Tennessee Valley Authority, 1101 Market Street, LP 5A, Chattanooga, Tennessee 37402-2801

September 19, 2008

10 CFR 52.79

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555

In the Matter of)
Tennessee Valley Authority)

Docket No. 52-014 and 52-015

BELLEFONTE COMBINED LICENSE APPLICATION – RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION – STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

Reference: Letter from Ravindra G. Joshi (NRC) to Andrea L. Sturgis (TVA), Request for Additional Information Letter No. 101 Related to SRP Section 2.5.4 for the Bellefonte Units 3 and 4 Combined License Application, dated August 5, 2008

This letter provides the Tennessee Valley Authority's (TVA) response to the Nuclear Regulatory Commission's (NRC) request for additional information (RAI) items included in the reference letter.

A response to each NRC request in the subject letter is addressed in the enclosure which also identifies any associated changes that will be made in a future revision of the BLN application.

If you should have any questions, please contact Phillip Ray at 1101 Market Street, LP5A, Chattanooga, Tennessee 37402-2801, by telephone at (423) 751-7030, or via email at pmray@tva.gov.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 19th day of Sep, 2008.

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Manager, New Nuclear Licensing and Industry Affairs
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Enclosure
cc: See Page 2

D085
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Enclosure
TVA letter dated September 19, 2008
RAI Responses

Responses to NRC Request for Additional Information letter No. 101 dated August 5, 2008
(51 pages, including this list)

Subject: Stability of Subsurface Materials and Foundations as detailed in the Final Safety Analysis Report

RAI Number	Date of TVA Response
02.05.04-01	This letter – see following pages
02.05.04-02	This letter – see following pages
02.05.04-03	This letter – see following pages
02.05.04-04	This letter – see following pages
02.05.04-05	This letter – see following pages
02.05.04-06	This letter – see following pages
02.05.04-07	This letter – see following pages
02.05.04-08	This letter – see following pages
02.05.04-09	This letter – see following pages
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02.05.04-16	This letter – see following pages
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02.05.04-19	This letter – see following pages
02.05.04-20	This letter – see following pages
02.05.04-21	This letter – see following pages

Enclosure
TVA letter dated September 19, 2008
RAI Responses

Associated Additional Attachments / Enclosures

Attachment 02.05.04-05A	5 pages
Attachment 02.05.04-06A	4 pages
Attachment 02.05.04-14A	1 page
Attachment 02.05.04-15A	1 page
Attachment 02.05.04-18A	2 pages
Attachment 02.05.04-19A	1 page
Attachment 02.05.04-21A	2 pages
Attachment 02.05.04-21B	2 pages

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-01

FSAR Section 2.5.4.1.3.1 describes cavities encountered in the borings at the Bellefonte Units 3 and 4 site. The largest cavity discontinuity encountered in the borings within the construction zone is 4 feet while the largest cavity discontinuity encountered in all borings is 8 feet. Please explain the control level on the physical dimension of those cavities outside the bore holes and any potential impact on the stability of the foundation and structure.

BLN RAI ID: 0981

BLN RESPONSE:

Data on the physical dimensions of the cavities at the BLN site is derived from logs of boreholes that penetrated cavities. Within the Units 3 and 4 power block construction zone, 16 of the 75 borings drilled (21%) encountered one or more cavities (FSAR Table 2.5-225). Dimensional data consist of the measured lengths of each cavity along the borehole axis, recorded on the borehole log. There is no direct observed data on the physical dimensions of portions of cavities beyond the boreholes, and of the overall shape of the cavities. The measured dimensions of the cavities encountered in boreholes are summarized in FSAR Figure 2.5-305 and Table 2.5-226. As the RAI notes, the largest cavities identified throughout the Bellefonte site measured 4 feet and 8 feet in height, and up to 4 feet in height for boreholes drilled in the Units 3 and 4 power block construction zone. Most identified cavities were less than 0.5 feet, measured along the length of the borehole axis.

Figure 2.5-306 of the FSAR shows the boreholes in the power block construction zone plotted in cross section, with the location of each cavity indicated by a dot. Cavities are concentrated near the top of rock and formation contacts. This figure illustrates the available data and our best interpretation as to the occurrence of cavities, but does not reveal the three-dimensional geometry of cavities.

Cavities in the epikarst zone form from the movement of meteoric water through the soil and upper layers of the rock. Many of the cavities encountered in boreholes at the BLN site occur within 5 ft. of the top of rock, thus can be considered part of the epikarst zone. These cavities form from diffuse, downward moving water, and consist of tabular enlarged joints and bedding planes, and vertical slots that open upward into the soil. They are typically filled with soil. Within the nuclear island, cavities in the epikarst zone are expected to be removed during excavation of the soil and weathered rock.

The second type of cavity forms from the vertical and lateral movement of groundwater through joints, fractures, and along bedding planes in the rock. Dissolution begins by enlarging joints and bedding planes, and continued flow of water leads to the development of conduits with lenticular to circular cross sections.

Groundwater recharge within the power block construction zone at the BLN site occurs through the soil. Water soaks in along the bottoms of swales in the undulating valley floor, then further concentrates within the epikarst zone, entering the rock through multiple enlarged joints and depressions. Within the power block construction zone there are no large through-going drainages; the recharge water is derived from local runoff. A small amount of water available for recharge results in relatively small conduits. Thus the conduits through the rock would be expected to be relatively small in comparison to those within an area where recharge occurred at a single point such as a sinking stream or sinkhole.

Bedrock structure refers to the physical properties of the rock including lithology, porosity, bedding, jointing, and fracturing. As detailed in FSAR Subsection 2.5.4, the BLN power block construction zone, sits on the interbedded limestones, dolomitic limestone, and argillaceous and silty limestones of the

Stones River Group. Strata dip about 15 degrees to the southeast. Vertical and subvertical joints cut across bedding planes. Groundwater moving through these rocks would be expected to alternately follow joints and bedding planes as it moves laterally through inclined strata. Solution cavities along joint and bedding plane-controlled flow paths tend to be self-enhancing, because of the large contrast in hydraulic conductivity between open conduits and surrounding intact rock. However, the frequency of argillaceous and silty beds, and restricted size of encountered cavities in the construction zone borings, suggest that the local bedrock condition of the middle Stones Group rock sequence underlying the Units 3 and 4 generally restricts the size of cavities to the dimensional range identified in borings.

