HITACHI

GE Hitachi Nuclear Energy

James C. Kinsey
Vice President, ESBWR Licensing

PO Box 780 M/C A-55
Wilmington, NC 28402-0780
USA

T 910 675 5057
F 910 362 5057
jim.kinsey@ge.com

MFN 06-407
Supplement 3

Docket No. 52-010

November 28, 2007

U.S. Nuclear Regulatory Commission
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Washington, D.C. 20555-0001

Subject: **Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – DCD Tier 2 Section 3.8 – Seismic Category I Structures - RAI Numbers 3.8-28 S02, 3.8-76 S02, 3.8-93 S02, 3.8-94 S02, 3.8-96 S02, 3.8-101 S02, 3.8-102 S02 and 3.8-103 S02**

Enclosure 1 contains GEH's response to the subject RAIs transmitted via email on May 24, 2007. GEH's original response was provided in the Reference 1 letter.

If you have any questions or require additional information, please contact me.

Sincerely,

James C. Kinsey

James C. Kinsey
Vice President, ESBWR Licensing

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NRO

Reference:

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,* dated 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,* dated February 1, 2007.

3. MFN 06-407, Supplement 2, 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 3.8-81 S01, 3.8-85 S01, 3.8-86 S01, 3.8-88 S01, 3.8-92 S01, 3.8-92 S01, 3.8-93 S01, 3.8-94 S01, 3.8-96 S01 and 3.8-99 S01,* dated March 26, 2007.

Enclosure:

1. MFN 06-407, Supplement 3 - Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification – DCD Tier 2 Section 3.8 – Seismic Category I Structures - RAI Numbers 3.8-28 S02, 3.8-76 S02, 3.8-93 S02, 3.8-94 S02, 3.8-96 S02, 3.8-101 S02, 3.8-102 S02 and 3.8-103 S02.

cc:	AE Cubbage	USNRC (with enclosure)
	GB Stramback	GEH/San Jose (with enclosure)
	RE Brown	GEH/Wilmington (with enclosure)
	DH Hinds	GEH/Wilmington (with enclosure)
	eDRF	0000-0068-8522

ENCLOSURE 1

MFN 06-407, SUPPLEMENT 3

Partial Response to NRC RAI Letter No. 38
Related to ESBWR Design Certification Application[1]

DCD Tier 2 Section 3.8 – Seismic Category I Structures

RAI Numbers 3.8-28 S02, 3.8-76 S02, 3.8-93 S02, 3.8-94 S02, 3.8-96 S02, 3.8-101 S02, 3.8-102 S02 & 3.8-103 S02

[1] Original Response, Supplement 1 and Supplement 2 previously submitted under MFNs 06-407, 06-407S1 and 06-407S2 without DCD updates are included to provide historical continuity during review.

NRC RAI 3.8-28

Provide additional details for the containment mechanical and electrical penetrations (other than Main Steam and Feedwater), including the number, types, geometry, analytical models used, loading, summary of results, comparison to Code allowables, and the current status of the design. Is the design final for all penetrations, or is this a COL applicant responsibility? If a COL applicant responsibility, where is this identified in the DCD? Include this information in DCD Section 3.8.2 and/or Appendix 3G.

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

Details for the containment mechanical and electrical penetrations (other than Main Steam and Feedwater) are not currently available and will be developed after the routing of piping and commodities, such as cable trays, ducts, etc., is laid out during detailed design. These containment penetrations will be designed to meet the ASME code.

DCD Tier 2 Subsection 3.8.2.4.1.3 will be revised in the next update as noted in the attached markup.

NRC RAI 3.8-28, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

Is this identified as a COL action item? Are any typical details available for staff review?

GE Response

Typical details for the containment mechanical and electrical penetrations will be added to DCD Tier 2.

DCD Impact

DCD Tier 2 Subsection 3.8.2.1.3 will be revised in the next update as noted in the attached markup.

DCD Tier 2 Section 3.8 will be revised in the next update as noted in the attached markups to include Figures 3.8-6 through 3.8-11.

NRC RAI 3.8-28, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The staff reviewed the applicant's proposed DCD changes and found them acceptable to address the design of typical mechanical and electrical penetrations. The staff confirmed that the proposed changes were incorporated in a formally submitted revision to the DCD. However, the staff is unclear how the typical design details will be implemented at the COL application stage. Therefore, the staff requests the applicant to identify and describe the COL action item to implement the typical mechanical and electrical penetration design details. This information needs to be documented in the DCD.

GEH Response

These mechanical and electrical penetrations are part of the containment boundary for which design commitment is an ITAAC as delineated in DCD Tier 1 Revision 4 Table 2.15.1-2, Item 2. Therefore, no COL action item is required in DCD Tier 2 for implementation of typical design details.

DCD Impact

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

NRC RAI 3.8-76

Provide information on Materials, Quality Control, and Special Construction Techniques for Other Seismic Category I Structures. This information is normally included in Section 3.8.4.6, but has not been provided in the ESBWR DCD. SRP 3.8.4 provides guidance as to the type of information that the staff expects to review.

Include this information in a new DCD Section 3.8.4.6.

GE Response

GE agrees to include the information on Materials, Quality Control, and Special Construction Techniques for Other Seismic Category I Structures in the DCD.

DCD Tier 2 Subsection 3.8.4.6 will be added as noted in the attached markup.

NRC RAI 3.8-76, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

SRP 3.8.4.I.6 requires, by reference to SRP 3.8.3.I.6, that welding of reinforcing bars (splices) comply with the applicable sections of the ASME Section III, Division 2, Code. The proposed markup in DCD Section 3.8.4.6 currently proposes to meet ACI 349-01 and applicable Regulatory Guides for splices. This needs to be addressed.

During the audit, GE agreed to revise the DCD to reflect the requirement of SRP 3.8.4.I.6 and 3.8.3.I.6.

GE Response

Welded bar splices are not intended to be used in the ESBWR design. If they are used, inspection and documentation requirements conform to ASME Code Section III, Division II consistent with SRP 3.8.3.I.6.c, which will be reflected in DCD Tier 2 Section 3.8.4.6.4.

DCD Impact

DCD Tier 2 Subsection 3.8.4.6.4 will be revised in the next update as noted in the attached markup.

NRC RAI 3.8-76, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The applicant's response regarding welding of reinforcing bars is too limited since it only adds a sentence under ESBWR DCD Section 3.8.4.6.4 Quality Control that states: For welding of reinforcing bars, inspection and documentation requirements conform to ASME Code Section III, Division 2 also. The staff's position is that welding of reinforcing bars should comply with all the applicable sections of the ASME Section III, Division 2, Code, not just those associated with inspection and documentation. This position applies to all seismic Category I concrete structures inside and outside containment. DCD Sections 3.8.3 and 3.8.4 need to be revised to address this position.

GEH Response

Welding of reinforcing bars complies with all the applicable requirements of ASME Code Section III, Division 2. This statement is included in DCD Tier 2 Revision 4 Subsection 3.8.4.6.3. The revised page 3.8-38 in DCD Tier 2 Revision 4 is attached. Welding of reinforcing bars is not applicable to DCD Tier 2 Subsection 3.8.3 because none of the structures included in this DCD Subsection are of reinforced concrete construction.

DCD Impact

No DCD change is 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.

No DCD change will be made in response to this RAI.

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

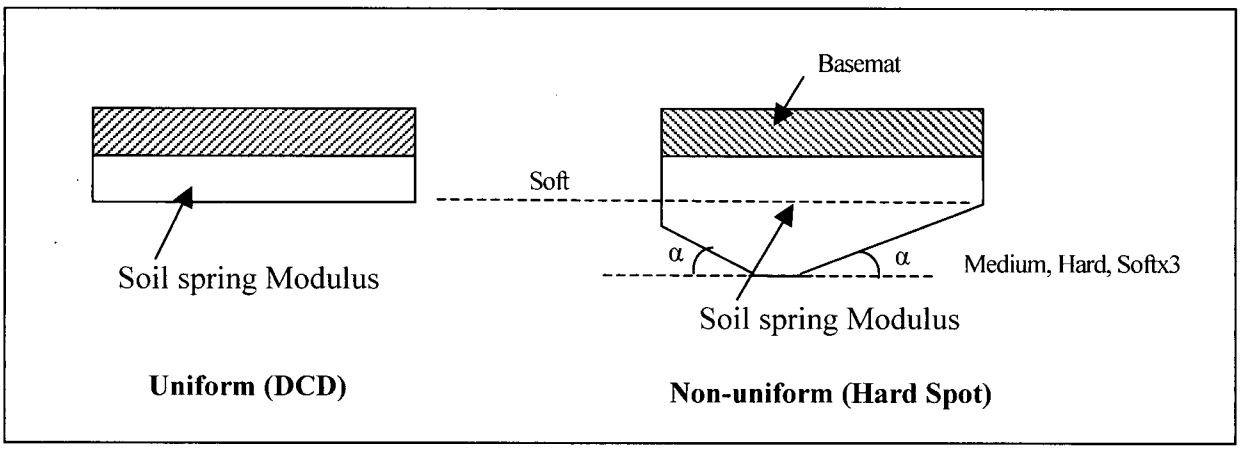


Figure 3.8-93(1) Soil conditions

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. 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.

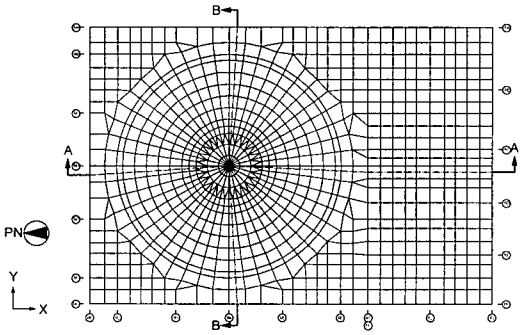
1. FB area poured (below the floor EL-6400, approximately)
2. Add exterior walls in RB
3. Add RCCV and B3F structure
4. Add pedestal

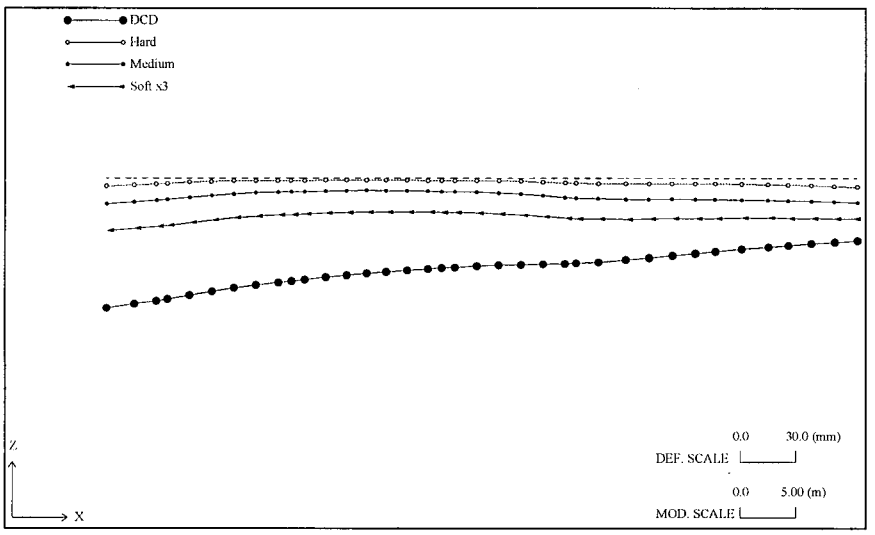
The analytical model has been extracted from global FEM model used for DCD design. Some modification has been applied to this as shown in Figure 3.8-93(4). This model is similar to the "Modified Truncated Model" provided to the NRC for confirmatory analysis. The height of structural members is limited 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:

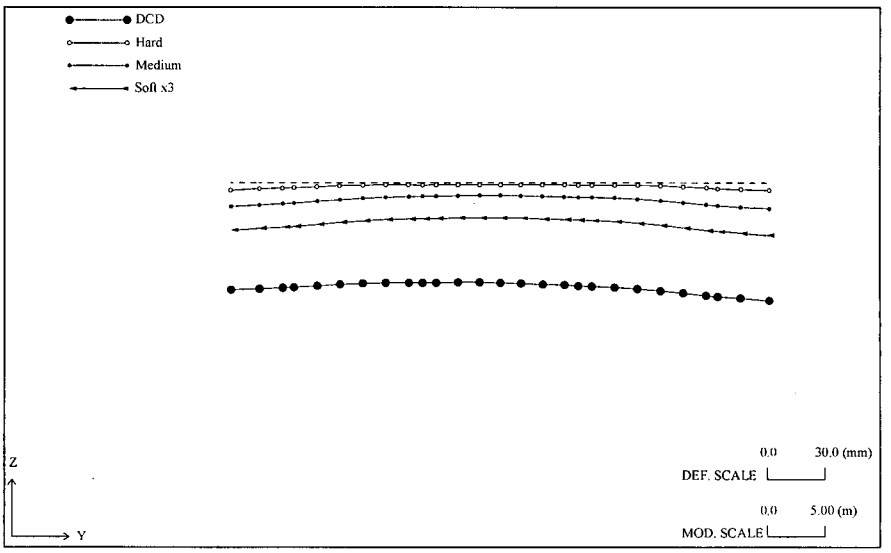
- FEM Model : Based on the "Modified Truncated Model (a part of Global FEM Model).
- Soil Condition : "Uniform", soft soil spring under the basemat
- Load : Dead Load

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.



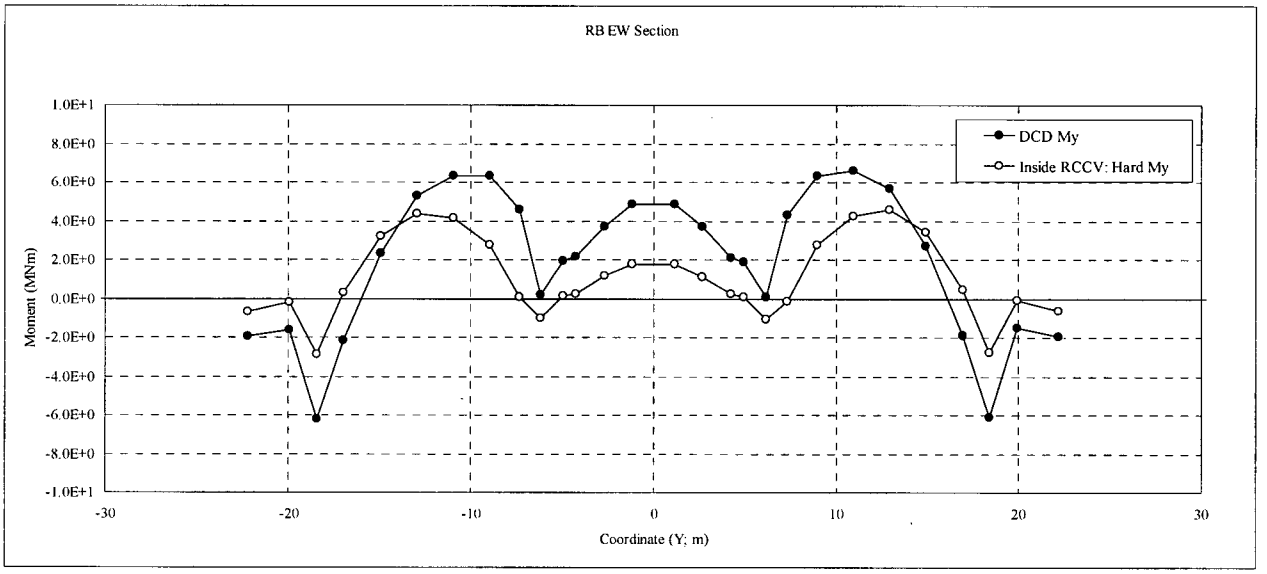


a) A-A Section

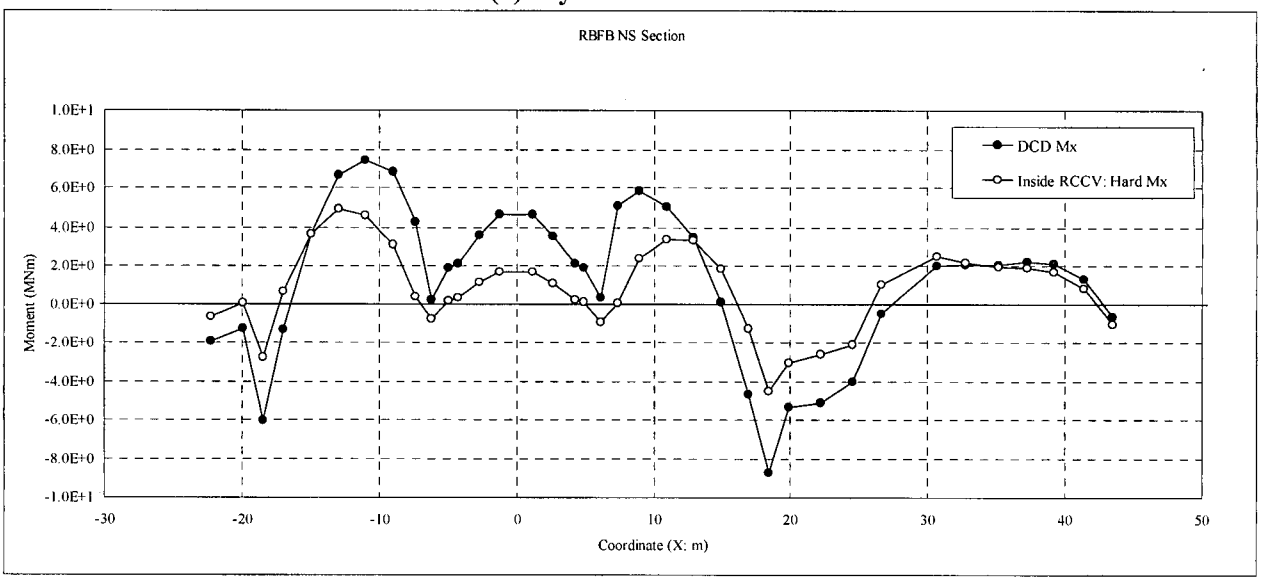


b) B-B Section

Figure 3.8-93(2) Comparison of Basemat Deformation



(a) My in B-B Section



(b) Mx in A-A Section

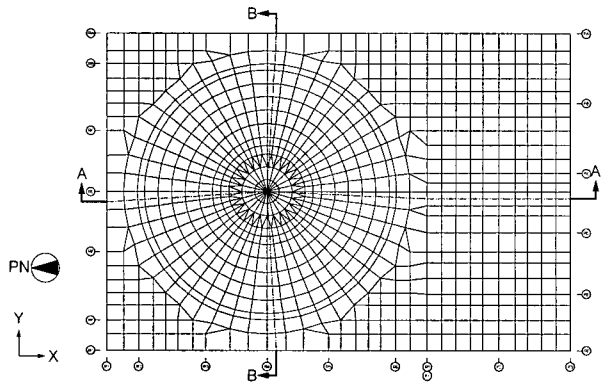
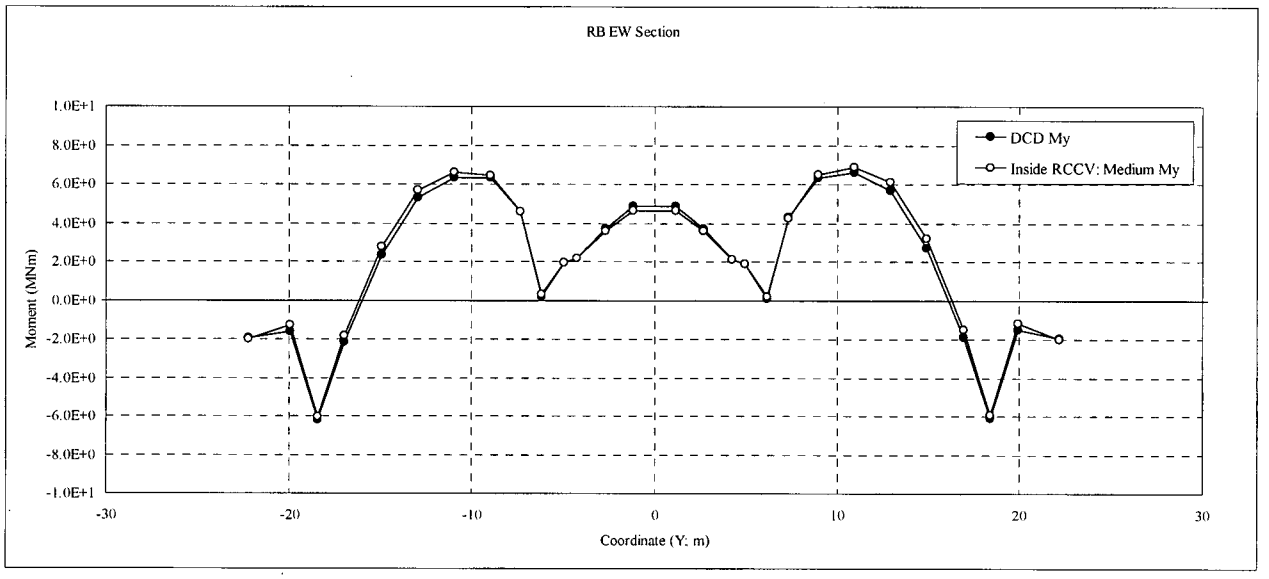
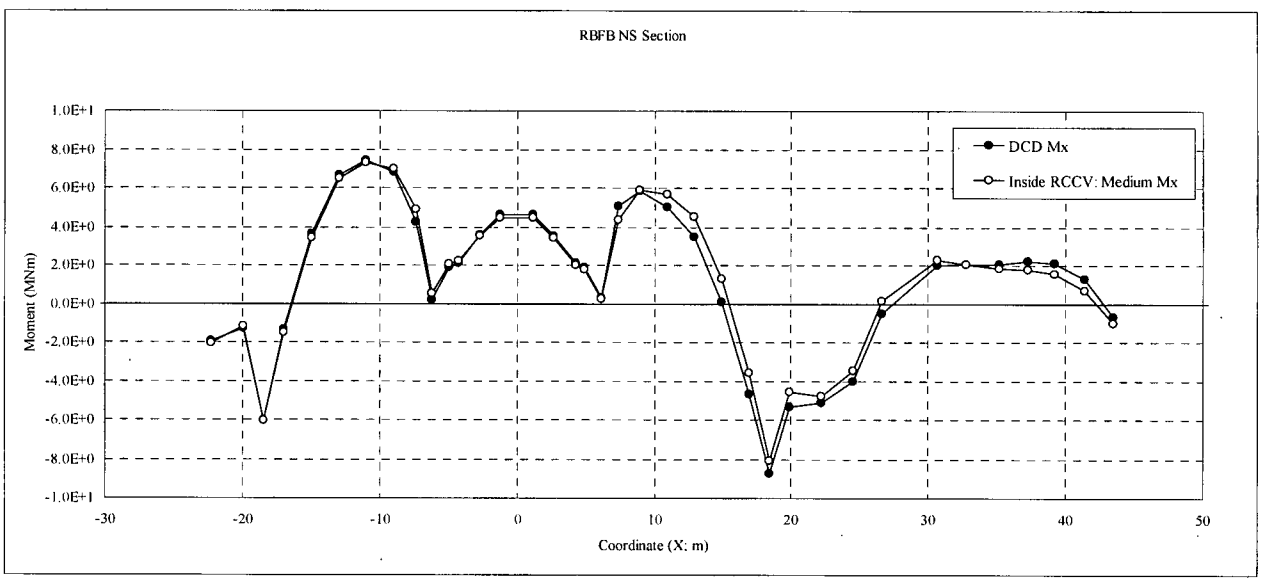


Figure 3.8-93(3)-a Comparison of Basemat Sectional Moments (Hard)



(a) My in B-B Section



(b) Mx in A-A Section

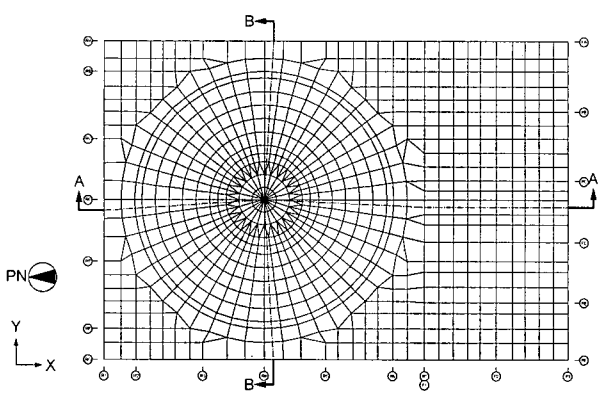
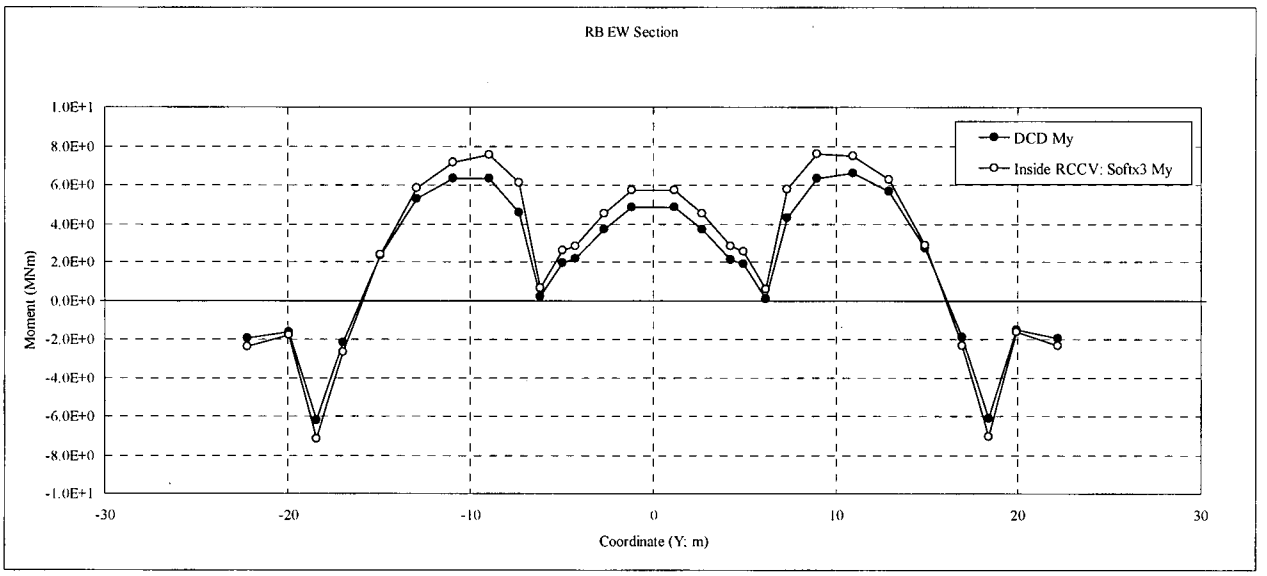
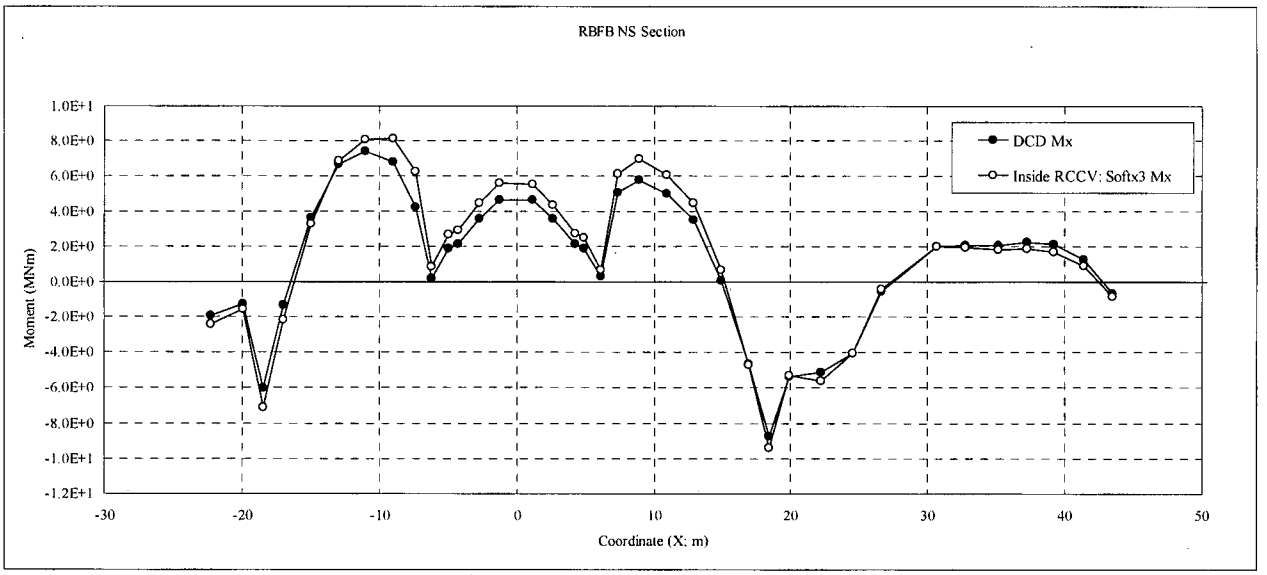


Figure 3.8-93(3)-b Comparison of Basemat Sectional Moments (Medium)



(a) My in B-B Section



(b) Mx in A-A Section

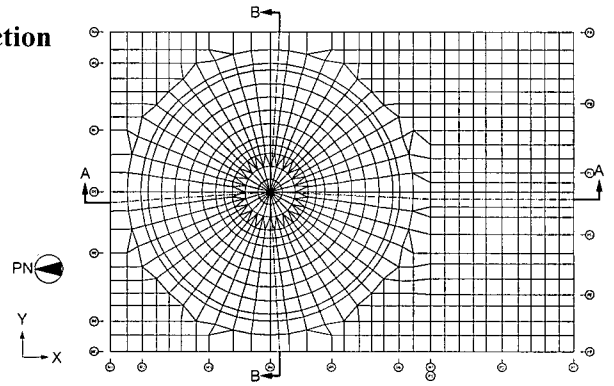


Figure 3.8-93(3)-c Comparison of Basemat Sectional Moments (soft x3)

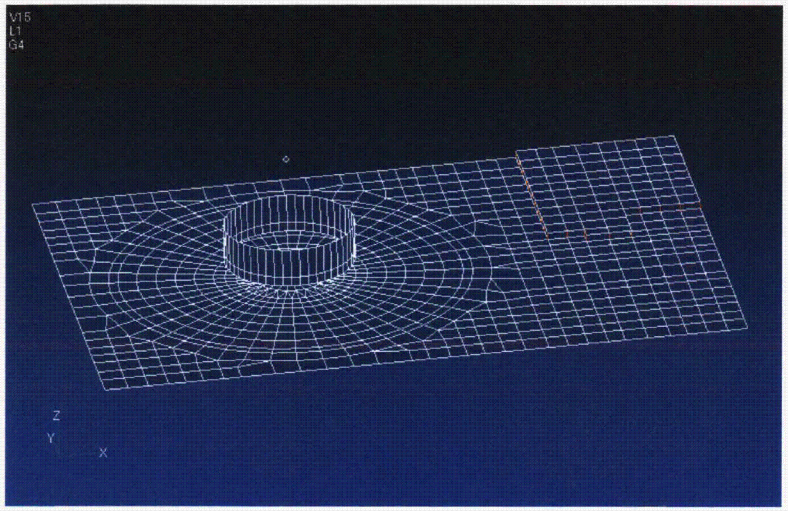


Figure 3.8-93(4)-a Case A (Step 1)

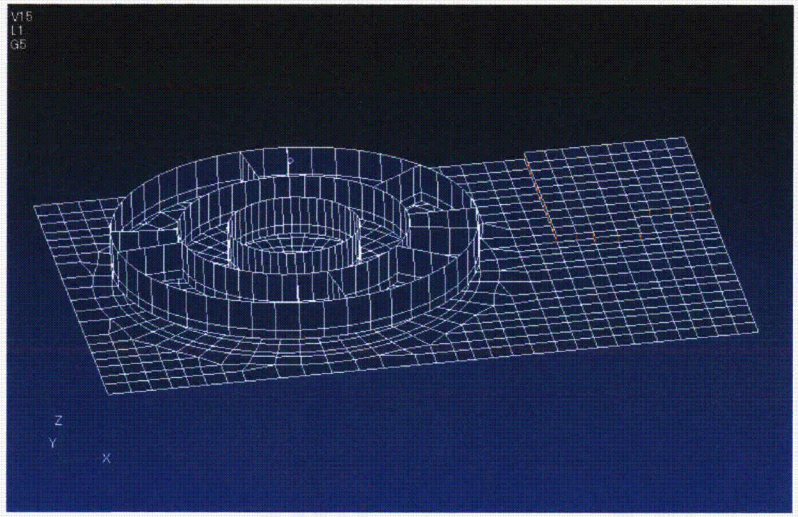


Figure 3.8-93(4)-b Case A (Step 2)

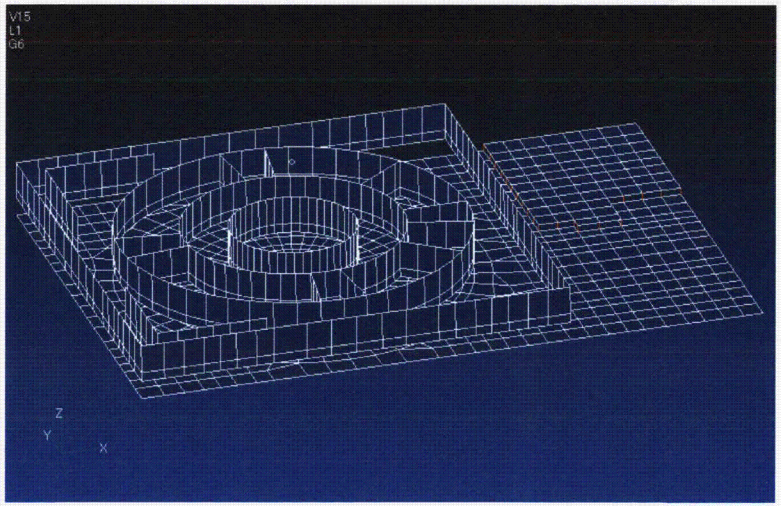


Figure 3.8-93(4)-c Case A (Step 3)

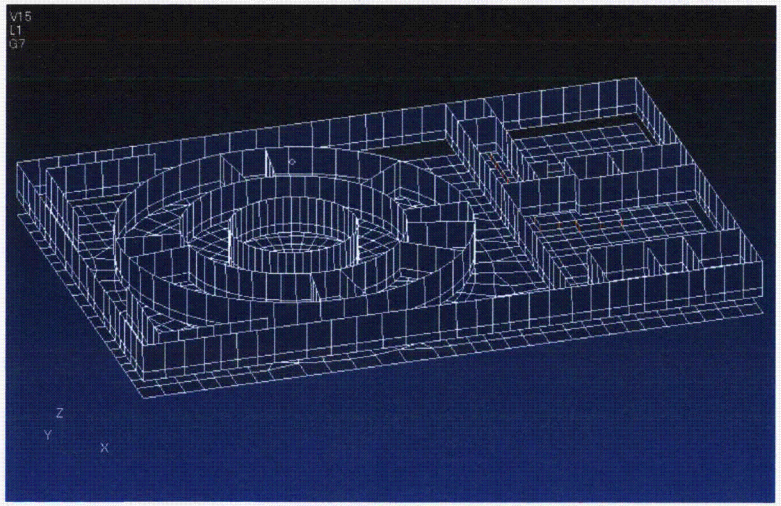


Figure 3.8-93(4)-d Case A (Step 4)

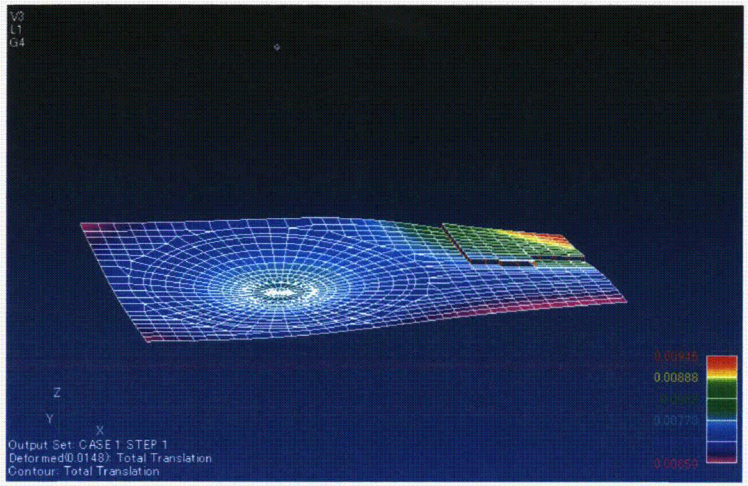


Figure 3.8-93(5) -a Deformation of basemat (CaseA - Step 1)

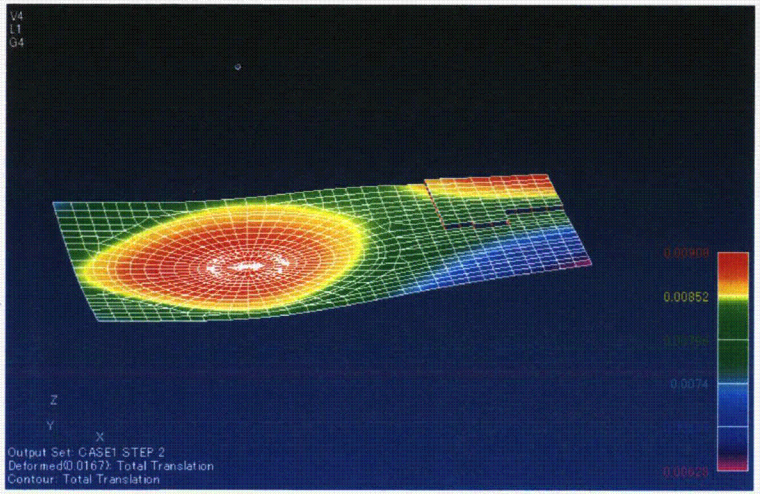


Figure 3.8-93(5) -b Deformation of basemat (CaseA - Step 2)

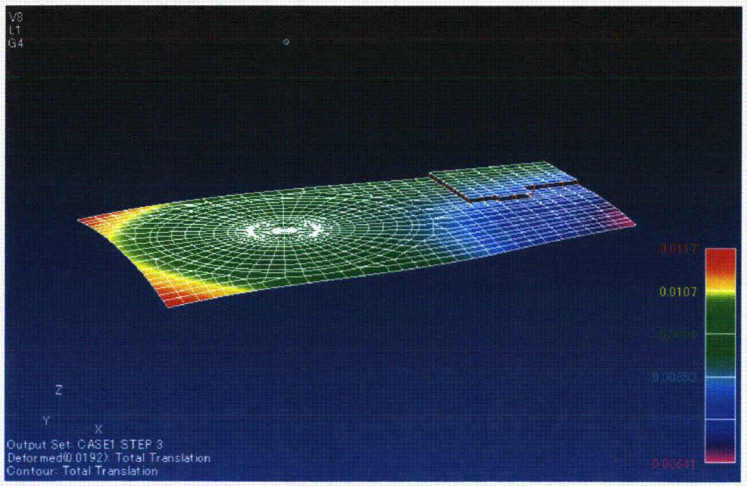


Figure 3.8-93(5) -c Deformation of basemat (CaseA - Step 3)

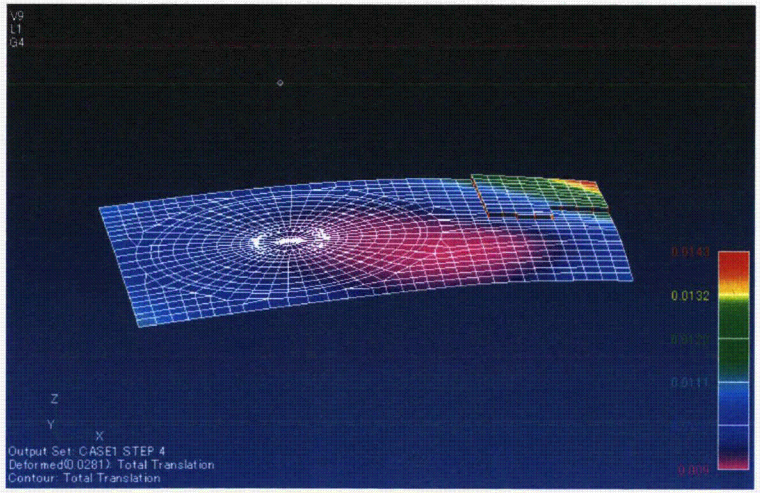


Figure 3.8-93(5) -d Deformation of basemat (CaseA - Step 4)

