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July 5, 2012

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, DC 20555-0001

Subject: Duke Energy Carolinas, LLC
William States Lee III Nuclear Station – Docket Nos. 52-018 and 52-019
AP1000 Combined License Application for the
William States Lee III Nuclear Station Units 1 and 2
Response to Request for Additional Information (eRAI 2563)
Ltr# WLG2012.07-01

References 1. Letter from Brian Hughes (NRC) to Peter Hastings (Duke Energy), Request for Additional Information Letter No. 068 Related to SRP Section 03.08.05 – Foundations for the William States Lee III Units 1 and 2 Combined License Application, dated April 16, 2009 (ML091050662)

2. Letter from Bryan Dolan (Duke Energy) to NRC Document Control Desk, Response to Request for Additional Information Letter No. 068 Related to SRP Section 03.08.05 (eRAI 2563) for the William Sates Lee III Units 1 and 2 Combined License Application, LTR#WLG2009.06-04, dated June 11, 2009 (ML091660230)

This letter provides supplemental information to Duke Energy's response (Reference 2) to the Nuclear Regulatory Commission's request for additional information (RAI), included in Reference 1, based on interactions with the NRC staff during the April 9-11, 2012 technical audit.

The supplemental information for the response to RAI 03.08.05-005 is addressed in Enclosure 1. Revisions to the previous RAI response are marked by revision bars in the right hand margin. This letter proposes no change to the Lee Nuclear Station Final Safety Analysis Report (FSAR). Changes to FSAR Figures 2.5.4-245 and -246 that were proposed in the previous RAI response (Reference 2) have been updated in the current version of the FSAR.

If you have any questions or need any additional information, please contact James R. Thornton, Nuclear Plant Development Licensing Manager (Acting), at (704) 382-2612.

Sincerely,

Christopher M. Fallon

Christopher M. Fallon
Vice President
Nuclear Development

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NRO

Enclosure:

1) Lee Nuclear Station Supplemental Response to Request for Additional Information (RAI), Letter No. 068, RAI 03.08.05-005

xc (w/out enclosure):
Frederick Brown, Deputy Regional Administrator, Region II

xc (w/ enclosure):
Brian Hughes, Senior Project Manager, DNRL

AFFIDAVIT OF CHRISTOPHER M. FALLON

Christopher M. Fallon, being duly sworn, states that he is Vice President, Nuclear Development, Duke Energy Carolinas, LLC, that he is authorized on the part of said Company to sign and file with the U. S. Nuclear Regulatory Commission this combined license application for the William States Lee III Nuclear Station, and that all the matter and facts set forth herein are true and correct to the best of his knowledge.

Christopher M. Fallon

Christopher M. Fallon, Vice President
Nuclear Development

Subscribed and sworn to me on July 5, 2012

Vickie M Prenatt

Notary Public

My commission expires: 5-11-2016

SEAL

VICKIE M PRENATT
Notary Public, North Carolina
Mecklenburg County
My Commission Expires
May 11, 2016

Lee Nuclear Station Supplemental Response to Request for Additional Information (RAI)

RAI Letter No. 068

NRC Technical Review Branch: **Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)**

Reference NRC RAI Number(s): **RAI 03.08.05-005**

NRC RAI:

A request for additional information, RAI 03.08.05-002, was previously submitted to obtain information regarding the design of fill concrete below the Unit 1 nuclear island basemat near the northwest corner. The slope of the continuous rock under this area is relatively steep.

The applicant responded on December 17, 2008. The response states that the sloping rock surface that receives fill concrete will be excavated to form benches in the continuous rock. The benches would provide a horizontal rock face to resist vertical foundation loads. As a result, shear through the fill and at the rock interface would not be a controlling failure mode. Revised Figures 2.5.4-245 and 2.5.4-246 were provided and showed the benches.

The use of benches appears to be a good way to provide a reliable load path to the rock. The text of the RAI response indicates benches will be formed at sloping rock. However the revised figures do not clearly require the formation of benches. Taking Note 2 of either figure literally, benches are required only if sloping rock does not meet the requirement for continuous rock (apparently RQD 65% per FSAR 2.5.4.11). Also, the angle at which a surface qualifies as "sloping" has not been established.

The RAI response also argues that shear stresses are not a controlling failure mode based on the calculations shown in the response for RAI 03.08.05-001. However, those calculations apply to a uniform layer of fill on non-sloping rock and may not be directly applicable.

The fill concrete is an essential part of the foundation load path. Without a quantitative stress analysis that accounts for the sloping rock geometry, it is not apparent to the Staff that the margin of safety for the fill concrete in the northwest corner is acceptable.

Thus, additional information is requested to clearly identify the engineering design process for the fill:

1. Please identify the allowable stress criteria for the design of the fill concrete (e.g., ACI-318-02 strength criteria for plain structural concrete).
2. Please describe the design analysis that has been or will be performed to verify that stress in the fill concrete is acceptable. Please identify the assumptions about the post-excavation geometry of the continuous rock, including the maximum slope of the rock.
3. Please describe the controls placed on the excavation process that will ensure that the as-built fill concrete geometry is within design assumptions. Also, please identify the post-excavation inspections that will be performed prior to placement of the fill concrete.

Duke Energy Supplemental Response:

During an NRC audit of the WLS COLA on April 10, 2012 Duke Energy was requested to address the following items.

- Provide additional justification for maximum coefficient of friction assumed in concrete fill design.
- Consider effects of nuclear island uplift (under SSE loading) and consequent increase in horizontal shear demand.
- Discuss appropriateness of the theory of elasticity approach.

This supplemental information provides the requested information.

1. The allowable stress criteria for the design of the fill concrete are in accordance with ACI 318-02 (2002) (Reference 1) and ACI 318-99 (1999) (Reference 2). ACI 318-02 (2002) requires that the specified concrete compressive strength f'_c cannot be less than 2500 lb/in^2 if the concrete is to be characterized as plain concrete for applying the provisions of ACI 318-02 (2002). The specified f'_c will be at least 2500 lb/in^2. ACI 318-02 (2002) does not contain details for the working stress method (without load factors). Earlier versions (e.g., ACI 318-99 (1999)) did contain certain provisions for the working stress method. ACI 318-02 (2002) states: "The Alternate Design Method of the 1999 code may be used in place of applicable sections of this code."