Cavity cross sections may be circular, lenticular, or irregular in shape, controlled in part by joints and the presence of less soluble beds. Cavities are likely to be concentrated between the top of rock and the historical groundwater table (>570 ft, elevation of the Tennessee River before impoundment).

In summary, the typical dimensions of cavities present at the BLN site may be approximately represented by the measured dimensions of the cavities penetrated by boreholes. As shown on FSAR Figure 2.5-305, most cavities are less than 0.5 ft. in length, with a small number of larger cavities up to a dimensional maximum of 4 ft. within the power block construction zone. Cavity shape is not known, but may form irregular pathways as they follow joints and bedding planes oriented vertically when following a joint and almost horizontal when following a bedding plane. They also may converge with other conduits. A conceptual model of karst in profile is presented in FSAR Figure 2.5-304.

Cavities within the rock below structural foundations are expected to be small and have no adverse impact on the stability of the structures at either nuclear island. Of the sixteen boreholes drilled within the nuclear islands, only a single borehole encountered a cavity below excavation grade (588.6 ft.); this cavity measured 0.1 ft in length. Experience at Bellefonte Units 1 and 2 (geologic mapping of excavation and Calyx holes), located in the same stratigraphic units as Unit 3 and in a similar geomorphic position, showed that rock conditions in the excavation were excellent and minor cavities were successfully remediated by typical dental concrete and limited grouting (FSAR Subsection 2.5.7, reference 201, BLN Units 1 & 2 FSAR, Figures 2.5-128 to -142.)

FSAR Subsection 2.5.4.1.3.4 discusses TVA's plan to conduct "geologic mapping and exploratory probing during construction to detail the full depth and configuration of dissolution depressions and cavities." The FSAR goes on to state that remediation of karst features will follow the methods described in Subsection 2.5.4.12, Techniques to Improve Subsurface Conditions. These measures provide verification of the absence of large cavities within the foundation excavation or near-surface of the foundation subgrade, and successfully treat small cavities that are encountered.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-02

FSAR Section 2.5.4 states that the bottom of the foundation basemat is located at an elevation of 588.6 ft. Please describe the minimum dimension of a cavity that could adversely impact both static and dynamic design for the basemat and the intersecting walls, and describe the possibility of encountering this kind of cavity beneath the basemat. In addition, please describe what procedures are to be used during excavation and construction to ensure that any cavities with potentially adverse impact to reactor safety do not exist below the foundation to ensure that the design of the basemat and intersecting walls will not be adversely impacted.

BLN RAI ID: 0982

BLN RESPONSE:

The RAI question posits that there may be a cavity below the level of the basemat that could not be spanned by the rock above the cavity and that this might cause loss of ground at the basemat level, thus affecting the basemat. In order to address this concern, the possible size of such a cavity is discussed below along with a discussion of the potential to discover and remediate cavities of that size. The purpose of foundation excavation examination and remediation is to check for conditions that would be detrimental to the performance of the basemat and to remediate those conditions.

The possible types of cavities that would be expected in the rock at the Bellefonte site are discussed in the response to NRC RAI No. 02.05.04-01. Cavities are expected to be concentrated above elevation 570 feet. Because the planned foundation level for the Unit 3 and 4 reactor basemats is 588.6 feet, the potential for encountering cavities is greatest within the rock approximately 20 feet of rock below the basemat elevation.

As indicated in the response to NRC RAI No. 02.05.04-01, cavities are typically portions of solution conduits that preferentially follow joints and bedding planes. Cavities are expected generally to have circular, ovaloid, slot-like or irregular cross sections. Cavities could be oriented horizontally or vertically, depending on the orientation of the controlling joints or bedding planes.

An initial evaluation of the potential size of cavities that might result in loss of ground below some portion of the reactor basemat (worst condition) used methods described in Obert, Duvall and Merrill (Reference 1). The analysis focused on tunnel excavations in bedded, competent rock and allowed an estimate of the maximum width of an unsupported tunnel roof (cavity) at different depths that could exist without exceeding the tensile stress of the rock above. The pertinent equation (page 24 of Reference 1) is:

$L = \text{Square Root } ((2Tt)/\gamma F)$, where:

L = roof span (least width),

T = tensile strength for layer above roof,

t = thickness of roof layer,

γ = rock unit weight, and

F = Factor of Safety.

The tensile strength of the rock was obtained from FSAR Table 2.5-236, and is conservative because the potential for blasting effects at the bearing level were included in the determination of the values in the table, whereas rock at depths below the basemat being considered herein would not be affected by the

excavation blasting. The values for tensile strength in FSAR Table 2.5-236 include consideration of the rock joints and fractures as well. For purposes of the evaluation, the lower bound tensile strength for the weaker argillaceous limestone was used, 0.020 kips per square inch (or 20 pounds per square inch.)

A rock unit weight of 169 pounds per cubic foot was used, based on results of the laboratory unconfined compressive strength testing described in FSAR Section 2.5.4.2.3.1.

For a cavity located with its top 5 feet below the basemat, the maximum unsupported width is about 9 feet for a Factor of Safety (FS) of 1 and only the weight of the rock above. Adding the static uniform bearing pressure (described in the DCD as 8600 pounds per square foot) to the weight of the rock, the maximum unsupported width decreases to about 4 feet at an FS=1.

The maximum unsupported width increases with depth below the basemat; for a depth of 20 feet below the basemat, the base of the zone where cavities are expected to be concentrated, the maximum unsupported width is about 14 feet, considering the static stress from the reactor basemat and an FS=1.

From the initial evaluation approach described above, a cavity within 5 feet of the basemat subgrade elevation and having a width less than 4 feet would not be expected to create a loss of support at the subgrade surface. The possibility of encountering such a cavity at the Bellefonte site is considered remote. As discussed in the response to NRC RAI No. 02.05.04-01, cavities are likely to be small. Also, as discussed in FSAR Subsection 2.5.4.1.3.1, cavities at the Units 1 and 2 power block locations were mostly in the upper 10 feet of rock and were removed by excavation.