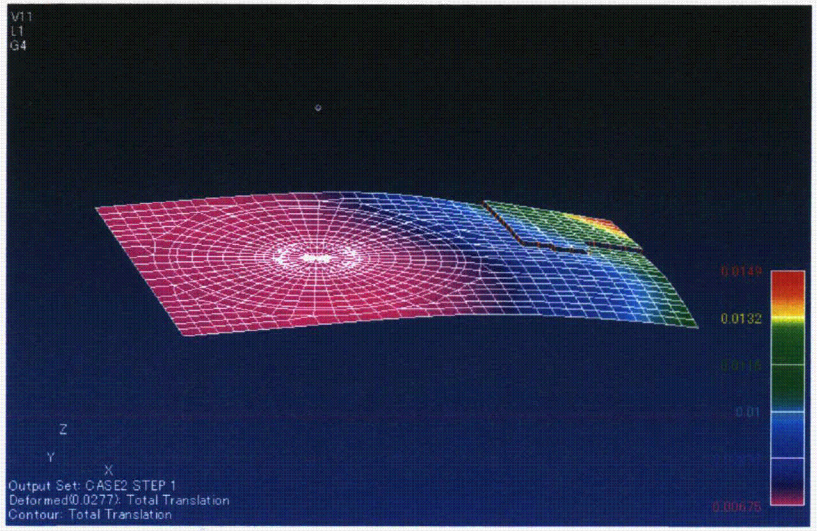


Figure 3.8-93(6) -a Deformation of basemat (CaseB - Step 1)

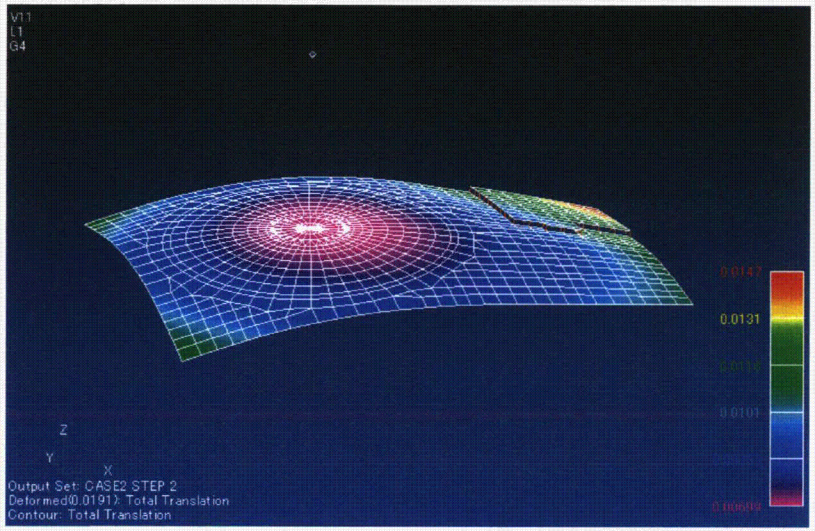


Figure 3.8-93(6) -b Deformation of basemat (CaseB - Step 2)

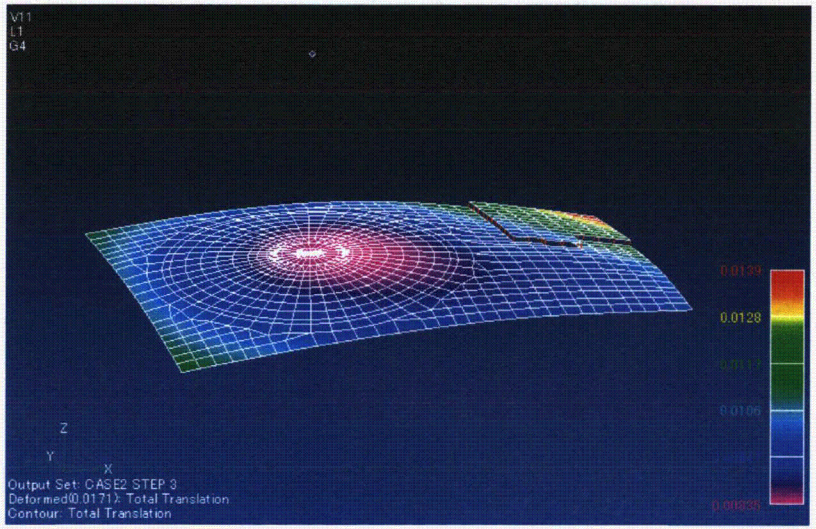


Figure 3.8-93(6) -c Deformation of basemat (CaseB - Step 3)

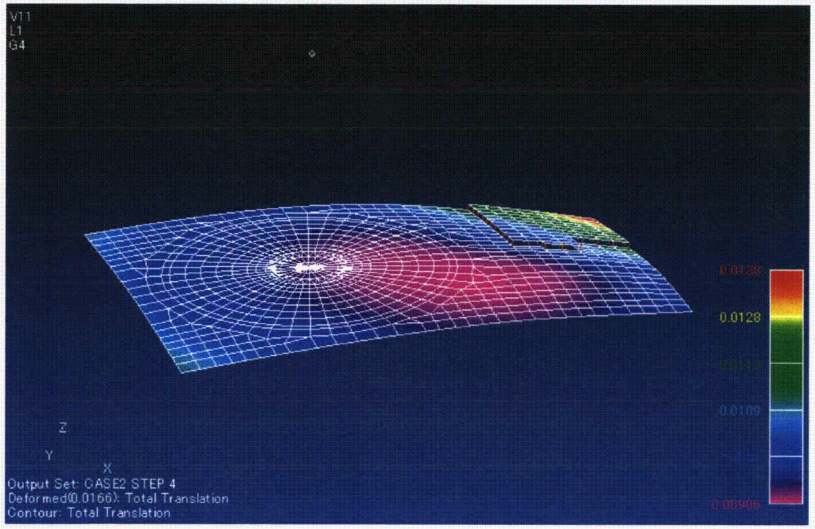
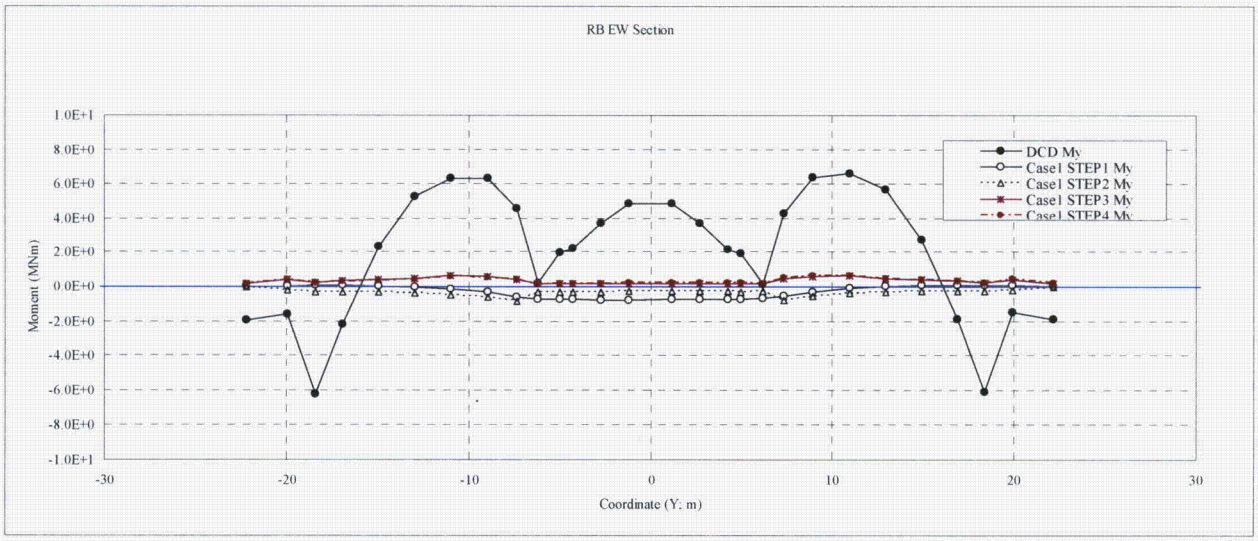
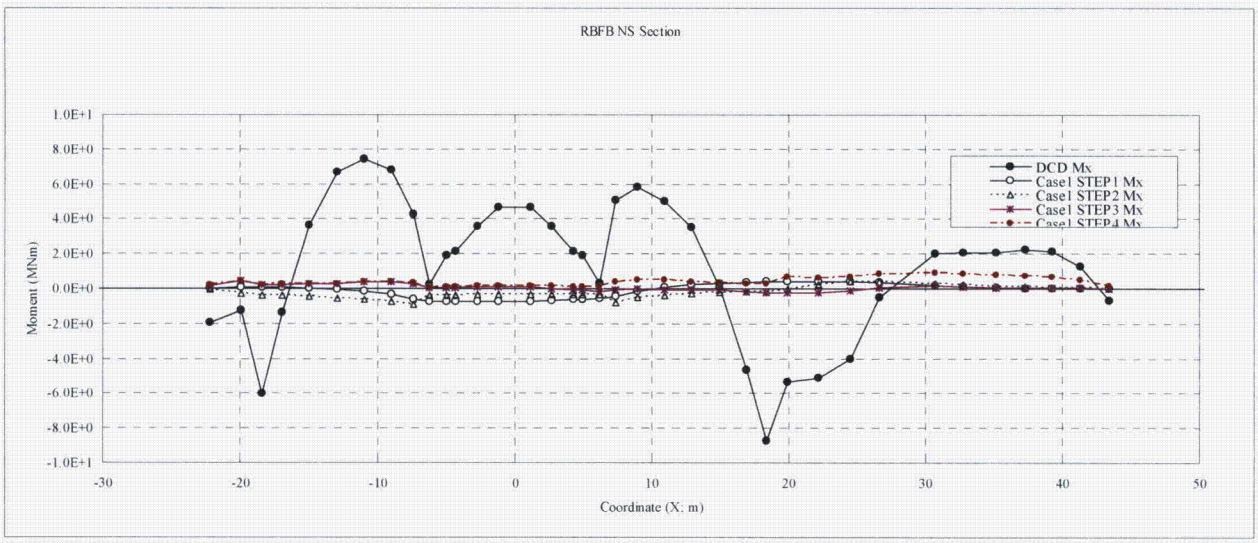


Figure 3.8-93(6) -d Deformation of basemat (CaseB - Step 4)





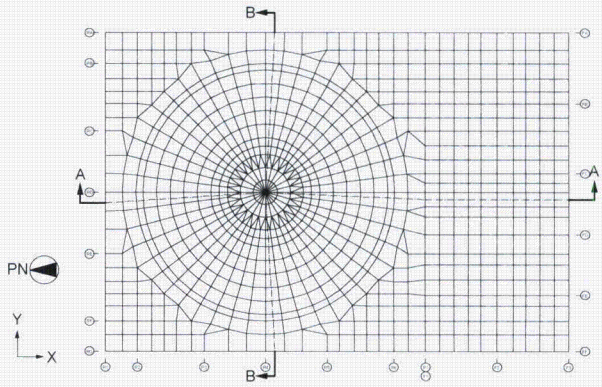
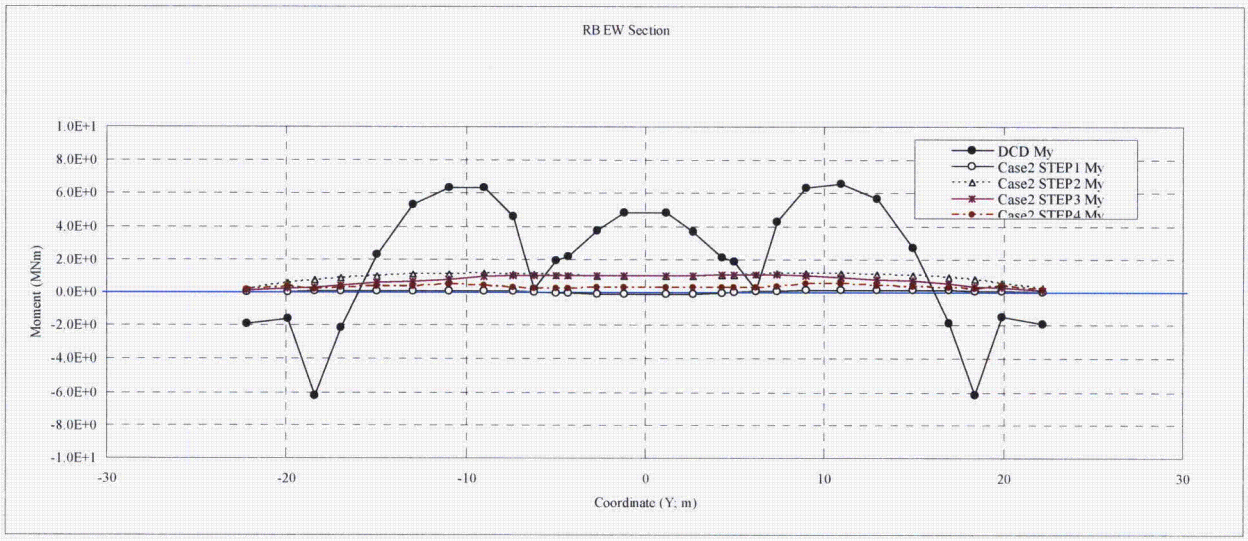
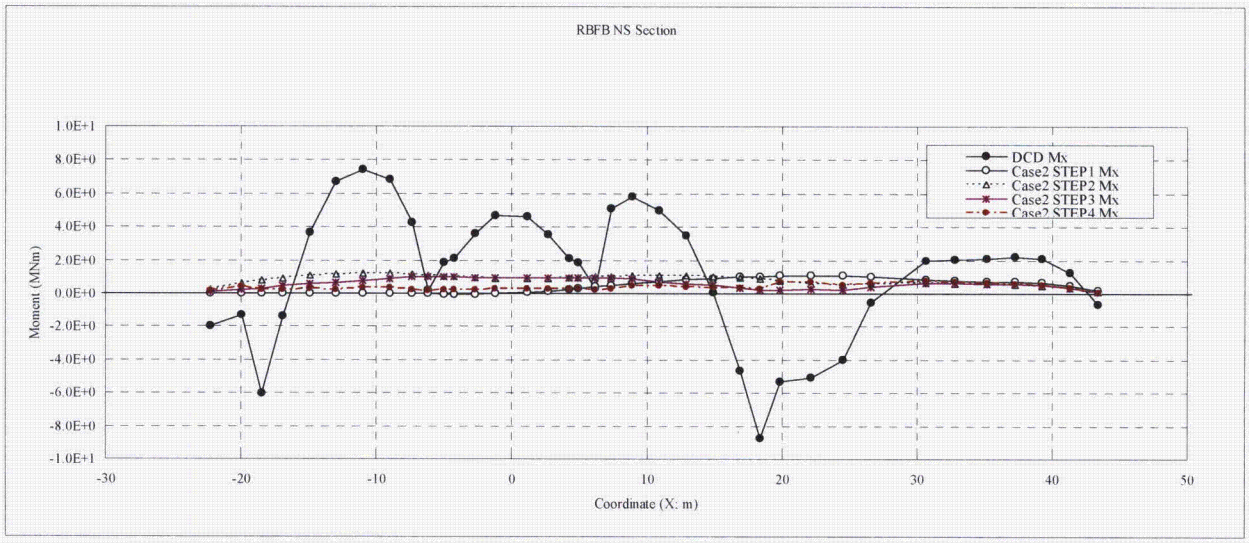


Figure 3.8-93(7) Comparison of Basemat Sectional Moments (Case A)





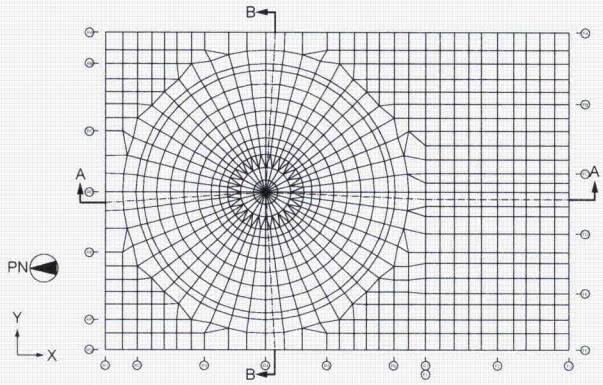


Figure 3.8-93(8) Comparison of Basemat Sectional Moments (Case B)

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

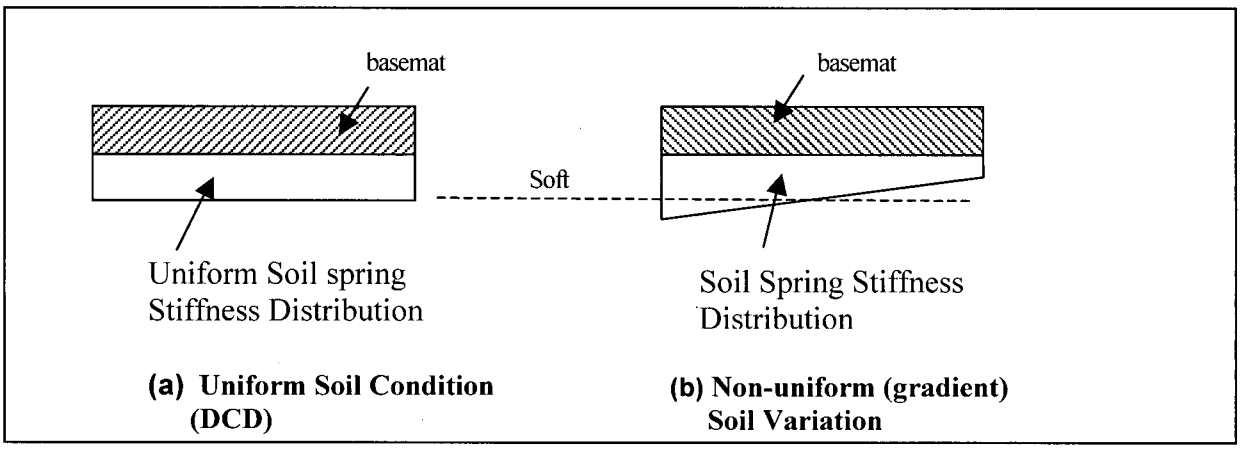


Figure 3.8-93 (9) Soil Conditions

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.)	

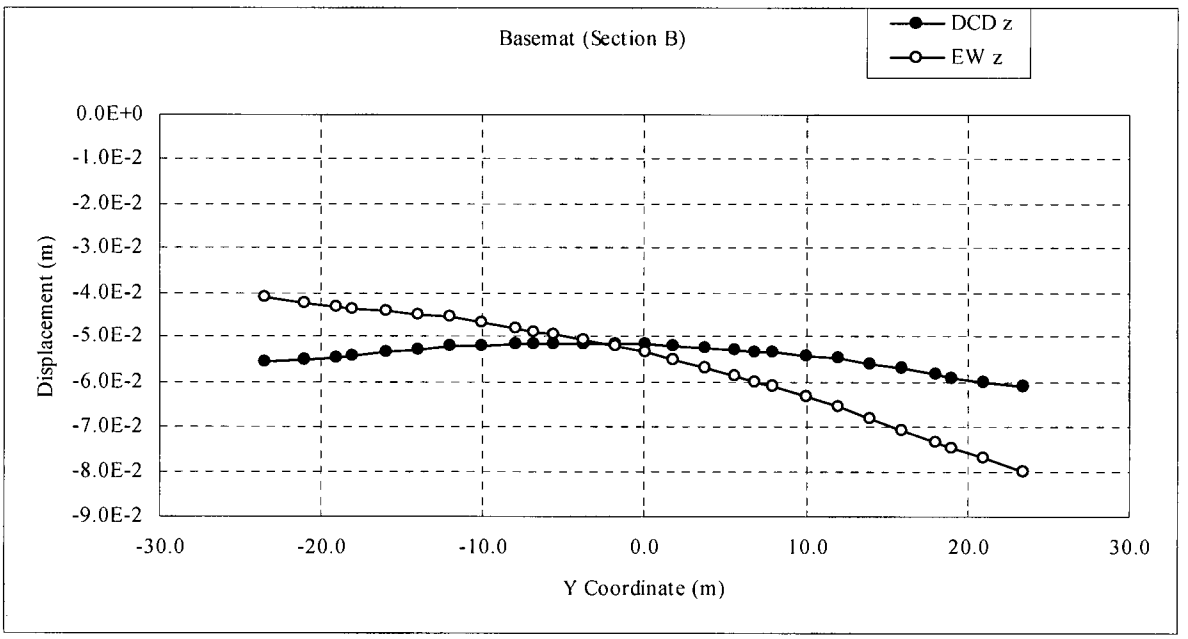


Figure 3.8-93 (10)-a Comparison of Settlement (RBFB, E-W direction)

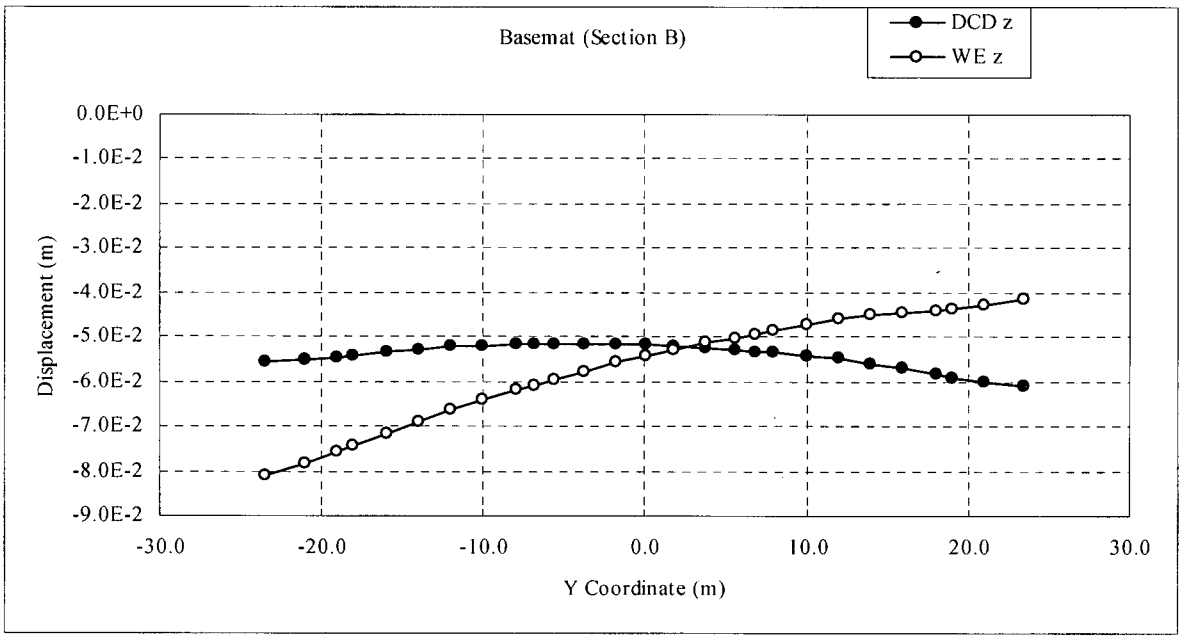
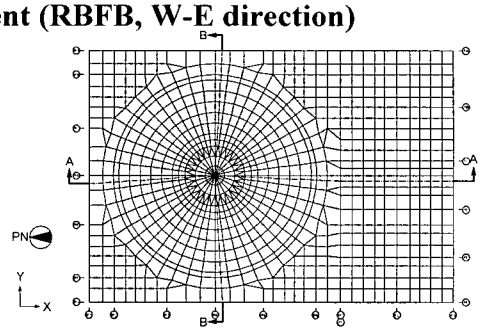


Figure 3.8-93 (10)-b Comparison of Settlement (RBFB, W-E direction)



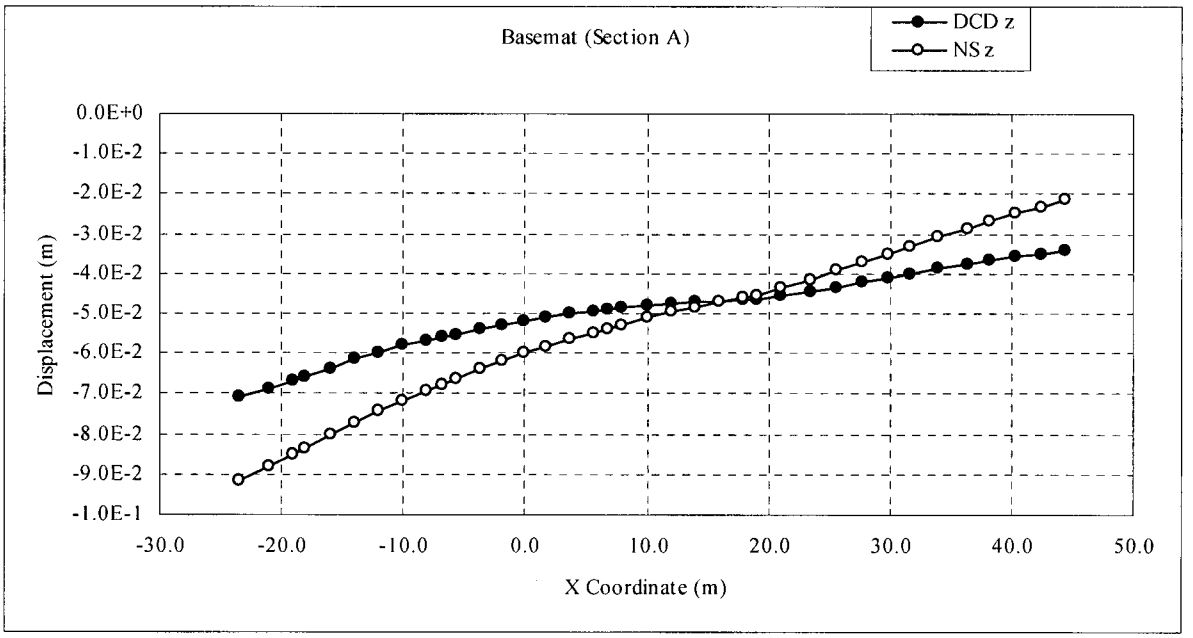


Figure 3.8-93 (10)-c Comparison of Settlement (RBFB, N-S direction)

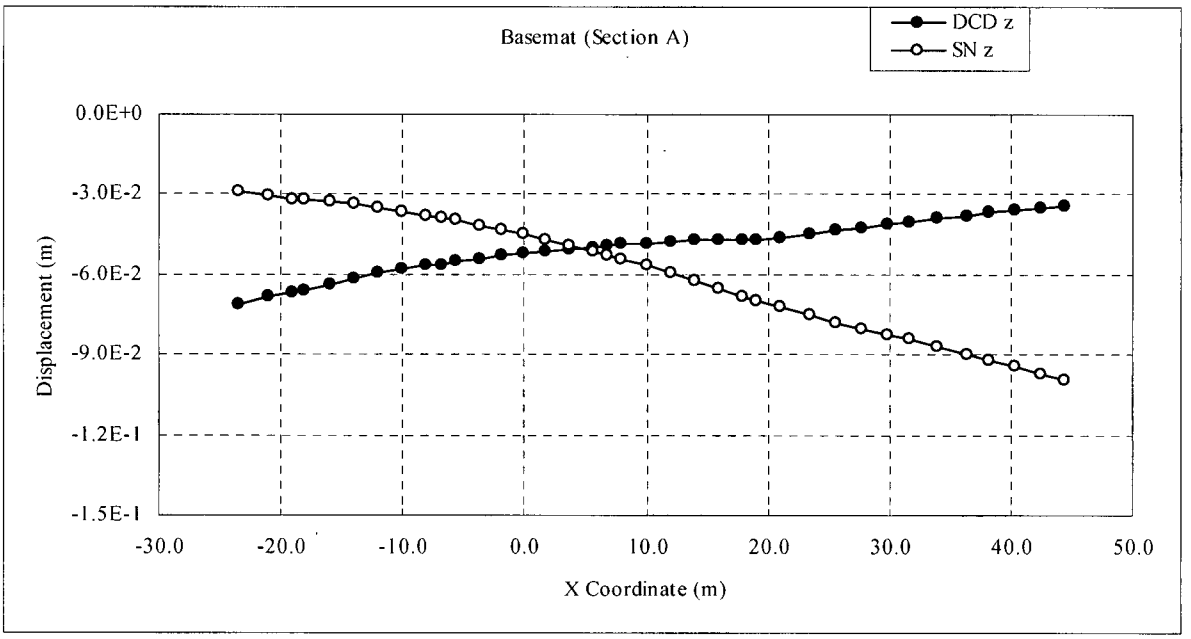
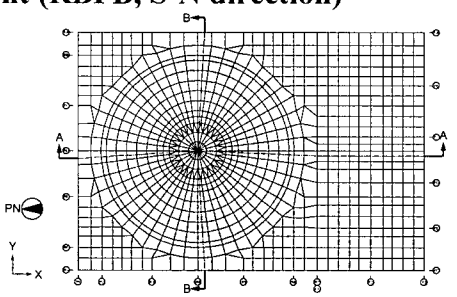


Figure 3.8-93 (10)-d Comparison of Settlement (RBFB, S-N direction)



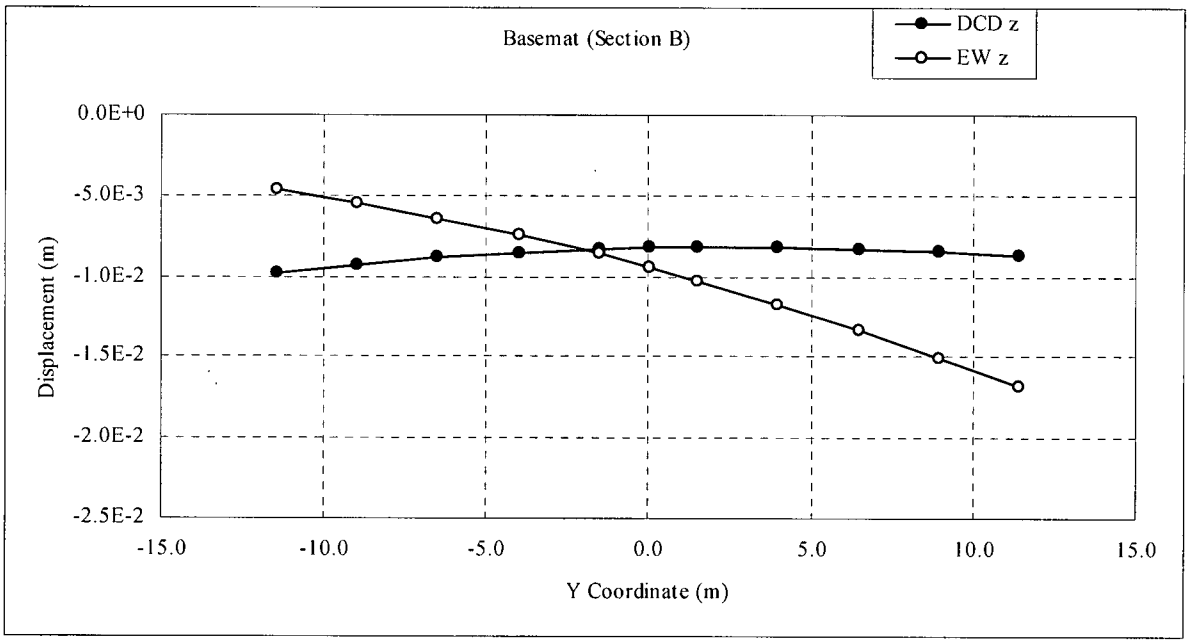


Figure 3.8-93 (11)-a Comparison of Settlement (CB, E-W direction)

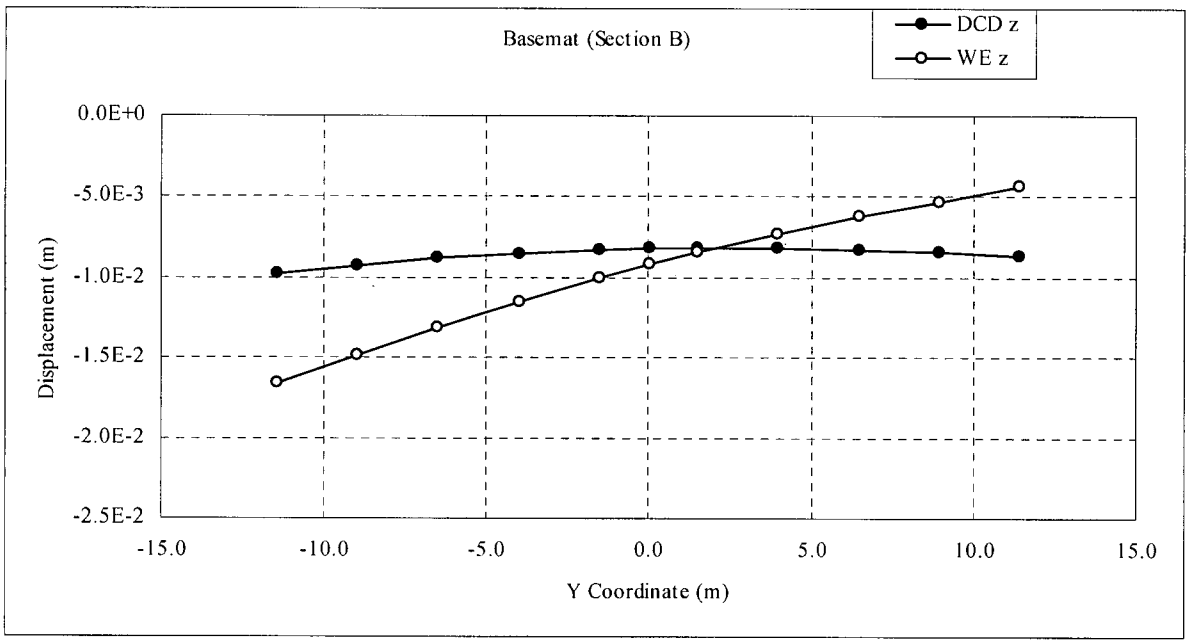
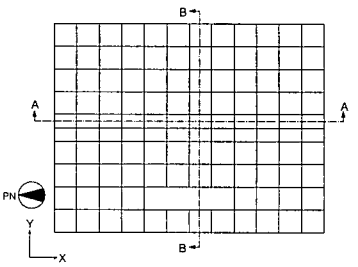


Figure 3.8-93 (11)-b Comparison of Settlement (CB, W -E direction)



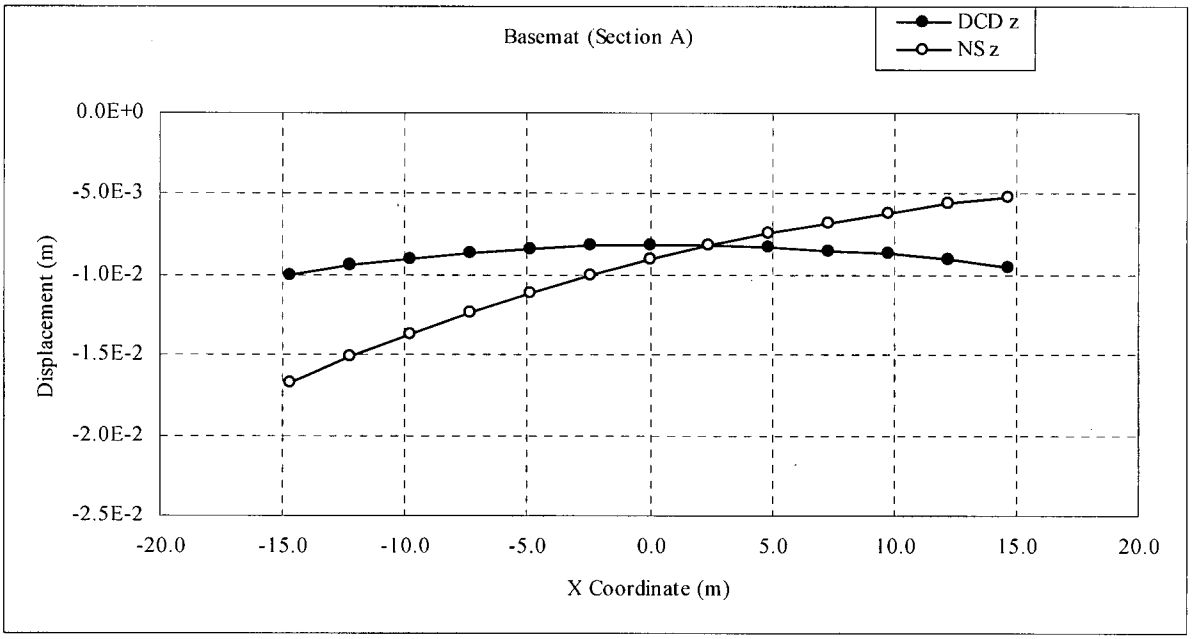


Figure 3.8-93 (11)-c Comparison of Settlement (CB, N-S direction)

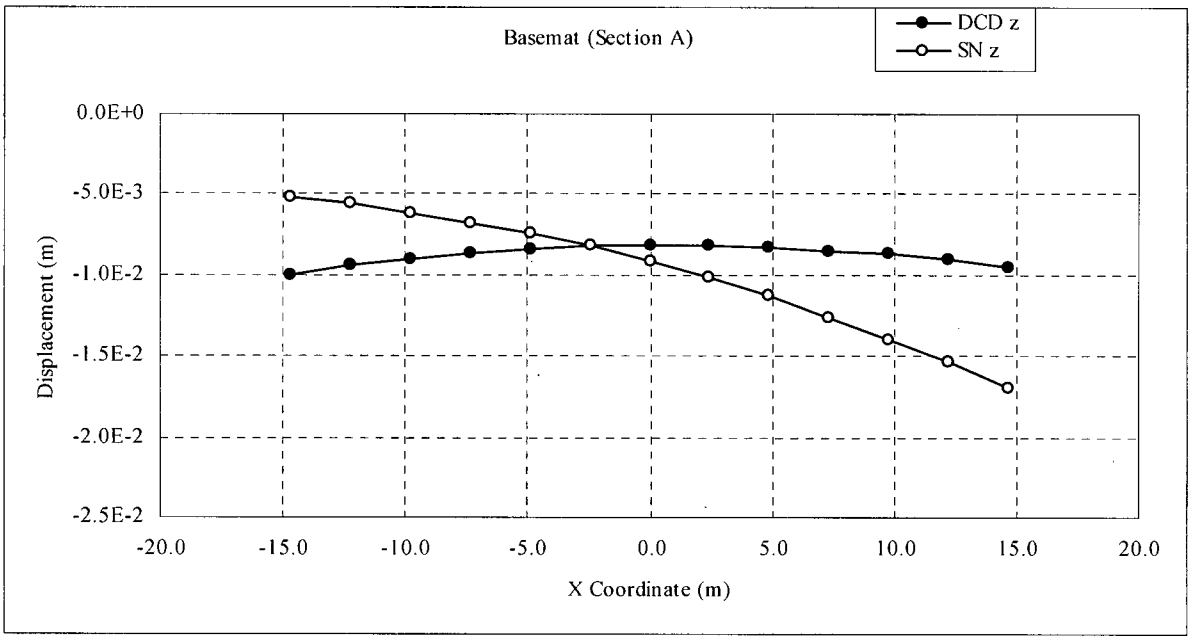
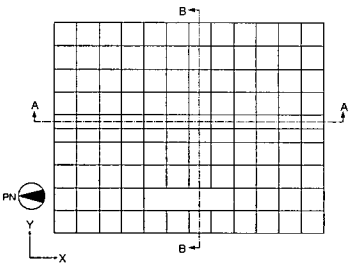
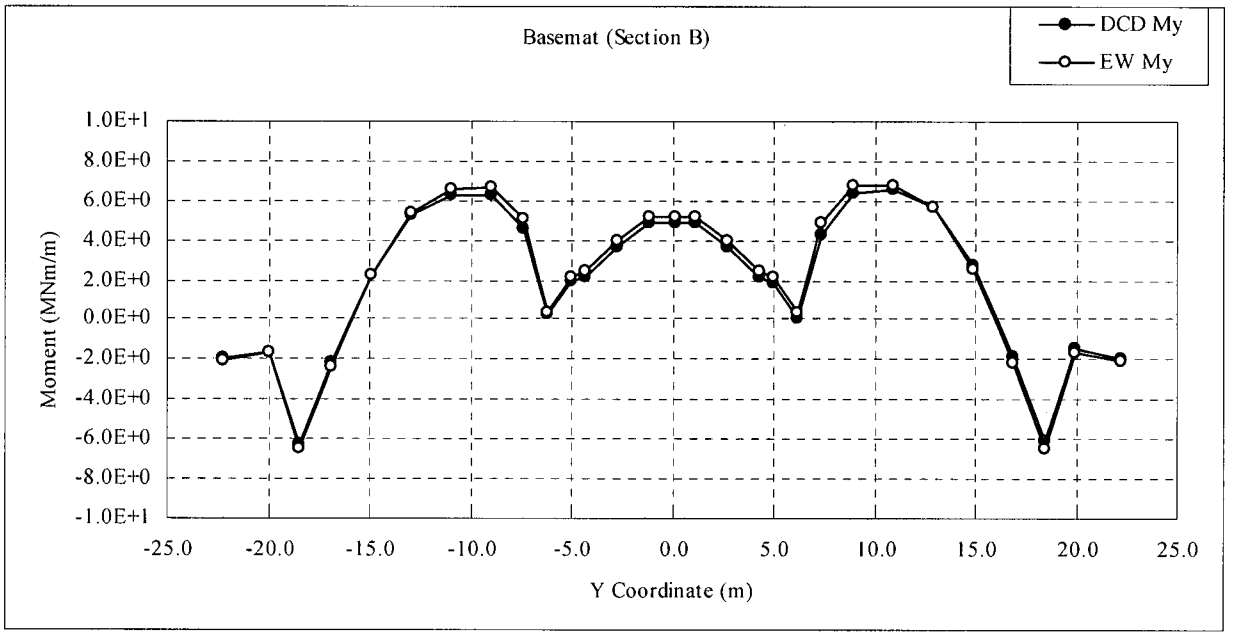
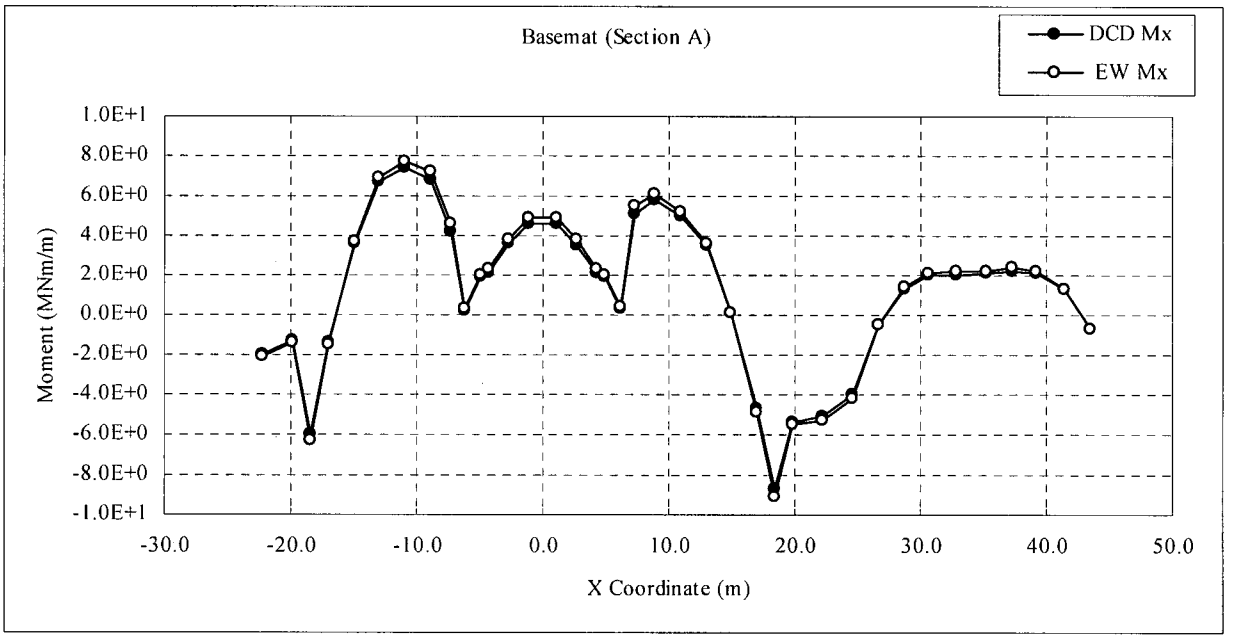