   Traditional working stress design provisions in Appendix A of ACI 318-99 (1999) specify that the allowable compressive stress in the plain concrete is $0.30\ f'_c$. If the supporting area A2 is larger than the loaded bearing area A1, the allowable stress may be multiplied by the square root of (A2÷A1), but not more than 2. This latter provision is conservatively ignored in assessing the allowable compressive stress on the fill concrete. The f'_c value (2500 lb/in^2) equates to an allowable bearing stress for this new concrete of 0.30 x 2500 lb/in^2 x 144 in^2/ft^2 = 108,000 lb/ft^2. This applies to normal stresses in the concrete (i.e., stresses perpendicular to a plane).

   Based on Section A.7.6 of ACI 318-99 (1999), the allowable shear stress that can be sustained by the new fill concrete for shearing through the concrete itself cannot exceed $0.55 \times 0.2\ f'_c\ A_c = 0.11\ f'_c\ A_c$, nor $0.55\ (800A_c) = 440\ A_c$. In these equations, f'_c is in lb/in^2 and A_c is the concrete area in in^2. These limits are 0.11 x 2500 lb/in^2 x 144 in^2/ft^2 = 39,600 lb/ft^2, which is less than the upper limit of 440 lb/in^2 x 144 in^2/ft^2 = 63,300 lb/ft^2. Based on the above, the allowable shear stress (allowable capacity) that can be sustained by the new fill concrete for shearing through the concrete itself is 39,600 lb/ft^2. This applies to shear stresses in the concrete along inclined planes of the rock slope.

   Section A.7.6 in ACI 318-99 (1999), plus Sections 11.7.4.3 and 11.7.5 in ACI 318-99 (1999), combine to give the coefficient of friction in shear (ultimate) = 1.0 on surfaces between concrete cast against hardened concrete with the surface intentionally roughened. Section 11.7.9 of ACI 318-02 (2002) specifies the requirement and criterion for intentionally roughening, which also are presented in FSAR Subsection 2.5.4.5.3.2. Reference 9 presents results of shear tests on concrete lift joint specimens with different surface parameters. The tests indicate that the coefficient of friction decreases with the increase in normal load. For joints that meet the definition of intentionally roughened joints, Reference 9 concludes "...a peak value of μ of 1.4 could be adopted for up to σ_n=1000 kPa, but μ = 1.1 would be adequate for all load levels, that is close to μ = 1.0 as suggested in the ACI code." This condition yields the ultimate shear stress at a locality on a horizontal lift line based on the normal (vertical) stress on the lift line at that location if sliding evaluation using the shear-friction mechanism is being considered. Table 2.9-1 in APP-GW-GLR-044, Technical Report Number 85 (TR-85), Revision 3 (Reference 3) requires a global factor of safety (FS)

= 1.1 for sliding resistance of the nuclear island under seismic loading. For illustration purposes, the applied shear stress at a locality on a horizontal lift line is compared to an allowable shear stress at that locality determined by dividing the ultimate shear stress (from the coefficient of friction in shear (ultimate) = 1.0) by the global FS = 1.1. This applies to shear stresses along horizontal planes.

The shear capacity of the foundation rock is controlled by the strength parameters of the rock. As mentioned in FSAR Subsection 2.5.4.10.1, the Hoek-Brown parameters of the rock mass were used to establish the Mohr-Coulomb parameters of friction angle and cohesion for the rock mass. The Hoek-Brown parameters of the rock mass used in conjunction with the equations for shear strength in Hoek, et al., 2002 (Reference 8) yielded the estimated lower bound rock mass cohesion value of 0.15 kips/in^2 (21,600 lb/ft^2) and the lower bound rock mass friction angle of 65 degrees for the analyzed confining stress value of 0.0524 kips/in^2. These are ultimate stress parameters. Table 7.4 of ASCE (1980) (Reference 4) recommends that the global FS = 1.5 under seismic loading of the nuclear island when evaluating the allowable bearing pressure based on the ultimate bearing pressure determined using the ultimate strength of the supporting materials. For illustration purposes, the applied shear stress at a locality in the rock adjacent to the rock slope is compared to an allowable shear stress at that locality determined by dividing the ultimate shear stress (from the cohesion and friction components of the ultimate rock strength) by the global FS = 1.5. Therefore, the allowable shear stress τ_n in rock = [21,600 lb/ft^2+tan(65 deg)×σ_n]/FS(= 1.5) where σ_n is the normal stress in the rock. This applies to shear stresses in the rock along inclined planes of the rock slope.

2. An analysis has been performed to verify that stress in the fill concrete is acceptable. The assumptions about the post-excavation geometry of the continuous rock, including the maximum slope of the rock are described below. The continuous rock begins to slope at the northwest corner of the Unit 1 nuclear island and slopes down to the north and to the west. The top of the continuous rock slope was kept at elevation 529.7 ft (20.8 ft below the base of the nuclear island) and a range of continuous rock slope angles was postulated from the flattest (4:1 H:V) to the steepest (0.25:1 H:V). The outer slope of the fill concrete begins 6 ft north and 6 ft west of the nuclear island base mat and slopes downward to the north and to the west at (0.5:1 H:V) and connects with the sloping continuous rock at elevation 500 ft as it is depicted on revised FSAR Figures 2.5.4-245 and 2.5.4-246, except where the postulated continuous rock slope is flat enough to intersect the outer slope at a higher elevation than shown on these figures. The plan view of the analyzed foundation model is depicted on Figure 1. The cross section geometric model is depicted on Figure 2, where alpha "α" is the symbol given to the rock slope inclination.

   TR-85, Revision 3, provides the foundation loading information. The safe shutdown earthquake (SSE) seismic base shear demand (for Hard Rock (HR) conditions) at generic elevation 60.5 ft (Lee Nuclear Station elevation 550.5 ft) is 123,750 kips (north-south) and 112,310 kips (east-west) per Table 2.4-2 of TR-85, Revision 3. These shear forces average over the base of the nuclear island foundation area to produce 3810 lb/ft^2 (north-south) and 3460 lb/ft^2 (east-west). The maximum SSE bearing pressure (vertical) is 35,000 lb/ft^2 per Section 2.5.4.2 of the AP1000 DCD, Revision 19. Note this maximum SSE bearing pressure is localized to the side of the nuclear island opposite the northwest corner (on the Shield Building's (Lee Nuclear Station) east edge, recalling that north at the Lee Nuclear Station site is opposite to north in the DCD figures). The SSE bearing pressure is less than 35,000 lb/ft^2 in other parts of the foundation. For analysis purposes herein, the horizontal SSE loads are applied either to the north or to the west, and the resulting stresses in each direction are combined with those from the vertical loading assumed as uniform over the

foundation. For this evaluation, using the maximum SSE vertical bearing pressure associated with the east side of the nuclear island as uniform over the foundation is very conservative.