The response to NRC RAI No. 02.05.04-08 discusses inclusion of geophysical methods in the foundation excavation monitoring program. Ground penetrating radar (GPR), electrical resistivity and microgravity surveys are expected to be used. A cavity with a width of 4 feet or more and located within 5 feet or 20 feet of the basemat subgrade would be expected to show an anomalous reading in the geophysical methods planned. A general guide for microgravity surveys is the 5:1 relationship, whereby a hypothetical minimum 10ft opening can be imaged at a 50-ft depth. Therefore, at a depth of 20 ft a minimum 4-ft cavity could be imaged. Anomalous readings would indicate a need for further checks using probe holes or core borings to define the source of the anomaly and bound its dimensions. The foundation improvement methods described in FSAR Subsection 2.5.4.12.6 would be suitable to remediate cavities identified.

In conclusion, the basemat design as described in DCD Section 3.8 and Table 2-1 allows for a uniform support from the foundation material. The potential for a weak or open portion of the subgrade resulting from a cavity located some depth below the subgrade surface to cause an impact on the DCD design is highly unlikely.

Reference

1. Obert, L., W. I. Duvall, and R. H. Merrill, "*Design of Underground Openings in Competent Rock*", Bulletin 587, Bureau of Mines, U. S. Department of the Interior, 1960.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-03

FSAR Section 2.5.4.1.3.2 indicates boring rod drops as much as 12 to 14 ft during the boring process. Please explain if such drops were related to cavities. Please also describe if any of the soft zones described in this Section extend to depths below the basemat.

BLN RAI ID: 0983

BLN RESPONSE:

The RAI cites that the FSAR indicates rod drops of between 12 to 14 ft. were encountered during the drilling process. The FSAR does not indicate rod drops of this magnitude. The RAI may be referring to the 12-ft. interval in boring B-1051 that includes a 5-ft. rod drop followed by 7 ft. of soft, wet mud. Boring B-1051 is located along the south perimeter wall of the Unit No. 4 turbine building, a structure that is not safety related. No rod drops of "12 to 14 ft." as listed in the RAI occurred, and measurable rod drops in the soil were not recorded in other COLA borings. A log for a non-COLA boring, drilled near the Units Nos. 3 and 4 construction zone for a 1987 soils investigation (SS-28), indicates SPT values of "0" and "3" which suggests very soft soil, or a possible localized cavity. Additionally, groundwater rose to within 12 ft. of the ground surface in the SS-28 boring, but groundwater was not encountered in surrounding 1987-vintage borings. However, the log does not indicate a rod drop in the boring.

With reference to the B-1051 occurrence, Subsection 2.5.4.1.3.2 of the FSAR specifically states, "...Boring B-1051 encountered stiff clays which began to soften at 27 ft. At 31 ft. the drill rods dropped to a depth of 36 ft. through a cavity, and from that depth to 43 ft. the drill penetrated soft, wet mud without reaching solid bedrock." As noted by the geologist logging this boring, the free fall rod drop was from 31 to 36 ft., with the wet mud encountered between 36 and 43 ft. The soil log of this boring shows that SPT sampling was conducted from 36 to 43 ft., with increasing blow counts with depth, until refusal was encountered at 43 ft.

The rod drop indicated on the B-1051 boring log as Weight of Rods [WOR] suggests either the presence of an open cavity (conservative interpretation), or very soft soil of a consistency that would not resist the drill rod advance, within the 5-ft interval between 31 and 36 ft. depth. This cavity/soft soil zone is irregular in shape and, as indicated by additional exploration conducted in this vicinity (inclined and vertical borings, CPT probes, and monitoring wells), is bounded by bedrock walls. Figure FSAR 2.5-309 shows the additional explorations completed around Boring B-1051. The additional explorations were located specifically to evaluate the lateral extent of the feature, and indicate maximum lateral dimensions on the order of about 10 by 10 ft. The B-1051 void/soft soil zone occurs within a larger bedrock surface depression that appears to be on the order of 50 to 100 ft. wide based on microgravity surveys. These explorations also confirm that the B-1051 feature does not extent northward to the safety-related nuclear island structure of Unit No. 4, but rather terminates about 300 ft. south of the nuclear island perimeter. Therefore, the B-1051 feature does not extend into the Unit No. 4 basemat zone, or pass below the basemat. Inclined Boring B-1052 passed below the B-1051 void/soft soil zone, and encountered limestone bedrock at a depth of 35 ft., and two small bedrock cavities at 45 ft. depth (measuring 1.1-ft. thick), and 60 ft. depth (measuring 1.6 ft. thick). Sound rock bridges occur above, and between, these localized cavities, suggesting that the B-1051 and B-1052 features do not join to form a large cavity.

The combined B-1051 rod drop and soft soil zone extends to a depth of 43 ft., or elevation 586.1ft., as shown on the boring log. The turbine building is supported on a deep pile foundation that derives bearing in sound rock below elevation 586.1 ft.; therefore this feature will not affect the stability of the non-safety Turbine Building structure.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-04

FSAR Section 2.5.4.1.3.3 describes the irregularity of the rock surfaces and contacts within the Construction zone and ascribes this irregularity to the variability of the dissolution weathering front created by the slow downward movement of erosive water through the soil and rock. Please explain how the irregularity of the rock surface beneath the foundation compares to the uniformity criteria presented in the AP1000 DCD.

BLN RAI ID: 0984

BLN RESPONSE:

The uniformity criteria are described under subsection 2.5.4.5.3, Site Foundation Material Evaluation Criteria in Revision 16 of the DCD. The DCD requirements are intended to ensure "...the foundation conditions do not have extreme variation within the nuclear island footprint." There are three criteria that are to be applied to the upper 120 feet of material below finished grade of the nuclear island footprint to demonstrate site uniformity:

1. The depth to soil/rock layer interfaces in the 120-foot depth profile should deviate no more than 5 percent from the average interface depth. If a deviation greater than 5 percent occurs, then the profile should be modified by adding additional layers/interfaces, or additional borings (to provide better resolution of the average layer interface depth(s)).
2. For layers exhibiting low strain shear wave velocity (Vs) greater than or equal to 2,500 feet per second (fps), the layer should have approximately uniform thicknesses and should have a dip no greater than 20 degrees, and the Vs within any layer should not vary from the average by more than 20 percent.
3. For a layer with a low strain Vs less than 2,500 fps, the layer should have approximately uniform thickness and should have a dip no greater than 20 degrees, and the Vs within the layer should not vary from the average by more than 10 percent.

Additionally, the DCD indicates that the key attribute for acceptability of the site for an AP1000 is the bearing pressure on the underside of the basemat. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site.