Figure 3.8-93 (11)-d Comparison of Settlement (CB, S-N direction)



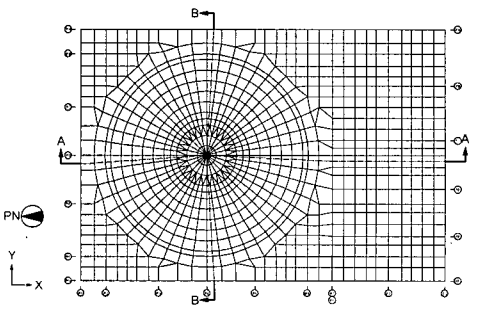


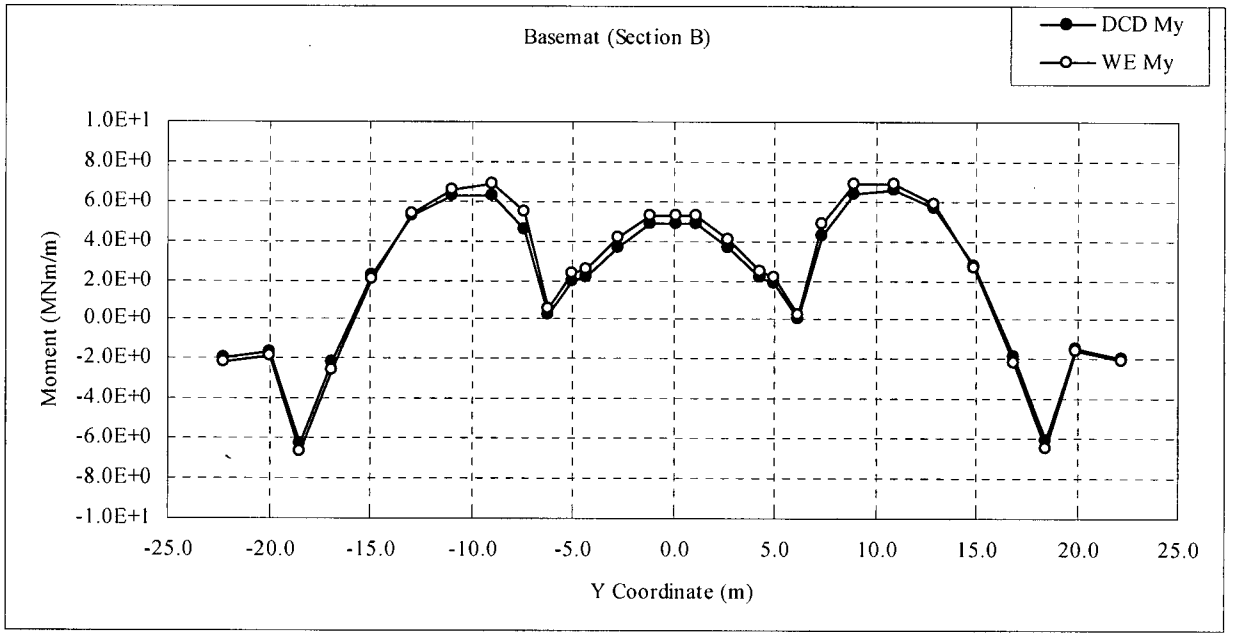
(a) My in E-W Section



(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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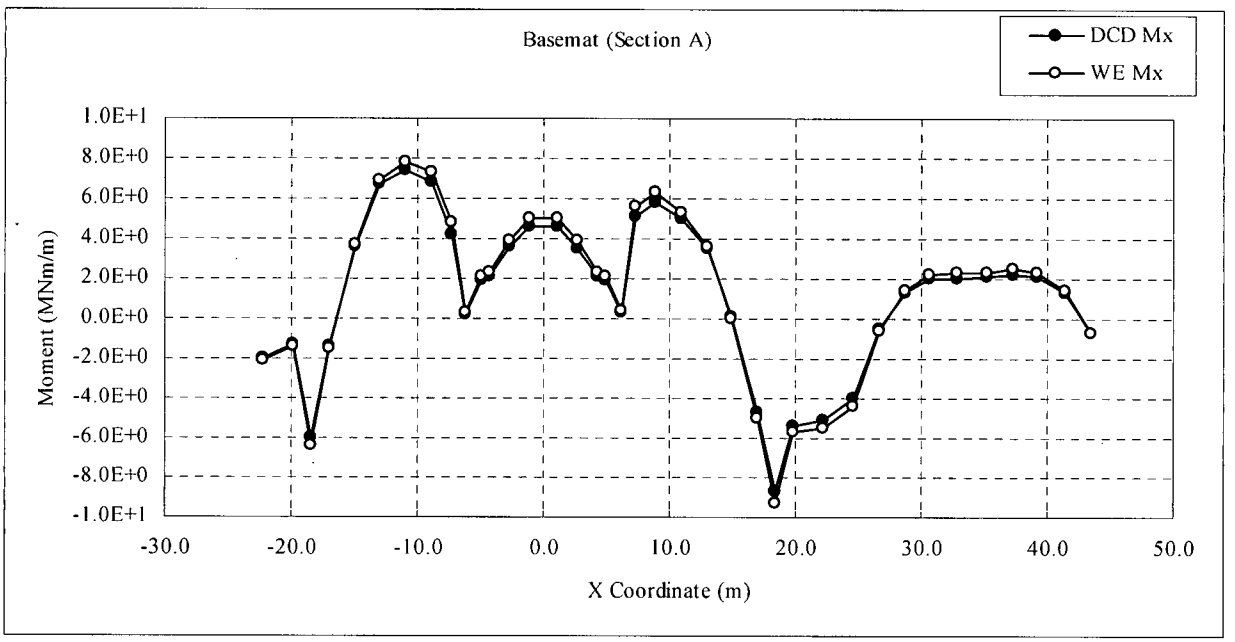
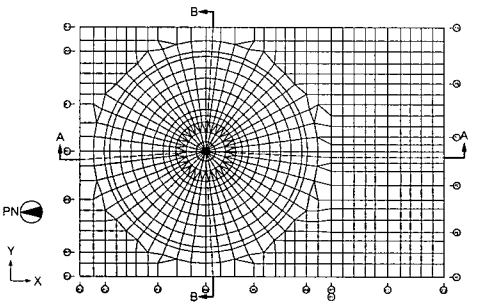
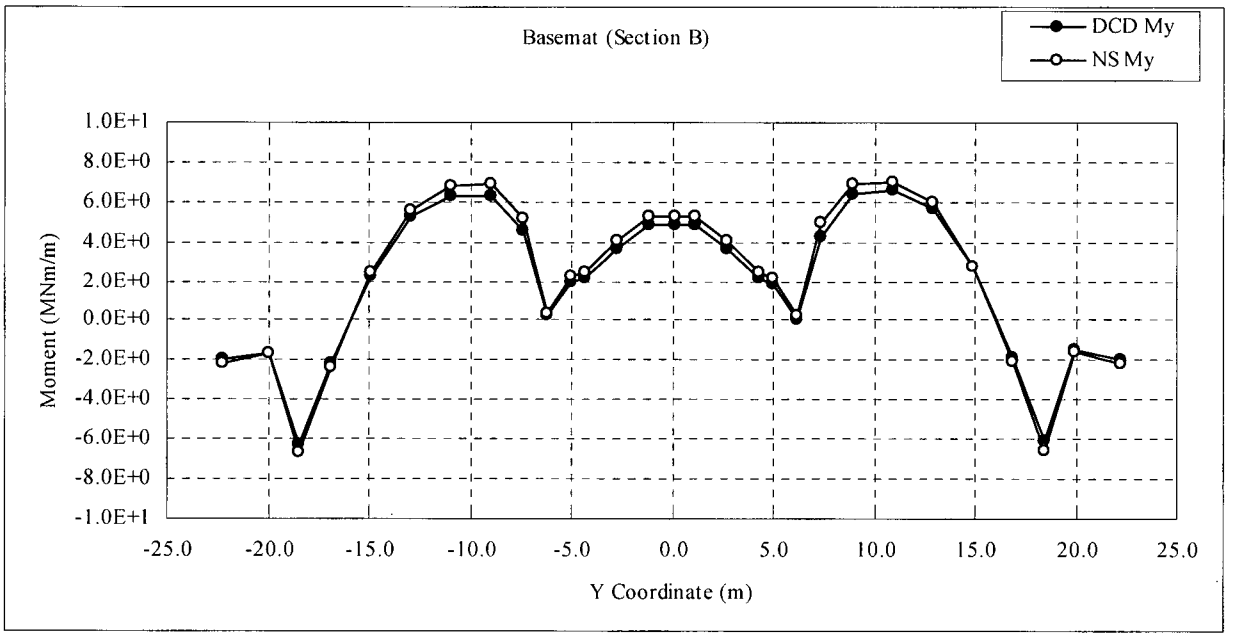


(b) Mx in N-S Section

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





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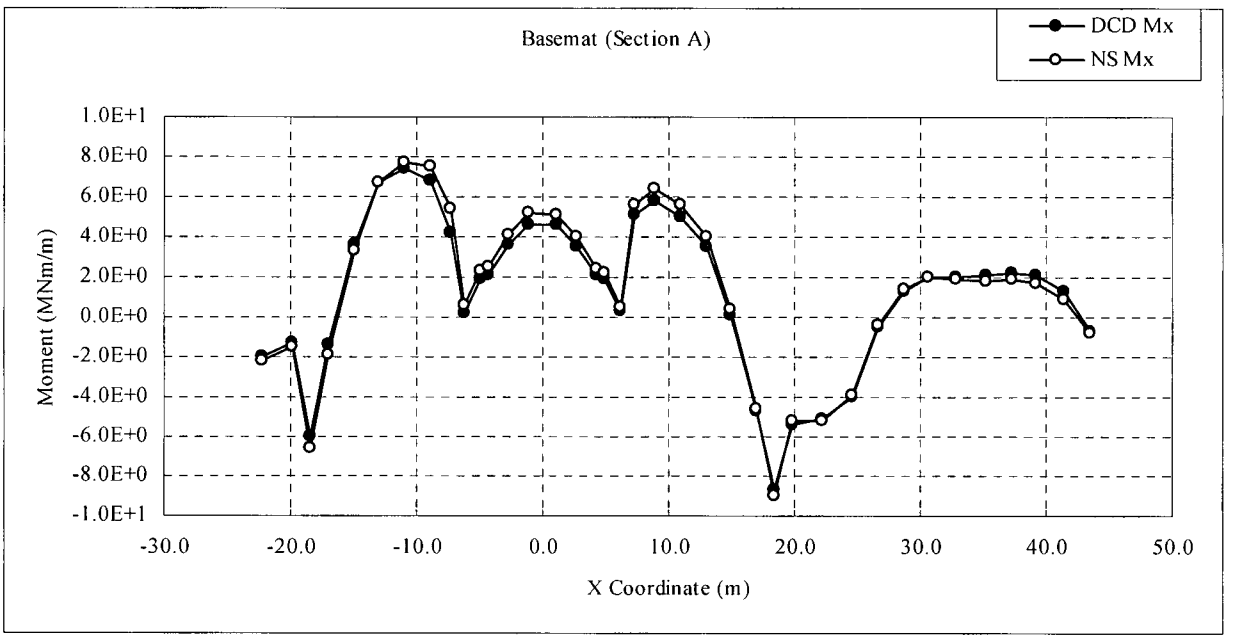
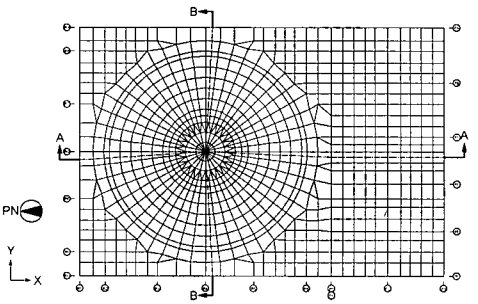
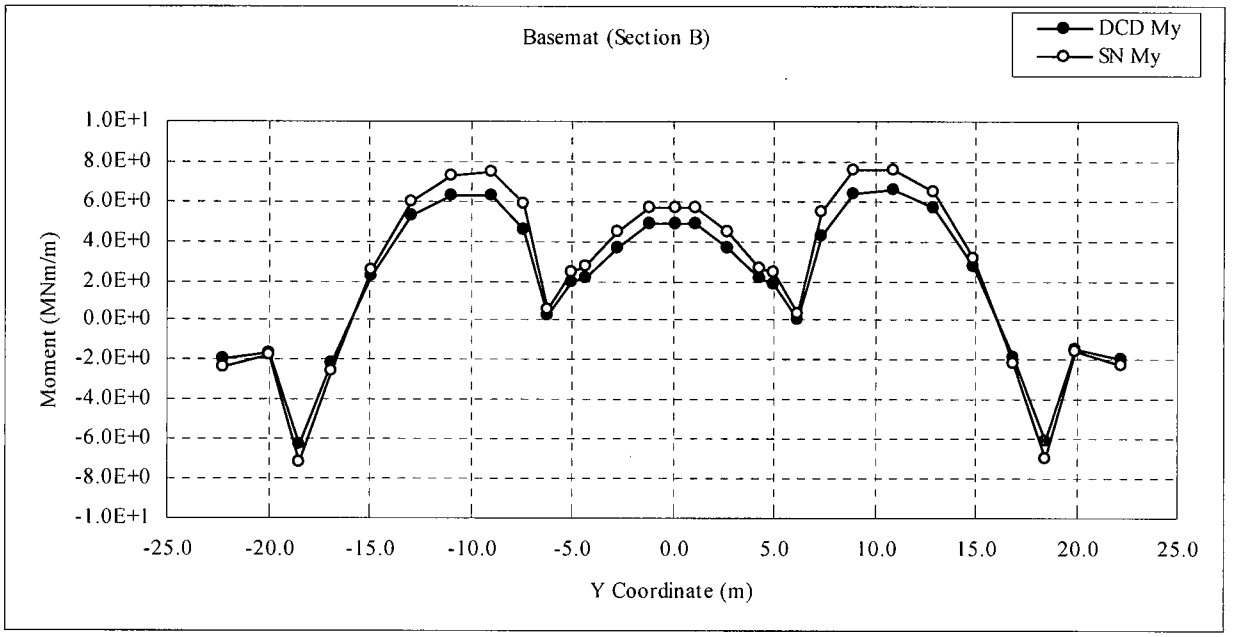


(b) Mx in N-S Section

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





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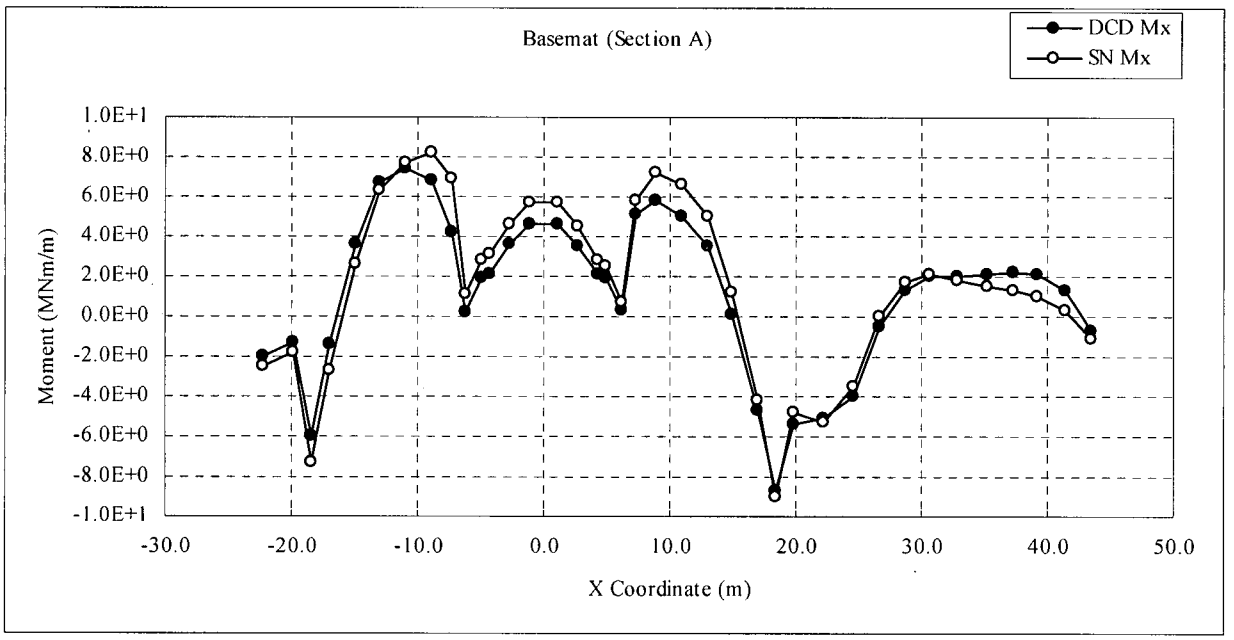
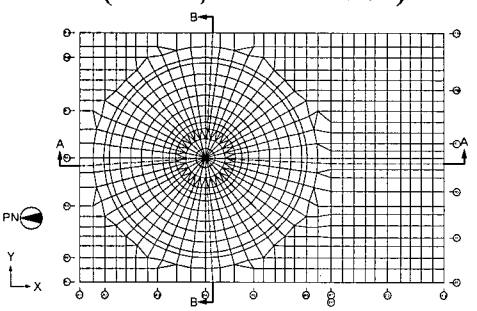
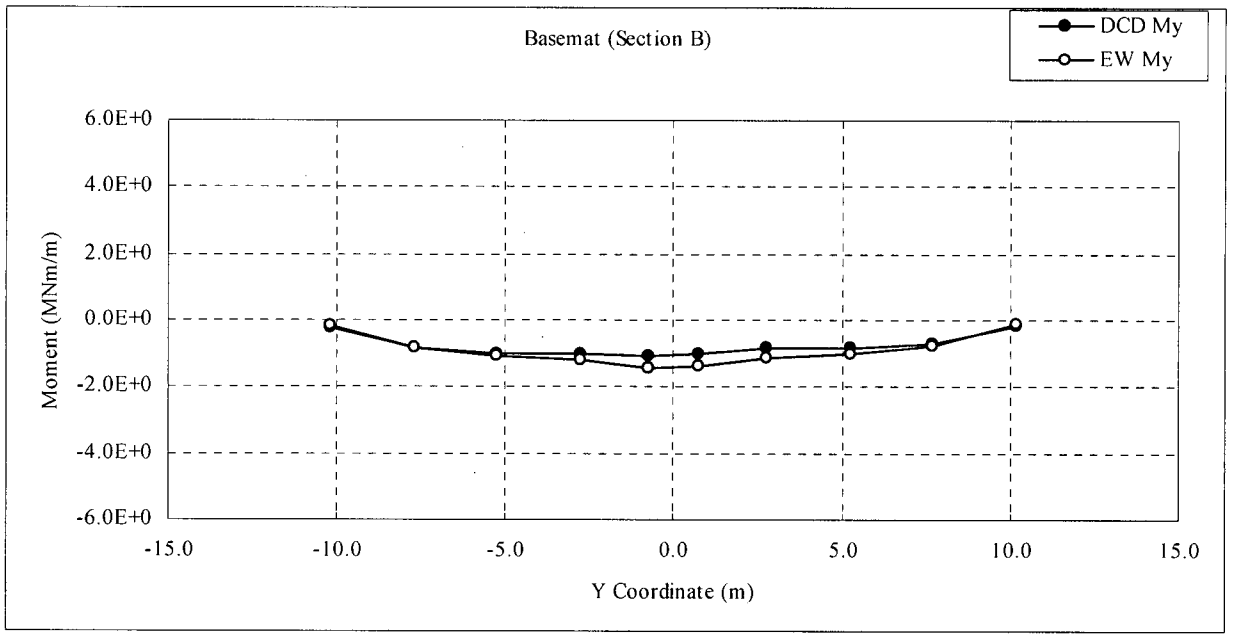


(b) Mx in N-S Section

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





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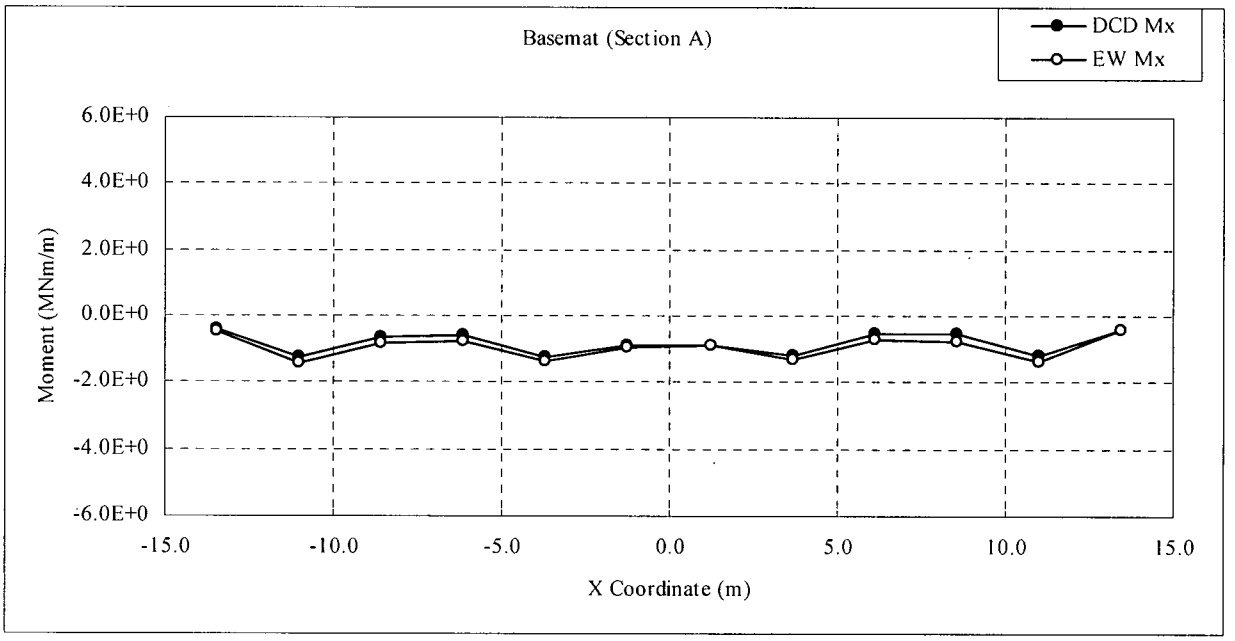
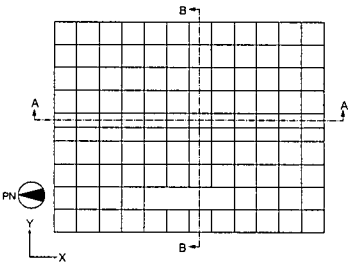
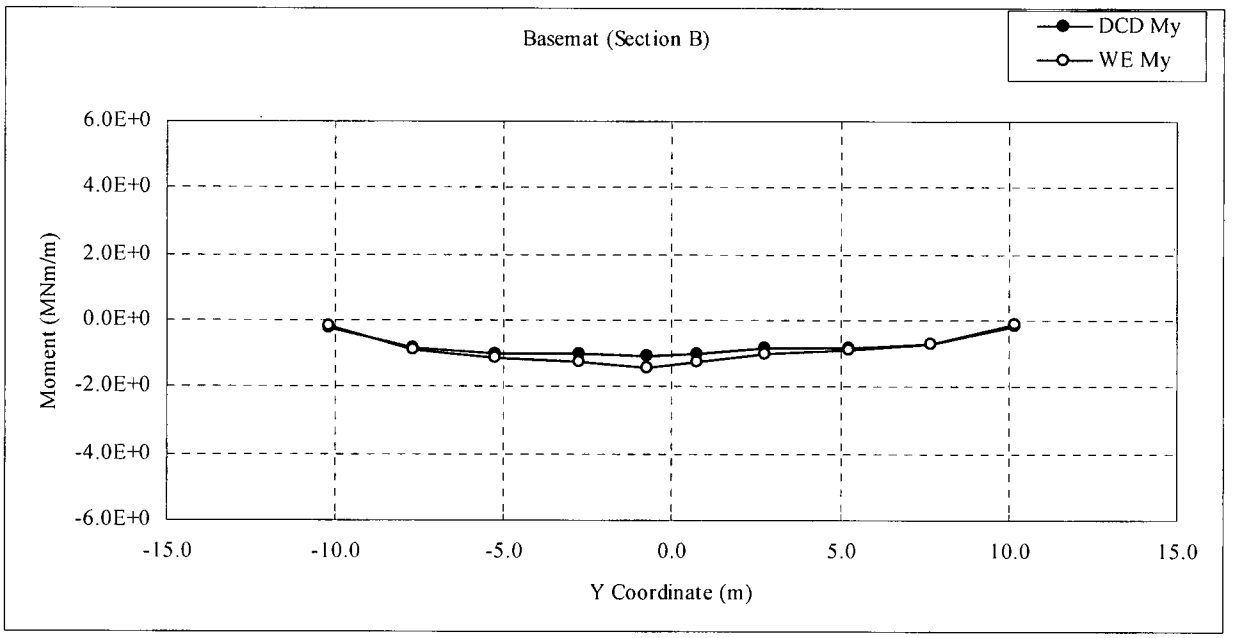


(b) Mx in N-S Section

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





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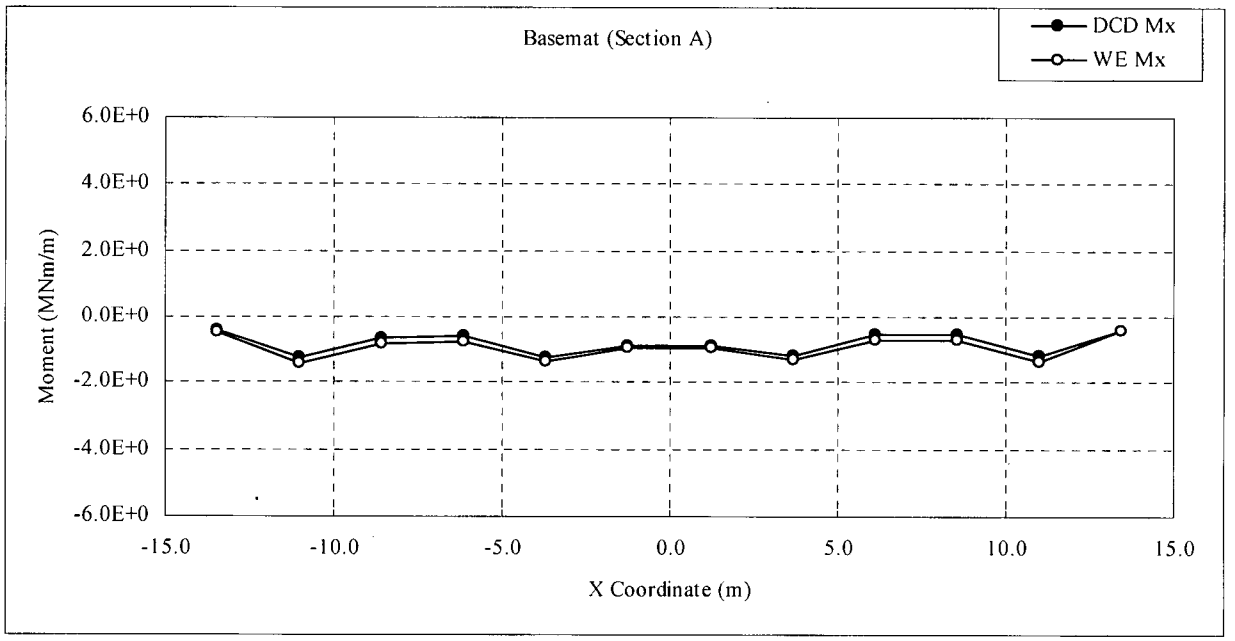
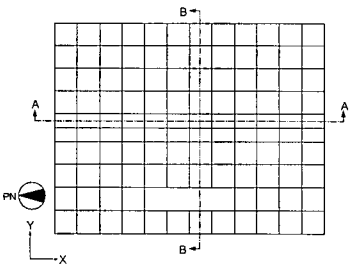
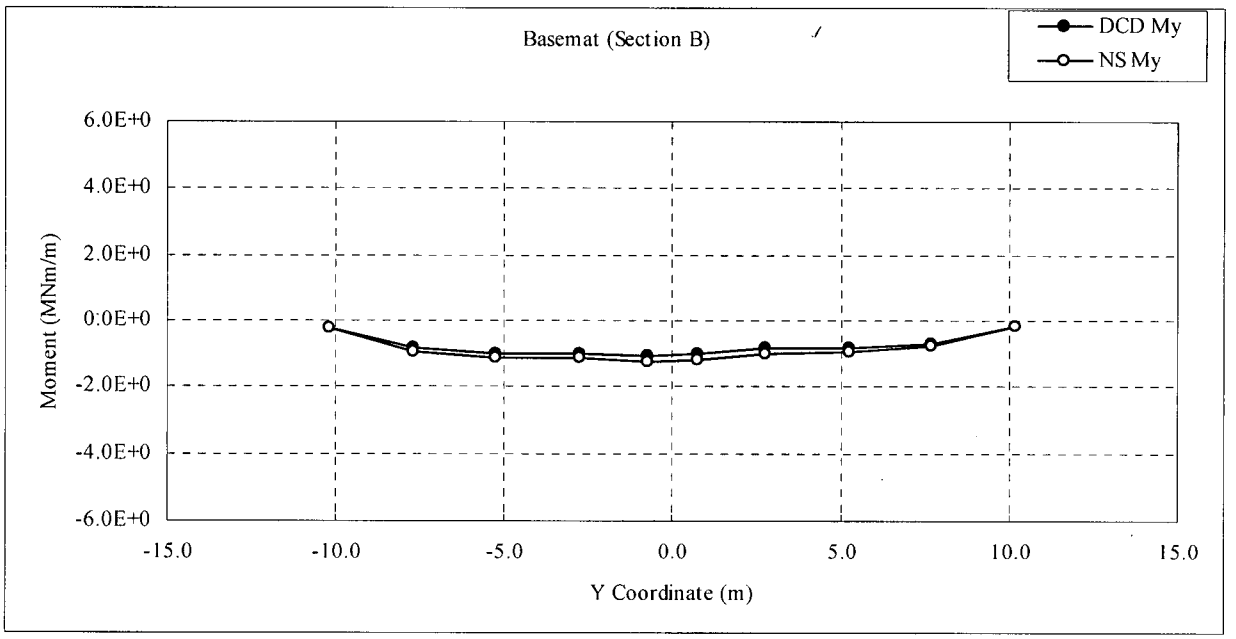


(b) Mx in N-S Section

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





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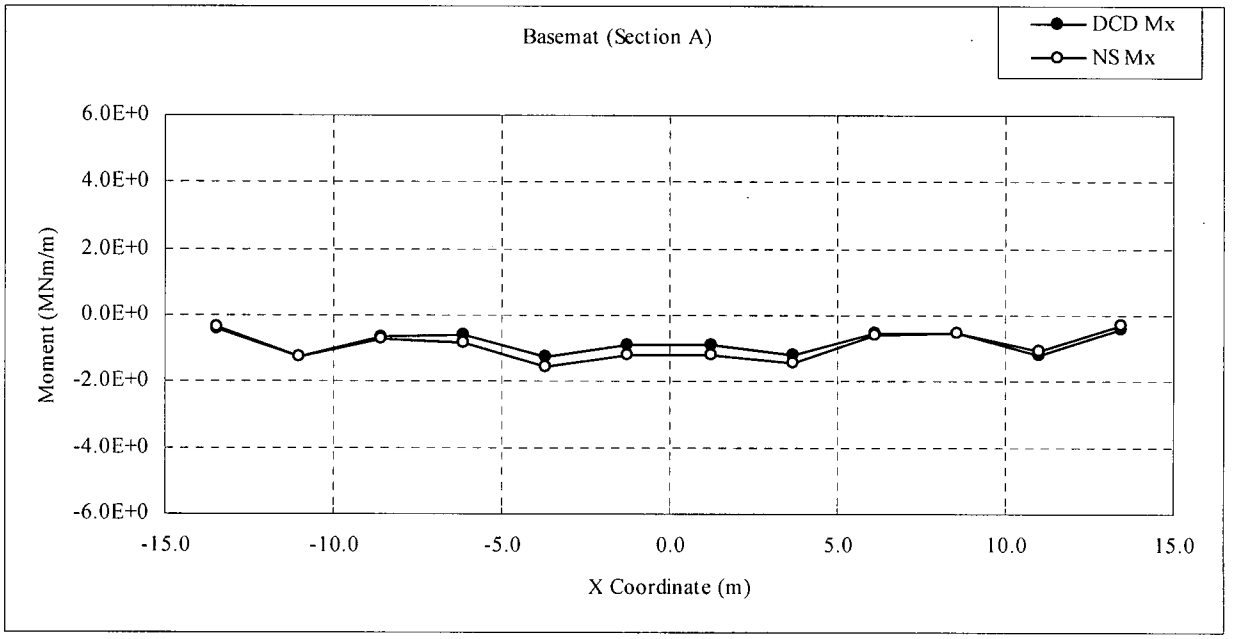
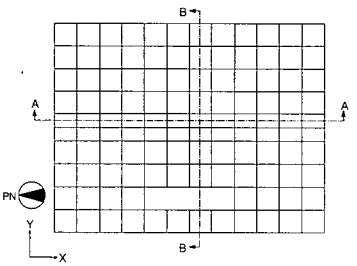
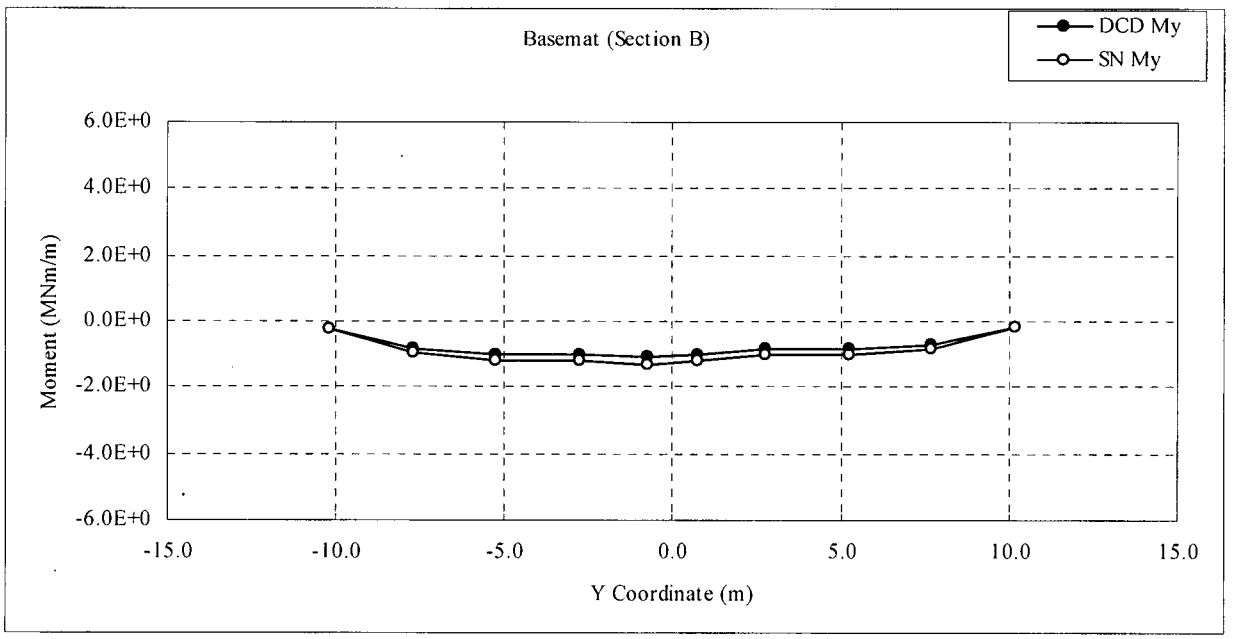


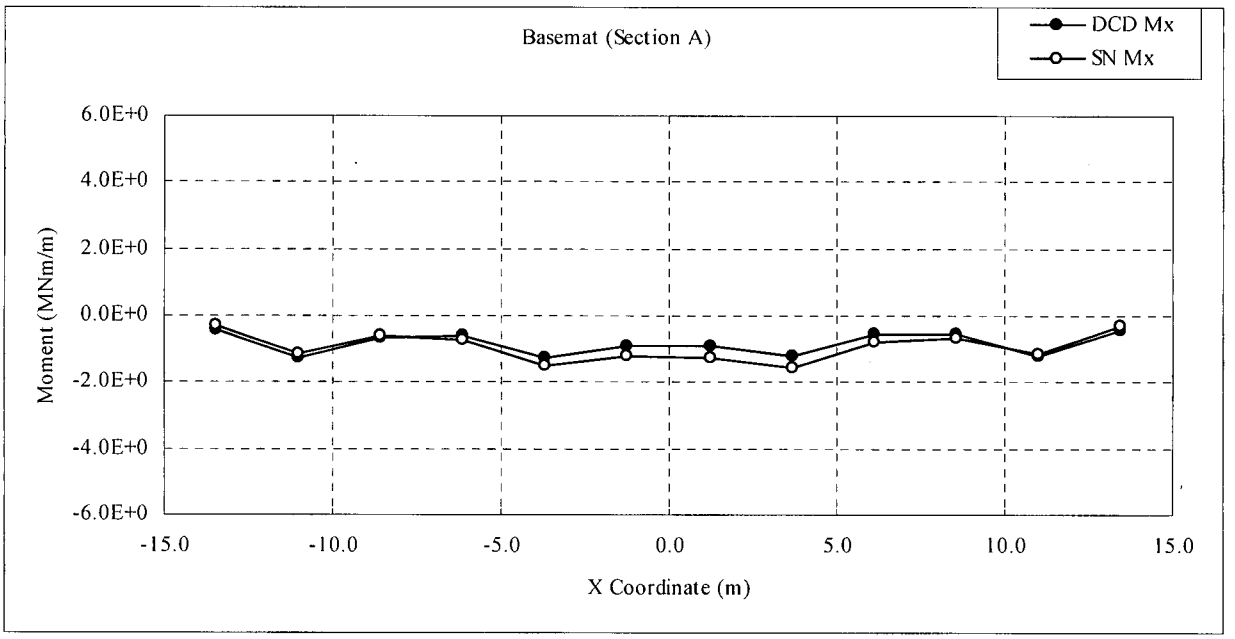
(b) Mx in N-S Section

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



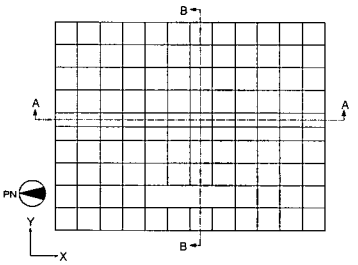


(a) My in E-W Section



(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).