The horizontal SSE loads for the Hard Rock (HR) condition are obtained from Table 2.4-2 of TR-85, Revision 3 (Reference 3). Review of Figure 2.4-6 (sheet 1) in Reference 3 indicates very minor liftoff conditions for the basemat under SSE loading and the HR condition. The analyses that generated Figure 2.4-6 in Reference 3 used the Certified Seismic Design Response Spectra (CSDRS) for the AP1000 as seismic input. In Reference 11, additional analyses have been performed using the site-specific Foundation Input Response Spectra (FIRS) for the Unit 1 nuclear island at WLS as seismic input. These additional analyses show that no uplift of the basemat occurs for this site-specific condition. Therefore, the entire area of the basemat distributes the loads into the foundation materials. The horizontal loads from the CSDRS input are conservatively used.

For loading to the north or to the west, an x-y-z rectangular foundation coordinate system, as depicted on Figures 1 and 2, was established for each direction with its origin at the corner of the nuclear island base mat and oriented so that the x-axis was always in the direction of the horizontal loading, with the z-axis always vertical and positive downward. Stresses were calculated at depths (z) below the bottom of the nuclear island ranging from 25 ft to the depth where each rock slope configuration intersected the outer slope of the concrete. For slope angles (alpha "α") equal to 1:1 (H:V) or steeper, the maximum depth was 50 ft; for flatter rock slope angles, the maximum applicable depth varied consistent with the depth where the rock slope angle intersects the outer slope (e.g. maximum depth = 25 ft for the flattest rock slope angle of 4:1 (H:V). Equations from the theory of elasticity that compute the stress in the supporting medium for point loads on the surface as found in Poulos and Davis (1974) (Reference 5) and in Som and Das (2009) (Reference 6) were used to compute the normal and shear stresses induced by the horizontal and vertical foundation loads.

The use of elastic theory for this purpose is appropriate because of the similarity of modulus values in the fill concrete and the adjacent rock mass that underlies the foundation after construction. From Reference 10, the fill concrete has a shear wave velocity of 7,500 ft/sec, and the adjacent rock material has a shear wave velocity of approximately 7,000 ft/sec (geometric mean). These similar values of shear wave velocity indicate the fill concrete and adjacent rock are compatible in their respective elastic modulus values, thus justifying the use of elastic theory to compute stresses.

The equations for induced stresses are for point loads. The foundation area (Figure 1) is subdivided into small square sub-areas with the center of each square serving as the coordinate location of a point load representing the area load associated with that square. Sub-area squares, five ft by five ft, are used. The foundation area is represented by 51 rows of sub-area squares in the north-south direction, and by 18 rows in the east-west direction, for a total of 918 sub-areas covering a total area of 22,950 ft^2 (255 ft (north-south) by 90 ft (east-west)).

For horizontal loading of the foundation, the point load in each square is given by:

$P = 3810\ lb/ft^2 \times 5\ ft \times 5\ ft = 95{,}250$ lbs applied in the north direction; and

$P = 3460\ lb/ft^2 \times 5\ ft \times 5\ ft = 86{,}500$ lbs applied in the west direction.

For vertical load, the point load in each square is given by:

$Q = 35{,}000\ lb/ft^2 \times 5\ ft \times 5\ ft = 875{,}000$ lbs at the center of each square.

The rectangular shape and size of the foundation in Figure 1 are somewhat smaller than the full nuclear island area, but were chosen for calculation convenience, justified by the concept that seismic loading to the north or to the west concentrates the vertical loading toward these sides (e.g., see Figure 2.6-8 of TR-85, Revision 3) and that applied loads on the foundation area that is located far from the northwest corner are not contributors to the stress in the sloping fill concrete in that locality.

To compute the stresses in the fill concrete at the northwest corner, horizontal load is applied either to the north or to the west. The coordinate location of each sub-area square with respect to the rectangular coordinate origin at the northwest corner is assigned and the x and y distance of each sub-area square is then adjusted to the horizontal location (denoted by x_a) along the x-axis (with $y_a = 0$) in the fill concrete being analyzed.

For all calculations, the concrete unit weight and Poisson's ratio were 145 lb/ft^3 and 0.17, respectively. The stresses induced by the weight of the adjacent granular backfill were calculated using equations in Carrier (2006) (Reference 7). The unit weight of the granular fill is shown on Figure 2.

Stresses in the x and z directions in the geometric model were computed at the outer slope and also along each of the postulated rock slope-fill concrete interfaces. The stresses along the postulated rock slope-fill concrete interfaces were then incorporated into equations from Poulos and Davis (1974) to compute the normal and shear stress combinations on the sloping interface. Equations in Poulos and Davis (1974) were also used to compute the principal normal stresses and the maximum shear stress for each locality on the sloping interface. This is analogous to use of the Mohr's circle for calculating stresses on inclined surfaces from given stresses in the x and z directions.

The results are plotted on Figures 3 through 6. The individual data points for each rock slope (4:1 H:V, etc.) correspond to specific depths (beginning at 25 ft with 5 ft increments) measured along the z-axis from the origin at the bottom of the nuclear island base mat and at various lateral distances (x_a) to the west or north along the x-axis (see the earlier discussion on why there are fewer depth data points for postulated rock slope angles flatter than 1:1 (H:V) than for steeper slopes). Shown on these figures are the criterion allowable stresses in the concrete and in the rock contained in the response to item (1) herein. The upper panel of Figures 3 through 6 is for the SSE horizontal loading applied to the west and the lower panel of Figures 3 through 6 is for the SSE horizontal loading applied to the north. Inspection of these figures shows the following:

- Revised Figure 3 depicts the normal (vertical) and shear stresses on horizontal planes along the outer slope of the fill concrete which is identified on Figure 2. Inspection of Figure 3 indicates the shear stresses on horizontal planes at the outer slope of the fill concrete are significantly below the descriptive criterion (local FS = 1.1).
- Revised Figure 4 depicts the normal (vertical) and shear stresses on horizontal planes at (adjacent to) the rock interface slope within the fill concrete for all the range of slope angles analyzed. Inspection of Figure 4 indicates the shear stresses on horizontal planes are significantly below the descriptive criterion (local FS = 1.1).
- Figure 5 depicts the normal and shear stresses on inclined planes on the rock interface for the range of rock slope angles analyzed. Inspection of Figure 5 indicates the shear stresses associated with each of the rock slope angles are significantly below both the allowable shear stress in the adjacent rock and the allowable shear stress in the fill concrete.

