE. GE needs to clarify that the values in Table 3G.1-58 represent the maximum soil bearing stress for all load combinations. GE also needs to explain whether the comparisons to the bearing pressures in Table 3.8-94(1) are for the same load combinations.

During the audit, GE provided a draft supplemental response to address the above. Regarding the first question, GE provided an acceptable response. GE needs to clarify the RAI response and the draft supplemental response regarding the comparison of the maximum bearing pressures reported in Table 3.8-94(1) to Table 3.G.1-58. GE also needs to explain why the toe pressures reported in Table 3G.1-58 are conservative when considering the variation of horizontal soil springs as discussed in RAI 3.8-93.

GE Response

The values in DCD Tier 2 Table 3G.1-58 represent the maximum soil bearing stress for all combinations calculated using the Energy Balance Method for the RB/FB (Reference 1). They are the maximum bearing stresses for the three generic soil conditions. The toe pressures presented in Table 3.8-94(1) are calculated using the global FE model for design seismic forces which envelope the responses of three soil conditions. The methods of analysis are different in the two calculations. Table 3.8-94(2) compares the maximum soil bearing pressures calculated by the Energy Balance Method and the linear FEM analysis. The results show that the Energy Balance Method is a more conservative method to use for the determination of soil bearing pressures. Note that the values obtained by the Energy Balance Method shown in Table 3.8-94(2) are the updated values for DCD Tier 2 Table 3G.1-58, due to the changes in seismic design loads, which have been included in DCD Tier 2 Revision 3.

Reference 1: Tseng, W.S. and Liou, D.D., "Simplified Methods for Predicting Seismic Basemat Uplift of Nuclear Power Plant Structures, Transactions of the 6th International Conference on SmiRT", Paris, France, August 1981

Table 3.8-94(2) Comparison of Maximum Bearing Pressure

		Site Condition (MPa)		
		Soft	Medium	Hard
Energy Balance Method		2.7	7.3	5.4
FEM	Linear	2.6	4.8	5.4
	Uplift*	-	-	5.4

* See response to NRC RAI 3.8-13 Supplement 1. The tension springs of linear cases are eliminated.

The variations of horizontal soil spring ("Hard Spot" and "Soft Spot" as shown in the response to NRC RAI 3.8-93, Supplement 1) are also considered in this study. Note that the DCD envelope is based on uniform soil conditions. Despite the fundamental difference in the treatment of the soil stiffness distribution, the maximum soil bearing pressures of the non-uniform soil condition are similar to those of the uniform soil condition.

Table 3.8-94(3) Maximum Bearing Pressure Under Non-Uniform Soil Condition

	Case	Max. Pressure (MPa)
FEM	Hard Spot*	3.8
	Soft Spot*	4.9

* See response to NRC RAI 3.8-93, Supplement 1. Stiffer area is Softx3 condition.

DCD Tier 2 Subsections 3G.1.5.5, 3G.1.6, Table 3G.1-58 and Table 3G.2-27 have been revised. The pages (pp. 3G-16, 3G-18, 3G-123 & 3G-215) revised in DCD Tier 2 Revision 3 for this response are attached.

DCD Impact

As stated above.

NRC RAI 3.8-96

DCD Section 3.8.5.5 presents two specifications of appropriate safety factors (SF) for foundation design. The SF against sliding indicates that sliding resistance is judged as the sum of both shear friction along the basemat and passive pressures induced due to embedment effects. However, the DCD does not indicate (1) how these effects are to consider consistent lateral displacement criteria (that is, the displacement effect on passive pressure is not the same as on friction development) and (2) how the effect of waterproofing is to impact the development of basemat friction capacity. DCD Section 3.8.5.5 needs to clearly indicate how these effects are incorporated into the standard plant design for the considered range of acceptable site conditions considered.

Include this information in DCD Section 3.8.5.5. In addition, (1) identify the applicable detailed report/calculation (number, title, revision and date, and brief description of content) that will be available for audit by the staff, and (2) reference this report/calculation in the DCD.

GE Response

a) As stated in the response to NRC RAI 3.7-35, SASSI analyses were performed to address the embedment effect. It was confirmed that the base shears calculated by the SASSI analyses, which consider the embedment effect, are less than those obtained by design seismic analyses that neglect the embedment effect. The use of higher base shears calculated without the beneficial effect of embedment is deemed conservative for the sliding evaluation without explicit consideration of consistent lateral displacement criteria for passive pressure and friction resistance.

b) Please see NRC RAI 3.8-89 for the response to impact of waterproofing.