With respect to application of the DCD uniformity criteria, the irregularity of rock weathering would be considered to be a non-uniform condition because of unpredictable and highly variable dip of the weathering front (typically exceeding a dip of 20 degrees), velocity differences between weathered and unweathered rock, and variable soft and hard spots.

However, as shown in FSAR Figure 2.5-339, the Units 3 and 4 nuclear island basemats are supported on fresh/slightly weathered, competent middle Stones River Group limestone bedrock below the weathering front. Weathered rock is to be removed completely from the foundation footprint, such that the 120-foot depth profile below the nuclear island referenced in the DCD is comprised of competent limestone bedrock below the weathering front. The competent limestone bedrock exhibits low strain shear wave velocity greater than 2,500 feet per second, and has individual beds of uniform thickness that dip less than 20 degrees. These conditions meet the uniformity criteria of the AP1000 DCD. Foundation preparation in FSAR Subsection 2.5.4.12.3 specifies that "Weathered discontinuities which are encountered during excavation of the foundation are cleaned to a minimum of two times their width or if the joint widens with depth cleaned downward farther until a wedging effect can be achieved with fill concrete". This

procedure treats any possible localized and small weathered features that may extend below the weathering front.

The combination of foundation embedment in competent rock, and foundation preparation approaches to treat possible localized deeper weathered features, eliminates non-uniformity related to the weathering front, with the end result that the foundation subgrade represents a uniform, competent bearing surface.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-05

AP1000 DCD (Rev.16) has site criteria for the lateral variability of the foundation supporting soils. Geologic units inside the Middle Stones River Formation demonstrate different geotechnical properties. For example, FSAR Figure 2.5-299 shows a sharp contrast between Units C and D in terms of their shear wave velocities. Due to the inclining layering at the site, different geologic units will appear beneath the same reactor foundation. For example, Middle Stones River Unit C will support the eastern part of the Unit 3 foundation and Unit D will support the western half of the foundation. Please explain if this subsurface lateral variability meets the minimum requirement of the AP 1000 DCD site criteria on lateral variability. Furthermore, please explain if the inclined geologic boundaries beneath the foundation basemat will affect the surface ground motion estimates and possibly introduce a seismic force concentration.

BLN RAI ID: 0985

BLN RESPONSE:

A comparison of geotechnical information is summarized below.

The subsurface lateral variability meets the minimum requirement of the AP1000 DCD site criteria on lateral variability as follows: AP1000 DCD Subsections 2.5.4.5 and 2.5.4.5.3 establish requirements to classify a site as "uniform" and limits on lateral variability, respectively.

The uniformity criteria are satisfied:

- Bellefonte Unit 3 shear wave velocity (Vs) is constant at 9800 fps and Unit 4 increases from 7800 fps to 9800 fps at 120 ft below ground.
- The limestone and argillaceous limestone dip uniformly to the southeast at an inclination of about 17 degrees which is less than AP1000 DCD limit of 20 degrees.

The lateral variability criteria are satisfied:

- For the Unit 3 basemat, bedrock Unit D exists under most of basemat with Unit C on the southeast side. The average Vs of Unit C and Unit D is 8400 fps. The differences of shear wave velocity between Units C and D and the averaged Vs are 17%, which are less than 20 percent variation limit stated in AP1000 DCD.
- For the Unit 4 basemat, bedrock Unit A underlies most of the basemat with Unit B on the northwest side. The average Vs of Unit A and Unit B is 8800 fps. The differences of shear wave velocity between Units A and B and the averaged Vs are 11%, which are less than 20 percent variation limit stated in AP1000 DCD.

Based on calculation results summarized in Table 1 below, Bellefonte Unit 3 and 4 site lateral variabilities are within the specified AP1000 DCD limits.

The Unit 3 foundation will consist of a relatively thin (1 to 5 ft) layer of leveling concrete emplaced on rock Units C and D (See Figure 1 of Attachment 02.05.04-05A). The leveling concrete will have a shear wave velocity comparable to the lower velocity Unit C which is less than the shear-wave velocity of Unit D.

Table 1 - Bellefonte Lateral Variability Comparison

	Bellefonte Unit 3		Bellefonte Unit 4	
Lateral Variability	Limestone	Vs. (fps)	Limestone	Vs. (fps)
Under Basemat	Unit D	9800	Unit A	7800
Under Basemat	Unit C	7000	Unit B	9800
Average		8400		8800
Percentage difference	Unit C vs. Average	17%	Unit A vs. Average	11%
AP1000 DCD limit		20%		20%

The Unit 3 foundation velocity structure produced by the dipping strata and the layer of fill concrete consists of two distinct velocities, Unit D with a shear-wave velocity of 10,000 fps, denoted V2, and Unit C and the fill concrete, both with a shear-wave velocity of 6500 fps, denoted V1. This value for V1 is selected to correspond to the lowest plausible mean estimate of V1 consistent with Figure 2 shown in Attachment 02.05.04-05A to obtain the most conservative estimate of the potential influence of V1 on site response and potential seismic force concentrations. These distinct shear-wave velocity regions always have the lower velocity material, V1, located above the higher velocity material, V2. Separating the two velocity units is a nearly flat interface between Unit D and the fill concrete under the western portion of the Unit 3 foundation and a 15 degree east-dipping interface between Units C and D beneath the eastern part of the Unit 3 foundation (shown in Figure 1 of Attachment 02.05.04-05A).

The second part of the question asks if inclined geologic boundaries beneath the foundation basemat will affect the surface ground motion estimates and possibly introduce a seismic force concentration. To answer this question, the potential for surface ground motion amplification associated with lower-velocity Unit C and the fill concrete is first evaluated and then the possibility of a force concentration associated with the inclined geologic boundaries beneath the foundation basemat is evaluated.

The maximum combined thickness of Unit C and the fill concrete between the basemat and Unit D at the eastern limit of the basemat is about h=12 ft. (see Figure 1 of Attachment 02.05.04-05A). The minimum resonance frequency associated with amplification occurs at the eastern limit of the basemat corresponding to the maximum value of h. The minimum resonance frequency associated with amplification is f=V1/(4*h)=135 Hz, using the quarter wavelength approximation (Boore and Brown, 1998; eqns. 2 and 5)(Reference 1). Consequently, any amplification associated with the lower-velocity Unit C portion of the basemat will occur at frequencies greater than 100 Hz, which exceeds the maximum frequency considered in probabilistic seismic hazard analyses (NRC Regulatory Guide 1.208, Page 7, last paragraph).