- Figure 6 depicts the maximum shear stress $[(\sigma_1 - \sigma_3)/2]$ plotted versus the average normal stress $[(\sigma_1 + \sigma_3)/2]$ in the fill concrete at (adjacent to) the rock interface slope. The orientation of the principal stress planes is variable and not necessarily parallel to the rock interface slope. The purpose of Figure 6 is to illustrate that the maximum shear stress in the fill concrete adjacent to any of the analyzed rock interface slopes is significantly below the allowable shear stress in the fill concrete.

The following are additional observations about the preceding analysis results:

- The 35,000 lb/ft^2 maximum SSE bearing pressure for the AP1000 is a localized value that occurs on the side of the nuclear island opposite the west edge and its northwest corner. This calculation applied the 35,000 lb/ft^2 as a uniform pressure over the foundation analysis area. This is very conservative.
- The horizontal SSE forces used in this calculation are from the CSDRS seismic loading. These loads are conservative (higher) compared to those that would be generated by the site-specific seismic loading for the WLS.
- For simplicity, the analysis considers the horizontal SSE forces going north or west, each combined with the maximum SSE bearing pressure in the vertical. The analyses are each for two earthquake directions at one time. This is conceptually equivalent to a load combination of 100/0/100 as compared to the 100/40/40 rule for considering three earthquake directions used in TR-85, Revision 3, (e.g., see Figure 2.6-8 of TR-85, Revision 3). The simple analyses herein exhibit significant margin from reaching the allowable stresses, and thus more complex analyses would not change the resulting conclusion that the stresses in the fill concrete are acceptable.

3. The results on Figures 3 through 6 indicate that stresses in the fill concrete are always considerably below the allowable values for all postulated rock excavation geometries evaluated. The calculated stresses are therefore always acceptable because there is a significant margin between the induced stresses and the allowable stresses for all excavation geometry. On this basis, there is no indicated need for special controls to be placed on the excavation process to ensure that the as-built fill concrete geometry is within design assumptions. FSAR Figures 2.5.4-245 and 2.5.4-246 clearly show that benches are required in the sloping continuous rock (but only in this northwest corner area to distinguish it from all other areas of the site). The angle at which a surface qualifies as "sloping", which is only relevant to this northwestern corner area, is not a critical limit; FSAR Figures 2.5.4-245 and 2.5.4-246 require benching for any continuous rock slope to support the fill concrete north and west of the northwest corner of Unit 1. The post-excavation observations and geological mapping to be performed prior to placement of the fill concrete are described in FSAR Subsection 2.5.4.5.3.1 and FSAR Subsection 2.5.4.12.

References:

1. ACI 318-02 (2002). Building Code Requirements for Structural Concrete and ACI-318R-02 (2002) Commentary.
2. ACI 318-99 (1999). Building Code Requirements for Structural Concrete and ACI-318R-99 (1999) Commentary.

3. Letter dated March 29, 2011 from R. F. Ziesing, Westinghouse Electric Company, to USNRC Document Control Desk, Transmittal of Technical Report APP-GW-GLR-044, Revision 3 (TR-85) (ML110900542).
4. ASCE, 1980. Structural Analysis and Design of Nuclear Plant Structures, prepared by Editing Board and Task Groups of the Committee on Nuclear Structures and Materials of the Structural Division of the American Society of Civil Engineers, Table 7.4 (Summary of typical safety margins used in foundation design of safety class structures).
5. Poulos, H. G. and Davis, E. H., (1974). "Elastic Solutions for Soil and Rock Mechanics". John Wiley and Sons, New York.
6. Som, N. N., and Das, S. C. (2009). Theory and Practice of Foundation Design, PHI Learning Pvt. Ltd., 2009, ISBN-978-81-203-2190-8, pages 78-79.
7. Carrier, W. D. (2006). "The Holl Halfspace: use with caution", Geotechnique 56, No. 9, pages 657-659.
8. Hoek, E., Carranza-Torres, C. and Corkum, B. (2002). Hoek-Brown Failure Criterion – 2002 Edition. 5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference: NARMS-TAC, 2002, pages 267-271.
9. Fronteddu, L., Leger, P., and Tinawi, R., 1998. "Static and dynamic behavior of concrete lift joint interfaces". ASCE, Journal of Structural Engineering, Vol. 124, No. 12, pp. 1418-1430, December.
10. Letter from Bryan J. Dolan (Duke Energy) to Document Control Desk, U.S. Nuclear Regulatory Commission (NRC), Duke Energy Response to RAI Letter No. 76 Related to SRP Section 02.05.02, Ltr# WL12010.03-04, dated March 12, 2010 (ML100760097).
11. Westinghouse APC_WLG_000107, Evaluation of AP1000 Basemat Lift-off and Bearing Pressure for Duke WS Lee 1&2, June 8, 2012.

Associated Revision to the Lee Nuclear Station Final Safety Analysis Report:

None

Attachments:

1. Lee Nuclear Station Supplemental Response to Request for Additional Information, RAI 03.08.05-005, Figures 1 through 6

Figure 1. Rectangular Foundation Area

Figure 2. Section Model at Northwest Corner Lee Nuclear Station Unit 1

Figure 3. Stresses in Fill concrete at Outer Slope (Revised)

Figure 4. Stresses in Fill Concrete Adjacent to Rock Interface (Revised)