(1) The applicable detailed reports/calculations that will be available for the NRC audit are:

26A6652, *RB FB Stability Analysis Report, Revision 2*, April 2006, which contains the stability calculations of the Reactor Building/Fuel Building.

26A6654, *CB Stability Analysis Report, Revision 2*, April 2006, which contains the stability calculations of the Control Building.

(2) Since this information exists as part of GE's internal tracking system, it is not necessary to add it to the DCD.

No DCD change was made in response to this RAI.

NRC RAI 3.8-96, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE needs to clarify the response to this RAI and revise Section 3.8.5.5 to be consistent with their response. Does GE calculate the SF against sliding by only considering the basemat shear friction? If not, GE needs to better explain the method used in the light of the question asked. GE also needs to explain (1) Do the exterior walls need to be designed for passive pressures as implied in the last sentence of item (a) of the response? (2) Are both base shear and passive pressures being relied upon for lateral restraint? (3) the friction coefficient used in the analysis and its technical bases, (4) how lift-off effects are captured in the sliding analysis, (5) the capacity of the mud mat to resist applied loads, and (6) what effect the use of chemical crystalline powder in the mud mat has on the assumed structural properties. Potential leaching of the mud mat due to groundwater is being reviewed under RAI 3.8-81.

During the audit, GE indicated the following:

(1) & (2) GE explained the answer to both is yes. The seismic stick model did not consider embedment effects while the stability calculations (soil sliding), using this shear force, did consider soil friction and soil passive pressure. However, the SASSI did consider soil embedment and it was shown that the resulting shear loads are smaller than those calculated by the seismic stick model. GE indicated that they will determine an appropriate method to consider the seismic shear force from the seismic stick model and/or SASSI analysis in their calculation of sliding stability calculation. The method used will ensure consistency of the deformation in developing the frictional soil resistance and soil passive pressure. Also, the design of the foundation walls will consider the appropriate pressures from the SASSI analysis and passive soil pressures used in the sliding stability calculations.

(3) GE will provide the reference for the static and dynamic coefficient of friction values. This would be needed if GE is not able to show that the soil frictional resistance alone can resist the seismic shear force.

(4) GE will provide additional justification to demonstrate that the effects of uplift are not significant.

(5) GE will expand on the description of the mud mat and provide the minimum applicable requirements (e.g., ACI Code).

(6) GE explained that this material has no deleterious effect on the concrete and has been used and approved at other NPPs.

GE Response

(1) & (2) Table 3.8-96(1) summarizes the evaluation results of the foundation sliding analyses for generic site conditions.

The seismic loads used in the evaluation are obtained by seismic response analysis using the lumped soil spring stick model (DAC3N analyses). Since the lumped soil spring model does not consider embedment effects, the resulting shear loads are larger than those calculated by SASSI analyses. The use of higher base shear is conservative for the foundation stability evaluation.

Sliding resistance is composed of the following:

- Friction force at the basemat bottom surface
- Cohesion force at the basemat bottom surface
- Passive soil pressure at the basemat side surface
  For the RB/FB and CB, the gap between the building and excavated soil is filled with concrete up to the top level of the basemat or higher. Since the basemat is constrained by rigid concrete backfill, the passive soil pressure is mobilized for the region.
- Passive soil pressure on walls
  The passive soil pressures considered are the envelope lateral soil pressures obtained from the elastic solution based on ASCE 4-98, Section 3.5.3.2 and SASSI analysis results, which are used in the wall design.

(3) Only the static coefficient of friction is used for stability evaluation. Coefficient of friction, μ, is calculated by the following equation.

$$\mu = \min(\tan\phi, 0.75)$$

where,

ϕ = Angle of internal friction (30° for soft and medium soil, 40° for hard soil).

The minimum angle of internal friction will be specified to be 30° in DCD Tier 2 Table 2.0-1 as a site requirement.

(4) Sliding resistance is composed of passive soil pressure, friction and cohesion forces at the basemat bottom. Uplift of the basemat has no effect on the passive soil pressure. The friction force at the basemat bottom is also not influenced by the uplift, because the friction force is calculated by (normal compressive force) x (friction coefficient). Because the basemat uplift has no effect on both the normal compressive force and friction coefficient, the resulting friction force is unchanged even if uplift occurs. As for the cohesion force, since it is calculated by (cohesion stress) x (contact area of basemat), the value is reduced if the basemat is uplifted. However, the contribution of the cohesion force to the total resistance is relatively small as shown in Table 3.8-96(1). The reduction of the cohesion force due to uplift has little impact on the total resistance.