The velocity structure beneath Unit 3 approximates a plano-concave lens that diverges collimated incident waves, in this case incident SH-waves. The qualitative effect of the dipping strata beneath Unit 3 is to defocus seismic forces in the foundation basemat where the dipping interface intersects the fill concrete. The velocity structure does not produce a convergence (focusing) of seismic forces in any other portion of the foundation. Snell's law is used to demonstrate that ray divergence in the fill concrete and Unit C near the intersection of the dipping interface with the fill concrete is never less than 5.3 degrees for the angles of incidence associated with incident shear waves (Figures 3 and 4 of Attachment 02.05.04-05A). The only systematic influence of the dipping strata beneath Unit 3 is to locally decrease seismic forces in the

foundation basemat. Consequently, the inclined geologic boundaries beneath the foundation basemat do not introduce a seismic force concentration in the basemat or fill concrete.

Reference

1. Boore, D. M. and L. T. Brown, 1998. Comparing shear-wave velocity profiles from inversion of surface-wave phase velocities with downhole measurements: Systematic differences between the CXW method and downhole measurements at six USC strong-motion sites, Seism. Res. Lett. 69, 222-229.

This response is PLANT SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Section 2.5.4.7.4 fourth paragraph will be revised from:

The geologic conditions satisfy the definition of a "Uniform" hard rock site specified in the DCD Section 2.5.4.5 for the nuclear island basemats. The limestone bedrock is regularly bedded with a gentle dip 15° to 17° inclination, and individual beds exhibit substantial uniformity in conditions both along strike and dip throughout the Units 3 and 4 power block construction zone. The weathered top of rock is irregular with local variations in depth to top of rock on the order of about 3 to 10 ft. typically, but is globally quite flat without an overall sloping surface.

To read:

The geologic conditions satisfy the definition of a "uniform" hard rock site specified in the DCD Subsection 2.5.4.5 for the nuclear island basemats. Specifically, for the Unit 3 basemat, bedrock Unit D exists under most of basemat with Unit C on the southeast side. The average Vs of Unit C and Unit D is 8400 fps. The differences of shear wave velocity between Units C and D and the averaged Vs are 17%, which are less than 20 percent variation limit stated in DCD Subsection 2.5.4.5. For the Unit 4 basemat, bedrock Unit A underlies most of the basemat with Unit B on the northwest side. The average Vs of Unit A and Unit B is 8800 fps. The differences of shear wave velocity between Units A and B and the averaged Vs are 11%, which are less than 20 percent variation limit stated in DCD Subsection 2.5.4.5. The limestone bedrock Units A, B, C, and D are regularly bedded with a gentle dip 15° to 17° inclination which is less than DCD Subsection 2.5.4.5 limit of 20°. The individual beds exhibit substantial uniformity in conditions both along strike and dip throughout the Units 3 and 4 power block construction zone. The weathered top of rock is irregular with local variations in depth to top of rock on the order of about 3 to 10 ft. typically, but is globally quite flat without an overall sloping surface.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

02.05.04-05A Four figures

Figure 1 - Vertical limits of excavation, Bellefonte Nuclear Plant, Unit 3 (from FSAR Figure 2.5-348a)

Figure 2 - Shear wave velocity stratigraphic column (from FSAR Figure 2.5-299)

Figure 3 - Calculation geometry, input parameters, and results for three SH-wave incident angles

Figure 4 - Divergence angle, ΔJ_e, as a function of SH-wave plane-wave incident angle for the Reactor Unit #3 fill concrete and basemat.

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-06

Based on seismic reflection profiles, FSAR Section 2.5.4.1.3.3 presents P-wave velocity (Vp) contours for two surfaces at 6000 and 14000 ft/sec, respectively (in FSAR Figures 2.5-312 and 313). You also implemented P-S suspension logging to measure the P wave velocity profiles at various borehole locations. Please provide a comparison of the wave velocity profiles obtained from these two different methods. Are these methods consistent in terms of P wave velocity measurements? FSAR Figure 2.5-339 shows a site stratigraphic profile across the site, including borehole locations and corresponding shear wave velocity profiles, as well as Units 3 and 4 foundation profiles. The figure demonstrates that the Unit 3 foundation excavation is located inside the bedrock with a shear wave velocity about 9000 fps and a P wave velocity of about 16000 fps. FSAR Figure 2.5-314, however, shows a large area underneath Unit 3 where the top of the 14000 fps (Vp) layer is below elevation 588.6 ft. Please explain the discrepancy.

BLN RAI ID: 0986

BLN RESPONSE:

The RAI notes a discrepancy between P-wave velocity (Vp) determination from P–S suspension logging in the boreholes and P-wave velocity determination computed from seismic refraction survey data. The two methods provide significantly different values of Vp for the same rock layer. The depth of the top of rock having a minimum of 14,000 fps is 2 to 8 feet deeper from the seismic refraction data compared to the P-S suspension data. Comparison of the 14,000 fps seismic refraction data to the top-of-competent rock determined from borehole logs and photographs shows discrepancies as much as 20 feet. The P-S suspension logging estimates of P-wave velocity are systematically slightly higher than the seismic refraction estimates.

The discrepancy between estimates to top of competent rock defined by the depth to a P-wave velocity of 14,000 fps based on seismic refraction measurements in FSAR Figure 2.5-314 and the estimates of depth to top of competent rock defined in FSAR Figure 2.5-339 based on borehole data are the results of several systematic factors in the acquisition, analysis, and interpretation of the seismic refraction data that tend to introduce a bias to overestimate refractor depths.

The frequency of the P-wave source tool employed by the P-S suspension logging is higher than that used by seismic refraction, resulting in higher velocities with P-S suspension logging for the same rock. The P-S suspension logging estimates of P-wave velocity are obtained in the 8000 Hz frequency range. As noted in Aki and Richards (1980) (Reference 1, pp. 212-214), head waves not only decay with distance as $r^{-1/2}L^{-3/2}$, where L is the refracted distance and r is the total path distance, but head wave amplitudes decrease linearly with increasing frequency. Consequently, while the 8-gauge shotgun source used in the seismic refraction profiles may excite P-waves to frequencies as high as 400-500 Hz (Miller et al., 1992) (Reference 2), the seismic refraction data generally produce refracted P-wave velocity estimates for frequencies < 250 Hz. Therefore, the effective frequency contents of the signal for the two techniques are significantly different.