Figure 5. Stresses on Inclined Planes on Rock Interface

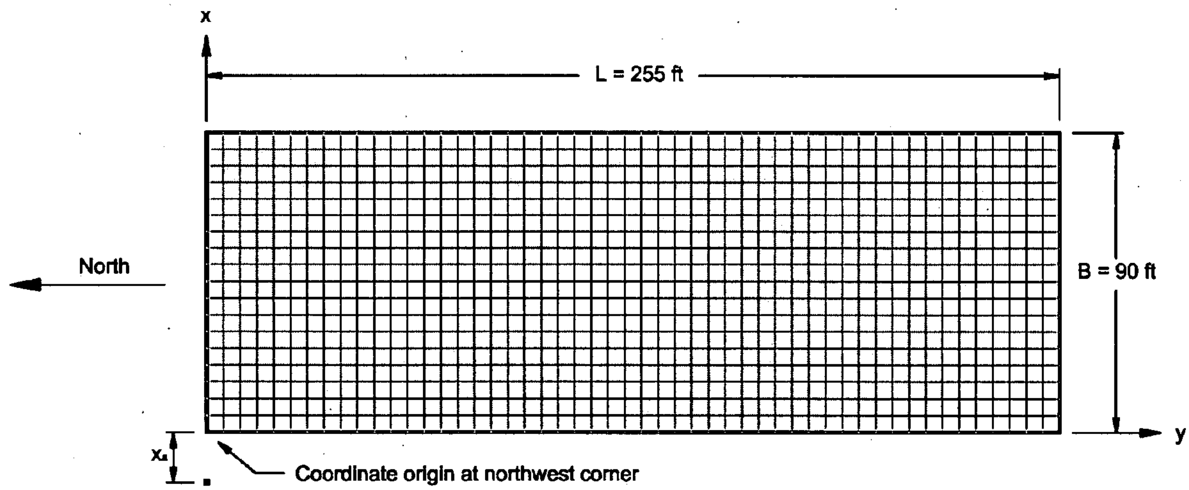
Figure 6. Maximum Shear Stresses in Fill Concrete Adjacent to Rock Interface

Attachment 1

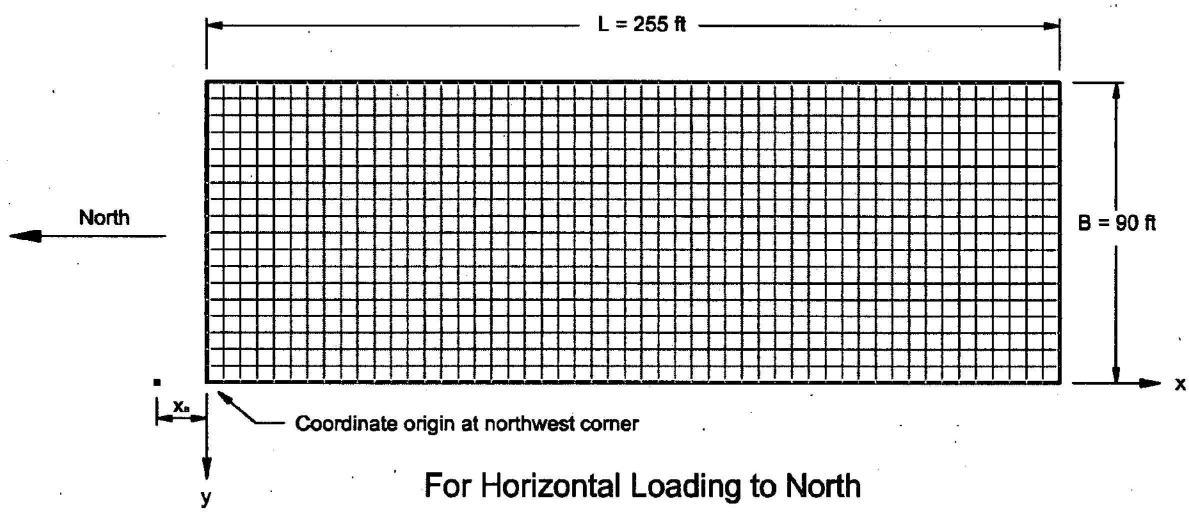
Lee Nuclear Station Supplemental Response to Request for Additional Information