(5) The mud mat construction is performed in accordance with the same standards and requirements as the basemat to avoid possibility of errors in the field.

(6) The crystalline powder used is the same material approved for use in AP-1000 and has no deleterious effect on concrete. It forms a substantial waterproofing barrier to prevent water infiltration or ex-filtration.

Table 3.8-96(1) Sliding Evaluation Results

(i) RBFB

Building width X	70.0 m					
Building width Y	49.0 m					
Total Weight	2360 MN					
Buoyancy	652 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	676 MN		1159 MN		1103 MN	
Minimum Vertical Load	1438 MN		1244 MN		1267 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*899*	*787*	*1462*	*1619*	*1486*	*1243*
Fub: Bottom Friction Force (MN)	830	830	718	718	950	950
Fc: Effective Cohesion Force (MN)	0	0	343	343	1166	1166
Fpb: Passive Pressure for Basemat (MN)	132	188	213	304	539	769
Fdsf: Passive Soil Pressure on Wall (MN)	440	644	440	644	440	644
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fdsf)	*1402*	*1663*	*1714*	*2010*	*3095*	*3530*
FS (=Fr/Fv)	**1.56**	**2.11**	**1.17**	**1.24**	**2.08**	**2.84**

(ii) CB

Building width X	30.3 m					
Building width Y	23.8 m					
Total Weight	173 MN					
Buoyancy	101 MN					
Soil Condition	Soft		Medium		Hard	
Vertical Seismic Load	72 MN		79 MN		100 MN	
Minimum Vertical Load	43 MN		40 MN		32 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*105*	*100*	*97*	*94*	*101*	*91*
Fub: Bottom Friction Force (MN)	25	25	23	23	24	24
Fc: Effective Cohesion Force (MN)	0	0	72	72	245	245
Fpb: Passive Pressure for Basemat (MN)	36	46	64	82	173	220
Fds: Passive Soil Pressure on Wall (MN)	58	74	58	74	58	74
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fds)	*119*	*145*	*218*	*251*	*500*	*563*
FS (=Fr/Fv)	**1.13**	**1.44**	**2.23**	**2.67**	**4.94**	**6.22**

Note:

1. Minimum vertical load: Wm = Wt – Fb – 0.4Fa
   where,
   Fb: Buoyancy due to groundwater
   Fa: Vertical seismic force
2. Bottom friction force: Fub = Wm* μ
   where,
   μ: friction coefficient
3. Fv and Fa are obtained by seismic lumped soil spring stick model analyses (DAC3N analyses)

DCD Tier 2 Table 2.0-1, Subsections 3G.1.5.5 and 3G.2.5.5 and Tables 3G.1-57 and 3G.2-26 have been revised. DCD Tier 2 Figures 3G.1-65 and 3G.2-15 have been added. The pages (pp. 2.0-3, 3G-16, 3G-123, 3G-189, 3G-194, 3G-215 & 3G-230) revised in DCD Tier 2 Revision 3 for this response are attached.

DCD Impact

As stated above.

NRC RAI 3.8-99

DCD Section 3.8.5.7 indicates that there are no testing or ISI requirements for the foundations. Has the applicant committed to RG 1.160 for monitoring of structures to meet the requirements of 10 CFR 50.65? If so, then modify DCD Section 3.8.5.7 to indicate this. If not, provide the technical basis in DCD Section 3.8.5.7.

GE Response

Regulatory Guide 1.160 will be referenced in a revised DCD Tier 2 Section 3.8.5.7 for monitoring of the Seismic Category I structures of the ESBWR listed in DCD Tier 2 Table 19.2-4.

A markup of DCD Tier 2 Section 3.8.5.7 was provided in MFN 06-407.

NRC RAI 3.8-99, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

The resolution of this RAI needs to be consistent with the outcome of the review performed for RAI 3.8-81.

During the audit, it was agreed that the resolution for RAI 3.8-81 will address this RAI. The revised wording in the DCD will address structures covered by DCD Sections 3.8.4 and 3.8.5.

GE Response

DCD Tier 2 Subsection 3.8.5.7 has been revised to reference NUREG-1801, 10 CFR 50.65 and RG 1.160.

Concrete specified in the ESBWR is watertight, and a crystalline powder admixture waterproofing is used in the foundation. See also the response to NRC RAI 3.8-96, Supplement 1, Item (6).

Settlements are similarly investigated at the start of the COL approval activities. Allowable differential settlements in the ESBWR are addressed in response to NRC RAI 3.8-93.