Batzle et al. (2006) (Reference 3) noted that significant discrepancies in velocity are often found when comparing sonic logs to checkshot or vertical seismic profile surveys. For instance, De et al. (1994) (Reference 4) find velocities 1%–7% higher in sonic logs, indicating that P-wave velocities increased with increasing frequency. Considerable effort has been expended reconciling velocity values observed with different velocity measurement techniques. Even in a completely homogeneous rock, frequency-dependent velocities, or dispersion, yield inconsistent values between measurements in different

frequency bands. To achieve measurement method consistency, Batzle et al. (Reference 3) used a forced deformation system in conjunction with pulse transmission to obtain elastic properties at seismic strain amplitude (10^{-7}) from 5 Hz to 800 kHz. They concluded that porous and permeable sands and carbonates are most likely to exhibit significant velocity dispersion, that is, frequency dependant velocity behavior. Consequently, given the saturated, porous, and permeable carbonate conditions at the site, the systematically higher estimates of P-wave velocity obtained using P-S suspension logging relative to seismic refraction estimates of P-wave velocity are in accordance with published results and expectations.

The seismic refraction profiling used a geophone spacing of 10 ft. Estimated refractor depth, z_r, is related to the crossover distance, x_{cross}, where the travel time curve slope switches from 1/v0, where v0 is the first layer velocity, to 1/v1, where v1 is the second layer refractor velocity as z_r =0.5*sqrt((v1-v0)/(v1+v0))* x_{cross} (Dobrin, 1976) (Reference 5, pages 296-298). Given a 10-ft. geophone spacing, there will be an average 5 ft. positive bias in the estimation of x_{cross} that corresponds to detecting the transition from the 1/v0 to 1/v1 branches of the travel time curve. For v0=6000 fps and v1=14000 fps and z_r =20 ft, x_{cross}=63.25 ft. A positive bias of 5 ft in x_{cross} will overestimate z_r by 1.6 ft.

Further, typical first-arrival travel time picking uncertainties of one-half to several milliseconds defer statistically-significant resolution of the change in travel time slope from 1/v0 to 1/v1 to distances > x_{cross}. This will smear the first-order velocity change between v0 and v1 into a gradient zone and create an additional systematic bias toward overestimating z_r.

Low velocity zones at the base of the soil will create an additional bias toward overestimating z_r (Reference 5, page 301). Blow count data from logs B-1010, B-1012, B-1035, B-1044, B-1049, and B-1055 indicate that low-velocity zones exist in soil in some regions just above competent rock. These low-velocity zones will increase travel times, thereby erroneously increasing estimated z_r.

Another factor of bias that has only recently been discovered is that head waves have frequency-dependent travel-time sensitivity kernels (Zhang et al., 2007) (Reference 6). In other words, the refracted rays have a finite cross sectional area, determined by the signal bandwidth, and sample a volume of subsurface material. Consequently, the geometric ray approximation (rays with infinitesimal cross sectional area) used in the seismic refraction velocity inversion will overestimate depth to refractors because the primary sensitivity of the head waves along the refractor are associated with waves propagating about half a wavelength below the top of the refractor and the geometric head-wave ray depth (Reference 6). For a P-wave head wave frequency of 250 Hz and velocities of 6000-14,000 fps, the half wavelengths are 12-28 ft. Further, the head-wave travel-time sensitivity kernels are most sensitive to low-velocity zones above the refractor near the piercing points of the geometric ray with the refractor. Low-velocity zones encountered near head-wave refractor piercing points cause the strongest travel time delays which in turn produce biases toward overestimating refractor depth.

Finally, Zhang et al. (Reference 6) found that upward undulations of refractors depress the travel-time sensitivity kernels below the undulation such that travel-time sensitivities are no closer to the refractor surface under an upward refractor undulation relative to a flat refractor. Consequently, the head-wave travel-time sensitivity kernels in Zhang et al. (Reference 6) show that the influence of the irregular refractor surface associated with the weathered top of rock serves to create a bias toward overestimating refractor depth relative to the actual irregular refractor depths.

As noted in the seismic refraction report, the top of relatively fresh rock likely corresponds to P-wave velocities in the 11,000-14,000 fps velocity range. The decision to use 14,000 fps for simplicity to contour and define the top of relatively fresh rock in FSAR Figure 2.5-314 likely results in overestimation of the depth to the top of rock in some regions, particularly where P-wave velocities of 11,000-12,000 fps would be more appropriate. Use of a P-wave velocity of 11,000-12,000 fps to define the top of rock would decrease estimated depths by several feet.

Analysis of the factors that influence estimated depth to seismic refractors shows that these factors bias toward overestimation of refractor depths. The head-wave frequency-dependent travel-time sensitivity

kernel investigations of Zhang et al. (Reference 6) represents the first investigation of finite frequency effects on seismic refraction data and interpretation, and was published after the seismic refraction analyses were completed. Quantifying the refractor depth biases associated with finite-frequency effects will require further research. It is not possible to provide a quantitative calculation of finite-frequency biases at this time, but the qualitative conclusion that can be drawn from Zhang et al. (Reference 6) is that a geometric ray interpretation of the seismic refraction data will result in the overestimation of refractor depth.

The cumulative effects of all refractor depth estimation biases is sufficient to explain the tendency of Figure 2.5-314, based on seismic refraction analyses of depth to refractor, to overestimate depths to top of rock by 0-20 ft relative to FSAR Figure 2.5-339, which is based on borehole data. Consequently, the most appropriate definition of depth to competent rock should be based on borehole data and actual conditions revealed during excavation, not on the seismic refraction data.