RAI 03.08.05-005

Figures 1 through 6



For Horizontal Loading to West



For Horizontal Loading to North

x_a = horizontal x-coordinate to analysis point

Figure 1. Rectangular Foundation Area

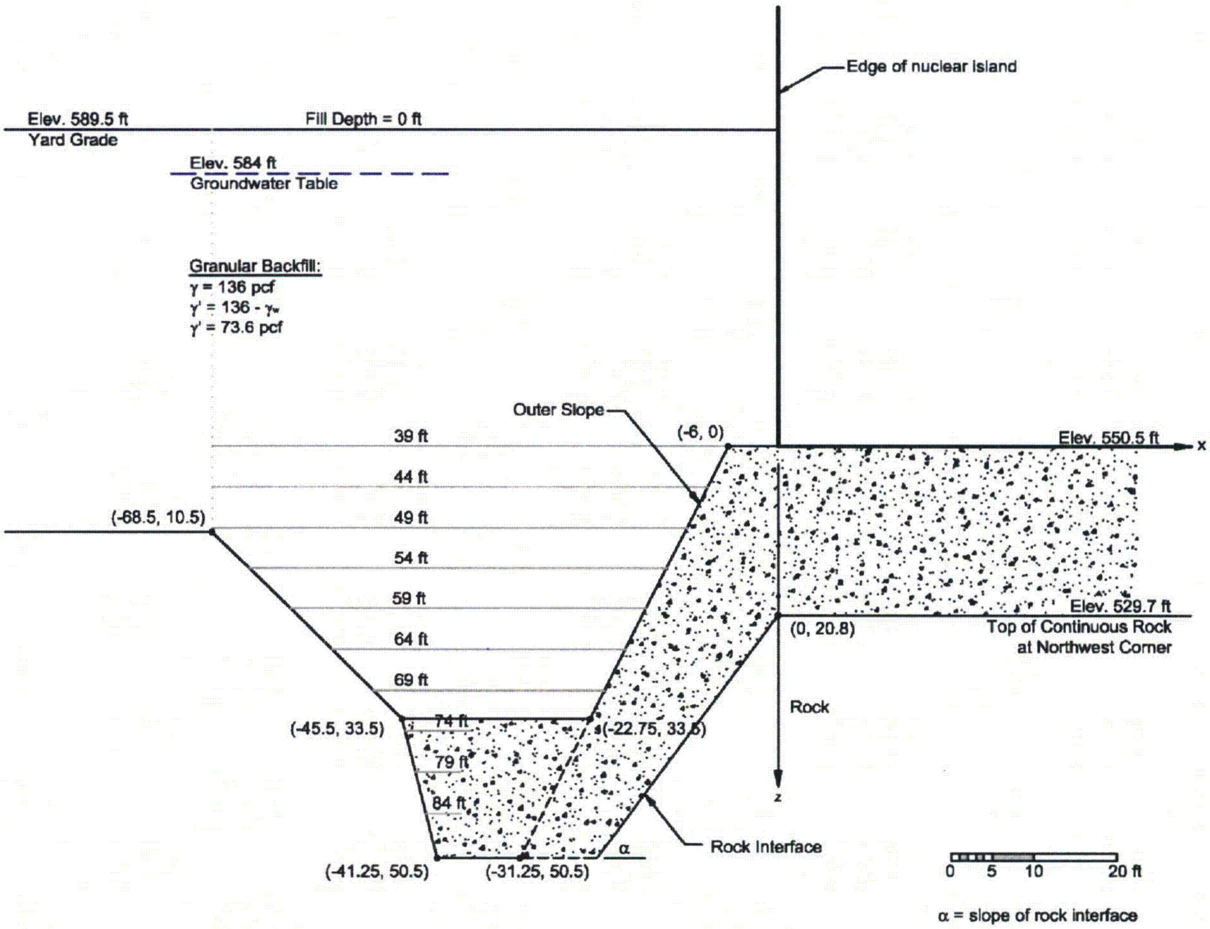
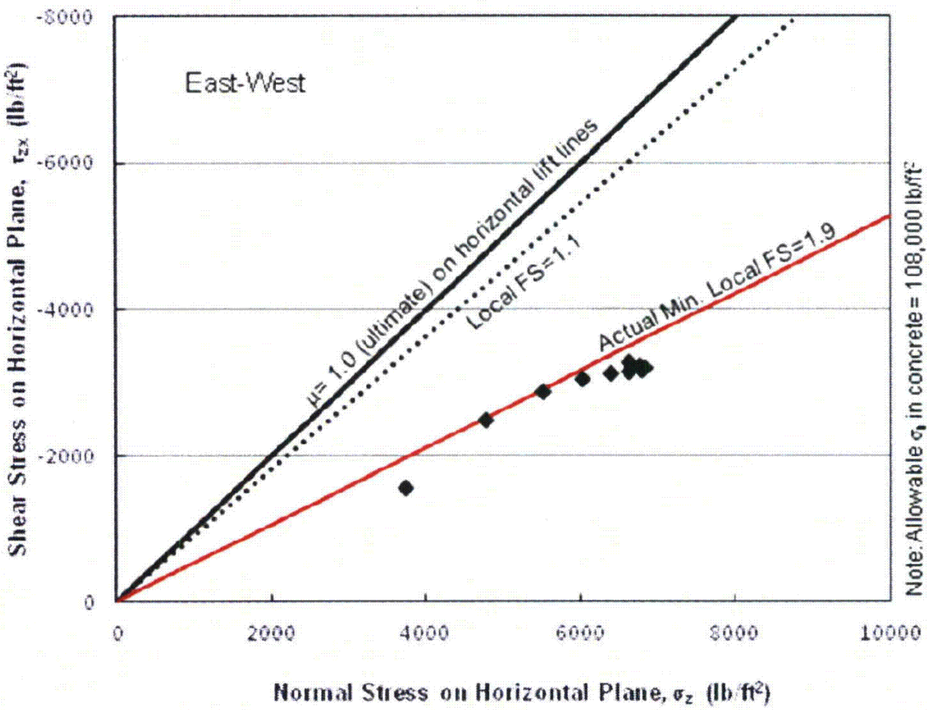


Figure 2. Section Model at Northwest Corner Lee Nuclear Station Unit 1



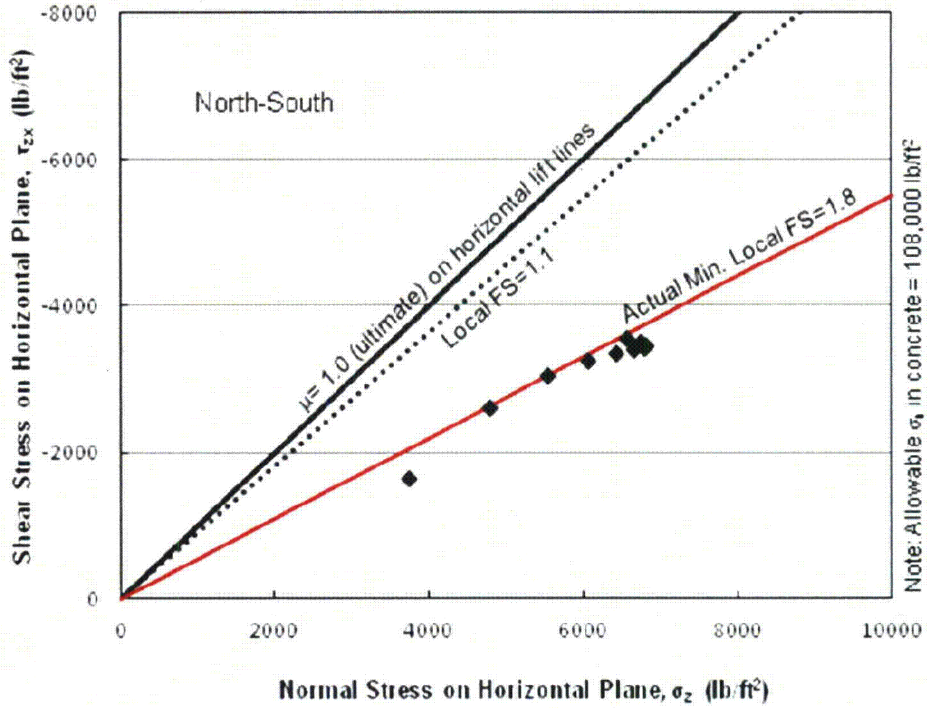
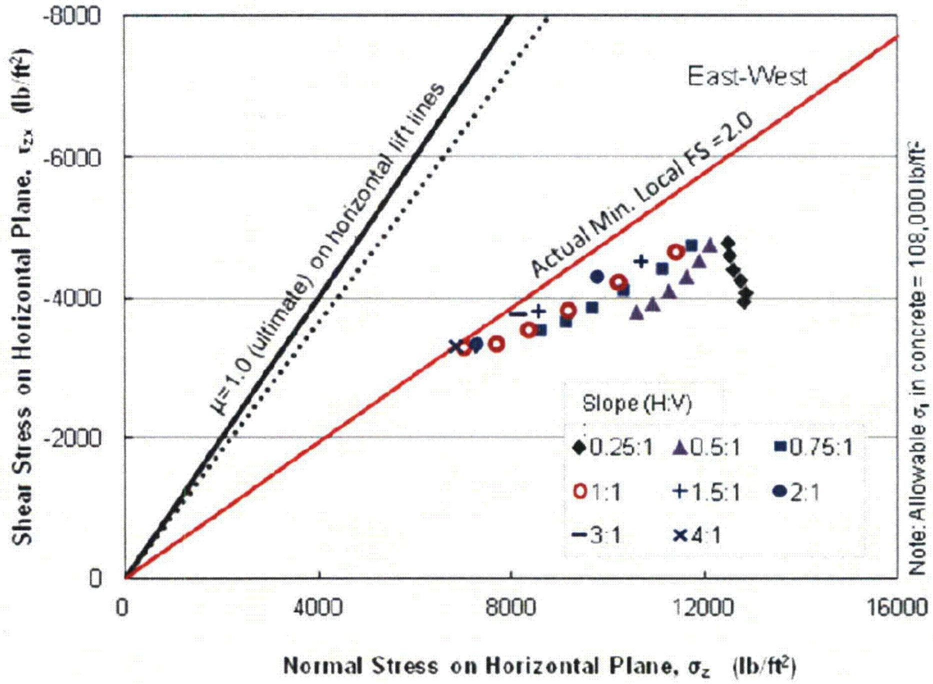