The page (p. 3.8-40) revised in DCD Tier 2 Revision 3 for this response is attached.

DCD Impact

As stated above.

Table 2.0-1

Envelope of ESBWR Standard Plant Site Design Parameters [1]

Maximum Ground Water Level:	0.61 m (2 ft) below plant grade
Extreme Wind:	**Seismic Category I and II Structures** - 100-year Wind Speed (3-sec gust): 67.1 m/s (150 mph) - Exposure Category: D **Non-Seismic Standard Plant Structures** - Extreme wind 49.2 m/s (110 mph)
Maximum Flood (or Tsunami) Level: [2]	0.3 m (1 ft) below plant grade
Tornado:	- Maximum Tornado Wind Speed: [3] 147.5 m/s (330 mph) - Maximum Rotational Speed: 116.2 m/s (260 mph) - Translational Velocity: 31.3 m/s (70 mph) - Radius: 45.7 m (150 ft) - Maximum Pressure Differential: 16.6 kPa (2.4 psi) - Rate of Pressure Change: 11.7 kPa/s (1.7 psi/s) - Missile Spectra: [3] Spectra I of SRP 3.5.1.4, Rev 2 applied to full building height.
Precipitation (for Roof Design):	- Maximum Rainfall Rate: [4] 49.3 cm/hr (19.4 in/hr) - Maximum Short Term Rate: 15.7 cm (6.2 in) in 5 minutes - Maximum Roof Load: [5] 2873 Pa (60 lbf/ft^2)
Ambient Design Temperature: [6]	**2% Exceedance Values** - Maximum: 35.6°C (96°F) dry bulb 26.1°C (79°F) wet bulb (coincident) 27.2°C (81°F) wet bulb (non-coincident) - Minimum: -23.3°C (-10°F) **1% Exceedance Values** - Maximum: 37.8°C (100°F) dry bulb 26.1°C (79°F) wet bulb (coincident) 27.8°C (82°F) wet bulb (non-coincident) - Minimum: -23.3°C (-10°F) **0% Exceedance Values** - Maximum: 46.1°C (115°F) dry bulb 26.7°C (80°F) wet bulb (coincident) 29.4°C (85°F) wet bulb (non-coincident) - Minimum: -40°C (-40°F)
Soil Properties:	- Minimum Static Bearing Capacity: [7] ≥ 718 kPa (15000 lbf/ft^2) - Minimum Shear Wave Velocity: [8] 300 m/s (1000 ft/s) - Liquefaction Potential: None under footprint of Seismic Category I or II structures. - Angle of Internal Friction ≥ 30 degrees

Table 2.0-1

Envelope of ESBWR Standard Plant Site Design Parameters (continued)

Seismology:	- SSE Horizontal Ground Response Spectra: [9]	See Figure 2.0-1
	- SSE Vertical Ground Response Spectra: [9]	See Figure 2.0-2
Hazards in Site Vicinity:	- Site Proximity Missiles and Aircraft:	$\leq 10^{-7}$ per year
	- Toxic Gases:	None *
	- Volcanic Activity:	None
* Maximum toxic gas concentrations at the Main Control Room (MCR) and Technical Support Center (TSC) HVAC intakes:	< toxicity limits	
Required Stability of Slopes: [10]	- Factor of safety for static (non-seismic) loading	1.5
	- Factor of safety for dynamic (seismic) loading	1.1
Maximum Settlement Values for Seismic Category I Buildings (see Subsections 3G.1.5.5.4 and 3G.2.5.5.1):		
Maximum Settlement at any corner of basemat	- Under Reactor/Fuel Building Mat	103 mm (4.0 inches)
	- Under Control Building	18 mm (0.7 inches)
Averaged Settlement at four corners of basemat	- Under Reactor/Fuel Building Mat	65 mm (2.6 inches)
	- Under Control Building	11 mm (0.4 inches)
Maximum Differential Settlement along the longest mat foundation dimension	- within Reactor/Fuel Building	77 mm (3.0 inches)
	- within Control Building	13 mm (0.5 inches)
Maximum Differential Displacement between Reactor/Fuel Buildings and Control Building		85 mm (3.3 inches)

Notes for Table 2.0-1:

(1) The design of the Radwaste Building uses a set of design parameters that are specified in Regulatory Guide 1.143, Table 2, Class RW IIa instead of the corresponding values given in this table.

(2) Probable maximum flood level (PMF), as defined in Table 1.2-6 of Volume III of Reference 2.0-4.