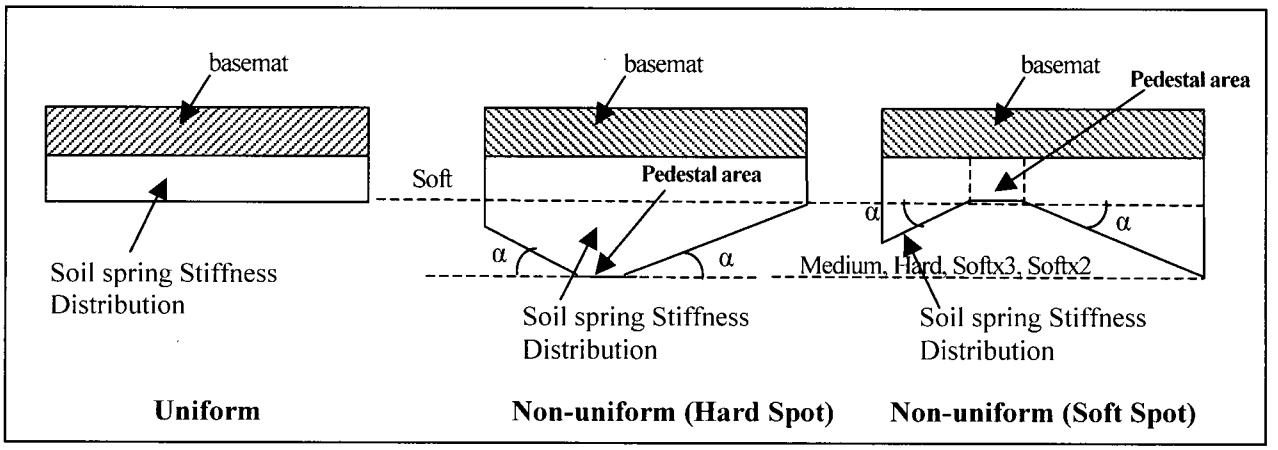


Figure 3.8-93 (14) Soil conditions

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.

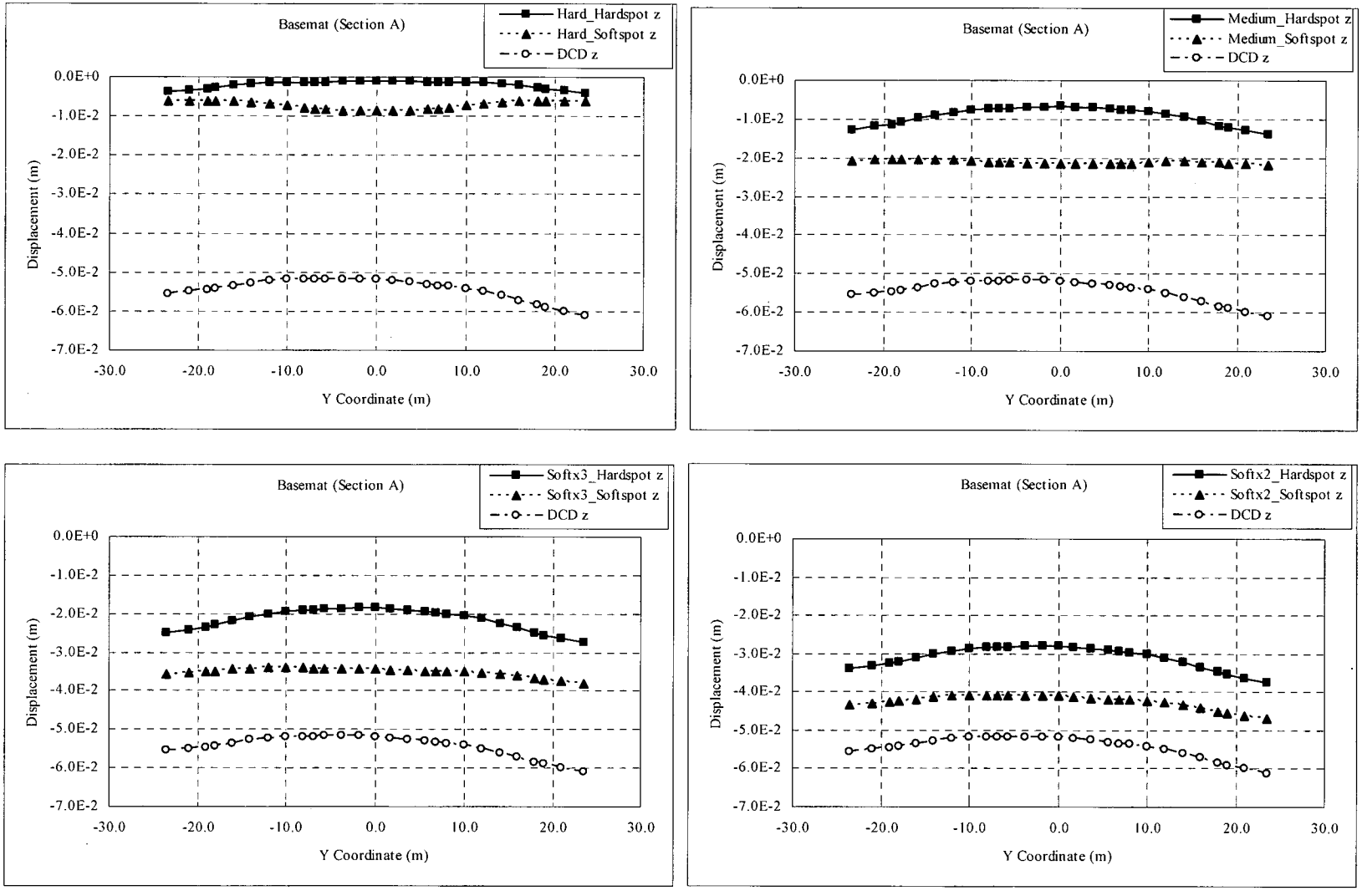


Figure 3.8-93 (15) -a Comparison of Basemat Deformation (E-W Section)

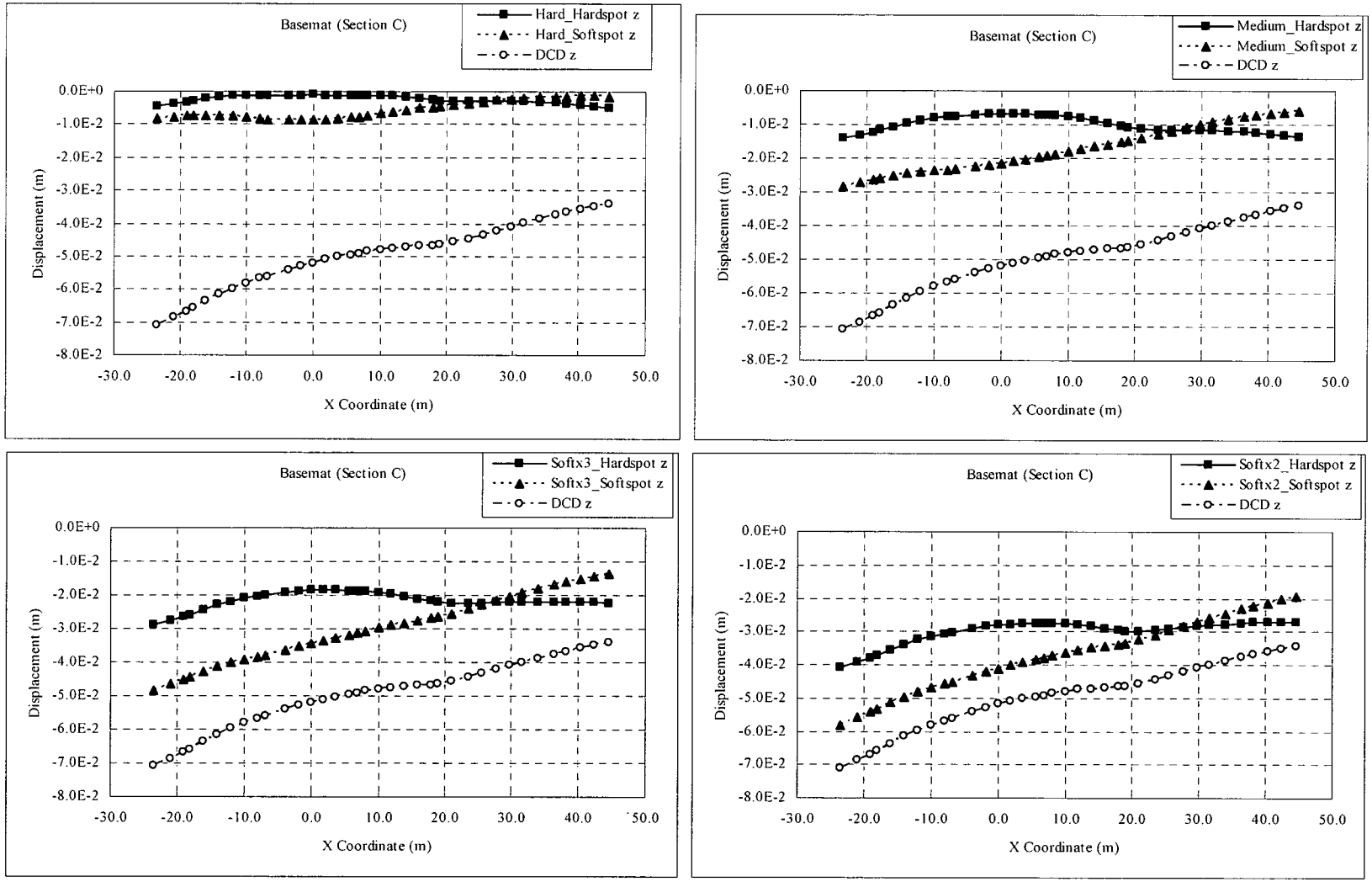
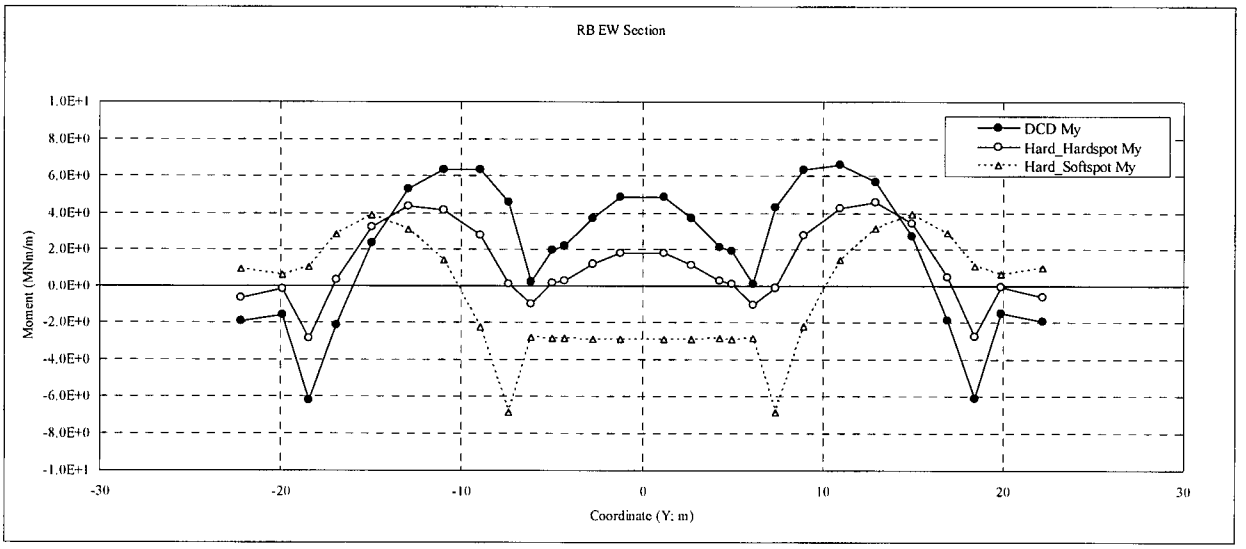
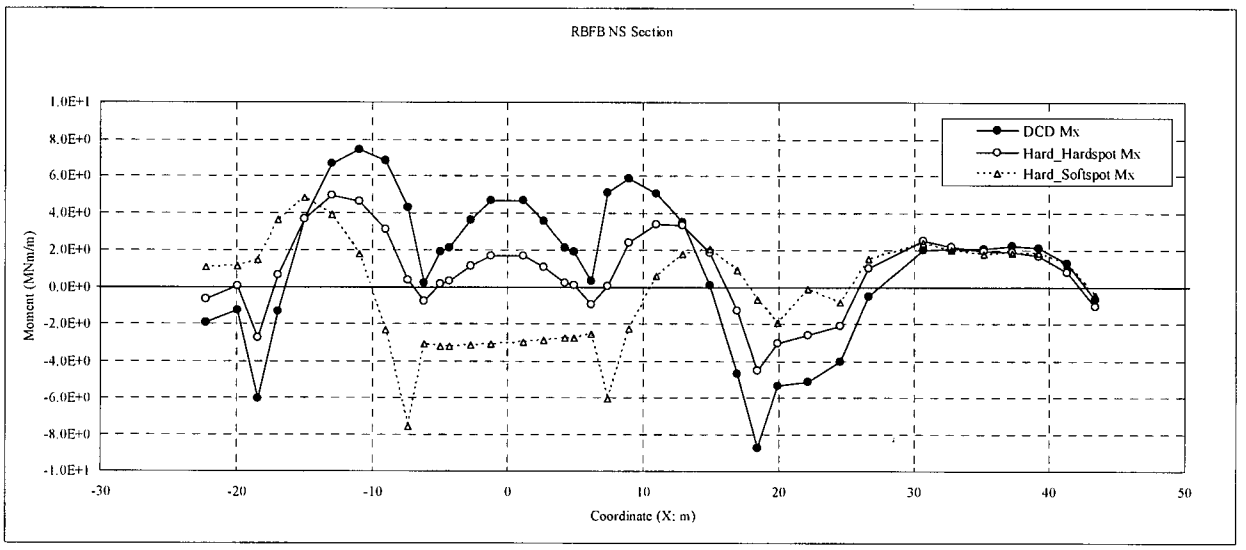


Figure 3.8-93 (15)-b Comparison of Basemat Deformation N-S Section)

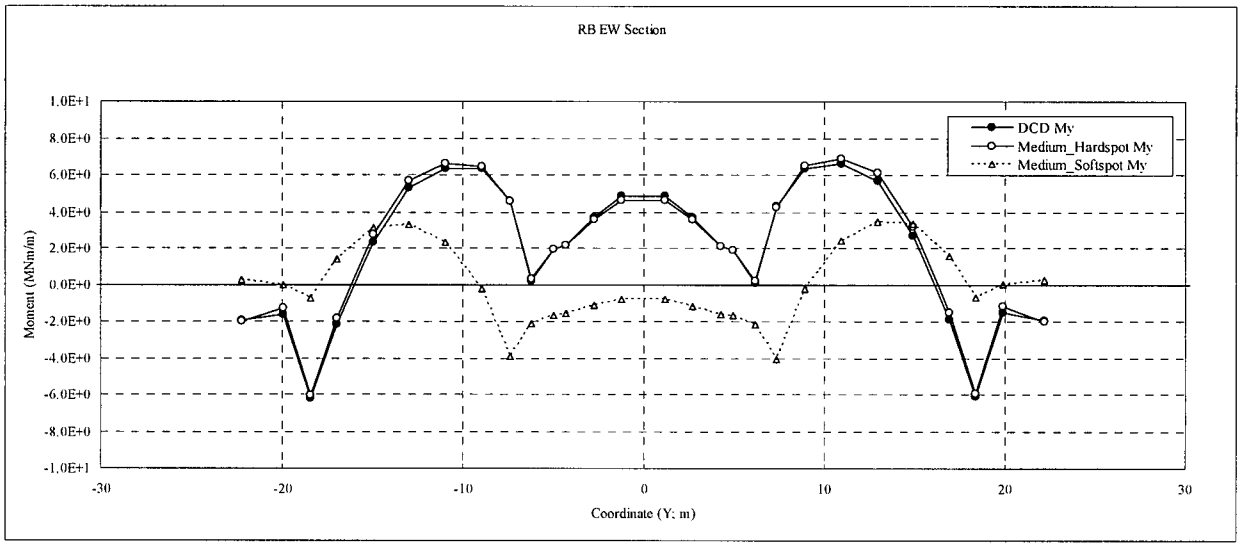


(a) My in E-W Section

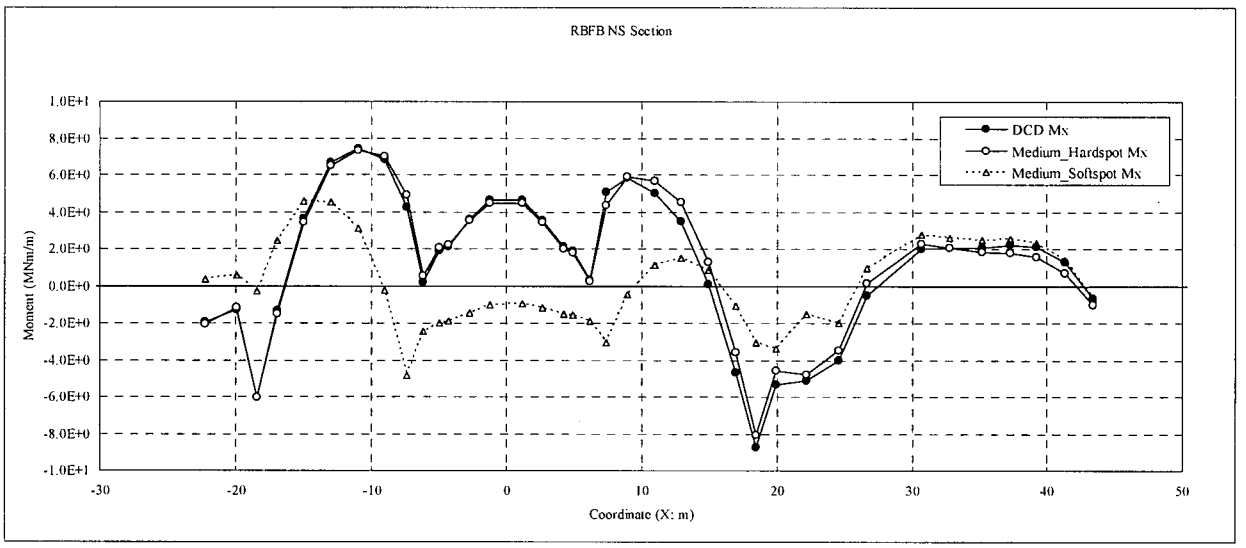


(b) Mx in N-S Section

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

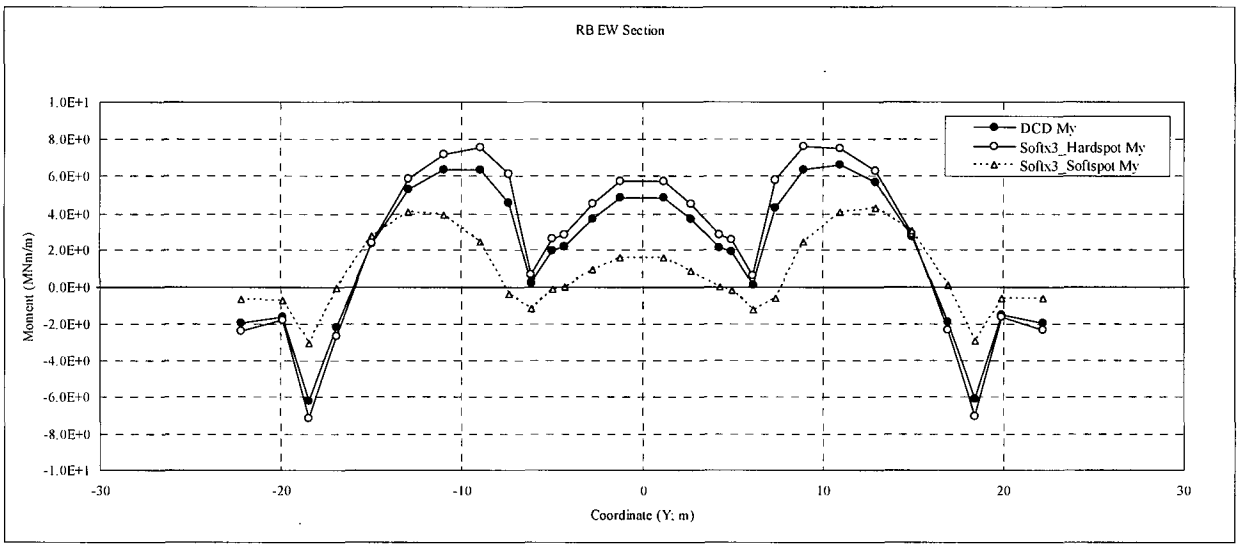


(a) My in E-W Section

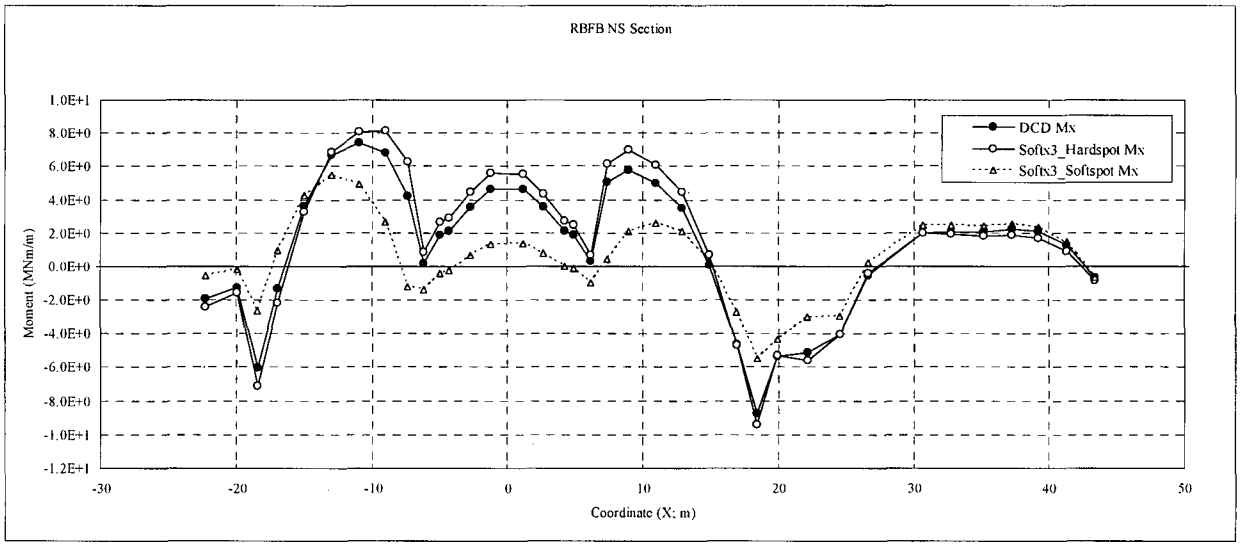


(b) Mx in N-S Section

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

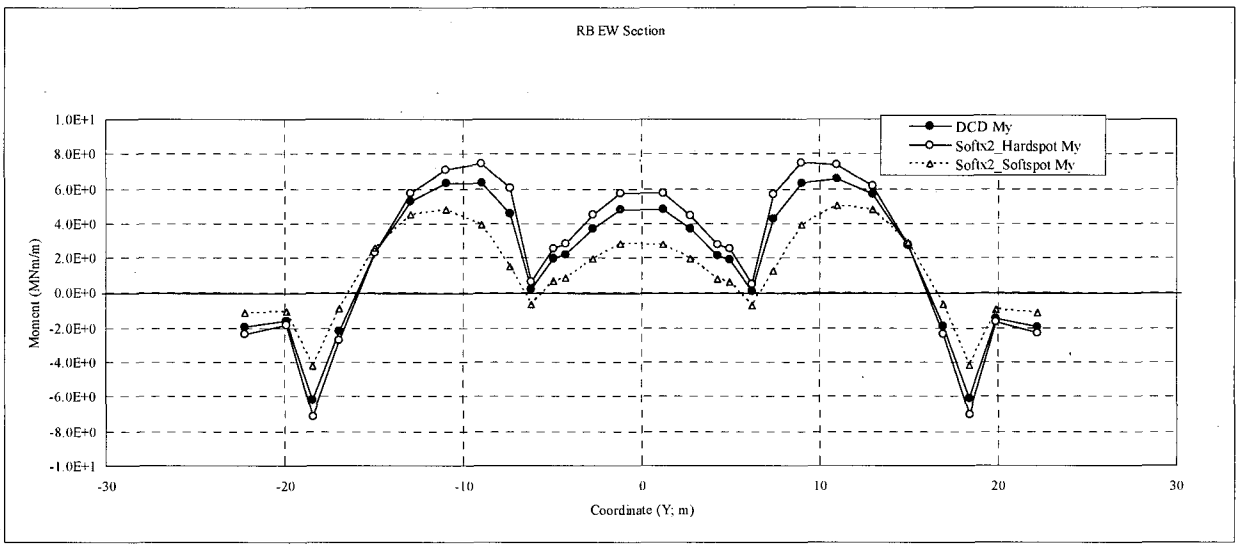


(a) My in E-W Section

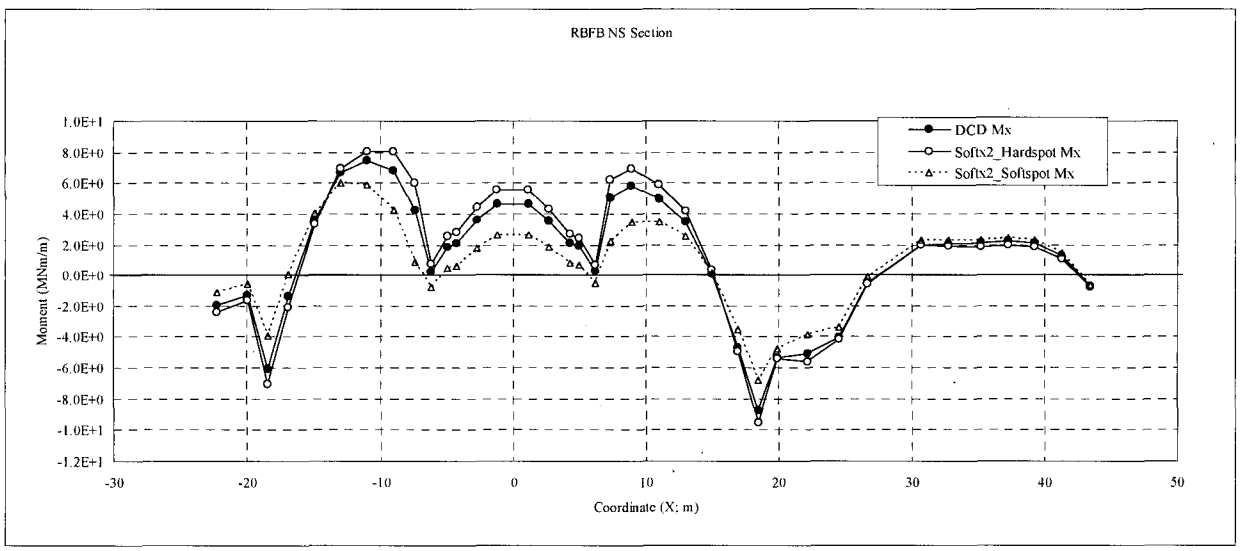


(b) Mx in N-S Section

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)

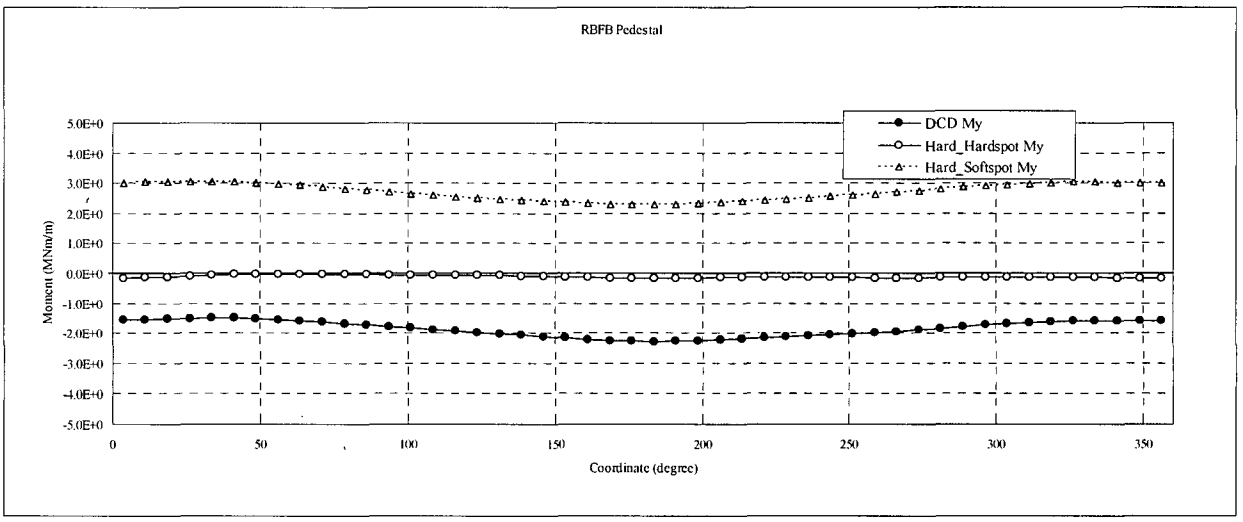


Figure 3.8-93 (17)-a Comparison of Sectional Moments (Pedestal, Hard)

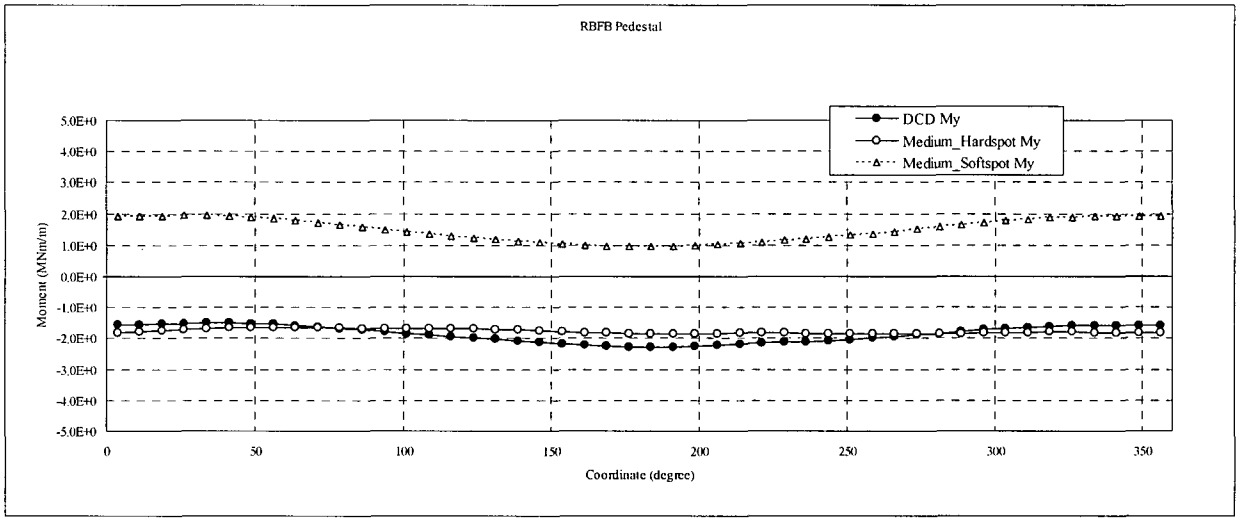


Figure 3.8-93 (17)-b Comparison of Sectional Moments (Pedestal, Medium)

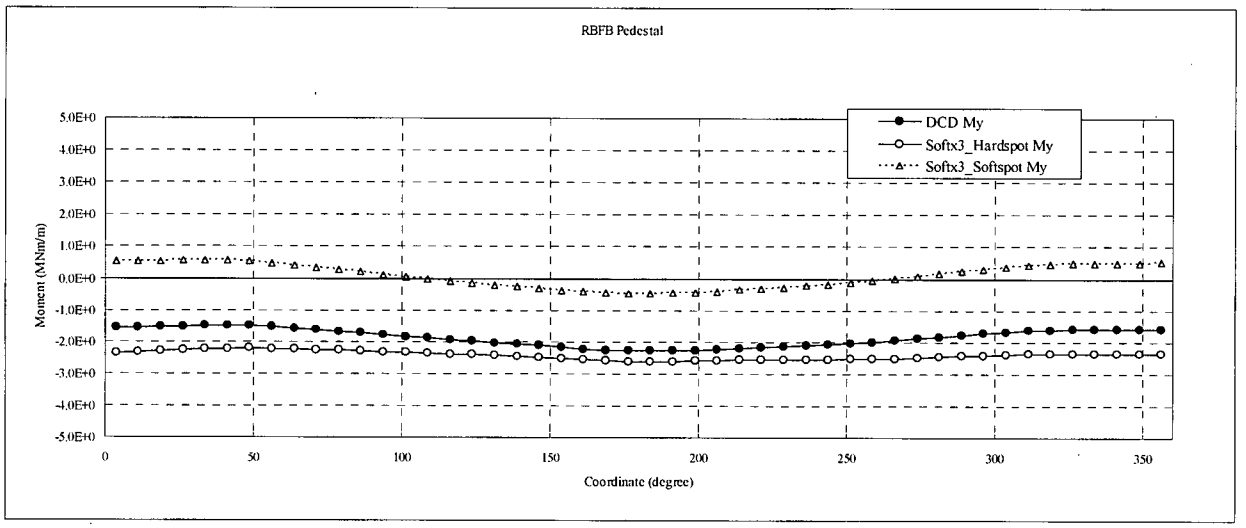


Figure 3.8-93 (17)-c Comparison of Sectional Moments (Pedestal, Soft x3)

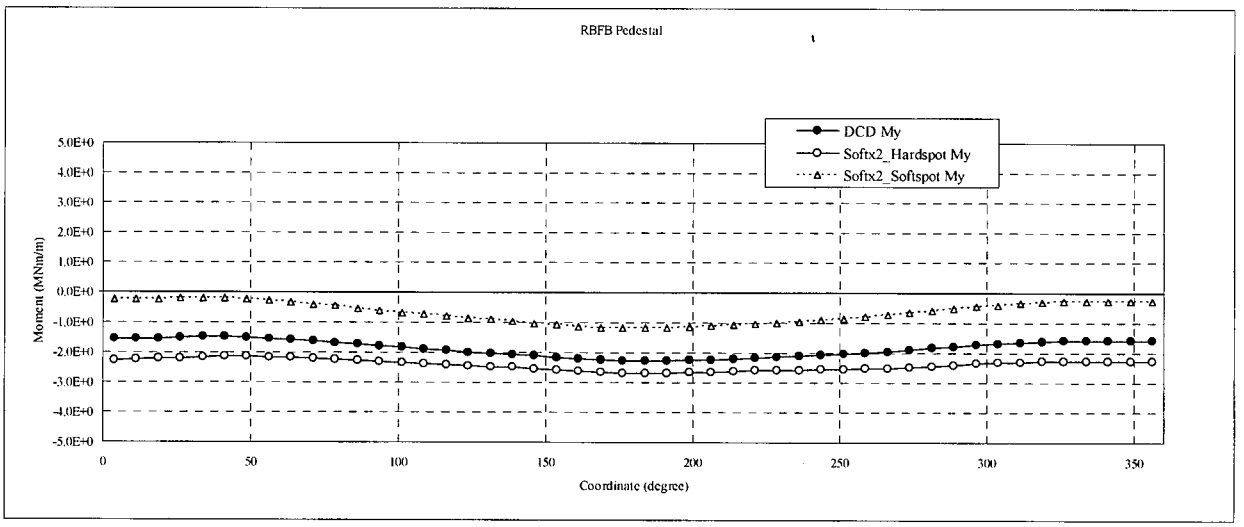


Figure 3.8-93 (17)-d Comparison of Sectional Moments (Pedestal, Soft x2)

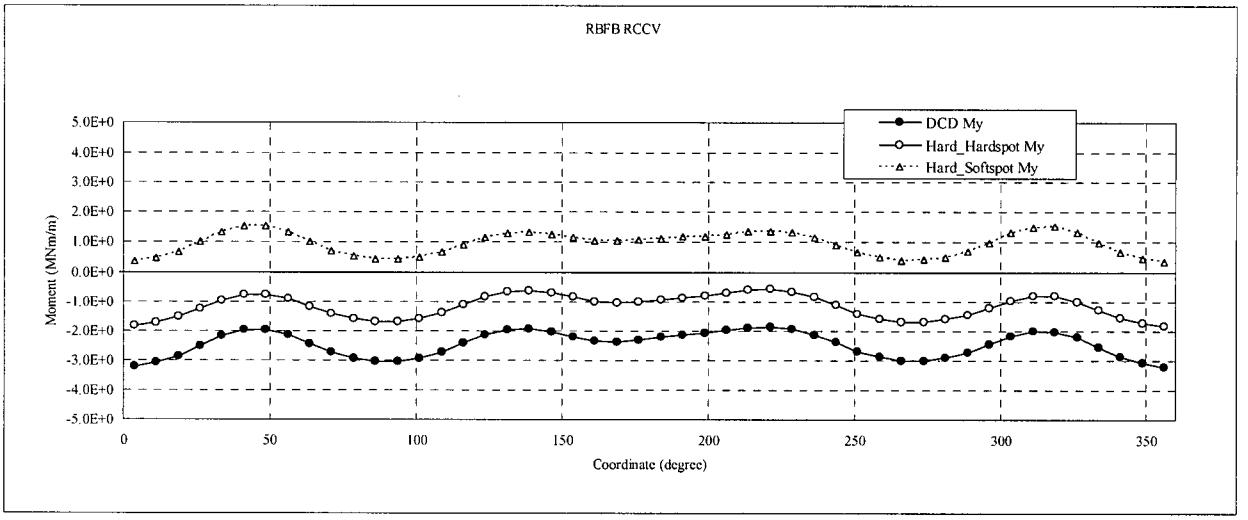


Figure 3.8-93 (18)-a Comparison of Sectional Moments (RCCV bottom, Hard)