References

1. Aki, K. and Richards, P.G., 1980, Quantitative seismology, theory and methods: San Francisco, W. H. Freeman and Company, v. 1,557 p.
2. Miller, R.D., Pullan, S.E., Steeples, D.W., and Hunter, J.A., 1992, Field comparison of shallow seismic sources near Chino, California: Geophysics, v. 57, no. 5, p. 693709.
3. Batzle, M.L., De-Hua Han, and Hofmann, R., 2006, Fluid mobility and frequency- dependent seismic velocity - Direct measurements: Geophysics, v. 71, no. 1, p. N1-N9.
4. De, G.S., Winterstein, D.F., and Meadows, M.A., 1994, Comparison of *P*- and S-wave velocities and Q's from VSP and sonic log data: Geophysics, v. 59, no. 10, p. 1512-1529.
5. Dobrin, M.B., 1976, Introduction to geophysical prospecting: U.S.A., McGraw-Hill, Inc., 3rd edition, 630 p.
6. Zhang, Z., Shen, Y., and Zhao, L., 2007, Finite-frequency sensitivity kernels for head waves: Geophysical Journal International, 171, p. 847-856.

This response is PLANT SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

1. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.1.3.3, will be revised from:

Seismic reflection 3D-modeling also provides a "first look" at excavation conditions. For Unit 4, both the top of the 6,000 fps Vp layer and the 14,000 fps Vp layer are above excavation grade of 589 ft. With the exception of any deep cavities or slots not imaged by the method, the Unit 4 excavation extends below most cavities and the weathered rock.

Unit 3, however, is located downslope, thus the ground surface and underlying bedrock surface occur at lower elevations. Seismic refraction 3D modeling shows areas beneath Unit 3 where the top of the unweathered rock, or the 14,000 fps Vp layer, is below excavation grade of 589 ft. (A Vp of 14,000 fps corresponds to a Vs of 8000 fps, given a Poissons ratio of 0.26.) Sufficient excavation beneath Unit 3 removes weathered rock and establishes the foundation on hard rock (Figure 2.5.4-218, 2.5.4-250 and 251).

To read:

Due to inherent limitations of the seismic refraction method, discussed in Subsection 2.4.4.1.2, the seismic refraction models do not provide precise elevations of the top of competent rock. Top of

competent rock is determined from the borehole data, and is defined here as the elevation below which rock core appears fresh, RQD is greater than 70%, geologists logs show no significant weathered intervals, and Vp measured from the borehole P/S logs (if available) exceeds 14,000 fps.

Contour maps of the top of competent rock for Units 3 and 4 (Figure 2.5-314), similar to the top of weathered rock (Figure 2.5-310), show an irregular surface below the soil and weathered rock overburden. Excavations are expected to extend below these irregularities such that both Units 3 and 4 nuclear islands will be founded entirely on competent rock. Foundation grade is 588.6 feet for both units. However, local deep depressions in the bedrock surface, not discovered by the existing borehole grid, may be present that would require additional localized excavation.

2. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.4.1.2, will be revised from:

Velocity profiles of the refraction survey data indicate subhorizontal velocity intervals. Some irregularity is observed which most likely represents variable weathering on the top of limestone bedrock. Figures 2.5-343, 2.5-344, and 244-345 display the velocity panels the correspond with Unit 3 and 4 nuclear islands, as well as geotechnical Profiles A-A', B-B', and C-C'. Correlation with the microgravity surveys on other profiles are presented on Figure 2.5-346.

To read:

Velocity profiles of the refraction survey data indicate subhorizontal velocity intervals. Some irregularity is observed which most likely represents variable weathering on the top of limestone bedrock. Figures 2.5-343, 2.5-344, and 244-345 display the velocity panels that correspond with Unit 3 and 4 nuclear islands, as well as geotechnical Profiles A-A', B-B', and C-C'. Correlation with the microgravity surveys on other profiles is presented on Figure 2.5-346.

A comparison of the depth of the various velocity intervals with borehole data shows that the top of the 6,000 fps seismic refraction layer correlates well (+/-5 feet) to the top of rock as determined from hollow stem auger refusal or SPT (Figure 2.5-310). However, the higher velocity intervals, especially the 14,000 fps layer, are anomalously deep when compared with borehole data.

The depth of the top of rock having a minimum of 14,000 fps is 2 to 8 feet deeper from the seismic refraction data compared to the P-S suspension logging data. Comparison of the 14,000 fps seismic refraction data to the top-of-competent rock determined from borehole logs and core photographs shows discrepancies as much as 20 feet. The P-S suspension logging estimates of P-wave velocity are systematically higher than the seismic refraction estimates.

The discrepancy is the result of several systematic factors in the acquisition, analysis, and interpretation of the seismic refraction data that tend to introduce a bias to overestimate refractor depths. These factors include (1) use of a lower frequency P-wave source in the seismic refraction survey relative to the suspension logs, in addition to the attenuation of the high frequency head-wave components with distance, which results in lower estimates of refraction velocities in rock, (2) the relatively wide (10 ft) geophone spacing employed which resulted in lower resolution of refractor depths, (3) the use of modeling software that assumes smooth velocity gradients instead of the abrupt interfaces seen in borehole logs, (4) the intermittent presence of low velocity zones at the base of the soil just above the top of rock which increases P-wave travel times, and (5) the presence of an irregular bedrock-soil interface which depressed the top of the refractor to a depth beneath the irregularities. Consequently, the most appropriate definition of depth to competent rock should be based on borehole data and actual conditions revealed during excavation, not the seismic refraction data.

3. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.4.1.3, will be revised from:

Seismic p-wave velocities of 6000 and 14,000 fps are most correlative with the top of weathered rock and the top of competent rock, respectively. The 6000 fps model is presented in Figure 2.5-312 and the 14,000 fps model is shown in Figure 2.5-313. Subsection 2.5.4.1.3.3 presents a thorough discussion of the interpretation of these results with respect to weathering of bedrock and karst.

To read:

Seismic P-wave velocity of 6000 fps is most correlative with the top of weathered rock. Figure 2.5-310 provides a comparison of the seismic refraction data and the borehole data. A more detailed 6000 fps model is presented in Figure 2.5-312, and the 14,000 fps model is shown in Figure 2.5-313. Subsection 2.5.4.1.3.3 presents a thorough discussion of the interpretation of these results with respect to weathering of bedrock and karst.

4. COLA Part 2, FSAR Chapter 2, Figure 2.5-314 will be revised as presented in Attachment 02.05.04-06A.

5. COLA Part 2, FSAR Chapter 2, Figure 2.5-348a will be revised as presented in Attachment 02.05.04-06A.

6. COLA Part 2, FSAR Chapter 2, Figure 2.5-348b will be revised as presented in Attachment 02.05.04-06A.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-06A	Revised FSAR Figures 2.5-314 and 2.5-348a & b

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-07

Section 2.5.4.1.3.3 describes weathering zones (Eastern and Western Anomaly Zones) extending to great depths (as much as 90 ft) in comparison to the design depth of the basemat (40 ft). Please demonstrate the locations for the two anomalies relative to the Unit 3 and 4 nuclear islands and explain if there is any impact from the weathering zones to the GMRS calculation.