(3) Maximum speed selected is based on NRC Interim Position on Regulatory Guide 1.76. Concrete structures designed to resist Spectrum I missiles of SRP 3.5.1.4, Rev. 2, will also resist missiles postulated in Draft Guide DG-1143.

(4) Based on probable maximum precipitation (PMP) for one hour over 2.6 km^2 (one square mile) with a ratio of 5 minutes to one hour PMP of 0.32 as found in Reference 2.0-3. Roof scuppers are designed to handle the PMP. When used in combination with snow pack, the roof and drainage design is for 2873 Pa (60 lbf/ft^2) extreme load. See also Table 3G.1-2.

(5) Maximum design roof load accommodates snow load and probable maximum winter precipitation in References 2.0-2 and 2.0-3. See also Table 3G.1-2.

(6) Zero percent exceedance values are based on conservative estimates of historical high and low values for potential sites. One and two percent exceedance values were selected in order to bound the values presented in Reference 2.0-4 and available Early Site Permit applications.

(7) At foundation level of Seismic Category I structures. See Subsections 3G.1.5.5, 3G.2.5.5 and 3G.3.5.5 for minimum dynamic bearing capacity for the Reactor, Control and Fuel Buildings, respectively.

(8) This is the equivalent uniform shear wave velocity (V_{eq}) at seismic strains after the soil property uncertainties have been applied. V_{eq} is calculated to achieve the same wave traveling time over the depth equal to the embedment depth plus 2 times the largest foundation plan dimension below the foundation as follows:

$$V_{eq} = \frac{\sum d_i}{\sum \frac{d_i}{V_i}}$$

where d_i and V_i are the depth and shear wave velocity, respectively, of the ith layer. The ratio of the largest to the smallest shear wave velocity over the mat foundation width at the foundation level does not exceed 1.7.

(9) Safe Shutdown Earthquake (SSE) design ground response spectra are defined as free-field outcrop spectra at the foundation level (bottom of the base slab) of Seismic Category I structures.

(10) Values reported here are actually design criteria rather than site design parameters. They are included here because they do not appear elsewhere in the DCD.

3.8.4.5.2 Control Building

The acceptance criteria for the design of the Control Building are same as the Reactor Building in Section 3.8.4.5.1.

3.8.4.5.3 Fuel Building

Same as the RB in 3.8.4.5.1.

3.8.4.5.4 Radwaste Building

Structural acceptance criteria and materials criteria for the RW is in accordance with Item 32 in Table 3.8-9 for Safety Class RW-IIa.

3.8.4.5.5 (Deleted)

3.8.4.6 Material, Quality Control and Special Construction Techniques

This subsection contains information related to the materials, quality control and special construction techniques used in the construction of the other Seismic Category I structures.

3.8.4.6.1 Concrete

Concrete material is the same as described in Section 3.8.1.6.1 with the following exception: The specified compressive strength is 34.5 MPa (5000 psi). Concrete is batched and placed according to ACI 349-01.

3.8.4.6.2 Reinforcing Steel

Reinforcing steel is the same as in Section 3.8.1.6.2.

3.8.4.6.3 Splices of Reinforcing Steel

Splices of reinforcing steel are the same as in Section 3.8.1.6.3 except that placing and splicing is in accordance with ACI 349-01.

3.8.4.6.4 Quality Control

Quality control is the same as in Section 3.8.1.6.5 except that the Construction Specification will reference ACI 349-01 and applicable Regulatory Guides. For welding of reinforcing bars, inspection and documentation requirements conform to ASME Code Section III, Division 2 also.

3.8.4.6.5 Special Construction Techniques

There is composite construction in the other Seismic Category I structures. Some of the components, such as rebar cages, are pre-assembled and lifted into place. As described in Section 3.8.4.1.1, the RB floor slabs are composed of reinforcing bars, steel plates, and concrete. Floor slab steel plates, which are reinforced by welded shapes, are assembled in discrete segments that are lifted into place. The steel plates are also used as formwork for concrete fill.

3.8.4.7 Testing and In-Service Inspection Requirements

Other Seismic Category I structures are monitored per NUREG-1801 and 10 CFR 50.65 as clarified in RG 1.160, in accordance with Section 1.5 of RG 1.160.

specific subgrade stiffness and calculated settlement on the design of the Seismic Category I structures and foundations is evaluated.

A detailed description of the analytical and design methods for the foundations of the RB including the containment, the CB and the FB is included in Appendix 3G.

3.8.5.5 Structural Acceptance Criteria

The main structural criteria for the containment portion of the foundation are to provide adequate strength to resist loads and sufficient stiffness to protect the containment liner from excessive strain. The acceptance criteria for the containment portion of the foundation mat are presented in Subsection 3.8.1.5. The structural acceptance criteria for the RB, CB and FB foundations are described in Subsection 3.8.4.5.