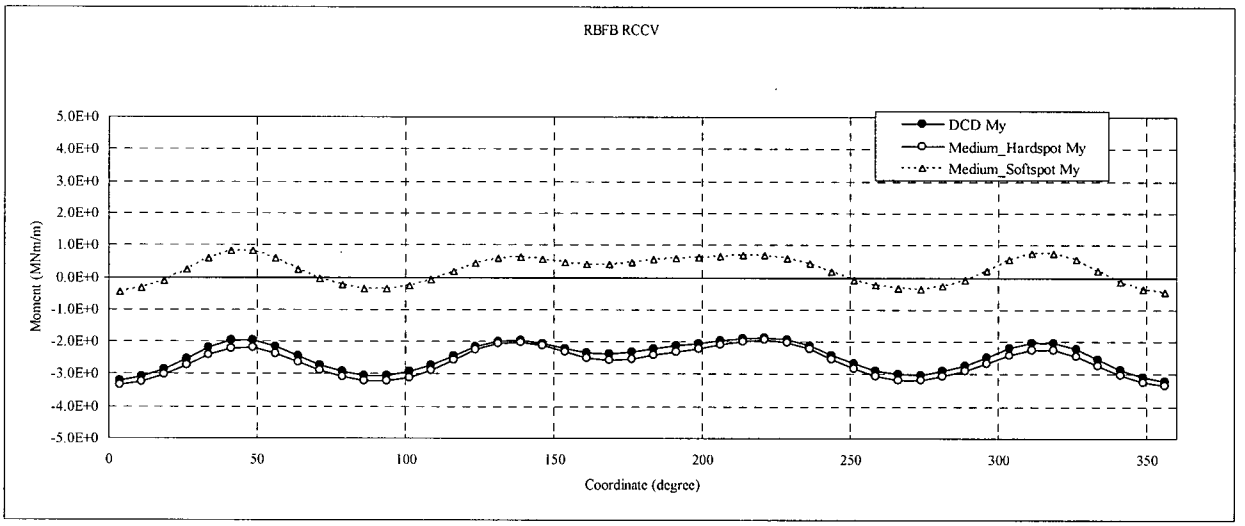


Figure 3.8-93 (18)-b Comparison of Sectional Moments (RCCV bottom, Medium)

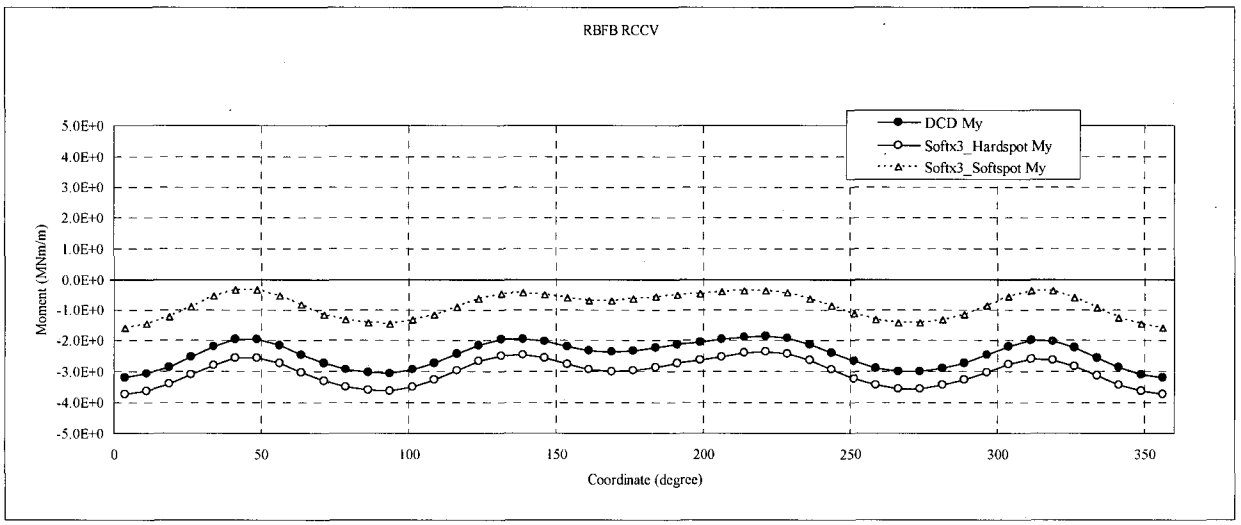


Figure 3.8-93 (18)-c Comparison of Sectional Moments (RCCV bottom, Soft x3)

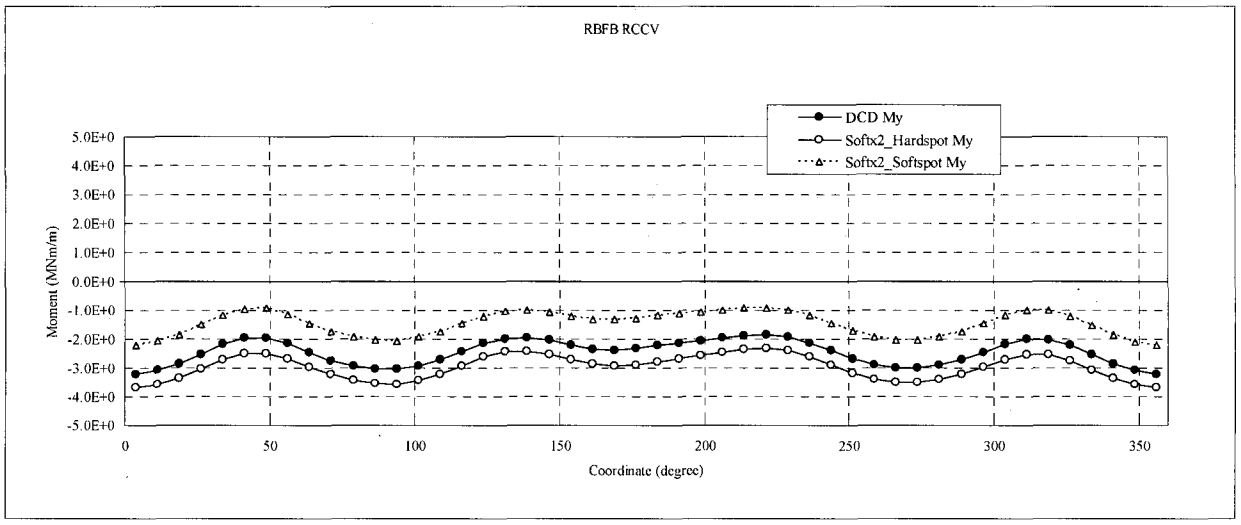


Figure 3.8-93 (18)-d Comparison of Sectional Moments (RCCV bottom, Soft x2)

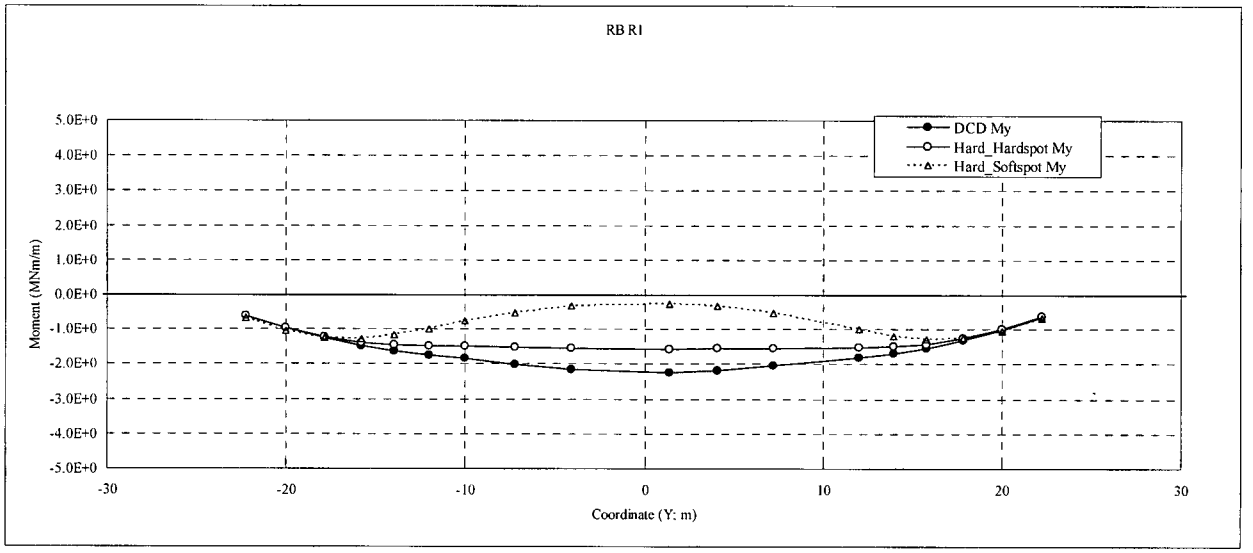


Figure 3.8-93 (19)-a Comparison of Sectional Moments (R1, Hard)

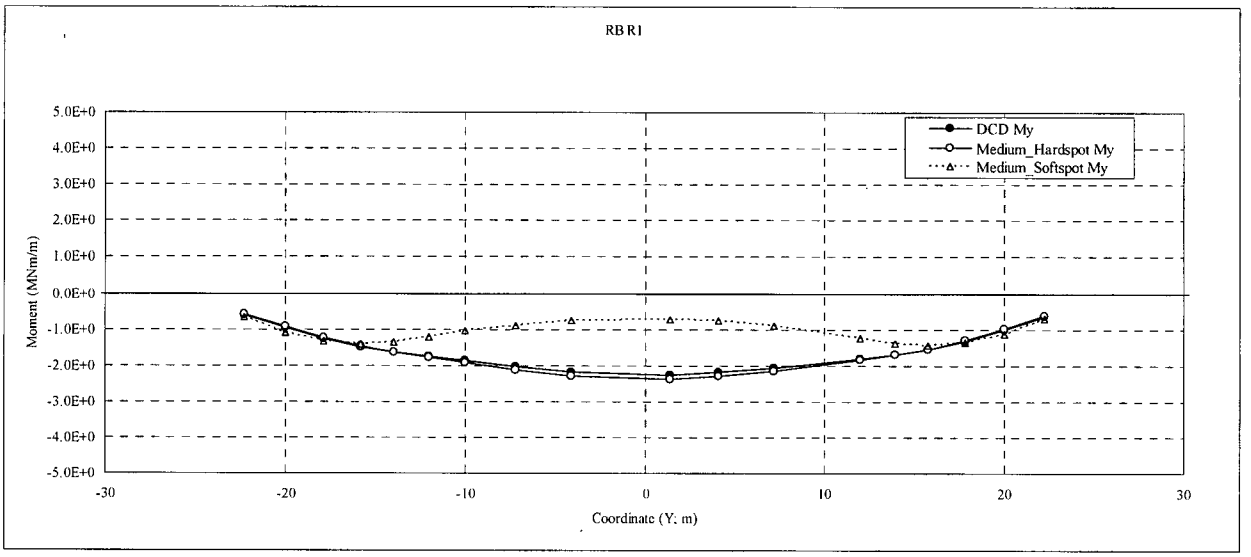


Figure 3.8-93 (19)-b Comparison of Sectional Moments (R1, Medium)

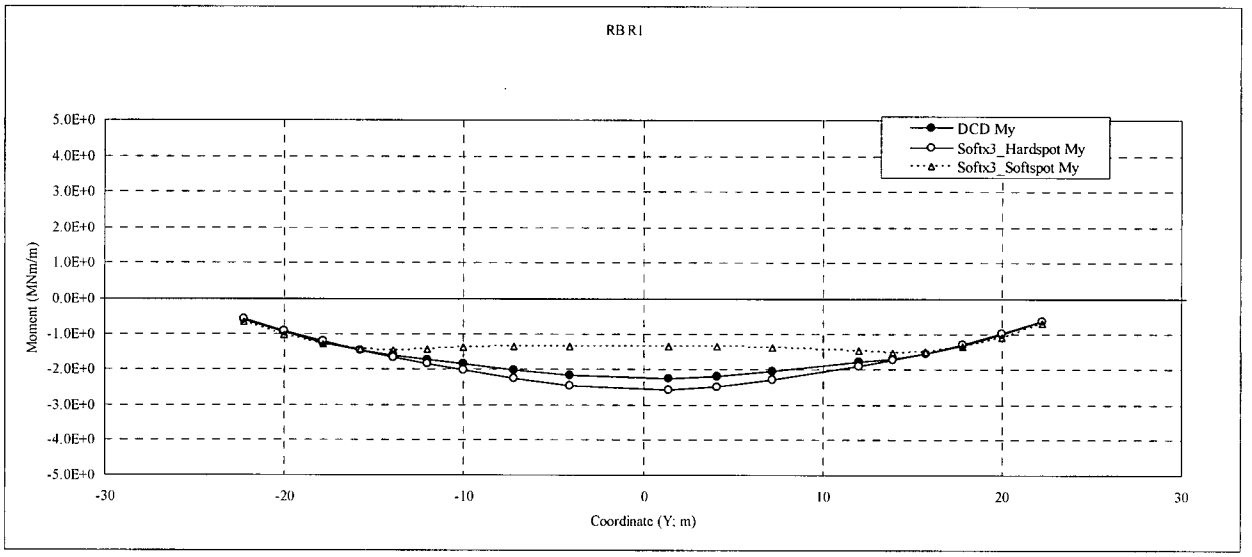


Figure 3.8-93 (19)-c Comparison of Sectional Moments (R1, Soft x3)

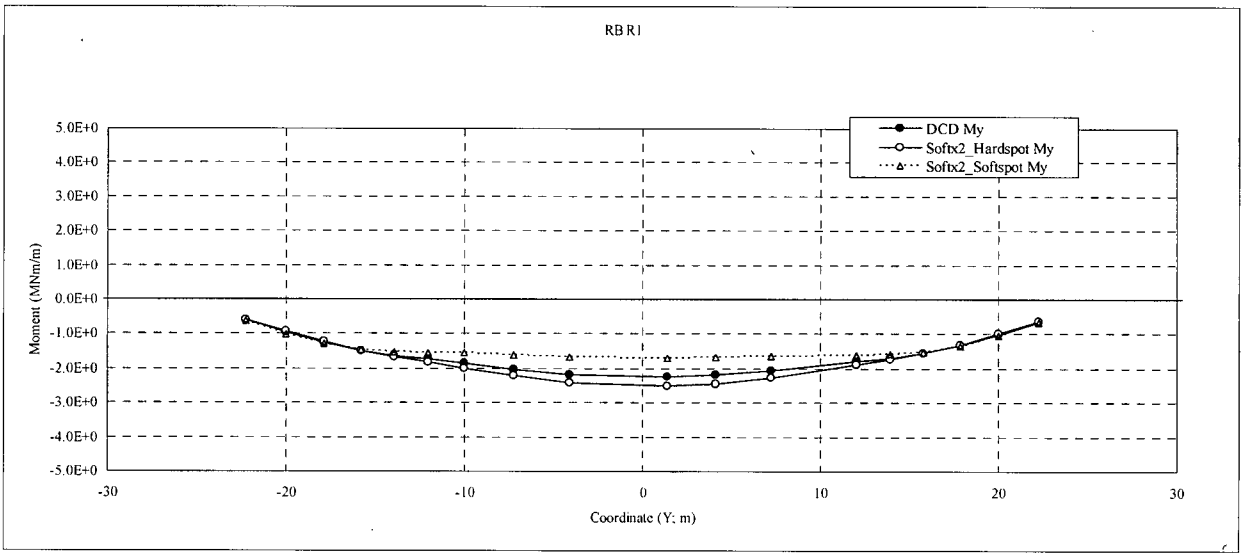


Figure 3.8-93 (19)-d Comparison of Sectional Moments (R1, Soft x2)

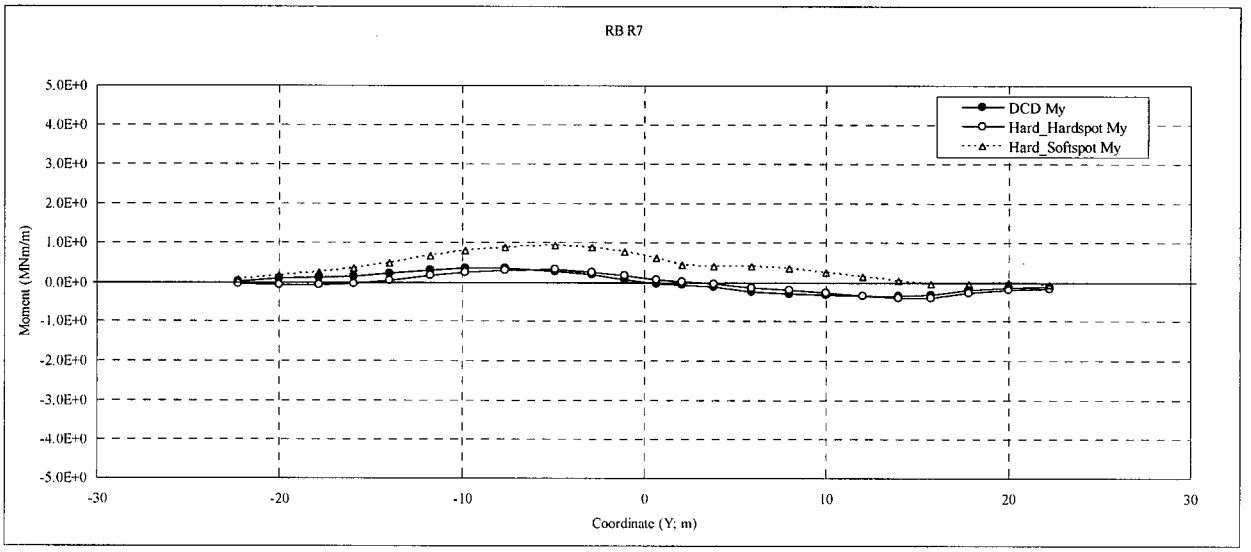


Figure 3.8-93 (20)-a Comparison of Sectional Moments (R7, Hard)

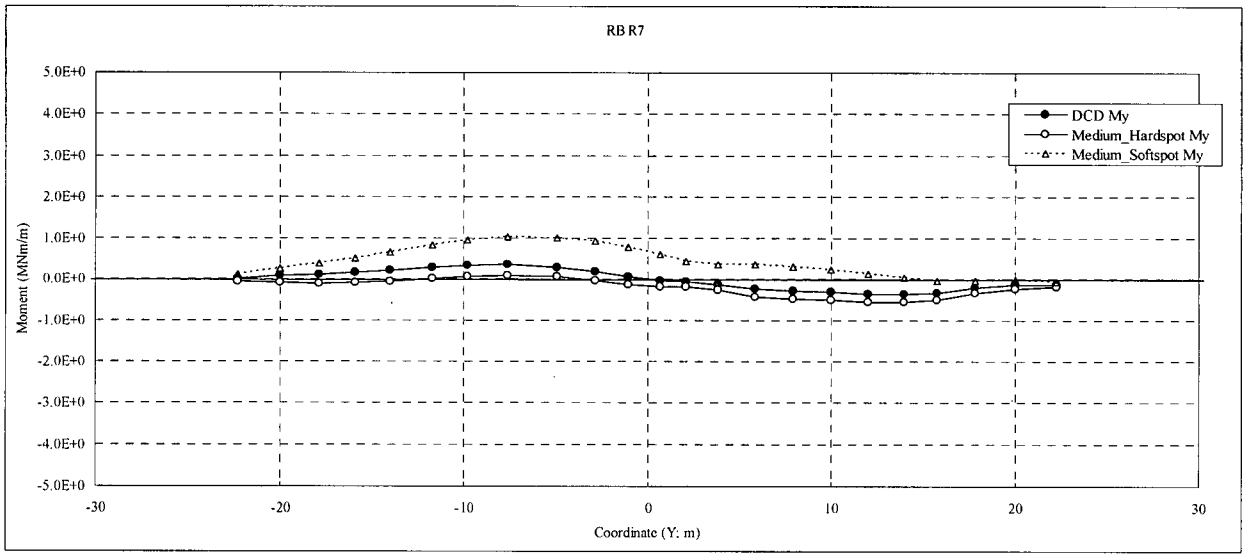


Figure 3.8-93 (20)-b Comparison of Sectional Moments (R7, Medium)

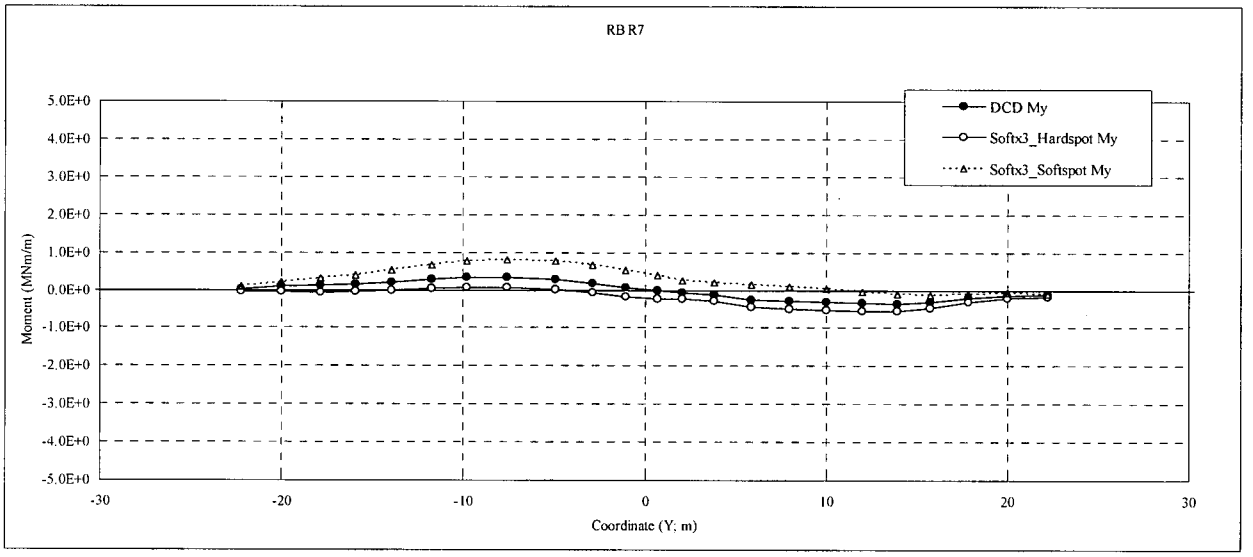


Figure 3.8-93 (20)-c Comparison of Sectional Moments (R7, Soft x3)

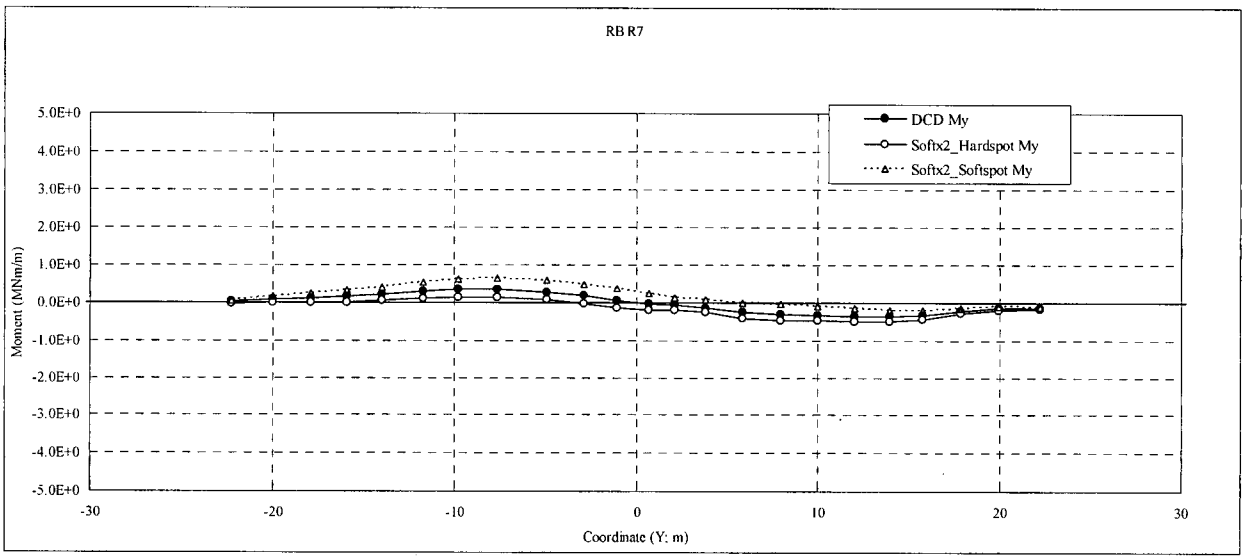


Figure 3.8-93 (20)-d Comparison of Sectional Moments (R7, Soft x2)

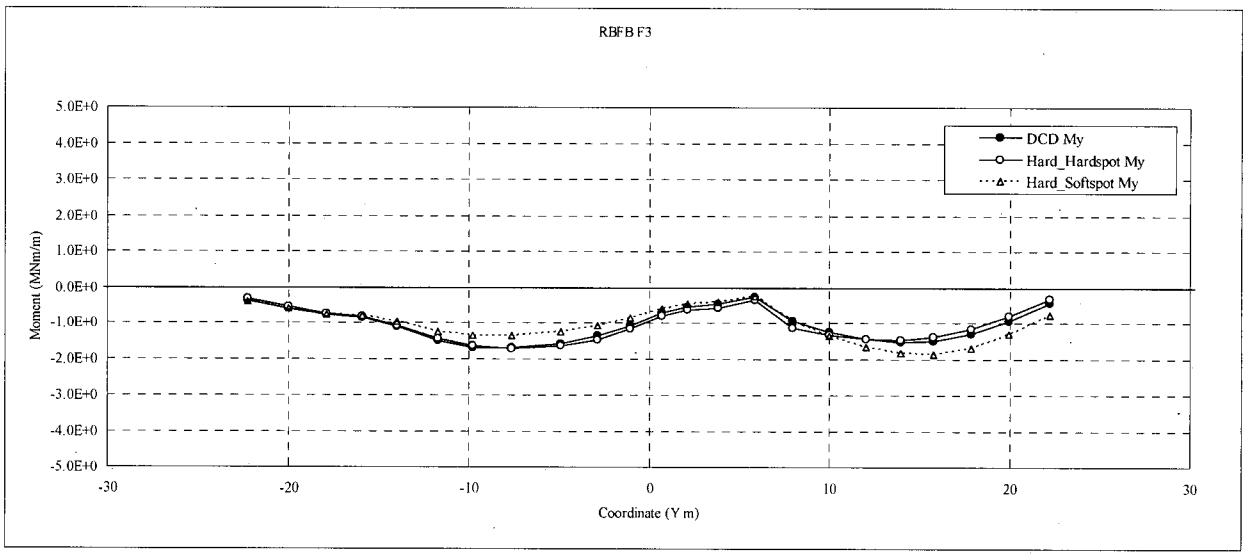


Figure 3.8-93 (21)-a Comparison of Sectional Moments (F3, Hard)

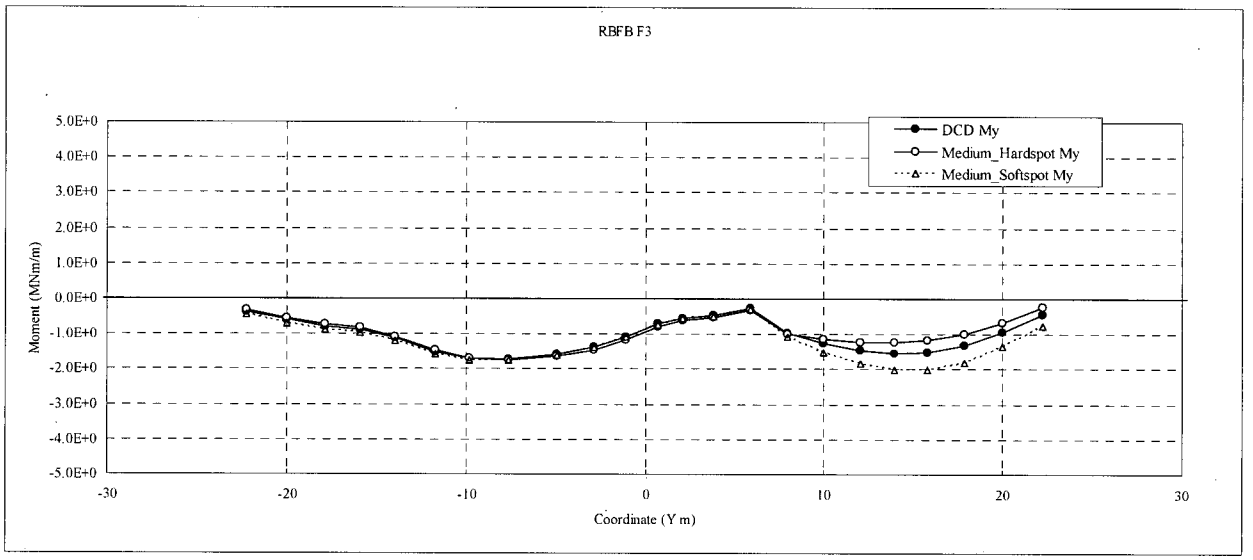


Figure 3.8-93 (21)-b Comparison of Sectional Moments (F3, Medium)

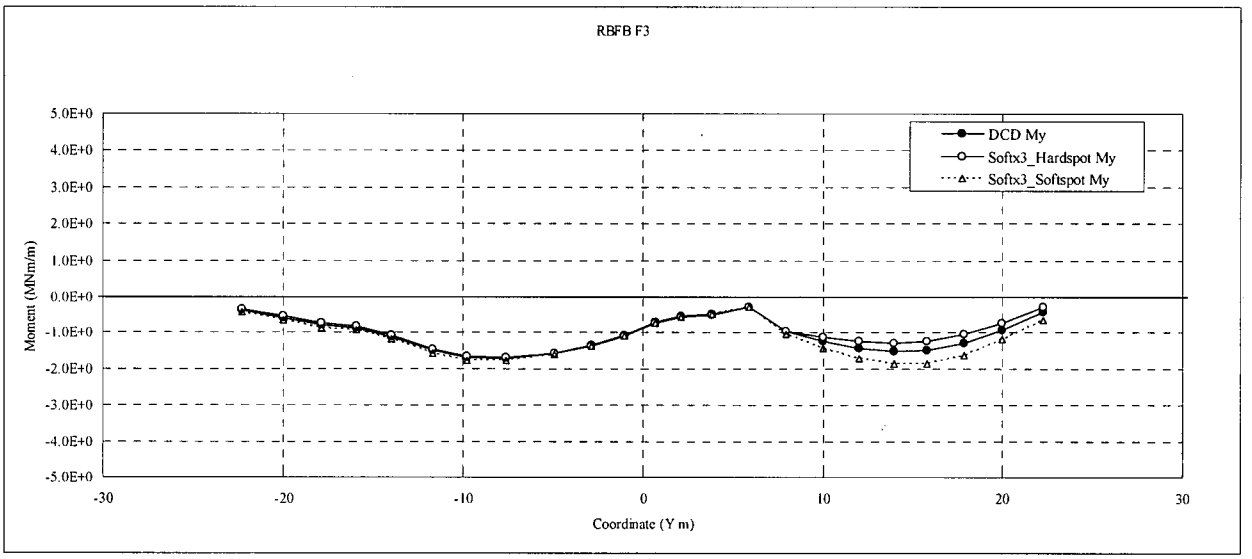


Figure 3.8-93 (21)-c Comparison of Sectional Moments (F3, Soft x3)

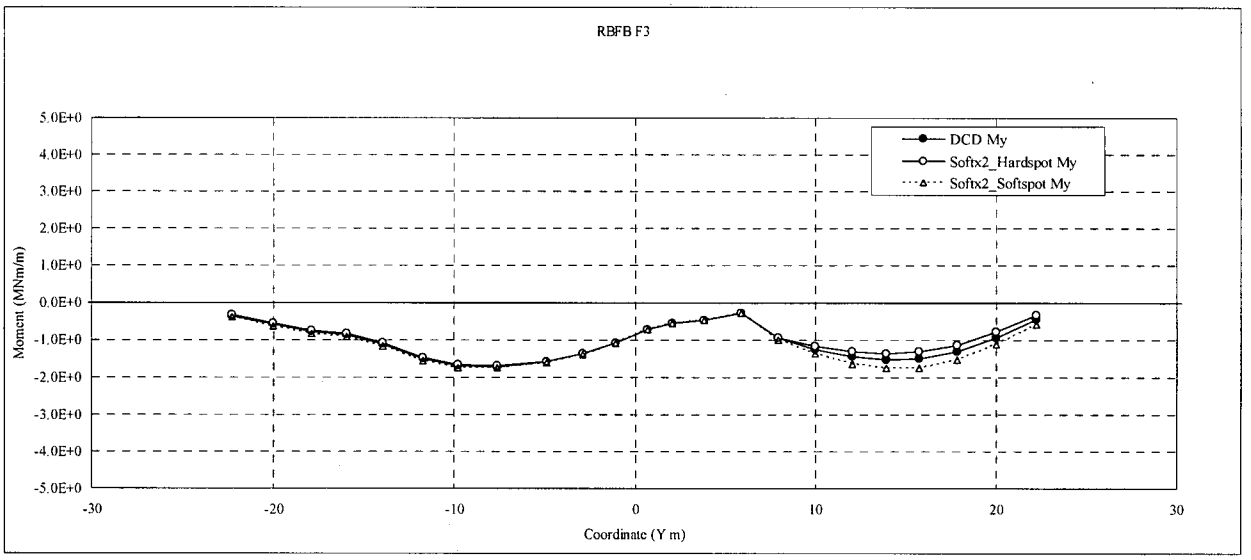


Figure 3.8-93 (21)-d Comparison of Sectional Moments (F3, Soft x2)

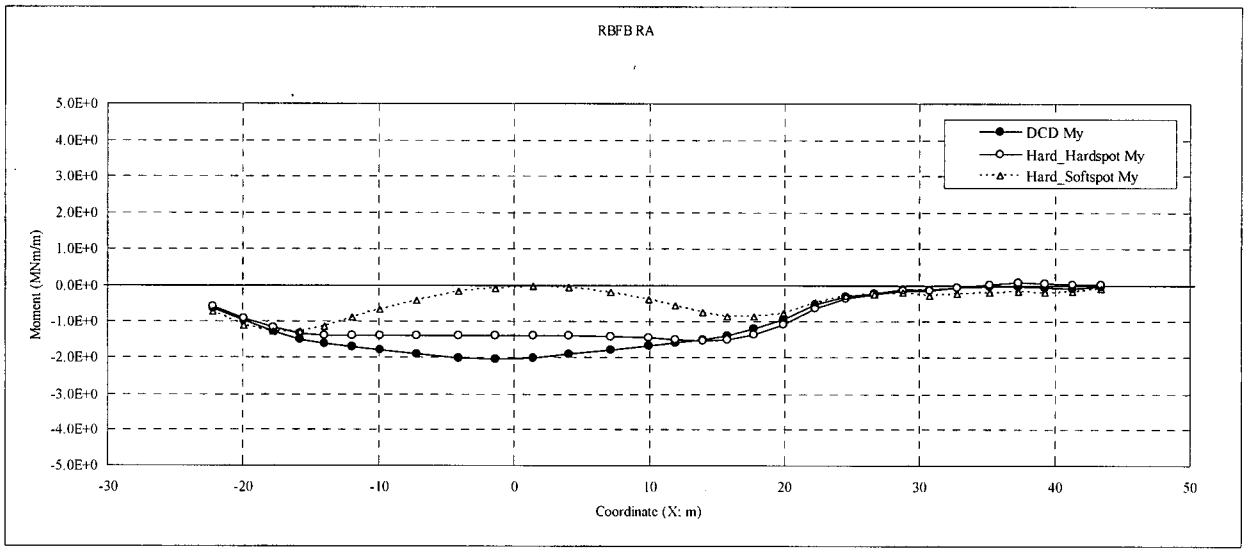


Figure 3.8-93 (22)-a Comparison of Sectional Moments (RA, Hard)

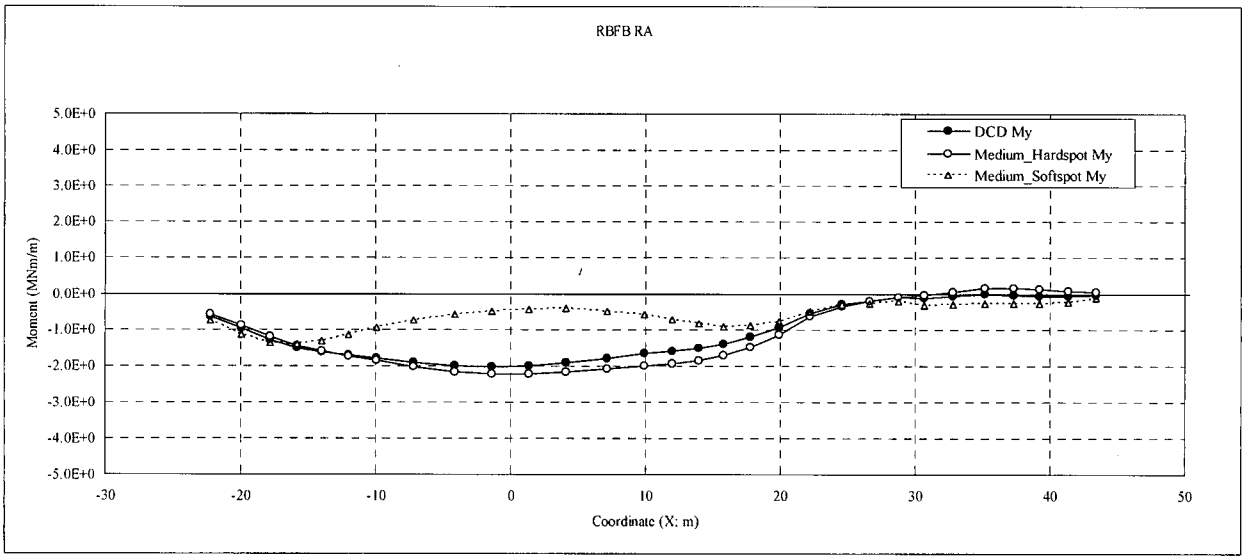


Figure 3.8-93 (22)-b Comparison of Sectional Moments (RA, Medium)

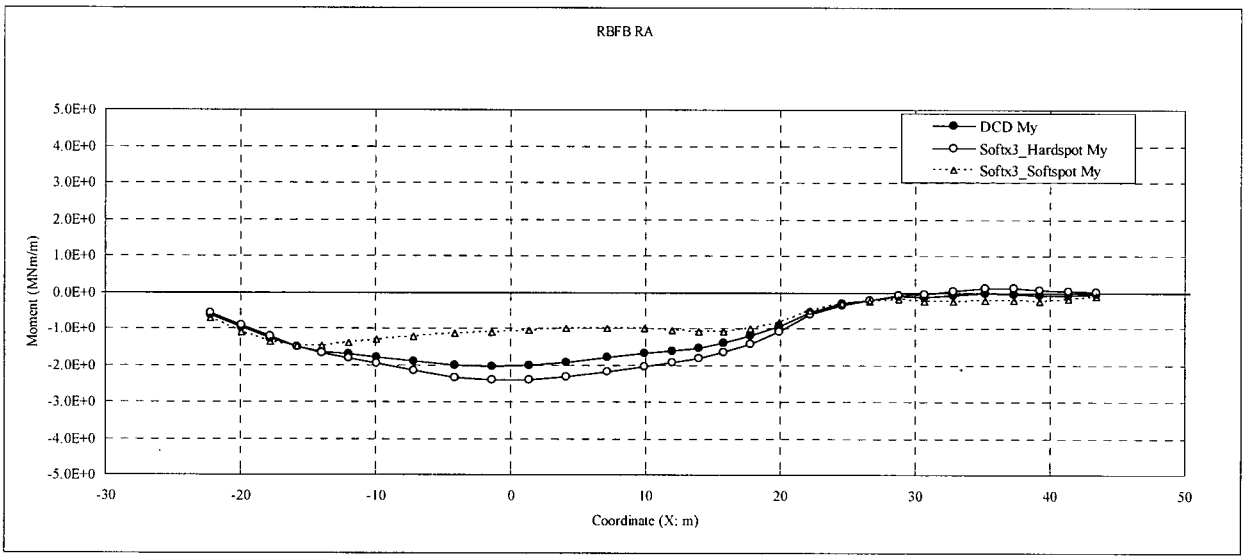


Figure 3.8-93 (22)-c Comparison of Sectional Moments (RA, Soft x3)