Figure 3. Stresses in Fill Concrete at Outer Slope



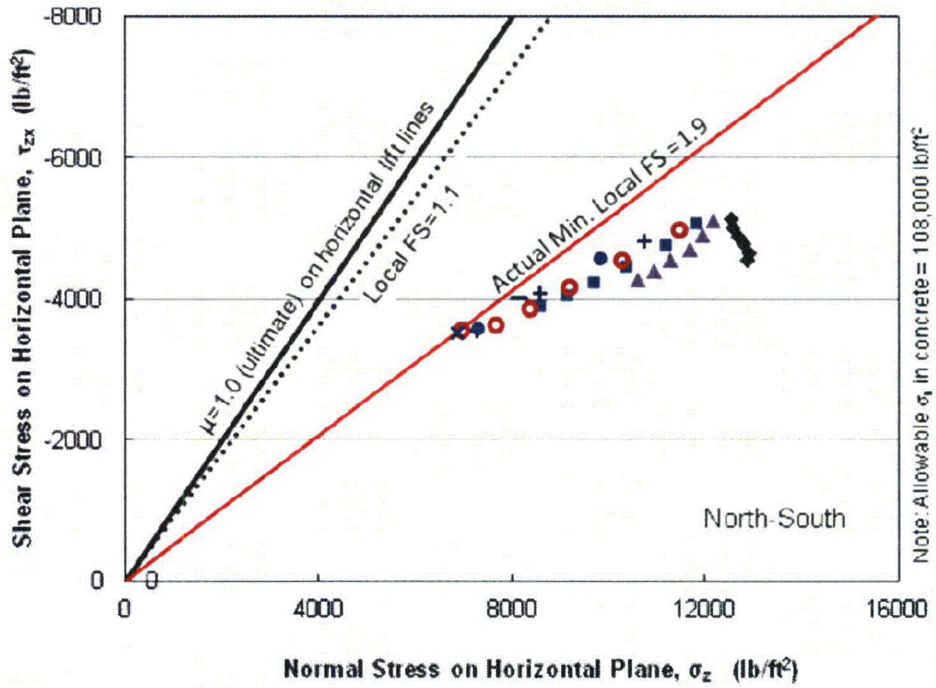
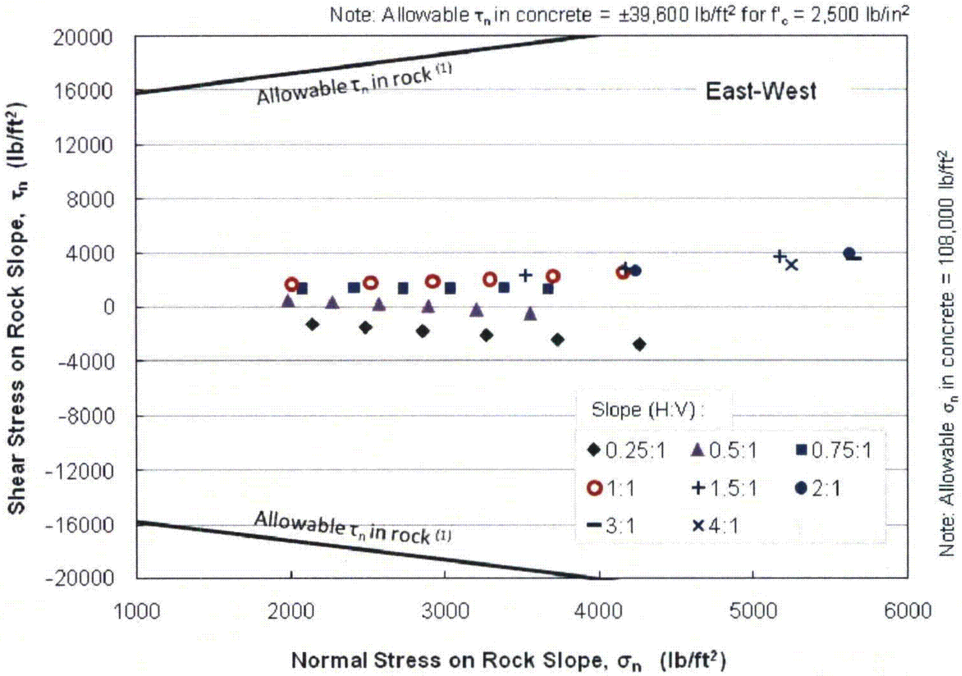
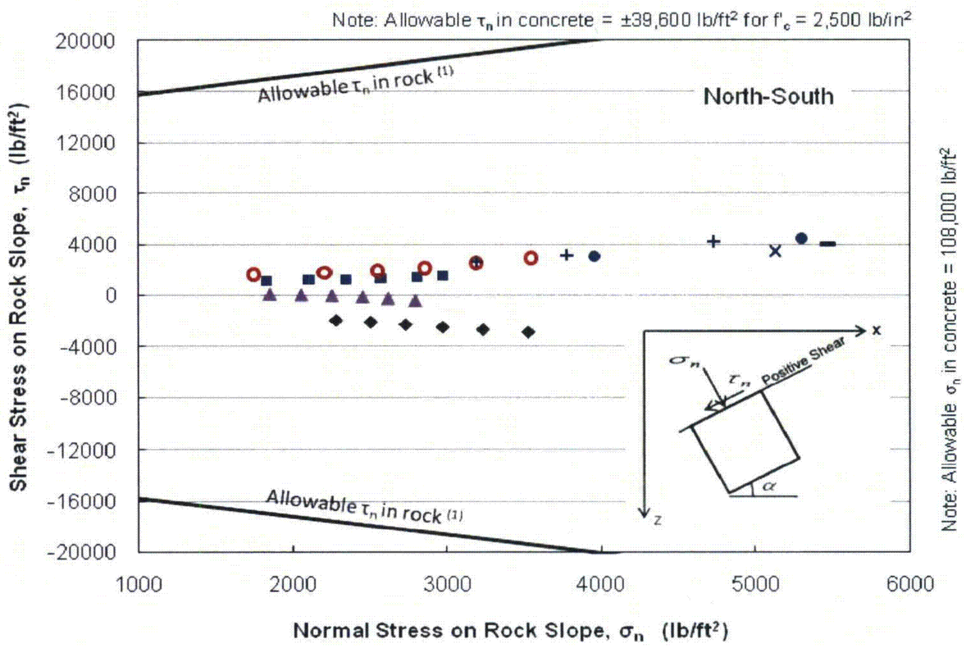