The allowable factors of safety of the ESBWR structures for overturning, sliding, and flotation are included in Table 3.8-14. The calculated factors of safety are shown in Appendix 3G for each foundation mat evaluated according to the following procedures.

The factor of safety against overturning due to earthquake loading is determined by the energy approach described in Subsection 3.7.2.14.

The factor of safety against sliding is defined as:

$$FS = (F_s + F_p)/(F_d + F_h)$$

where F_s and F_p are the shearing and sliding resistance, and passive soil pressure resistance, respectively. F_d is the maximum lateral seismic force including any dynamic active earth pressure, and F_h is the maximum lateral force due to loads other than seismic loads.

The factor of safety against flotation is defined as:

$$FS = F_{DL}/F_B$$

where F_{DL} is the downward force due to dead load and F_B is the upward force due to buoyancy.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The foundations of Seismic Category I structures are constructed of reinforced concrete using proven methods common to heavy industrial construction. For further discussion, see Subsection 3.8.1.6.

3.8.5.7 Testing and In-Service Inspection Requirements

The foundations of Seismic Category I structures are monitored per NUREG-1801 and 10 CFR 50.65 as clarified in RG 1.160, in accordance with Section 1.5 of RG 1.160.

3.8.6 COL Information

None.

3G.1.5.4.3.5 Main Steam Tunnel Floors and Walls

Section 31 is selected for the MS tunnel wall (Element #150122) and slabs (Elements #96611 and #98614). The MS tunnel is composed of the reinforced concrete structures as described in Subsection 3G.1.5.4.3.3.

The maximum rebar stress is found to be 220.7 MPa (32.0 ksi) in Table 3G.1-51, and the maximum transverse shear force is found to be 0.50 MN/m (2.86 kips/in) against the shear strength of 1.55 MN/m (8.85 kips/in).

3G.1.5.5 Foundation Stability

The Reactor Building, the concrete containment and the Fuel Building share a common foundation. The stabilities of the foundation against overturning, sliding and floatation are evaluated. The energy approach is used in calculating the factor of safety against overturning.

The factors of safety against overturning, sliding and floatation are given in Table 3G.1-57. All of these meet the acceptance criteria given in Table 3.8-14. In the sliding evaluation the gap between the building and excavated soil is backfilled with concrete up to the top level of the basemat as shown in Figure 3G.1-65.

The maximum bearing stresses shown in Table 3G.1 58 are evaluated using the Energy Balance Method (Reference 3G.1 2). In order to verify the results, toe pressures obtained by the FE analyses using the RB/FB global model are compared with the values in Table 3G.1 58. As a result, the bearing pressures calculated by the Energy Balance Method envelop the pressures of FE analyses.

A series of parametric analyses are performed to verify the assumptions and results of the global FE analysis is used as the baseline for the basemat design.

- Lateral variations of soil stiffness are evaluated using the global FE model. Analyses are performed assuming "Hard spot" and "Soft spot" under the RPV Pedestal area.
- Construction loads are evaluated in the design of the basemat. The analyses focus on the response of the basemat during the early stage of construction when it could be susceptible to differential loading and deformations.
- The analyses are performed to confirm acceptability of allowable total and differential settlement that are specified over the length of the foundation.

Details are provided in Subsections 3G.1.5.5.2 through 3G.1.5.5.4.

3G.1.5.5.1 Effect of Basemat Uplift

As described in Appendix 3G.1.4.2, the foundation soil is represented by elastic soil springs which resist both compression and tension. However, actual foundation soil cannot bear tensile force. This difference may have an influence on the stresses in the basemat, if the basemat is uplifted due to design loads. Therefore, analyses to evaluate the effect of potential uplift of the basemat are performed using the RB/FB global FE model shown in Figure 3G.1 8.

An iterative approach is used. Based on the result from the initial analysis, the tension capability is removed in the next iteration for those springs that are in tension. This iterative process is continued until there are no more springs in tension.

Analyses are performed for the horizontal SSE loads. Figures 3G.1 60 through 3G.1 64 show the comparison of the sectional deformations of the basemat and the bending moments generated in the basemat respectively at the final step of iteration. In the area close to the RCCV wall, bending moments are higher than that of the linear analysis results; however the resulting stresses in the concrete and reinforcement for the design "SSE + LOCA" load combination are still below the code allowables with large margins as shown in Table 3G.1 59. Therefore, it can be concluded that the effect of uplift is negligible to the linear analysis using the global FE model.