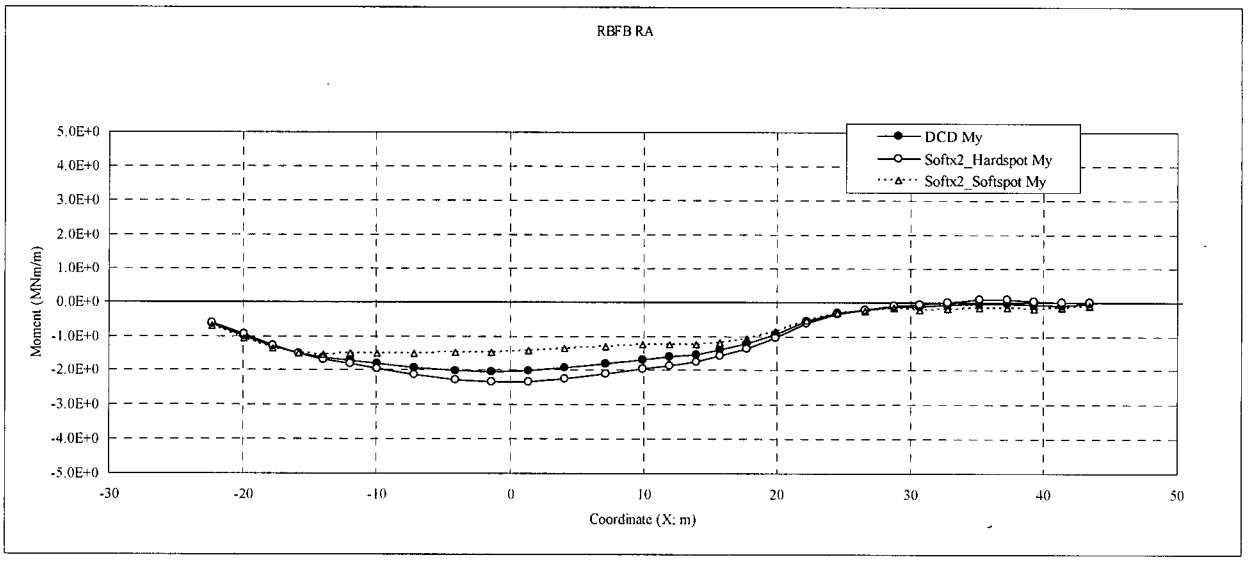


Figure 3.8-93 (22)-d Comparison of Sectional Moments (RA, Soft x2)

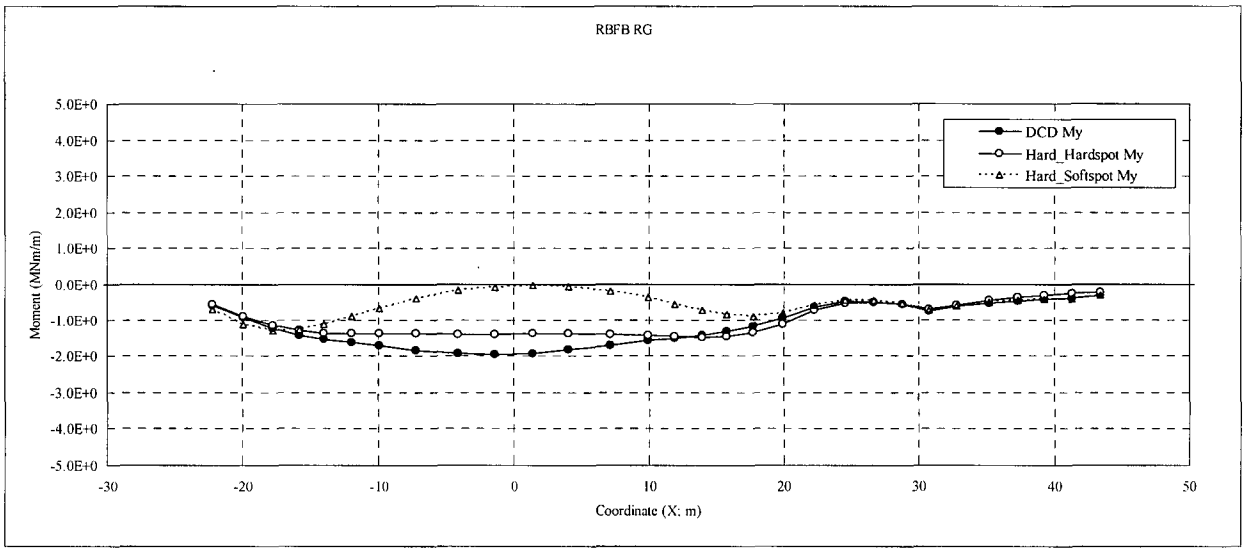


Figure 3.8-93 (23)-a Comparison of Sectional Moments (RG, Hard)

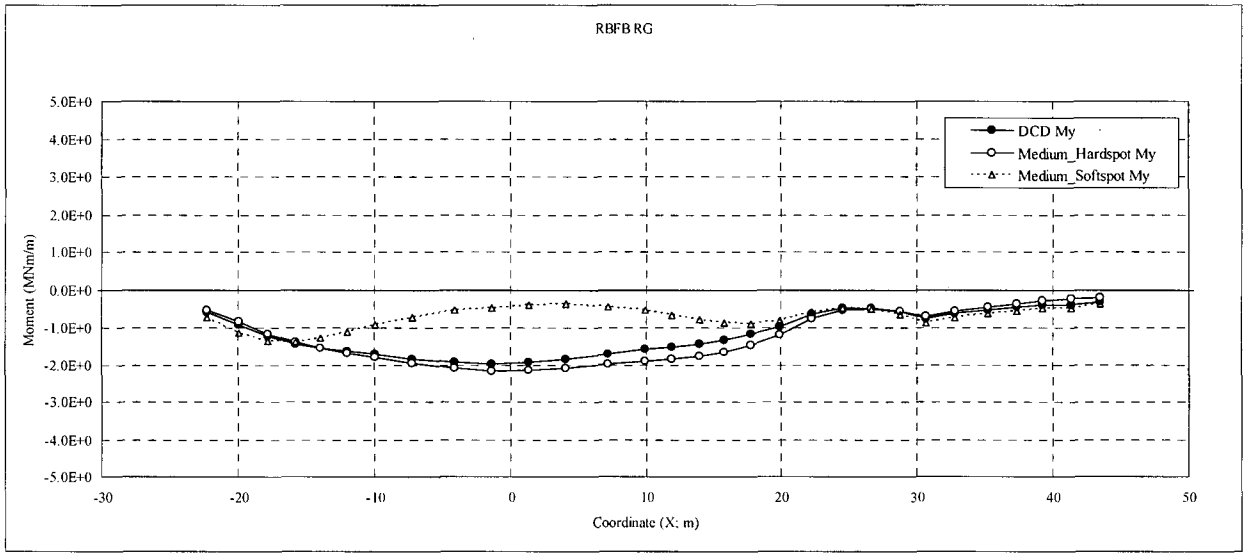


Figure 3.8-93 (23)-b Comparison of Sectional Moments (RG, Medium)

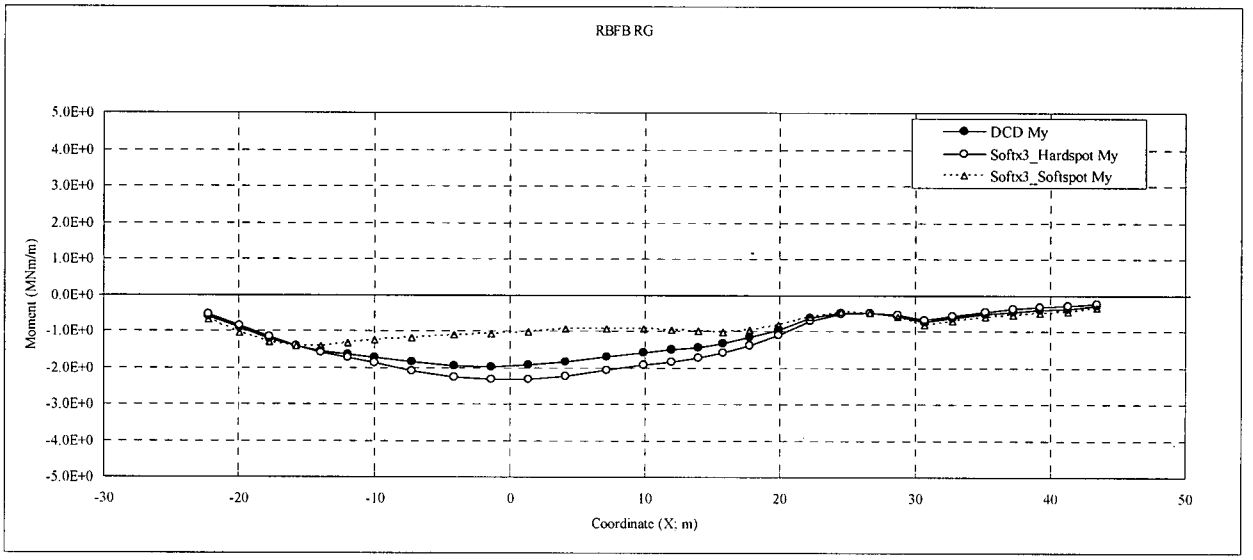


Figure 3.8-93 (23)-c Comparison of Sectional Moments (RG, Soft x3)

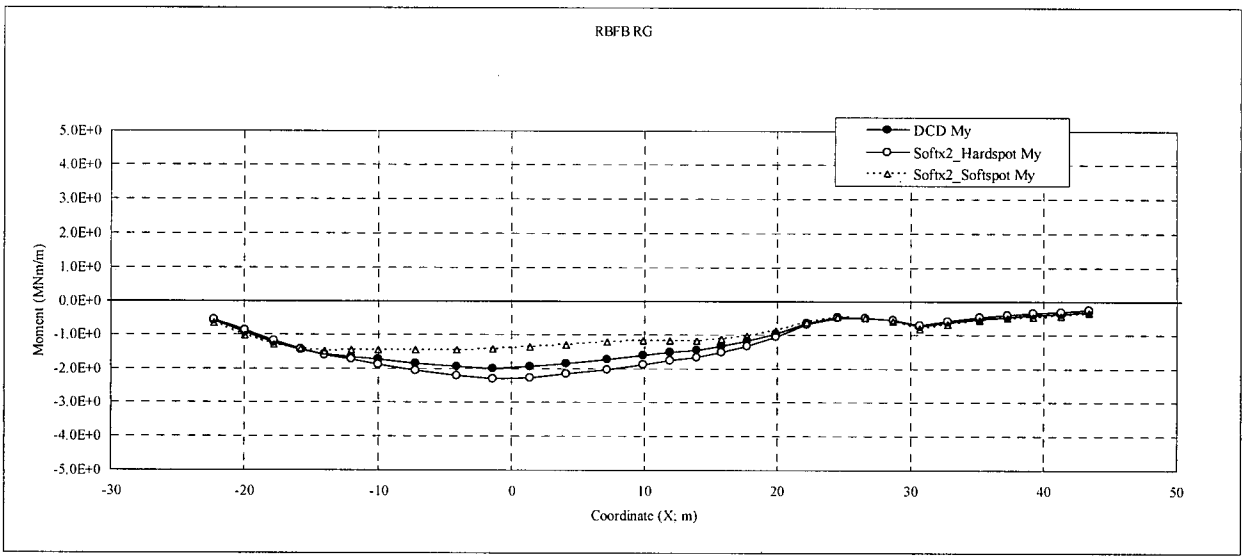


Figure 3.8-93 (23)-d Comparison of Sectional Moments (RG, Soft x2)

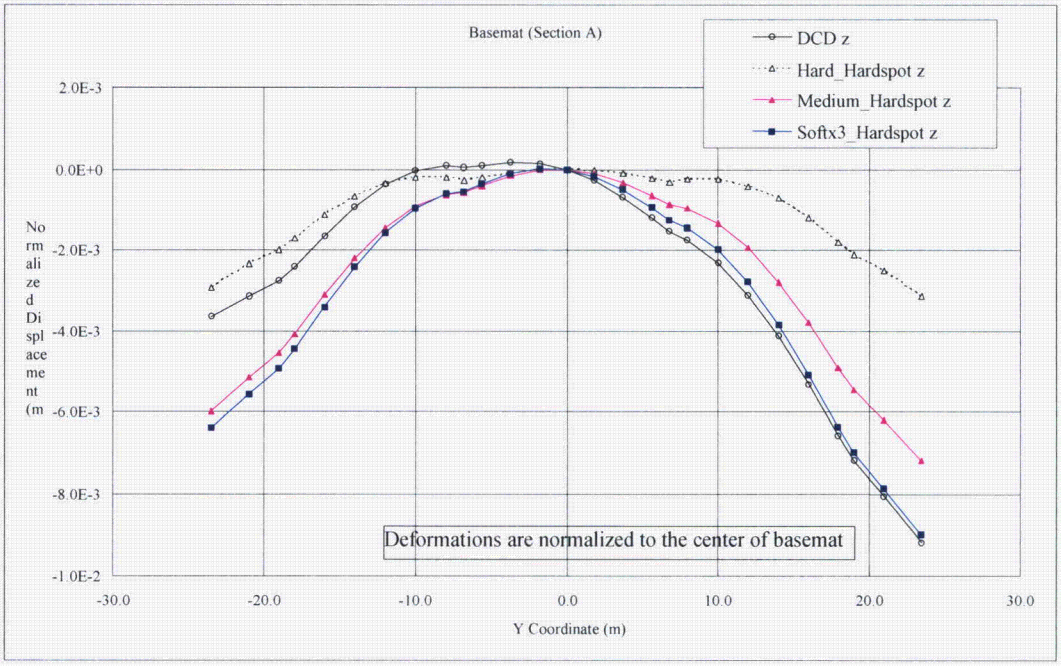


Figure 3.8-93 (24)-a Comparison of Normalized Basemat Displacement (E-W Section)

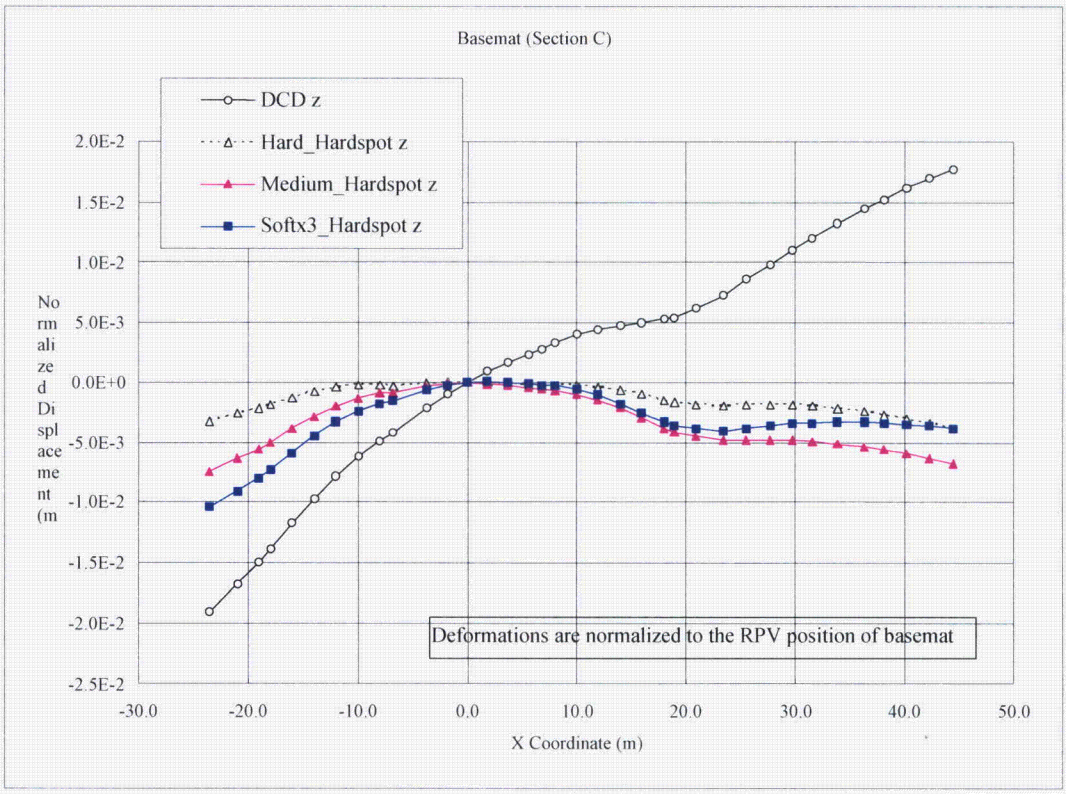


Figure 3.8-93 (24)-b Comparison of Normalized Basemat Displacement (N-S Section)

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 $0.1f_c' = 2.76$ 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

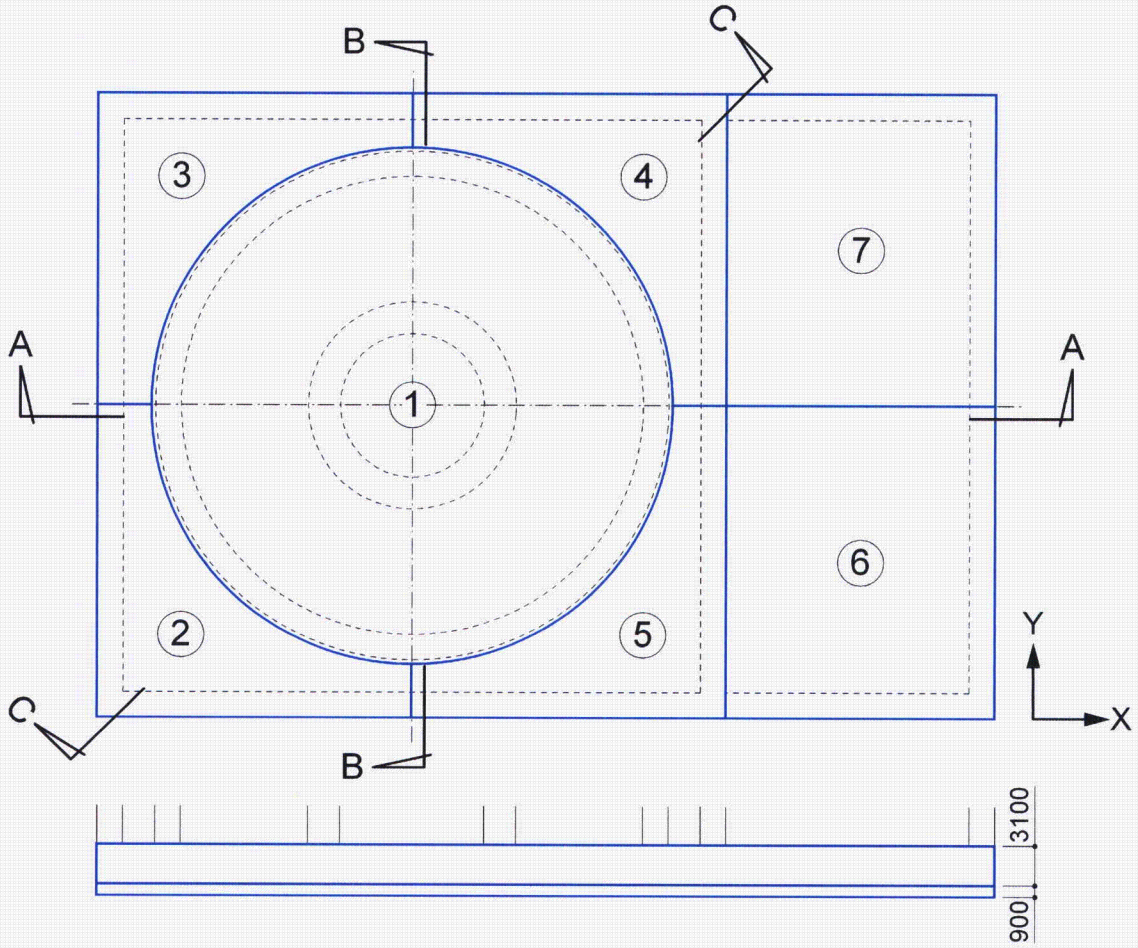
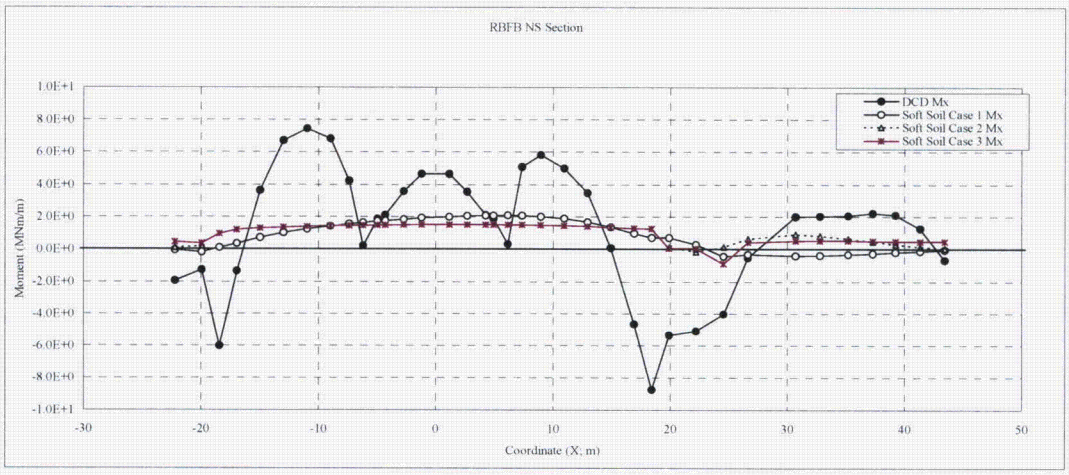
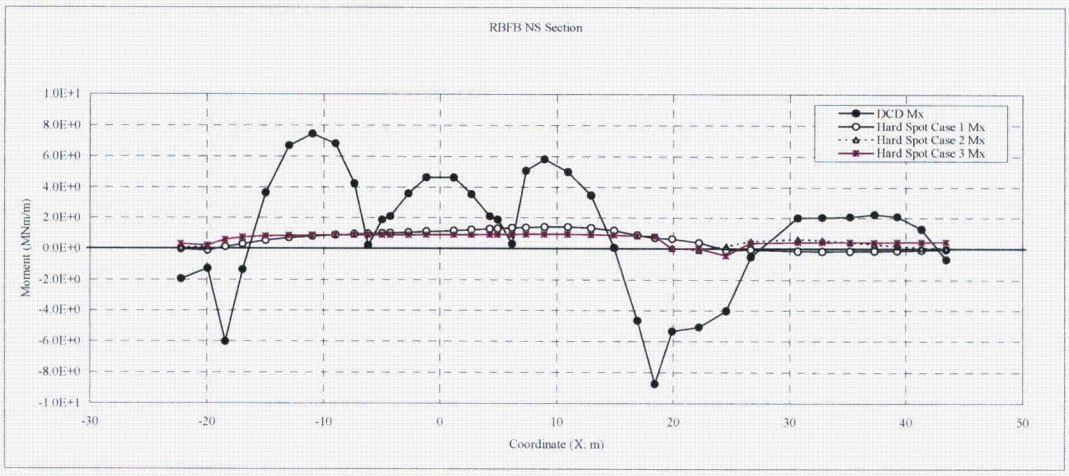


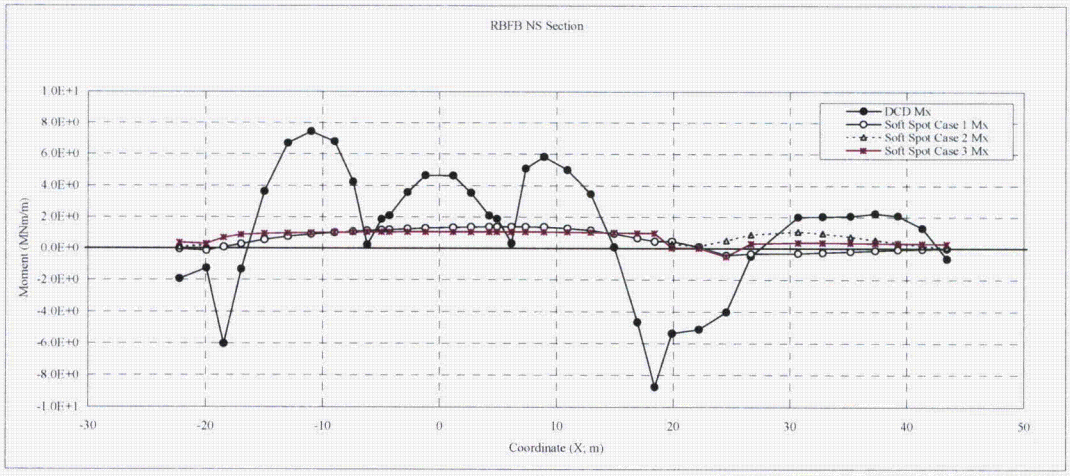
Figure 3.8-93(25) Zone of Basemat for Concrete Pour



(a) Soft Soil

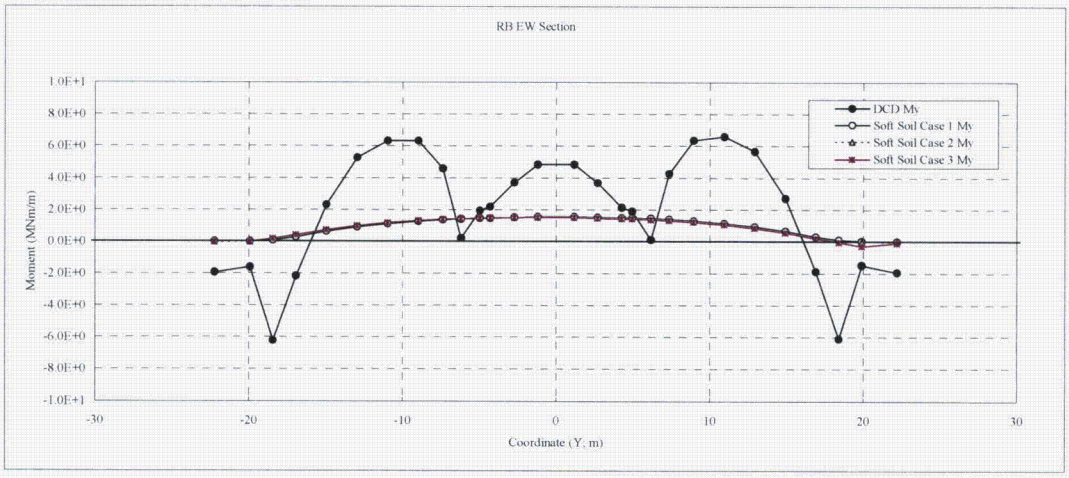


(b) Hard Spot

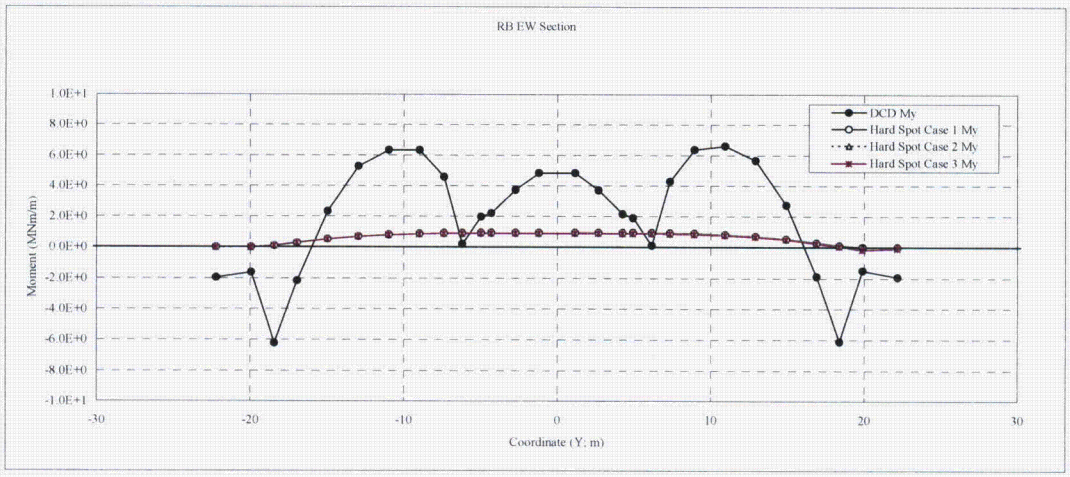


(c) Soft Spot

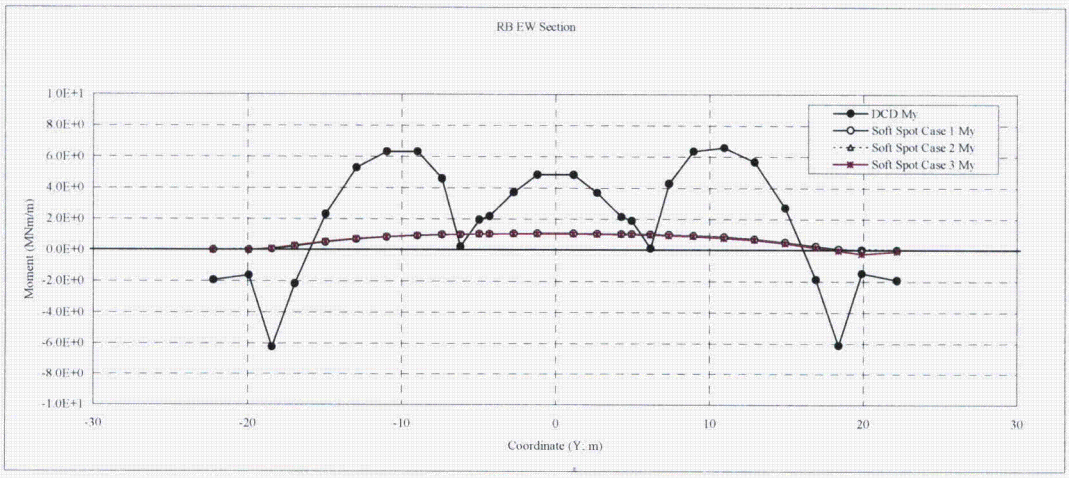
Figure 3.8-93(26) Comparison of Basemat Moment (Section A: Moment in X-dir., Mx)



(a) Soft Soil

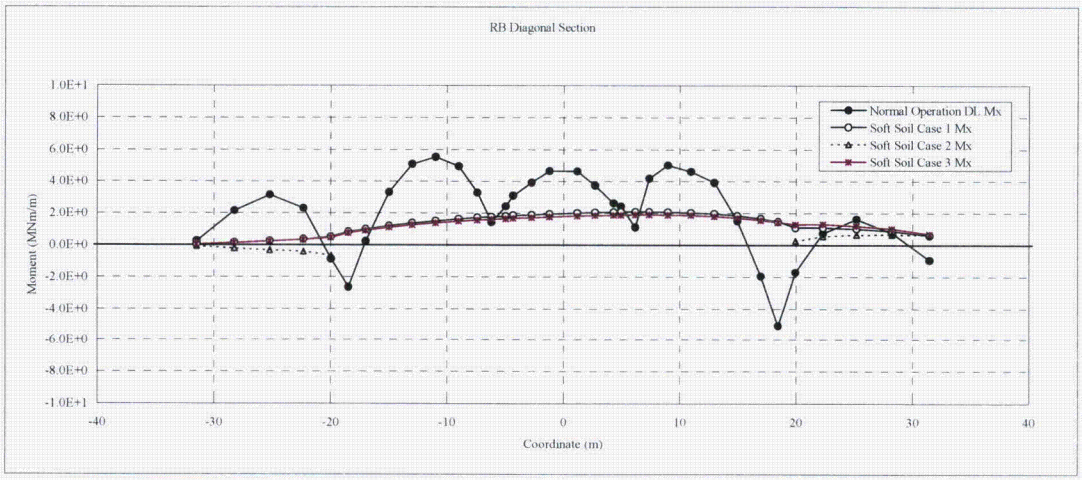


(b) Hard Spot

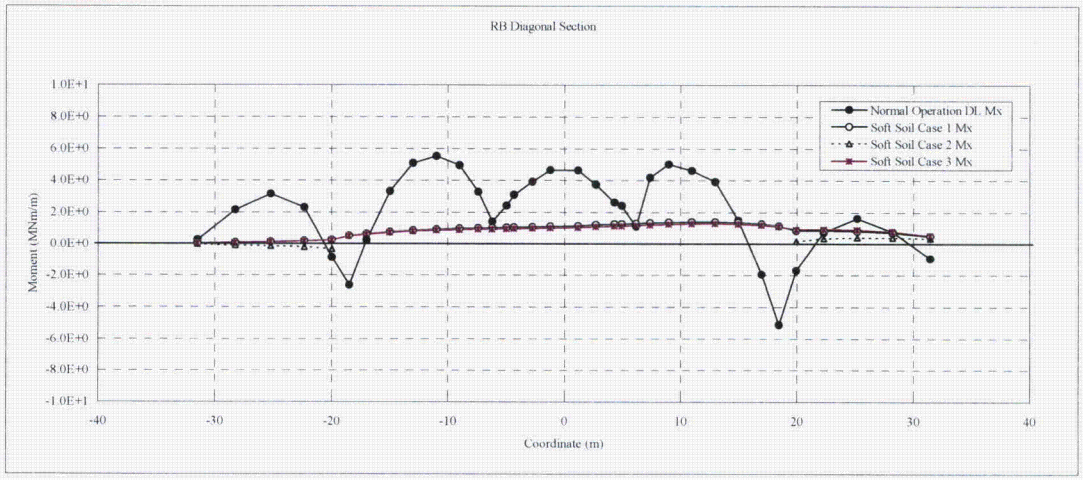


(c) Soft Spot

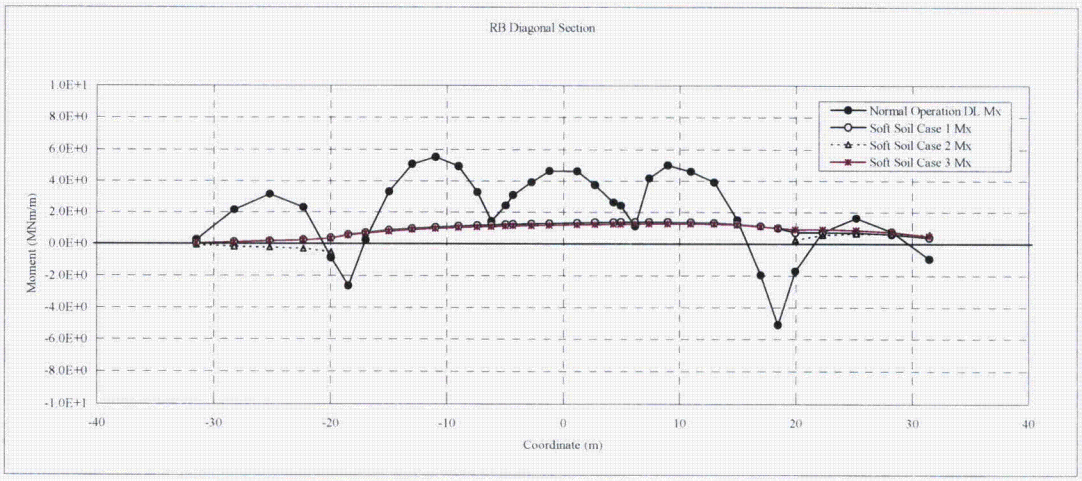
Figure 3.8-93(27) Comparison of Basemat Moment (Section B: Moment in Y-dir., My)



(a) Soft Soil

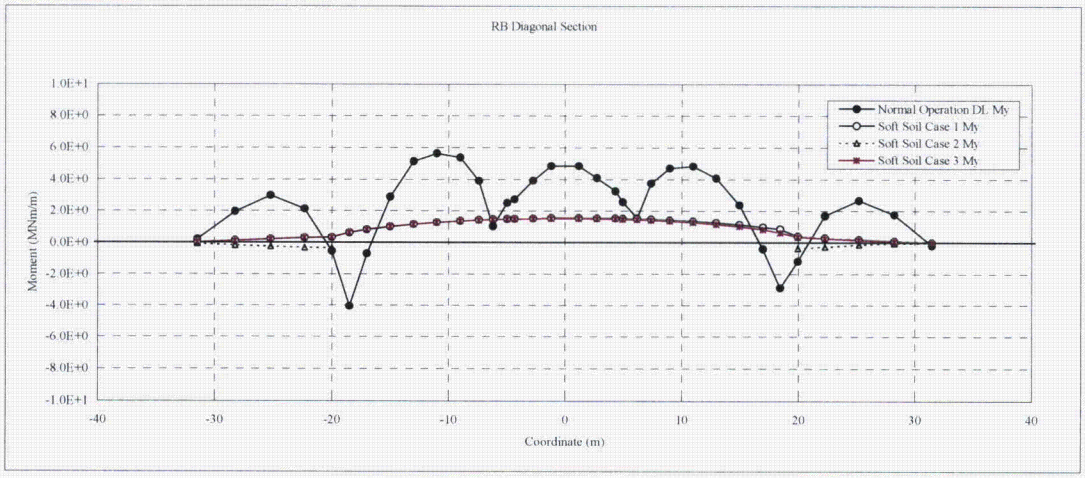


(b) Hard Spot

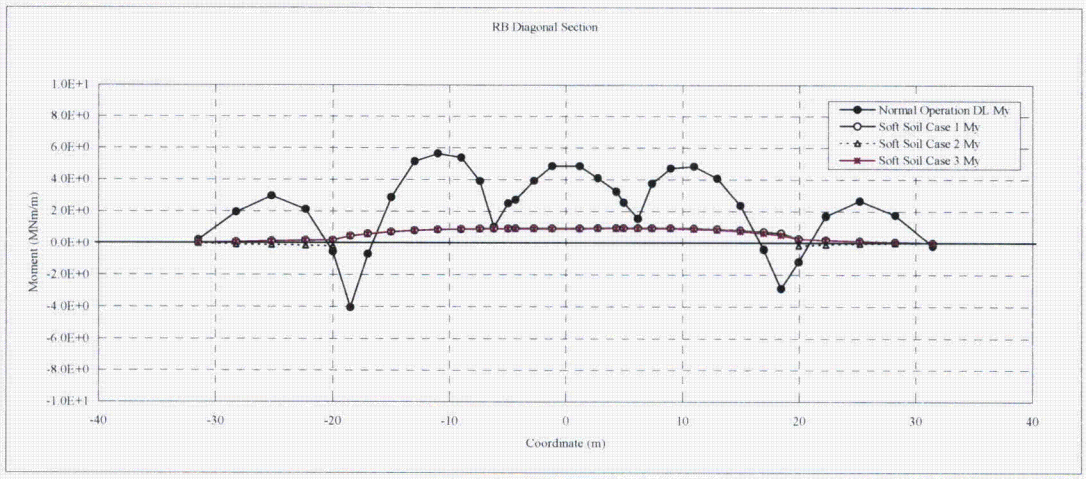


(c) Soft Spot

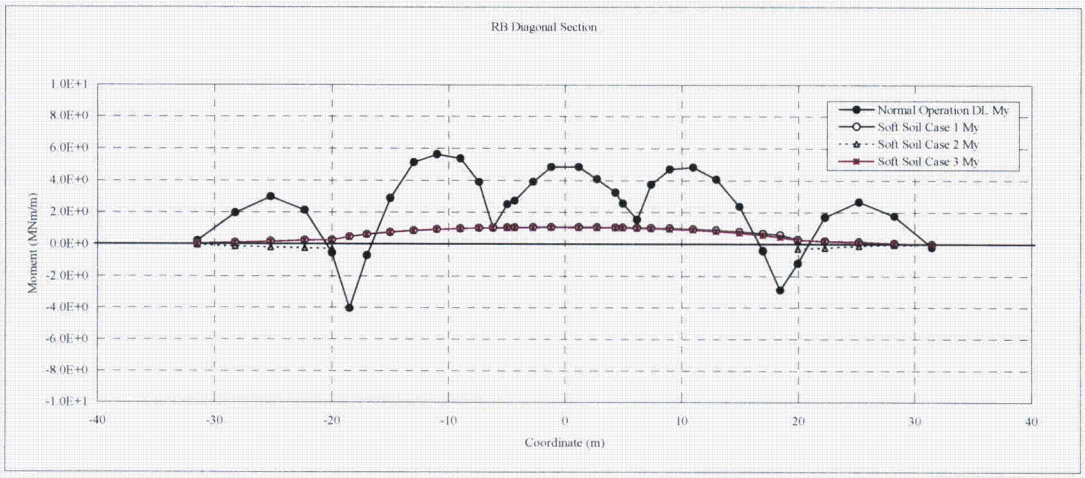
Figure 3.8-93(28) Comparison of Basemat Moment (Section C: Moment in X-dir., Mx)