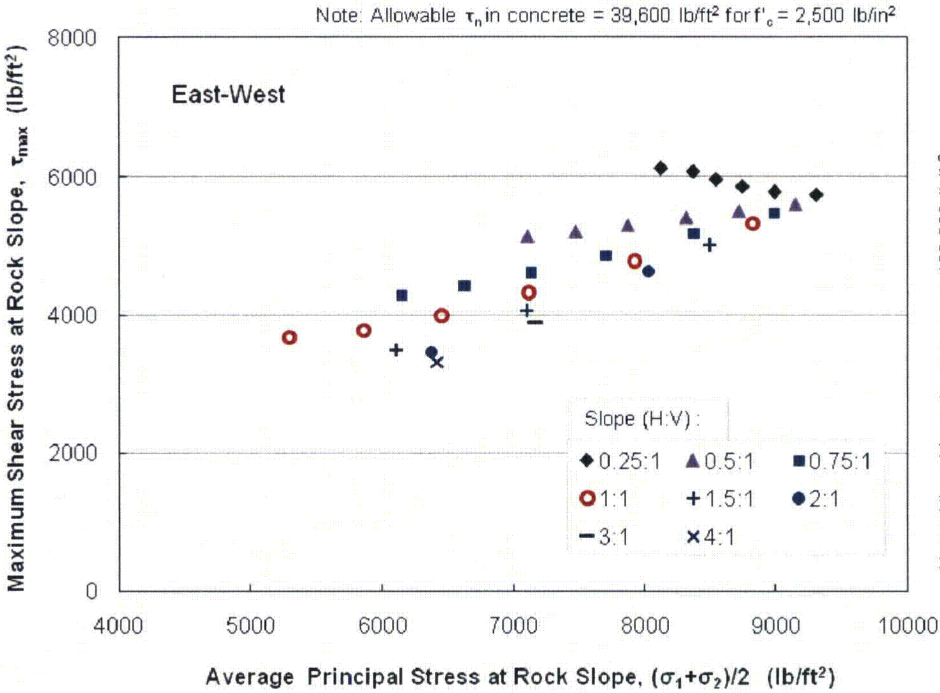
Figure 4. Stresses in Fill Concrete Adjacent to Rock Interface





(1) Allowable shear stress τ_n in rock =[21,600 lb/ft²+tan(65 deg)×σ_n]/FS(=1.5)

Figure 5. Stresses on Inclined Planes on Rock Interface



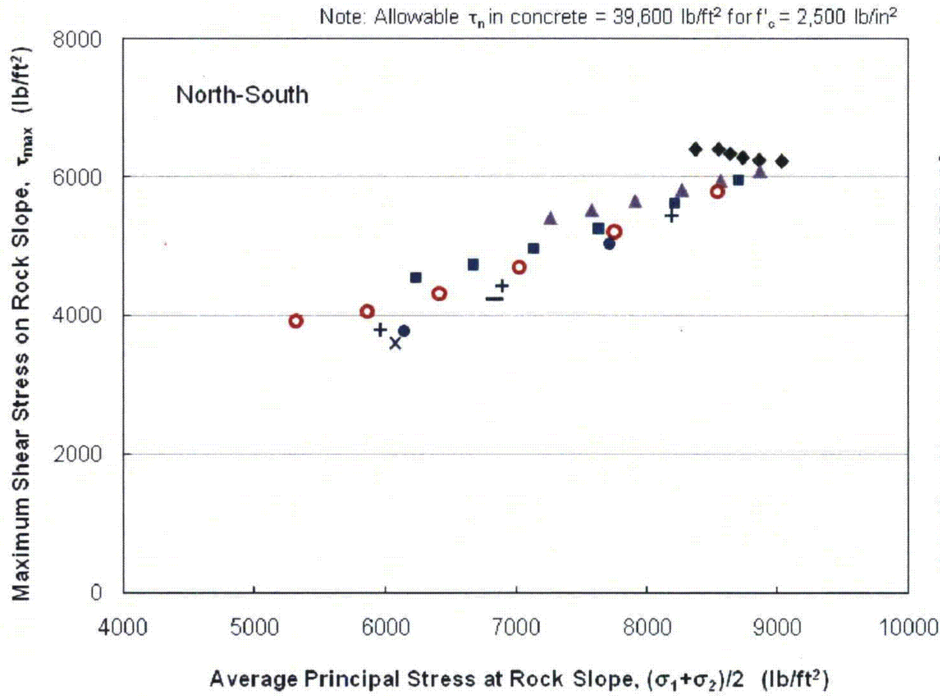


Figure 6. Maximum Shear Stresses in Fill Concrete Adjacent to Rock Interface