3G.1.5.5.2 Effect of Horizontal Variation of Soil Spring

To account for potential horizontal variation of foundation soil stiffness over the basemat width, stiff soil springs are considered under the RPV Pedestal area assuming linear variation to the edge of the mat. The RPV Pedestal was selected because it produces the largest clear span for the mat and is likely to be the first structure constructed on the mat. This is used as the "Hard Spot". In addition, the inverted variation, i.e. softer soil springs assumed under the RPV Pedestal area, is also considered and called the "Soft Spot". Based on the analysis results for these soil conditions, some of the "Soft Spot" case results predict larger mat bending moments than the uniform soil condition. However, the DCD design envelopes the results of horizontal variation of soil spring as long as the ratio of spring stiffness at the basemat center to that at the basemat edge does not exceed 3. This spring stiffness ratio converts to $\sqrt{3}$ (1.7) for the corresponding shear wave velocity ratio.

3G.1.5.5.3 Effect of Construction Sequence

The basemat design is checked against the loads expected during construction of the basemat. The RB/FB basemat is divided into 7 zones for concrete pour and these zones are investigated for three possible construction sequences. The moment differences between sequences considered are negligibly small in comparison with the moments used in the basemat design. In addition to basemat construction sequence, the impact of the building structures construction sequence, i.e., RPV Pedestal, RCCV and walls, on the basemat design is also investigated. The evaluation results confirm that the building structures construction sequence has negligible effect on the basemat design. These studies include horizontal soil spring variations, "Hard Spot" and "Soft Spot" as described in Subsection 3G.1.5.5.2.

3G.1.5.5.4 Foundation Settlement

The basemat design is checked against the normal and differential settlement of the RB/FB. It is found that the basemat can resist the maximum mat foundation corner settlement of 103 mm (4.0 in.) and the settlement averaged at four corners of 65 mm (2.6 in.). The allowable differential settlement specified in Section 2.0 is 77 mm (3.0 in.) across the basemat under linearly varying stiffness of soil condition (gradient condition). The estimated differential settlement between buildings (RB/FB and CB) is 85 mm (3.3 in.).

3G.1.5.6 Tornado Missile Evaluation

The minimum thickness required to prevent penetration and concrete spalling is evaluated. The methods and procedures are shown in Section 3.5.3.1.1. The minimum thickness required is less than the minimum 1000 and 700 mm thickness provided for the RB external walls and roof, respectively.

3G.1.6 References

3G.1-1 Burns & Roe, "State-of-the-Art Report on High Temperature Concrete Design," prepared for US. Department of Energy, Document No. DOE/CH/94000-1, November 1985.

3G.1-2 Tseng, W.S. and Liou, D.D., "Simplified Methods for Predicting Seismic basemat Uplift of Nuclear Power Plant Structures, Transactions of the 6^{th} International Conference on SmiRT", Paris, France, August 1981.

Table 3G.1-57

Factors of Safety for Foundation Stability

Load Combination	Overturning		Sliding		Floatation	
	Required	Actual	Required	Actual	Required	Actual
D + H + E'	1.1	111.1	1.1	1.17	--	--
D + F'	--	--	--	--	1.1	3.48

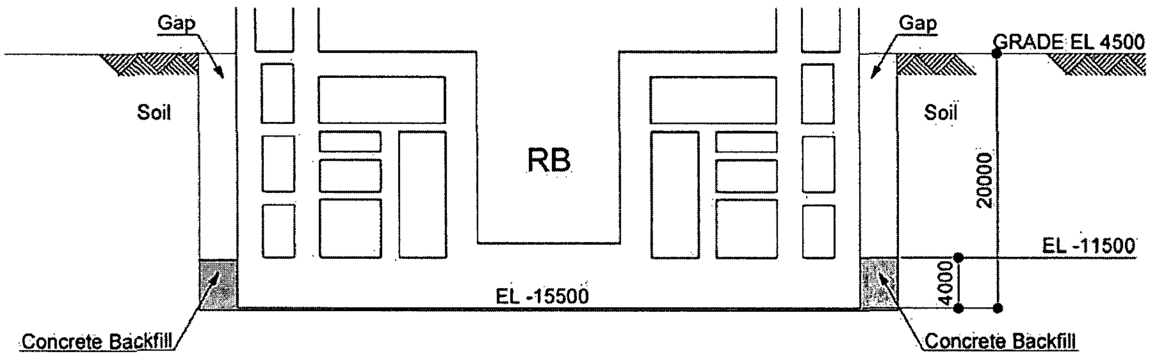
Where,
D = Dead Load
H = Lateral soil pressure
E' = Safe Shutdown Earthquake
F' = Buoyant forces of design basis flood

Table 3G.1-58

Maximum Soil Bearing Stress Involving SSE

	Site Condition*		
	Soft	Medium	Hard
Bearing Stress (MPa)	2.7	7.3	5.4

* See Table 3A.3-1 for site properties.