(a) Soft Soil

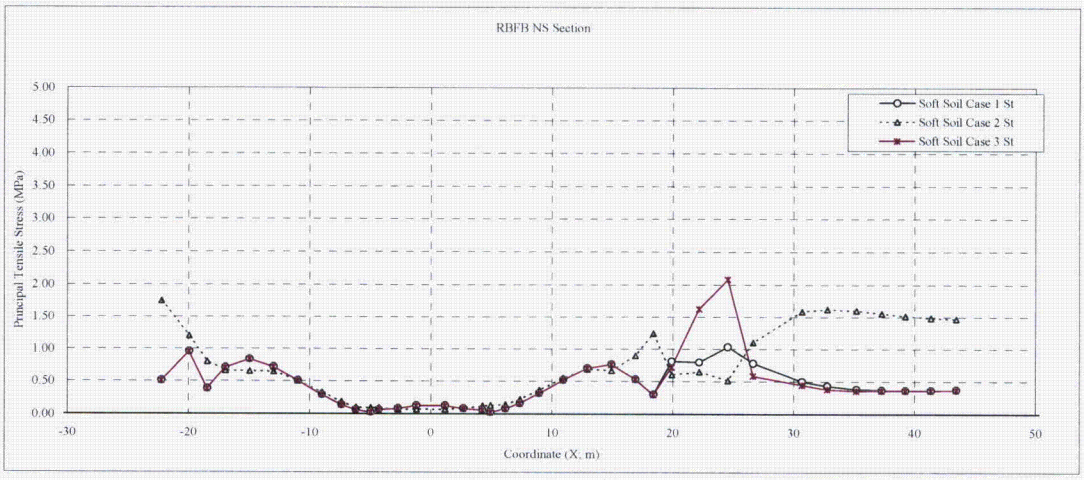


(b) Hard Spot

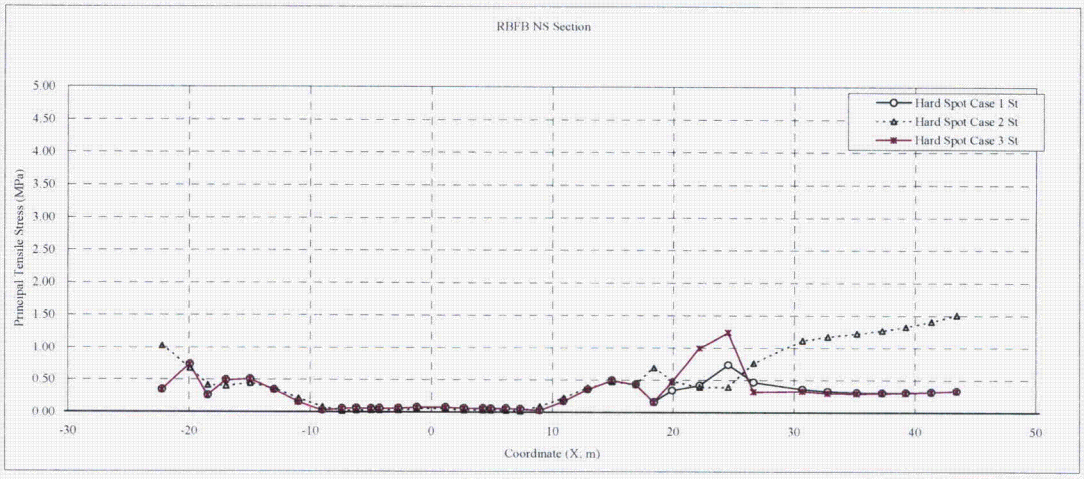


(c) Soft Spot

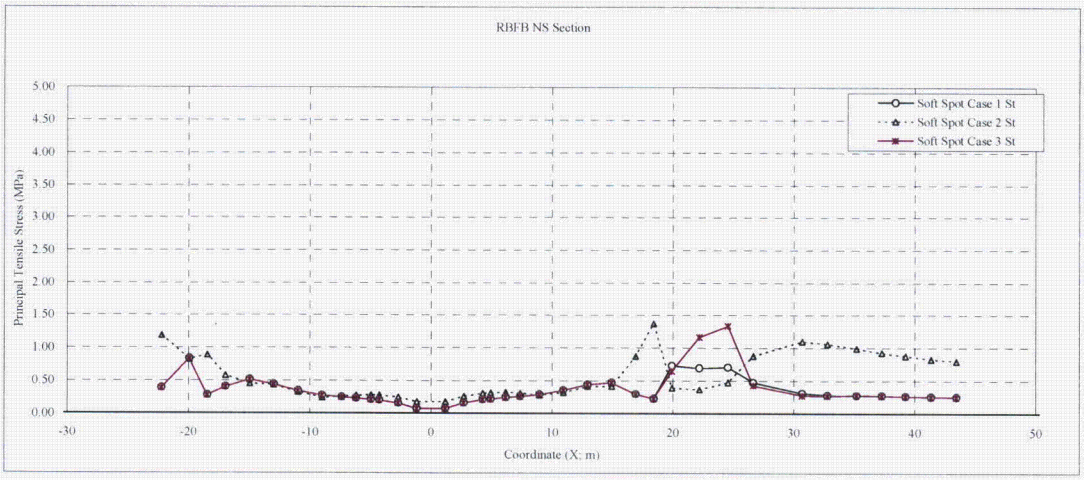
Figure 3.8-93(29) Comparison of Basemat Moment (Section C: Moment in Y-dir., My)



(a) Soft Soil

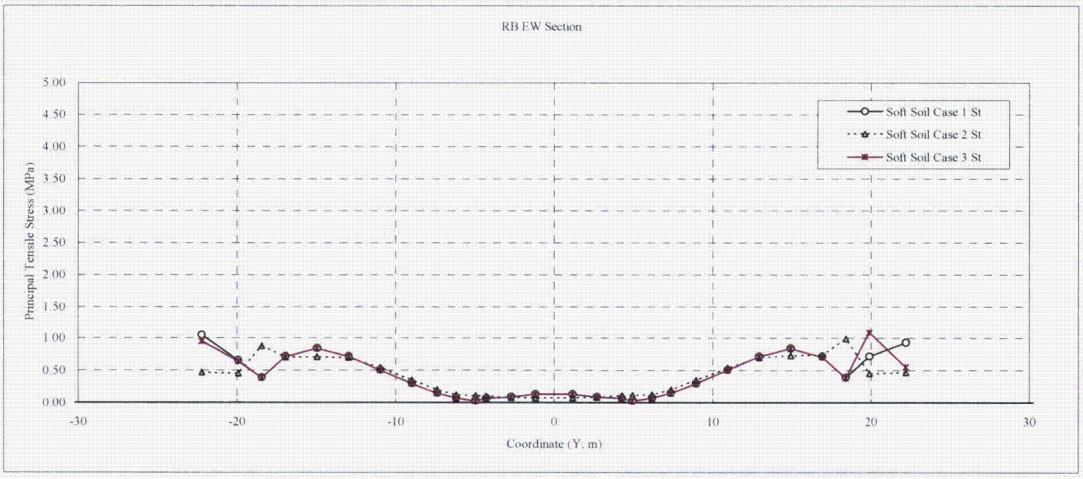


(b) Hard Spot

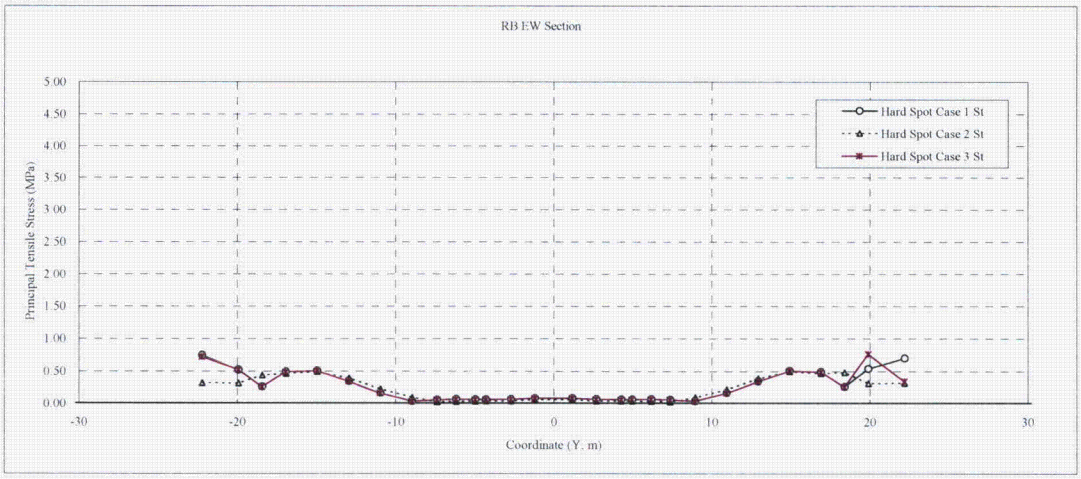


(c) Soft Spot

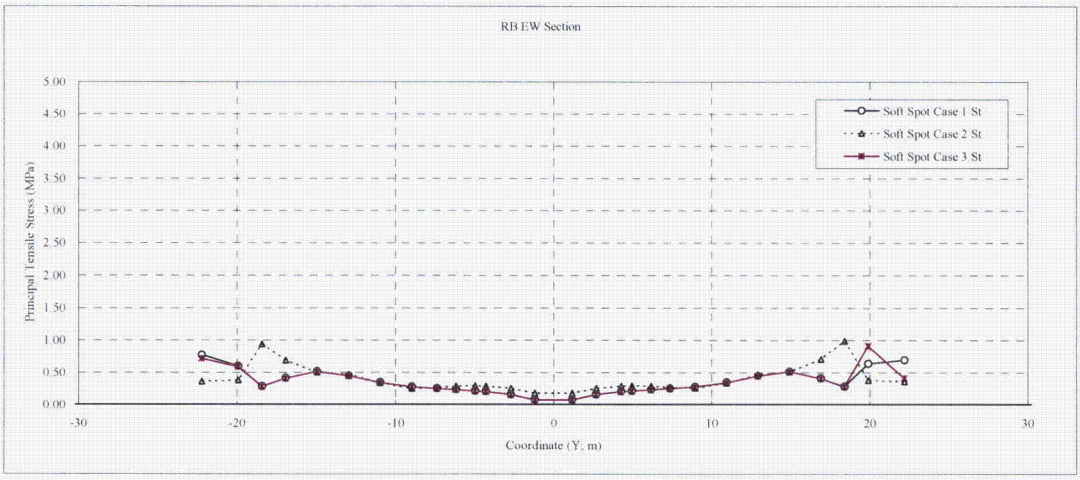
Figure 3.8-93(30) Comparison of Principal Tensile Stress in 0.9 m Thick Slab (Section A)



(a) Soft Soil

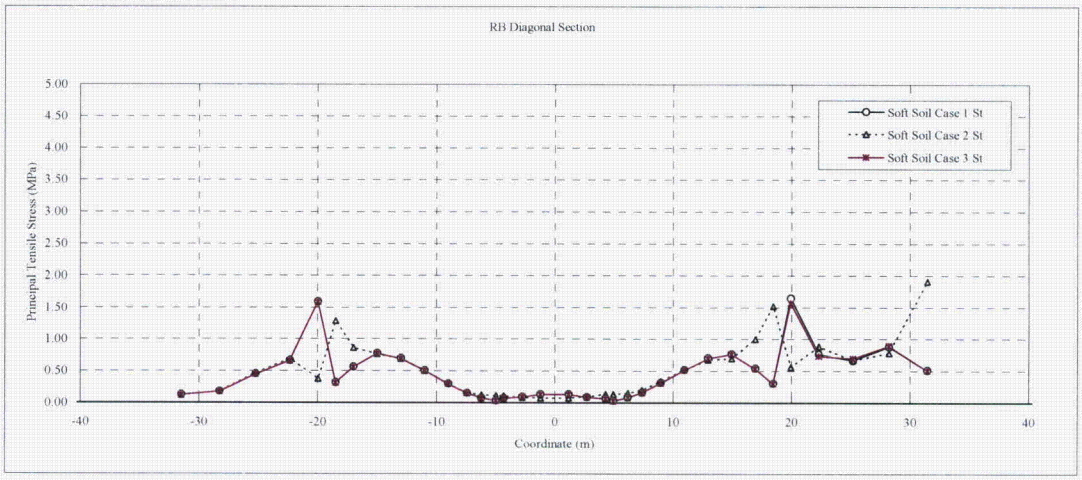


(b) Hard Spot

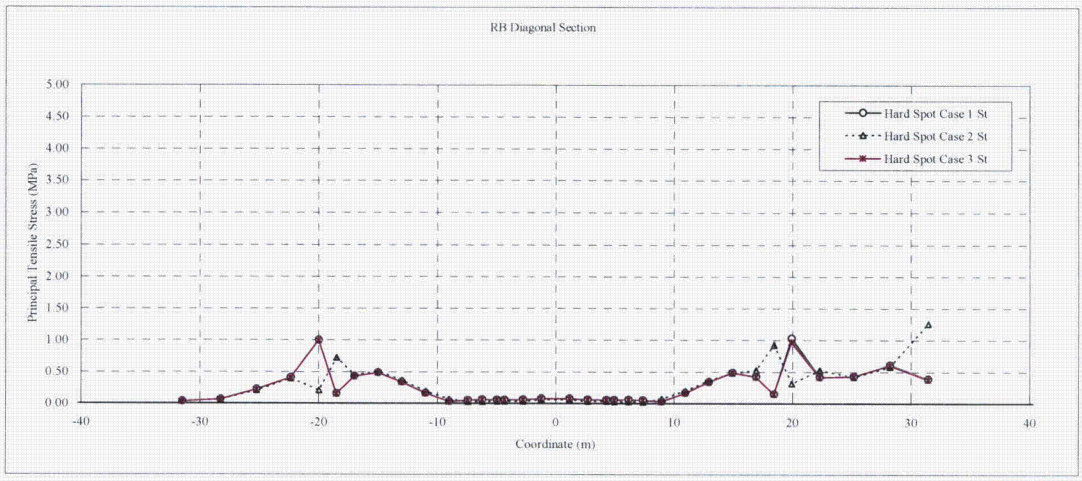


(c) Soft Spot

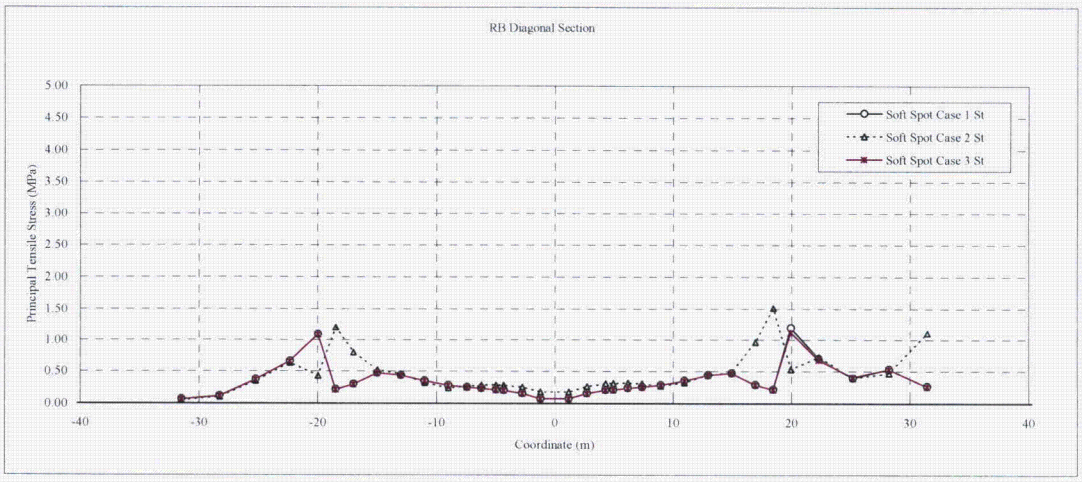
Figure 3.8-93(31) Comparison of Principal Tensile Stress in 0.9 m Thick Slab (Section B)



(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

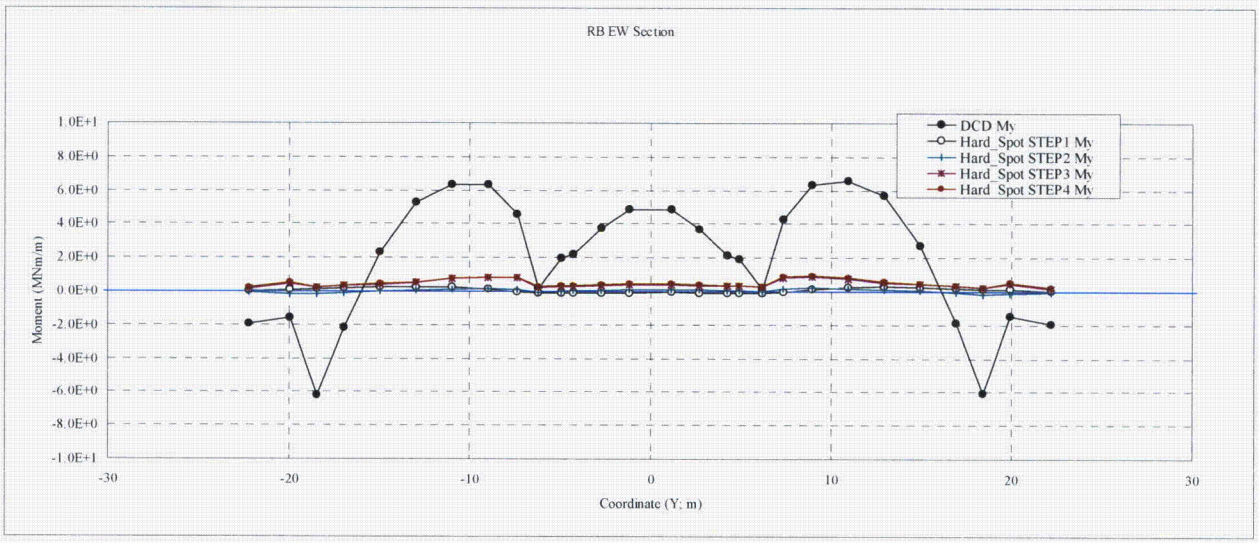
The analytical model has been extracted from the global FEM model used for the DCD design. The dead loads are generated within the program considering element thickness and density of concrete. The analytical conditions are as follows:

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

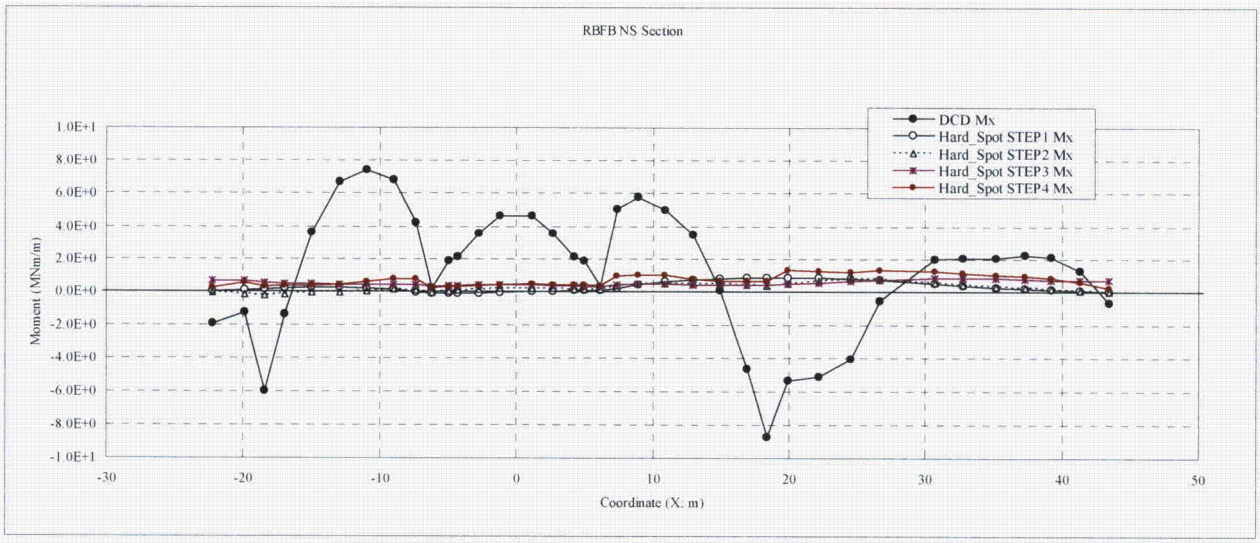
- Soil Condition: "Hard Spot" and "Soft Spot" are shown in Figure 3.8-93 (14). Using the Softx3 is considered as stiffer soil springs under the basemat.
- Load: Dead Load

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

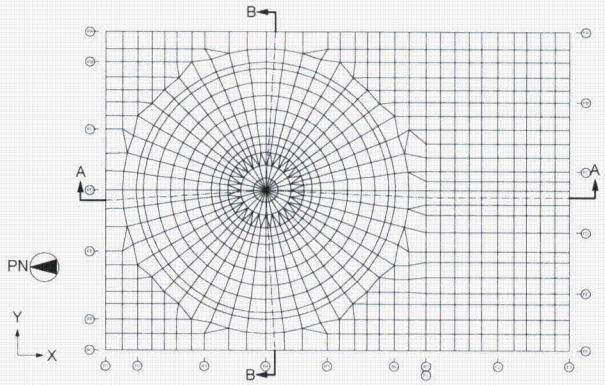
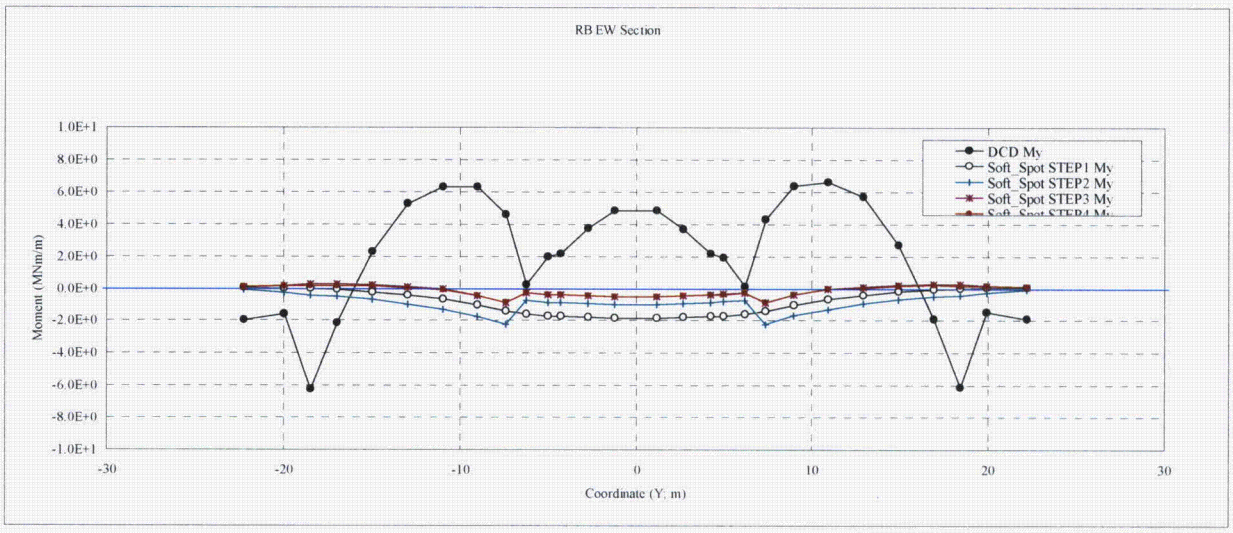
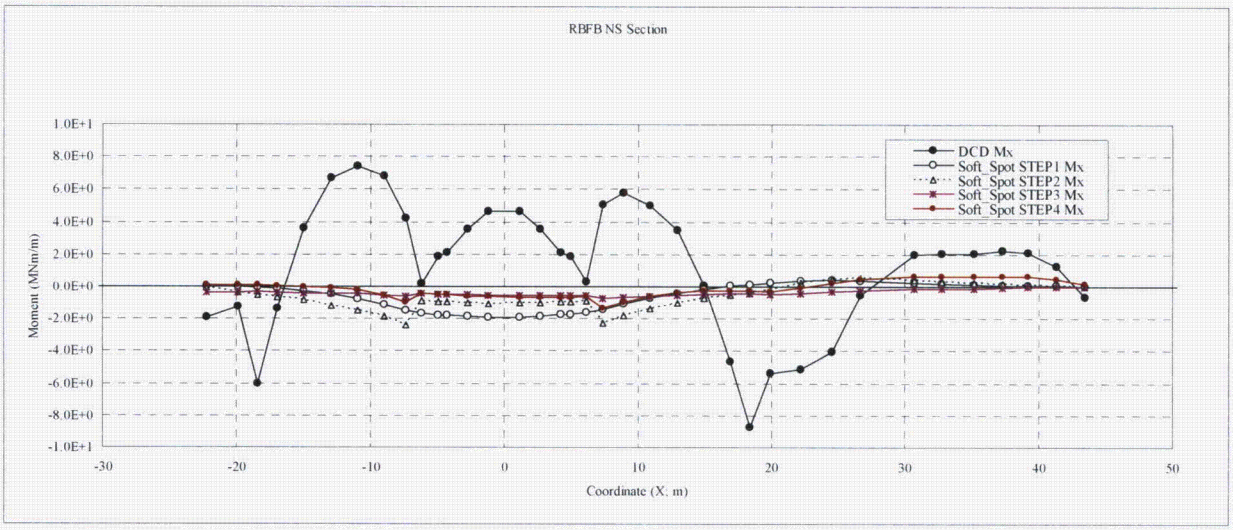


Figure 3.8-93(33) Comparison of Basemat Sectional Moments (Hard Spot)



(a) E-W Section



(b) N-S Section

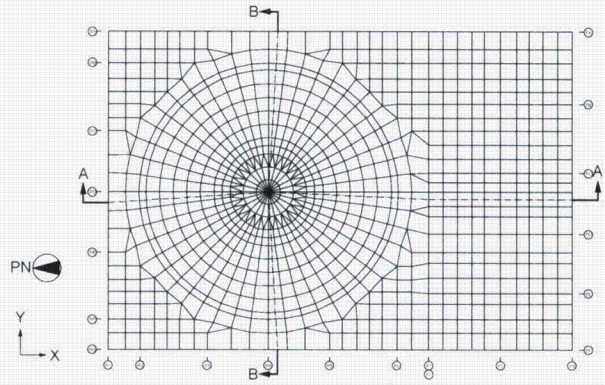


Figure 3.8-93(34) Comparison of Basemat Sectional Moments (Soft Spot)

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 3.8-93, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The staff requests the applicant to address the following:

(1) It is noted that the settlement values in DCD Tier 2 Table 2.0-1 are the same as those specified in DCD Tier 1 Table 5.1-1. The DCD should clearly state that the COL applicant must estimate the settlement by an analysis using actual site conditions and show that they are acceptable when compared to the values specified in DCD Tier 1 Table 5.1-1.

(2) DCD Tier 1 Table 5.1-1 and DCD Tier 2 Table 2.0-1 now requires that the ratio of the largest to the smallest shear wave velocity over the mat foundation width at the foundation level should not exceed 1.7. Clarify that this requirement is imposed to ensure that the bending moments on the basemat do not exceed the design allowable for horizontal soil spring variations that may vary by a factor of 3 from the basemat center to the basemat edge. Also describe how such a variation in shear wave velocity over the mat foundation was considered in the seismic analysis of the RB/FB and Control buildings. If this variation was not considered in the seismic soil structure interaction (SSI) analysis provide the appropriate criteria for the permissible variation in shear wave velocity to be used by the COL applicant, along with the technical bases for the criteria. The criteria in DCD Tier 1 Table 5.1-1 and DCD Tier 2 Table 2.0-1 should be revised accordingly.

(3) In response to this RAI, the applicant has studied several construction sequences and concluded that they have no effect on the basemat design. However, it is difficult to conclude that the worst loading condition has been considered. The criteria in DCD Tier 1 Table 5.1-1 and DCD Tier 2 Table 2.0-1 should require the COL applicant to review the construction sequences considered by GE in the design of the RB/FB and CB buildings. If the COL applicant proposes to use a construction sequence that is substantially different than that studied by GE, the COL applicant should be required to demonstrate that their proposed sequence does not cause a problem.

(4) Figure 3.8-93 (12)-d compares basemat moments resulting from differential settlement that are higher than the moments used for the DCD design condition. Explain why these higher moments are acceptable.

(5) As stated by the applicant in the response, Figure 3.8-93(16)-c indicates that the Softx3 case exceeds the base case (DCD) under the "Hard Spot" condition. The applicant states that for the design allowable, slightly less than 3x soft or hard conditions is used. However, Figure 3.8-93(16)-d indicates that the Softx2 case exceeds the base case (DCD) under the "Hard Spot" condition with results that appear to be identical to those in Figure 3.8-93(16)-c. Clarify what is meant by "slightly less than 3x soft or hard conditions is used." Also compare the moments shown in these two figures with the basemat moment design capacities in both directions across the entire basemat and

provide the technical justification for predicted moments that are higher than the design allowable.

GEH Response

(1) In DCD Tier 2 Subsection 2.0-1-A, the COL applicant referencing the ESBWR DCD is required to demonstrate that the site characteristics, which include settlement values, of a given site fall within ESBWR DCD design parameter values shown in DCD Tier 2 Table 2.0-1. Site-specific settlement values are compared to the settlement values specified in DCD Tier 2 Table 2.0-1 to show that they are bounded. This is performed for the COL application. Settlement values have been deleted from DCD Tier 1 Table 5.1-1 in Revision 4. This is consistent with ABWR and AP1000 DCD Tier 1 in which settlement is not an ITAAC item.

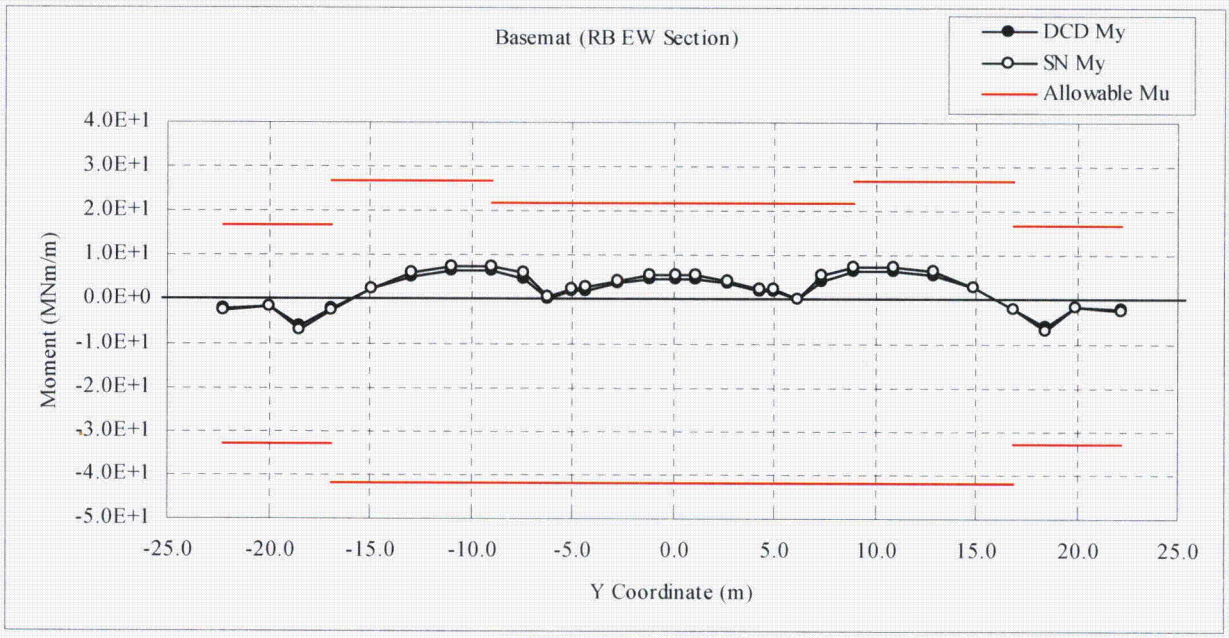
(2) The intent of the 1.7 factor for shear wave variation is to ensure that the bending moments on the basemat do not exceed the design allowable for horizontal soil spring variations that may vary by a factor of 3 from the basemat center to the basemat edge.

Horizontal variation of shear wave velocity is not considered in the seismic SSI analysis. Such variation is not a common practice and is not required since the purpose of the SSI analysis is to predict global structural response. In the SSI analysis the effect of soil uncertainties is typically taken into account in the form of 3 soil profiles, best estimate, lower bound and upper bound. A wide range of shear wave velocity variations is considered in the ESBWR Standard Plant design SSI analysis for generic sites. Therefore, no additional criteria for permissible horizontal variation of shear wave velocity for SSI analysis are required for COL to meet. The existing criteria stating "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" in DCD Tier 2 Table 2.0-1 are sufficient.

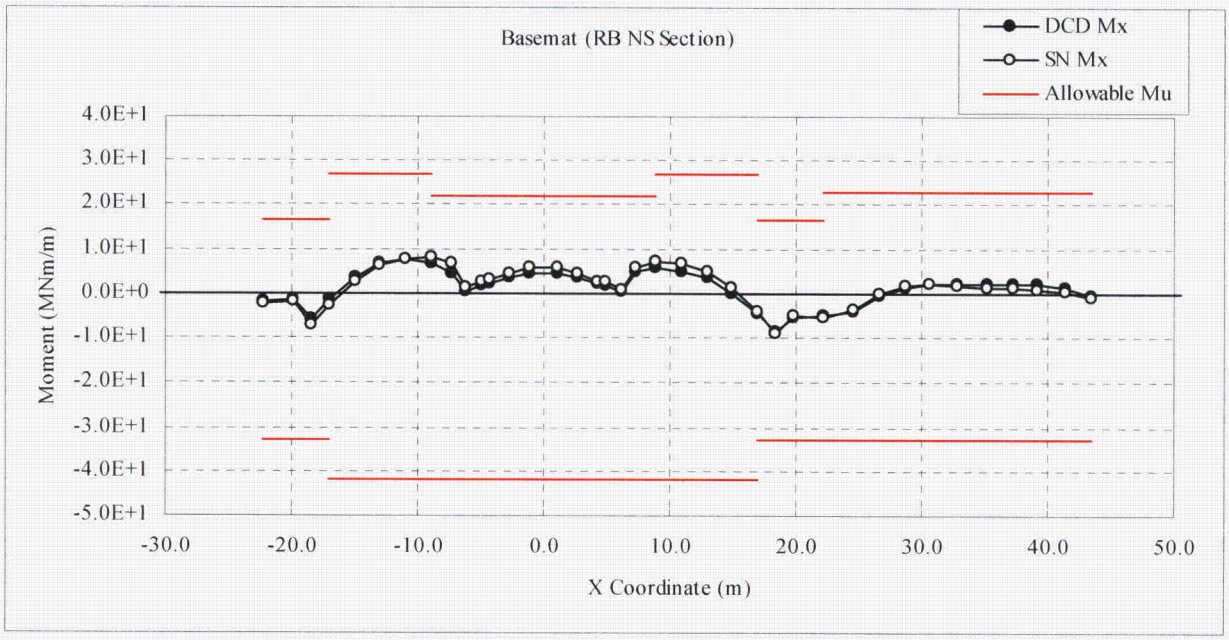
(3) The following sentence will be added to DCD Tier 2 Subsection 3G.1.5.5.3: "The actual construction sequence, if substantially different from the sequences considered in the design, is ensured not to adversely impact the basemat design".

(4) The higher moments resulting from differential settlement are acceptable since they are well within the allowable moment capacities, calculated using beam theory, at various sections of the basemat. Figure 3.8-93(35) is an update to Figure 3.8-93(12)-d that shows the allowable moment capacities.

(5) The statement "slightly less than 3x soft or hard conditions is used" is intended to mean that the soft×3 case is selected as a limiting horizontal variation of soil springs since the soft×3 and soft×2 results presented in Figures 3.8-93(16)-c and 3.8-93(16)-d are essentially the same. Figures 3.8-93(36) and 3.8-93(37) are updates to Figures 3.8-93(16)-c and 3.8-93(16)-d, respectively, that show the allowable moment capacities, calculated using beam theory, at various sections of the basemat. As shown, the predicted basemat moments for the soft×3 and soft×2 cases are well within the design capabilities.

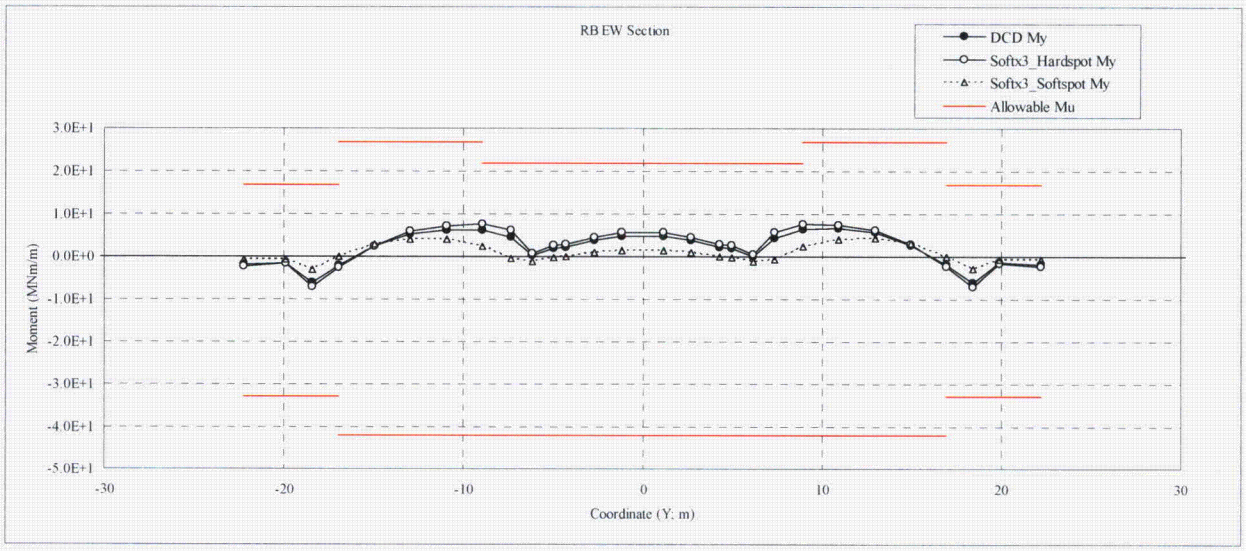


(a) My in E-W Section

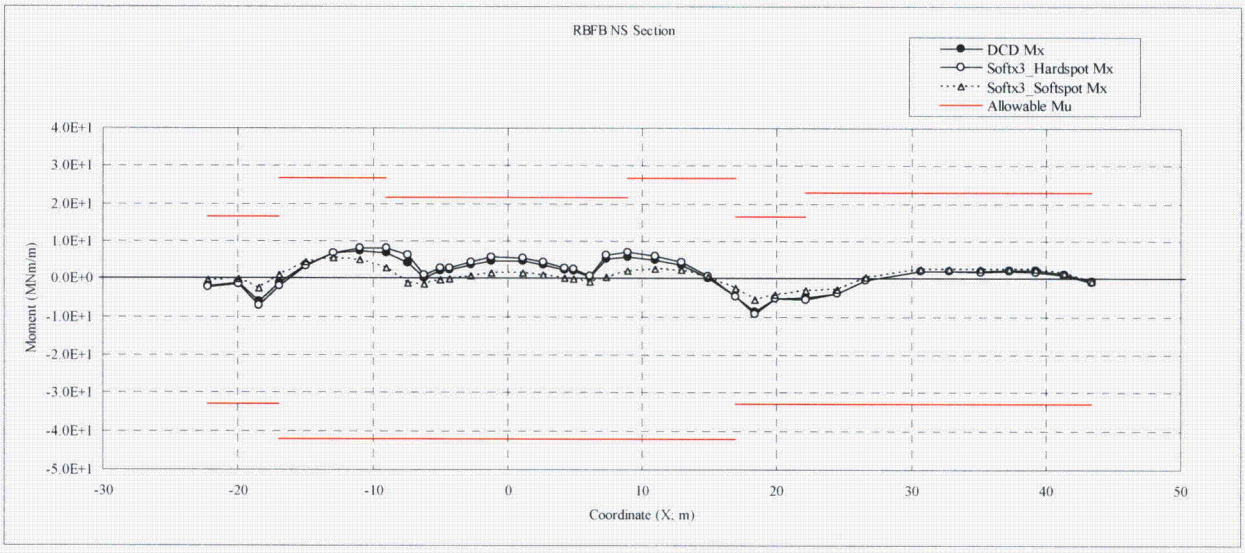


(b) Mx in N-S Section

Figure 3.8-93(35) Comparison of Basemat Sectional Moments (RBFB, S-N direction)

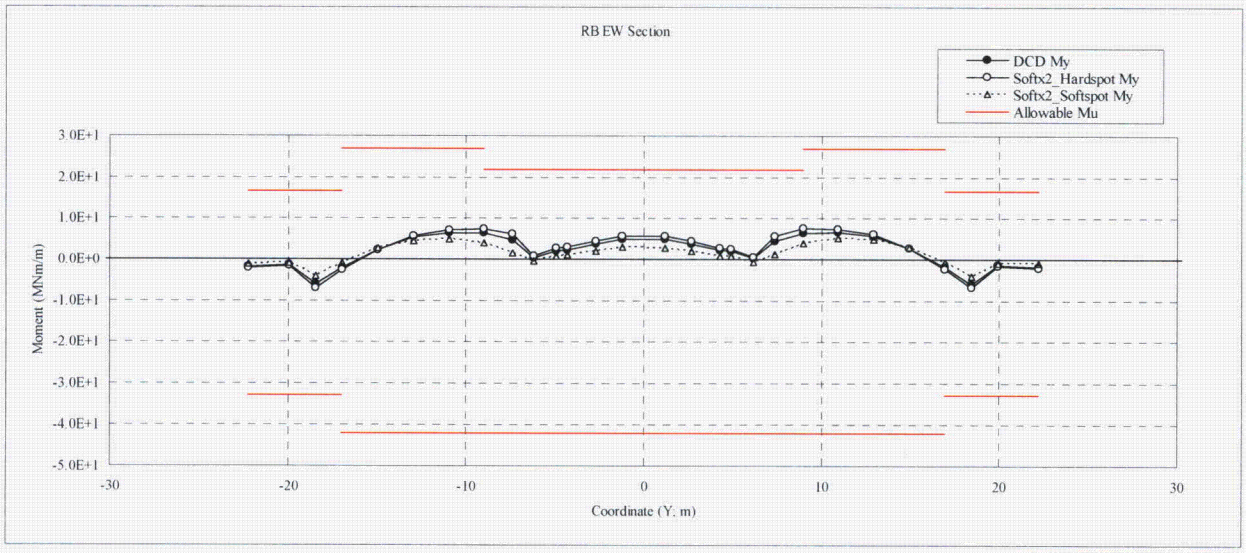


(a) My in E-W Section

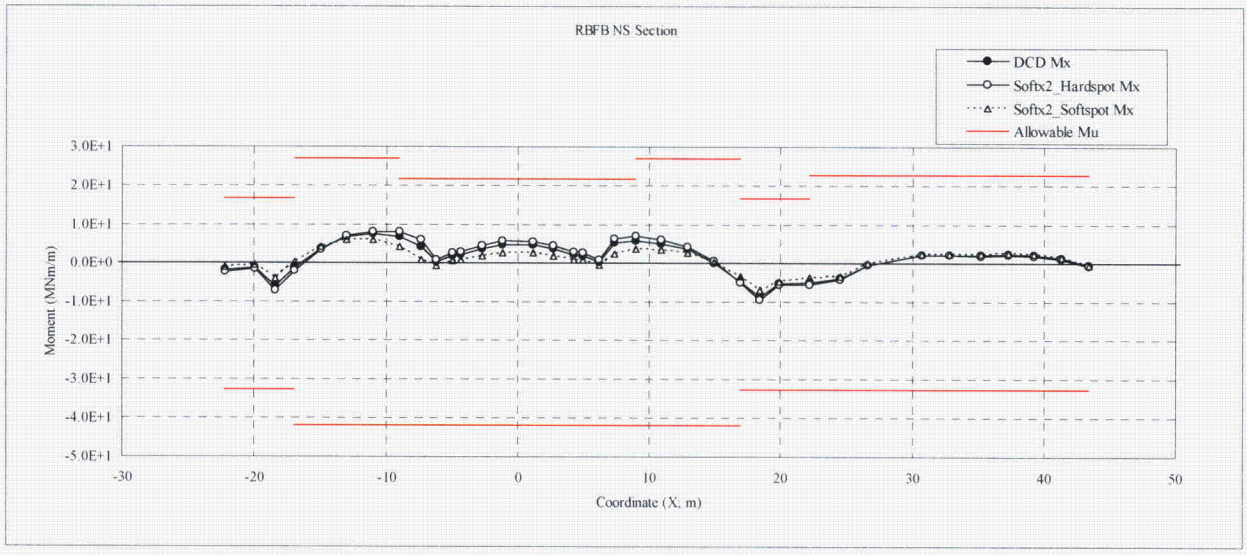


(b) Mx in N-S Section

Figure 3.8-93(36) Comparison of Basemat Sectional Moments (soft x3)



(a) My in E-W Section



(b) Mx in N-S Section

Figure 3.8-93(37) Comparison of Basemat Sectional Moments (soft x2)

DCD Impact

DCD Tier 2 Subsection 3G.1.5.5.3 will be revised in the next update as noted in the attached markup.