Note: Backfill method for gap and excavation method (e.g., vertical cut, open cut) will be determined considering actual site conditions

Figure 3G.1-65. Concrete Backfill in Sliding Evaluation

3G.2.5.5 Foundation Stability

The stabilities of the CB foundation against overturning, sliding and floatation are evaluated. The energy approach is used in calculating the factor of safety against overturning.

The factors of safety against overturning, sliding and floatation are given in Table 3G.2-26. All of these meet the acceptance criteria given in Table 3.8-14. In the sliding evaluation the gap between the building and excavated soil is backfilled with concrete up to the top level of the basemat as shown in Figure 3G.2-15.

Maximum soil bearing stress is found to be 256 kPa due to dead plus live loads. Maximum bearing stresses for load combinations involving SSE are shown in Table 3G.2-27 for various site conditions.

3G.2.5.5.1 Foundation Settlement

The basemat design is checked against the normal and differential settlement of the CB. It is found that the basemat can resist the maximum settlement at mat foundation corner of 18 mm (0.7 in.) and the settlement averaged at four corners of 11 mm (0.4 in.). The allowable differential settlement specified in Section 2.0 is 13 mm (0.5 in.) across the basemat under linearly varying stiffness of soil condition (gradient condition). The estimated differential settlement between buildings (RB/FB and CB) is 85 mm (3.3 in.).

3G.2.5.6 Tornado Missile Evaluation

The CB is shown in Figure 3G.2-3. The minimum thickness required to prevent penetration and concrete spalling is evaluated. The methods and procedures are shown in Section 3.5.3.1.1.

Table 3G.2-26

Factors of Safety for Foundation Stability

Load Combination	**Overturning**		**Sliding**		**Floatation**	
	Required	Actual	Required	Actual	Required	Actual
D + H + E'	1.1	86.1	1.1	1.13	--	--
D + F'	--	--	--	--	1.1	1.66

Where,

D = Dead Load

H = Lateral soil pressure

E' = Safe Shutdown Earthquake

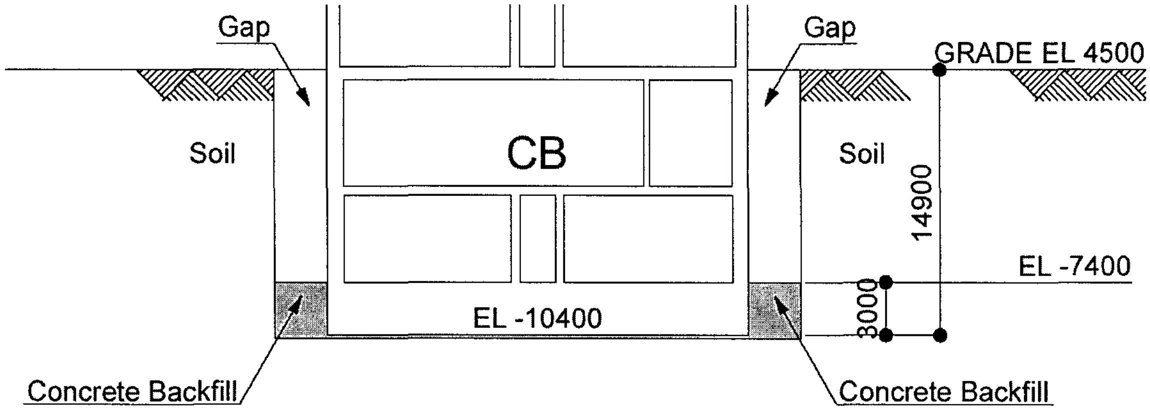
F' = Buoyant forces of design basis flood

Table 3G.2-27

Maximum Soil Bearing Stress Involving SSE

	Site Condition*		
	Soft	Medium	Hard
Bearing Stress (MPa)	2.2	2.2	2.7

* See Table 3A.3-1 for site properties.



Note: Backfill method for gap and excavation method (e.g., vertical cut, open cut) will be determined considering actual site conditions.

Figure 3G.2-15. Concrete Backfill in Sliding Evaluation