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.

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

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

NRC RAI 3.8-94, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The staff requests the applicant to address the following:

(1) The bearing stresses reported in DCD Tier 2 Table 3G.1-58 for soft, medium and hard site conditions are 2.7 MPa (56.4 ksf), 7.3 Mpa (152.6 ksf) and 5.4 MPa (112.9 ksf). These values are extremely large compared to known soil and rock capacities. Explain how the COL applicant will satisfy this criteria. Also explain why the bearing stress reported for the medium site condition (7.3 MPa) is higher than the hard site condition (5.4 MPa).

(2) Explain how the COL applicant is to use the maximum bearing pressures reported in DCD Tier 2 Table 3G.1 58 and Table 3G.2 27 when conditions for a specific site fall between the tabulated values for soft, medium and hard site conditions.

(3) Footnote 7 to DCD Tier 2 Table 2.0-1 references DCD Tier 2 Subsections 3G.1.5.5, 3G2.5.5 and 3G.3.5.5 for the minimum dynamic bearing capacities for the Reactor, Control and Fuel Building, respectively. However, Footnote 7 to the corresponding DCD Tier 1 Table 5.1-1 only states "At foundation level of Seismic Category I structures." Explain why the minimum dynamic bearing capacities are not clearly specified as Tier 1 information.

(4) The response to RAI 3.8-94 states that variations in the horizontal soil spring were considered and concludes that the maximum soil bearing pressures of the nonuniform soil condition are similar to those of the uniform soil condition. Results for maximum bearing pressure under non-uniform soil conditions are presented in Table 3.8-94(3). To complete the response, for the nonuniform soil conditions considered in Table 3.8-94(3), provide comparisons of the bending moments across the basemat in both directions that demonstrate that the DCD design moments bound the moments for the nonuniform soil condition.

GEH Response

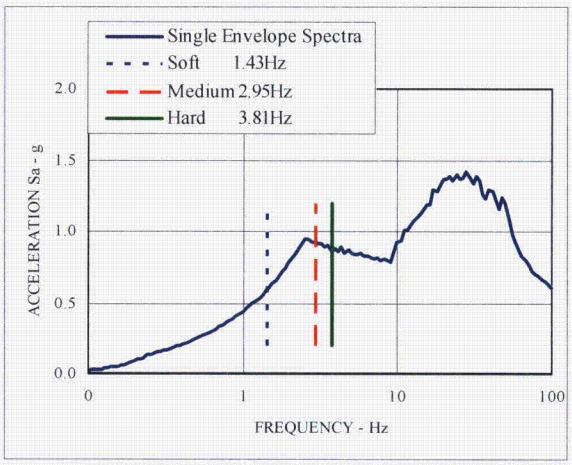
(1) Confirmation of bearing capacity is a COL item as stated in DCD Tier 2 Table 2.0-1. The higher bearing stress at the medium site condition is due to the higher spectral acceleration of the input ground motion response spectra at the SSI fundamental frequencies as shown in Figure 3.8-94(1) in comparison with other site conditions for each direction. Consequently, the envelope of the soil reaction forces, which are the basis for calculating the bearing pressures, are the largest at the medium site as shown in Table 3.8-94(4).

(2) When specific site conditions fall between the cases specified, the larger value within the applicable range applies. Alternatively, a linearly interpolated value may be used and is clarified in footnotes to DCD Tier 2 Revision 4 Tables

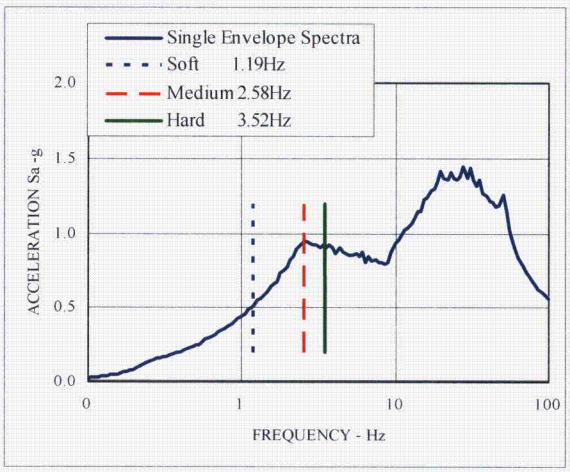
3G.1-58 and 3G.2-27. The revised pages 3G-123 and 3G-228 in DCD Tier 2 Revision 4 are attached.

(3) Minimum dynamic bearing capacities have been included in DCD Tier 1 Revision 4 Table 5.1-1. The revised page 5.1-3 in DCD Tier 1 Revision 4 is attached.

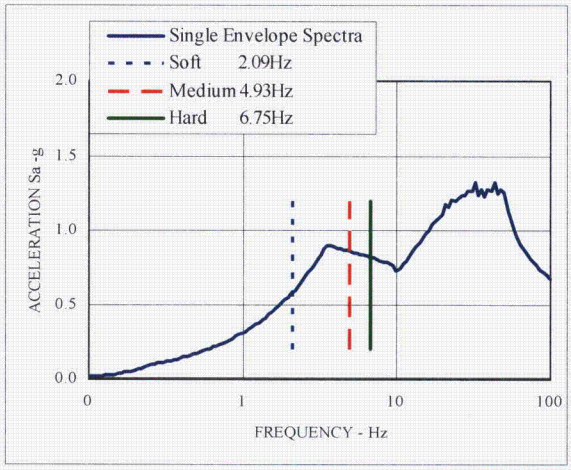
(4) Table 3.8-94(3) is a summary of the analyses results presented in the response to NRC RAI 3.8-93, Supplement 1. The comparisons of the bending moments across the basemat were provided in Figure 3.8-93(16)-c. In that figure Hard Spot case is higher than DCD condition. The allowable bending moment at the top surface of the basemat is 16.7 MNm/m using the rebar ratio (0.321%) shown in DCD Tier 2 Table 3G.1-50. Therefore, it is concluded that the hard spot results do not affect section design in the DCD. Also, DCD Tier 2 Tables 3G.1-51 through 3G.1-55 show rebar and concrete stresses. These calculated stresses are sufficiently lower than Code allowable limits.



(a) NS Direction



(b) EW Direction



(c) Vertical Direction

Figure 3.8-94(1) Input Motion Spectra and RBFB SSI Fundamental Frequencies

Table 3.8-94(4) Soil Spring Reaction for RBFB Seismic Model

	Envelope Soil Reaction	
Soft	N(MN)	M(MNm)
NS	-	22094
EW	-	31999
V	676	-
Bearing Pressure	2.7 Mpa	
Medium	N(MN)	M(MNm)
NS	-	48131
EW	-	58908
V	1148	-
Bearing Pressure	7.3 Mpa	
Hard	N(MN)	M(MNm)
NS	-	50238
EW	-	47061
V	1003	-
Bearing Pressure	5.4 Mpa	

DCD Impact

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

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 will be 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 3.8-96, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The applicant has not used a consistent set of criteria to determine the safety factor against sliding and also needs to provide the technical bases for some of the parameters used in the analysis results that are presented. The staff requests the applicant to address the following:

(1) The fourth bullet in the list of items that comprise the sliding resistance is identified as "passive soil pressure on walls." This terminology is misleading since the information included under this item is the elastic lateral soil pressure. If passive soil pressures are being credited to provide sliding resistance, explain how these pressures are calculated and confirm that the walls are designed to resist these forces. If elastic lateral soil pressures on the walls are being credited to provide sliding resistance, it is not consistent to use these elastic soil pressures with the passive soil pressures at the basemat side surface. Also, explain how the passive soil pressures are calculated for the basemat side surface.

(2) Passive soil pressure at the basemat side surface is being credited to provide sliding resistance, which means that the static friction resistance at the bottom of the basemat is overcome. Therefore, explain why a dynamic coefficient of friction is not used to calculate the friction force at the basemat bottom surface.

(3) How has GE determined that there are sufficient soil sites that would have an angle of internal friction of 30 degrees or greater? What would a COL applicant be required to do if a site has a soil friction angle of less than 30 degrees?

(4) Provide a description of the formulations used to calculate the cohesion resisting forces and discuss how the material properties were determined for the analysis.

(5) Provide the technical basis for assuming that medium soils with an angle of internal friction of 30 degrees would also have the effective cohesion resisting forces reported in the analysis results in Table 3.8-96(1). Why is the cohesion value in Table 3.8-96(1) equal to zero for soft soils?

(6) Provide the technical basis for assuming that the hard soil/rock conditions have the effective cohesion resisting forces reported in the analysis results in Table 3.8-96(1).

(7) Why does the response indicate that the cohesion force contribution to total force is small when Table 3.8-96(1) shows that it is quite large for hard soils? For the RBFB medium soil condition, a small change in the cohesion force could result in a factor of safety of less than 1.1. In the light of these observations, further justification is needed to support the statement that the reduction of the cohesion due to uplift has little impact on the total resistance.

(8) Describe the COL requirements for the backfill material for the gap shown in Figures 3G.1-65 and 3G.2-15. Will the backfill material be required to have a stiffness defined by its shear wave velocity which is at least equal to the shear wave velocity of the surrounding insitu soil? If not, explain why not. Also, clarify that the backfill material will completely fill the gap above the concrete backfill to the grade level.

(9) The footnote in Table 3.8-96(1) implies that the 100-40-40 three directional combination method was used for the sliding evaluation. The data in the tables above the footnote, however indicate that a two dimensional (one horizontal and one vertical) check was made for calculating the factor of safety. In this evaluation the bottom friction force is derived based on the total vertical load consisting of dead weight minus the buoyancy effect minus 0.40 times the vertical seismic force. Since a simplified two dimensional approach (i.e., N-S & Vertical and then E-W & Vertical) is being used to demonstrate the factors of safety against sliding and overturning, the 100-40-40 rule is not considered to be appropriate. The typical approach that is utilized for checking sliding and overturning in accordance with the SRP 3.8.5 requirements is to use the dead load minus the buoyancy effect and then subtract the full vertical seismic load for the N-S & Vertical check and the E-W & Vertical check. If any other method is utilized, then GE needs to provide the technical justification for the approach. Note that 90% of the dead load (including the buoyancy effect) should be utilized as specified in footnote 1 of DCD Table 3.8-15, which is also in accordance with ACI 349 requirements.

GEH Response

(1) In the calculations shown in Table 3.8-96(1), elastic lateral soil pressures on the walls were credited to provide sliding resistance. This is conservative for sliding evaluation since actual passive pressures, if mobilized, would be higher. Wall design is based on elastic lateral soil pressures. As discussed in the response to Item (4), the required factor of safety can be satisfied without considering the sliding resistance from the elastic lateral soil pressures. Passive pressure is mobilized on the side surface of the basemat since the basemat is constrained by rigid concrete backfill. The passive pressure at the basemat side is calculated using the following equations:

$$P_p = k_p \gamma' H + \gamma_w H_w + k_p q + 2C\sqrt{k_p}$$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

where,

k_p = Passive pressure coefficient
H = Height of soil column
H_w = Height of water column
γ' = Effective weight of soil. Use buoyant unit weight below water table and moist unit weight above water table.
γ_w = Unit weight of water
q = Magnitude of surcharge load per unit area

ϕ = Angle of internal friction of soil
C = Cohesion

The stress in the basemat generated by passive soil pressures is 2.45 MPa for the Hard site condition and is less than 10% of the concrete compressive strength. The stress is acceptable for the basemat design.

(2) The shear strength of soil, i.e., the resistance at the basemat bottom, is composed of friction and cohesion. It is generally recognized that the strength of soil for dynamic loads is larger than that for static loads. Therefore, calculations using static coefficient of friction, i.e., calculations based on the static strengths, are conservative.

(3) Table 2-6 from Reference 1 shows that a 30° angle of internal friction is a reasonable lower bound for competent soil material. A site-specific sliding evaluation would be performed if the angle of friction of the site-specific foundation material is lower than 30°. In DCD Tier 2 Subsection 2.0-1-A, the COL applicant referencing the ESBWR DCD is required to demonstrate that the site characteristics, which includes angle of internal friction, of a given site fall within ESBWR DCD design parameter values shown in DCD Tier 2 Table 2.0-1.

TABLE 2-6 Representative values for angle of internal friction ϕ

	Type of test*		
Soil	Unconsolidated-undrained U	Consolidated-undrained CU	Consolidated-drained CD
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28–34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		30–35°
Clay	0° if saturated	3–20°	20–42°

* See a laboratory manual on soil testing for a complete description of these tests, e.g., Bowles (1986*b*).

Notes:

1. Use larger values as γ increases
2. Use larger values for more angular particles
3. Use larger values for well-graded sand and gravel mixtures (BGW, SW)
4. Average values for
   Gravels: 35–38°
   Sands: 32–34°

(4) In Reference 1 it is stated that the ultimate bearing capacity, q_u, can be nine times cohesion, c. In the same reference, it is suggested to use 0.5 to 0.7 of c for sliding stability evaluations. That is, the cohesion used for sliding evaluations, c', can be evaluated by the following equation as a function of the ultimate bearing capacity:

$$c' = 0.5 \times q_u / 9 = q_u / 18$$

The expected ultimate bearing capacities of the ESBWR design need to be larger than the maximum soil bearing stresses summarized in the DCD Tier 2 Table 3G.1-58 for the RBFB and Table 3G.2-27 for the CB, respectively. These are the demand pressures.

Assuming the demand pressures are the actual ultimate bearing capacities, the associated cohesions can be conservatively evaluated by substituting the maximum soil bearing stresses into q_u in the above equation. The resulting cohesions are summarized in Table 3.8-96(2). The sliding stability evaluations were updated using these cohesions. The results are shown in Table 3.8-96(3). The calculated factors of safety (FS) satisfy the allowable value of 1.1. In DCD Tier 2 Revision 4, Tables 3G.1-57 and 3G.2-26 were revised in accordance with the results in Table 3.8-96(3). The revised pages 3G-123 and 3G-228 in DCD Tier 2 Revision 4 are attached.

In the calculations in Table 3.8-96(3), the elastic lateral soil pressures on the walls discussed in Item (1) above are conservatively neglected. The passive pressure utilized is only at the basemat side as described Item (1) above.

(5) See response to Item (4) where cohesion is taken to be a function of the ultimate bearing capacity.

(6) See response to Item (4) where cohesion is taken to be a function of the ultimate bearing capacity.

(7) According to the basemat uplift analysis results, which are shown in the DCD Tier 2 Figures 3G.1-60 and 3G.1-61, the ratios of contact area of the basemat are about 80% and 85% for N-S and E-W directions, respectively. Since the cohesion is effective at the contact area only, it is reduced in proportion to the ratio of contact area. The FS listed in Table 3.8-96(3) have sufficient margins for the reduced contact area of 80%.

(8) The shear wave velocity of the backfill material is not required to be at least equal to that of the surrounding in situ soil. This is because lateral soil/backfill was neglected in the design basis seismic analysis using the lumped-mass soil spring approach (DCD Tier 2 Subsection 3A.5.1). This approach was confirmed to be conservative as compared to the results of the SASSI analysis taking into account embedment (DCD Tier 2 Subsection 3A.8.7). The gap is completely filled with compacted engineered backfill material. This statement is included in footnotes to DCD Tier 2 Revision 4 Figures 3G.1-65 and 3G.2-17. The revised pages 3G-189 and 3G-245 in DCD Tier 2 Revision 4 are attached.

(9) Alternate sliding stability is performed for the three dimensional seismic loads in accordance with the 100-40-40 rule.

Applied horizontal seismic forces and sliding resistances are schematically shown in Figure 3.8-96(1). Among the resistances, the basemat bottom friction and cohesion act in the direction of the resultant seismic force and their magnitudes are the same as those in the 2-dimensional evaluation.

Resistances due to the passive soil pressures applied to the basemat side surfaces are evaluated as follows:

Soil pressures are applied perpendicular to the basemat. The component in the direction of the seismic force is calculated by the following equation:

$$F = F_x \cos\theta + F_y \sin\theta \quad \text{.....(1)}$$

From the equilibrium of forces in the direction perpendicular to the seismic forces, the following equation needs to be satisfied:

$$F_x \sin\theta = F_y \cos\theta \quad \text{..... (2)}$$

By substituting Eq. (2) into Eq. (1), the following equations are obtained:

$$F_1 = \left(\cos\theta + \frac{\sin^2\theta}{\cos\theta}\right)F_x = \left(\cos\theta + \frac{1-\cos^2\theta}{\cos\theta}\right)F_x = \frac{F_x}{\cos\theta} \quad \text{.....(3a)}$$

or

$$F_2 = \left(\sin\theta + \frac{\cos^2\theta}{\sin\theta}\right)F_y = \left(\sin\theta + \frac{1-\sin^2\theta}{\sin\theta}\right)F_y = \frac{F_y}{\sin\theta} \quad \text{..... (3b)}$$

F_1 and F_2 reach their maximum values when F_x and F_y are equal to the resultant forces due to passive soil pressures. As a result, the resistance due to passive soil pressures is obtained by the following equations:

$$\begin{aligned} F_{pb1} &= F_{pbx}/\cos\theta \\ F_{pb2} &= F_{pby}/\sin\theta \\ F_{pbm} &= \min(F_{pb1}, F_{pb2}) \end{aligned} \quad \text{..... (4)}$$

where,

F_{pbx}, F_{pby} : Forces due to passive soil pressures in X and Y directions, respectively

The evaluation results are shown in Tables 3.8-96(4) and 3.8-96(5). The calculated factors of safety are similar to those in Table 3.8-96(3) for the two-dimensional approach using 40% of vertical seismic forces. Therefore, the use of 0.4 vertical seismic component in the two dimensional approach (i.e., N-S & Vertical and then E-W & Vertical) is justified for design evaluation.

As for dead load consideration, SRP 3.8.5 has no requirements for dead load reduction in sliding evaluation. The uncertainties in dead load are implicitly accounted for in the required minimum factor of safety. The 90% reduction specified in footnote 1 of DCD Tier 2 Table 3.8-15 and ACI 349 is for design of structural members only and therefore it does not apply to the foundation sliding evaluation. However, the 90% reduction is conservatively considered in the calculations shown in Table 3.8-96(3) and in Tables 3.8-96(4) and 3.8-96(5).

Reference:

1. Bowles, Joseph E. Foundation Analysis and Design. 4th Edition. New York: McGraw-Hill, 1988.

Table 3.8-96(2) Cohesions Based on Maximum Soil Bearing Pressure

Building	RBFB			CB		
Soil Condition	Soft	Medium	Hard	Soft	Medium	Hard
Max. Soil Bearing Stress (MPa)	2.7	7.3	5.4	2.8	2.5	2.4
Cohesion coefficient (MPa)	0.15	0.41	0.30	0.16	0.14	0.13

Table 3.8-96(3) Updated Sliding Stability Evaluation Results

<RB>

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	1202 MN		1008 MN		1031 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*899*	*787*	*1462*	*1619*	*1485*	*1243*
Fub: Bottom Friction Force (MN)	694	694	582	582	773	773
Fc: Effective Cohesion Force (MN)	514	514	1391	1391	1029	1029
Fpb: Passive Pressure for Basemat (MN)	132	188	213	304	539	769
Fdsf: Passive Soil Pressure on Wall (MN)	0	0	0	0	0	0
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fdsf)	*1340*	*1397*	*2186*	*2277*	*2341*	*2572*
FS (=Fr/Fv)	**1.49**	**1.78**	**1.50**	**1.41**	**1.58**	**2.07**

<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	91 MN		83 MN		90 MN	
Minimum Vertical Load	18 MN		22 MN		19 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
Fv: Horizontal Seismic Force (MN)	*124*	*124*	*109*	*118*	*115*	*122*
Fub: Bottom Friction Force (MN)	11	11	12	12	14	14
Fc: Effective Cohesion Force (MN)	112	112	100	100	96	96
Fpb: Passive Pressure for Basemat (MN)	36	46	64	82	173	220
Fdsf: Passive Soil Pressure on Wall (MN)	0	0	0	0	0	0
Fr: Sliding Resistance (=Fub+Fc+Fpb+Fds)	*159*	*169*	*177*	*195*	*283*	*331*
FS (=Fr/Fv)	**1.28**	**1.36**	**1.63**	**1.64**	**2.46**	**2.71**

Table 3.8-96(4) Sliding Evaluation Results for 3-dimensional Inputs: 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	1202 MN		1008 MN		1031 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
<3-dimenaional Evaluation> 1.0*NS+0.4*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*899*	*315*	*1462*	*648*	*1485*	*497*
Fvr: Resultant Seismic Force (MN)	*953*		*1599*		*1566*	
Fub: Bottom Friction Force (MN)	*694*		*582*		*773*	
Fc: Effective Cohesion Force (MN)	*514*		*1391*		*1029*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*142*	*507*	*229*	*819*	*580*	*2072*
Fpbm=min(Fpb1, Fpb2) (MN)	*142*		*229*		*580*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*1350*		*2203*		*2382*	
FS (=Fr/Fv)	**1.42**		**1.38**		**1.52**	
<3-dimenaional Evaluation> 0.4*NS+1.0*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*360*	*787*	*585*	*1619*	*594*	*1243*
Fvr: Resultant Seismic Force (MN)	*865*		*1721*		*1378*	
Fub: Bottom Friction Force (MN)	*694*		*582*		*773*	
Fc: Effective Cohesion Force (MN)	*514*		*1391*		*1029*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*355*	*203*	*573*	*328*	*1450*	*829*
Fpbm=min(Fpb1, Fpb2) (MN)	*203*		*328*		*829*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*1411*		*2301*		*2631*	
FS (=Fr/Fv)	**1.63**		**1.34**		**1.91**	

Table 3.8-96(5) Sliding Evaluation Results for 3-dimensional Inputs: 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	91 MN		83 MN		90 MN	
Minimum Vertical Load	18 MN		22 MN		19 MN	
	NS dir	EW dir	NS dir	EW dir	NS dir	EW dir
<3-dimenaional Evaluation> 1.0*NS+0.4*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*124*	*49*	*109*	*47*	*115*	*49*
Fvr: Resultant Seismic Force (MN)	*133*		*118*		*125*	
Fub: Bottom Friction Force (MN)	*11*		*12*		*14*	
Fc: Effective Cohesion Force (MN)	*112*		*100*		*96*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*39*	*123*	*69*	*221*	*187*	*594*
Fpbm=min(Fpb1, Fpb2) (MN)	*39*		*69*		*187*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*162*		*182*		*297*	
FS (=Fr/Fv)	**1.21**		**1.54**		**2.38**	
<3-dimenaional Evaluation> 0.4*NS+1.0*EW+0.4*V						
Factored Horizontal Seismic Force (MN)	*50*	*124*	*43*	*118*	*46*	*122*
Fvr: Resultant Seismic Force (MN)	*133*		*126*		*130*	
Fub: Bottom Friction Force (MN)	*11*		*12*		*14*	
Fc: Effective Cohesion Force (MN)	*112*		*100*		*96*	
Fpb1, Fpb2: Passive Pressure for Basemat (MN)	*97*	*49*	*173*	*88*	*466*	*237*
Fpbm=min(Fpb1, Fpb2) (MN)	*49*		*88*		*237*	
Fr: Sliding Resistance (=Fub+Fc+Fpbm)	*172*		*201*		*348*	
FS (=Fr/Fv)	**1.29**		**1.59**		**2.67**	

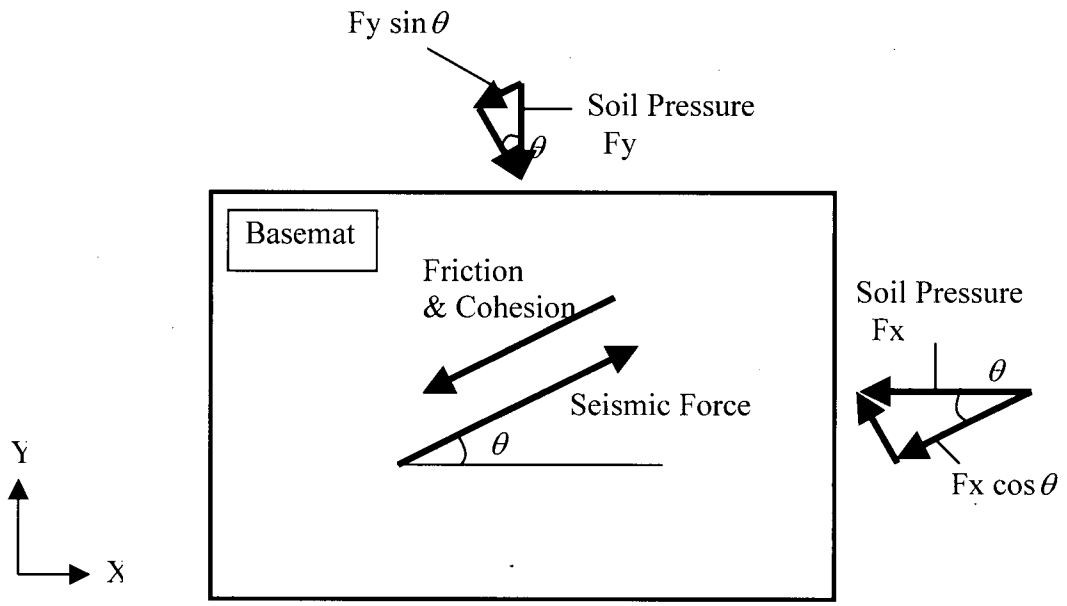


Figure 3.8-96(1) Horizontal Forces in Sliding Evaluation (Basemat Plan)

DCD Impact

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

NRC RAI 3.8-101

DCD Section 3.8.5.2 implies that two separate sets of codes, standards, and specifications were used for the common RCCV/RB/FB foundation. Was the common foundation supporting the RCCV, RB, and FB actually designed to two different sets of codes, standards and specifications, as indicated, or was a uniform design basis employed? If two different design bases were employed, explain how this was implemented and justify the jurisdictional boundary.

Include this information in DCD Section 3.8.5.2. 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

Section designs of the portions, which are included in the RCCV, are performed in accordance with the ASME code, and other portions outside of containment are designed in accordance with ACI 349.

The loads and load combinations that cover both codes are considered for the whole basemat for conservatism.

See also response to NRC RAI 3.8-4.

No DCD change was made in response to this RAI.

NRC RAI 3.8-101, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE's response to RAI 3.8-101 does not adequately address RAIs 3.8-101, -102, and -103. There is no discussion of how jurisdictional boundaries have been evaluated. How are loads and load combinations that cover both codes considered for the whole basemat? Were the code-specific acceptance criteria applied to the whole basemat, for the code-specific load combinations? Was there redundancy of evaluation, to effectively qualify the whole basemat in accordance with both codes?

During the audit, it was agreed that this issue is being addressed under RAI 3.8-4.

GE Response

See response to NRC RAI 3.8-4, Supplement 1 for further clarification of jurisdictional boundaries.

DCD Impact

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

NRC RAI 3.8-101, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

This RAI relates to the jurisdictional boundary between the containment and other Category I structures. This issue is discussed under RAI 3.8-4, which is currently unresolved. When resolved, DCD Section 3.8.5.2 will need to be revised to reflect the resolution.

GEH Response

Please see the response to NRC RAI 3.8-4, Supplement 2.

DCD Impact

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

NRC RAI 3.8-102

DCD Section 3.8.5.3 implies that two different sets of loads and load combinations were used for design of the common RCCV/RB/FB foundation. For the common foundation supporting the RCCV, RB and FB, explain how two different sets of loads and load combinations were implemented and justify the jurisdictional boundary.

Include this information in DCD Section 3.8.5.3. 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

Refer to the response to NRC RAI 3.8-101.

No DCD change will be made in response to this RAI.

NRC RAI 3.8-102, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE refers to response to RAI 3.8-101. See staff assessment of response to RAI 3.8-101. During the audit, it was agreed that this issue is being addressed under RAI 3.8-4.

GE Response

See response to NRC RAI 3.8-4.

DCD Impact

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

NRC RAI 3.8-102, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

This RAI relates to the jurisdictional boundary between the containment and other Category I structures. This issue is discussed under RAI 3.8-4, which is currently unresolved. When resolved, DCD Section 3.8.5.3 will need to be revised to reflect the resolution.

GEH Response

Please see the response to NRC RAI 3.8-4, Supplement 2.

DCD Impact

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

NRC RAI 3.8-103

DCD Section 3.8.5.5 describes the structural acceptance criteria for foundations and states that the containment portion follows DCD Section 3.8.1.5, and the rest of the foundations follow DCD Section 3.8.4.5. Was the common foundation supporting the RCCV, RB, and FB actually designed to two different sets of structural acceptance criteria, as indicated, or was uniform structural acceptance criteria employed? If two different structural acceptance criteria were employed, explain how this was implemented and justify the jurisdictional boundary.

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

Refer to the response to NRC RAI 3.8-101.

No DCD change will be made in response to this RAI.

NRC RAI 3.8-103, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE refers to response to RAI 3.8-101. See staff assessment of response to RAI 3.8-101. During the audit, it was agreed that this issue is being addressed under RAI 3.8-4.

GE Response

See response to NRC RAI 3.8-4.

DCD Impact

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

NRC RAI 3.8-103, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

This RAI relates to the jurisdictional boundary between the containment and other Category I structures. This issue is discussed under RAI 3.8-4, which is currently unresolved. When resolved, DCD Section 3.8.5.5 will need to be revised to reflect the resolution.

GEH Response

Please see the response to NRC RAI 3.8-4, Supplement 2.

DCD Impact

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

Table 5.1-1

Envelope of ESBWR Standard Plant Site Design Parameters

Soil Properties:	- Minimum Static Bearing Capacity:	
	Reactor/Fuel Building:	699 kPa (14,600 lbf/ft^2)
	Control Building:	292 kPa (6,100 lbf/ft^2)
	Fire Water Service Complex:	165 kPa (3,450 lbf/ft^2)
	- Minimum Dynamic Bearing Capacity:	
	Reactor/Fuel Building:	
	Soft:	2700 kPa (56,400 lbf/ft^2)
	Medium:	7300 kPa (152,500 lbf/ft^2)
	Hard:	5400 kPa (112,800 lbf/ft^2)
	Control Building:	
	Soft:	2800 kPa (58,500 lbf/ft^2)
	Medium:	2500 kPa (52,300 lbf/ft^2)
	Hard:	2400 kPa (50,200 lbf/ft^2)
	Fire Water Service Complex (FWSC):	
	Soft:	440 kPa (9,200 lbf/ft^2)
	Medium:	540 kPa (11,300 lbf/ft^2)
	Hard:	670 kPa (14,000 lbf/ft^2)
	- Minimum Shear Wave Velocity:	300 m/s (1000 ft/s)
	- Liquefaction Potential:	
	Seismic Category I Structures	None under footprint of Seismic Category I structures resulting from site-specific SSE
	- Angle of Internal Friction	≥ 30 degrees
Seismology:	- SSE Horizontal Ground Response Spectra:	See Figure 5.1-1
	- SSE Vertical Ground Response Spectra:	See Figure 5.1-2
Hazards in Site Vicinity:	- Site Proximity Missiles and Aircraft:	≤ 10^{-7} per year
	- Toxic Gases:	None *
	- Volcanic Activity:	None
* Maximum toxic gas concentrations at the Main Control Room (MCR) HVAC intakes	< toxicity limits	
Required Stability of Slopes:	- Factor of safety for static (non-seismic) loading	1.5
	- Factor of safety for dynamic (seismic) loading	1.1

3.8.4.5.2 Control Building

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

3.8.4.5.3 Fuel Building

The acceptance criteria for the design of the Fuel Building are same as the Reactor Building in Subsection 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 Fire Water Service Complex

The acceptance criteria for the design of the FWSC are same as the Reactor Building in Subsection 3.8.4.5.1.

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 other Seismic Category I structures.

3.8.4.6.1 Concrete

Concrete material is the same as described in Subsection 3.8.1.6.1 with the following exception: Concrete is batched and placed according to ACI 349-01.

3.8.4.6.2 Reinforcing Steel

Reinforcing steel is the same as in Subsection 3.8.1.6.2.

3.8.4.6.3 Splices of Reinforcing Steel

Splices of reinforcing steel are the same as in Subsection 3.8.1.6.3 except that placing and splicing is in accordance with ACI 349-01. Welding of reinforcing bars complies with all the applicable requirements of ASME Code Section III, Division 2.

3.8.4.6.4 Quality Control

Quality control is the same as in Subsection 3.8.1.6.5 except that the Construction Specifications 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

Composite construction is used in other Seismic Category I structures. Some of the components, such as rebar cages, are pre-assembled and lifted into place. As described in Subsection 3.8.4.1.1, the composite structures, which are composed of reinforcing bars, steel plates, and concrete, are used for most RB floor slabs. The structural pieces are assembled in discrete segments and are lifted into place. Concrete is poured in-place. See Figure 3.8-12.

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.

The actual construction sequence, if substantially different from the sequences considered in the design, is ensured not to adversely impact the basemat design.

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 relative displacement between two corners along the longest dimension of the building basemat calculated under linearly varying soil stiffness is 77 mm (3.0 in). The estimated differential settlement between buildings (RB/FB and CB) is 85 mm (3.3 in). These values are specified as maximum settlements in Table 2.0-1.

3G.1.5.6 Tornado Missile Evaluation

The minimum thickness required to prevent penetration, concrete spalling and scabbing is evaluated. The methods and procedures are shown in Subsection 3.5.3.1.1. The minimum thickness required is less than the minimum 1000 mm (39.4 in) and 700 mm (27.6 in) 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 6th 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	1.41	--	--
D + F'	--	--	--	--	1.1	3.48

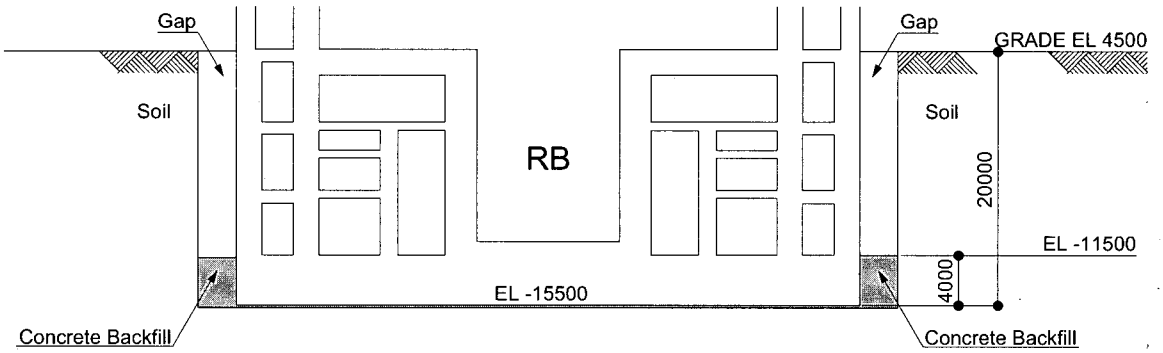
Where,
D = Dead Load
H = Lateral soil pressure
E' = Safe Shutdown Earthquake
F' = Buoyant forces of design basis flood

Table 3G.1-58

Maximum Soil Bearing Stress Involving SSE

	Site Condition*		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
Bearing Stress (MPa)	2.7	7.3	5.4

* See Table 3A.3-1 for site properties. For site specific application, use the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level.



Note: Backfill method for gap and excavation method (e.g., vertical cut, open cut) is determined considering actual site conditions. The gap is completely filled with compacted engineered backfill material.

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

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	62.5	1.1	1.16	--	--
D + F'	--	--	--	--	1.1	1.85

Where,

D = Dead Load

H = Lateral soil pressure

E' = Safe Shutdown Earthquake

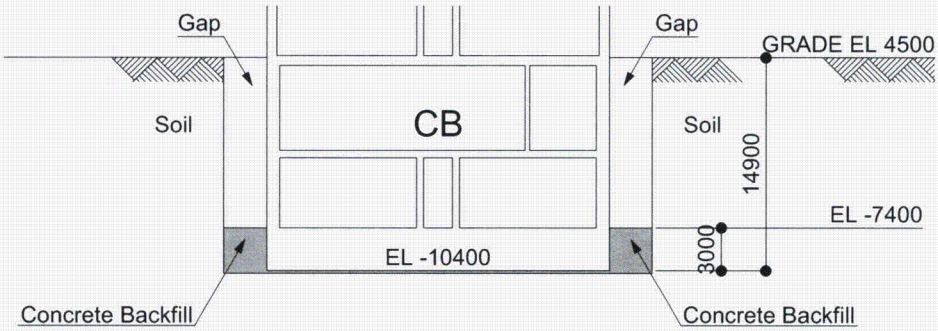
F' = Buoyant forces of design basis flood

Table 3G.2-27

Maximum Soil Bearing Stress Involving SSE

	Site Condition*		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
Bearing Stress (MPa)	2.8	2.5	2.4

* See Table 3A.3-1 for site properties. For site specific application, use the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level.



Note: Backfill method for gap and excavation method (e.g., vertical cut, open cut) is determined considering actual site conditions. The gap is completely filled with compacted engineered backfill material.

Figure 3G.2-17. Concrete Backfill in Sliding Evaluation