BLN RAI ID: 0987

BLN RESPONSE:

The Western and Eastern Anomaly Zones were identified during prior explorations (FSAR reference 399) at a different proposed plant location termed the "Southern Site." The southern site is located approximately 3000 feet south of the BLN Units Nos. 3 and 4 construction zone, as indicated on FSAR Figure 2.5-201. The Eastern Anomaly Zone occurs along strike, and within the same Stones River Group rock unit, as that occupied by the BLN Unit 3 reactor area. However, a significant difference between the two areas is that the Eastern Anomaly Zone is located on one of the most-prominent topographic lineaments mapped at the site, Lineament #2 (FSAR Figure 2.5-291) that may represent the abandoned eroded paleo channel of Town Creek. No such lineaments are identified at the Unit No. 3 location. Extensive subsurface exploration demonstrates that the degree and depth of rock weathering in the BLN Unit No. 3 (and Unit No. 4) area are much less than for the Eastern Anomaly at the Southern Site. The more-extensive and deeper weathering in the Eastern Anomaly Zone is likely due to enhanced weathering occurring along the Lineament #2 feature, and possible prior scour or weathering caused by former occupation by the paleo channel of Town Creek

The Western Anomaly Zone in the ESP Southern Site is also developed within the Stones River Group bedrock; however, when projected northeasterly along strike, the same rock units fall west of BLN Unit No. 3 site. Additionally, the Western Anomaly Zone occurs along Lineament #2, and likely exhibits localized more-severe and deeper bedrock weathering for the same reasons discussed above for the Eastern Anomaly Zone.

Because the Southern Site Eastern and Western Anomaly zones, and Lineament #2, are over 3000 ft from the BLN site, they do not impact the stability of the Units Nos. 3 and 4 AP1000 basemats. Sound rock is encountered at, or near, the elevation of the AP1000 basemats. In the case where possible localized deeper weathering is encountered, minor overexcavation and fill concrete will be used to provide stable bearing in sound, slight- to fresh bedrock.

This response is PLANT SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-08

FSAR Section 2.5.4.1.3.3 states that "most of the rock containing cavities was ... removed, leaving only isolated cavities at depth. Cavities encountered at the base of the excavation were small and were grouted." If the same procedure will be used in the construction of Units 3 and 4, please state whether such a visual inspection can guarantee that no larger cavities exist below the excavation limit. Provide a detailed description of the grouting program. In addition, please explain if the grouting procedure should be included in the ITAAC or post-COL activities. Are there any special criteria to be used for specification of dental grouting? Please explain the basis for saying "to clean all cavities down to the minimum depth of two times the width," and provide more detail to describe the "wedging effect" mentioned in the FSAR.

BLN RAI ID: 0988

BLN RESPONSE:

A visual inspection of the exposed foundation bearing surface cannot determine the presence or absence of cavities below the exposed surface if there is no surface expression of the cavity. However, the processes for examining the exposed foundation surface are more than a visual look. The foundation review is discussed in FSAR Subsection 2.5.4.12.6 as consisting of thorough examination and observation by appropriately trained and qualified plant personnel with augmentation by soundings, test holes and similar methods. Further discussion with persons familiar with TVA foundation inspection practices (past and present) indicates additional geophysical tools have a place in the foundation examination program for limestone areas. Co-incident use of ground penetrating radar, electrical resistivity and microgravity surveys has been done recently on a dam foundation area.

The response to NRC RAI No. 02.05.04-01, discusses depth, size and shape of cavities that might exist at Bellefonte. As presented in that response, cavities are generally expected to be concentrated above elevation 570 feet. Cavities are also expected generally to have circular, ovaloid, slot-like or irregular cross sections and to be controlled by the network of joints, fractures and bedding planes. The response to NRC RAI No. 02.05.04-01 also discusses size of cavities, concluding they are expected to be small.

The planned base of the nominal foundation excavation for the nuclear island is elevation 588.6 feet; thus the most likely zone of cavities is within the upper 20 feet of rock below the foundation level. The geophysical techniques described above are capable of examining the rock within a zone of that thickness if conducted from the foundation level.

A detailed description of the grouting program is requested in the RAI. Grouting is an activity that will be part of the foundation treatment if indications of open seams, joints or cavities below the level of practical surface treatment methods are indicated. A grouting program is typically specific to the size and type of feature, so the layout of grout injection points, spacing and grouting pressures are adjusted based on what is observed and on what amount of grout volume is accepted in a particular grout injection point compared to adjacent points. The grouting program for Units 1 and 2 is described in BLN Units 1 & 2 FSAR (FSAR Subsection 2.5.7, Reference 201). For Units 3 and 4, a similar grouting program is incorporated into FSAR Subsection 2.5.4.12.3 as shown in the Application Revisions section below.

Dental concreting is intended to fill in cavities, open seams, joints, fractures, depressions resulting from removal of rock fractured by blasting and similar small features. The definition of "small" is a relative term. Typically, features treated with dental concrete are localized, unconnected areas with obvious physical edges. Undulations in rock surfaces due to sloping bedding planes would create areas to be addressed by fill concrete.

Cleaning out fractures and joints down to a depth equal to twice the width is normal engineering practice for rock foundations (Reference 1).

Wedging refers to creating a plug in a sloping or vertical opening such that the opening is getting narrower with depth and the plug is prevented from sliding down by the narrowing of the opening.

There is small potential of the grouting processes (as described in the Application Revisions section below) to impact the safety related structures. As such, the grouting processes are appropriate to be included in the NRC construction inspection portion of the post-COL activities. An ITAAC for this process is not considered necessary, and would be inconsistent with the criteria for an ITAAC in FSAR Section 14.3. As discussed in that section, ITAAC are intended to verify the as-built configuration and performance characteristics of structures, systems, and components, rather than the equipment or process used for installation or construction.

Reference

1. Wyllie, Duncan C., Foundations on Rock, Chapman & Hall, 1992, page 307.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

1. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.5.5, 3rd sentence will be revised from:

Geologic maps of the excavation sides and the bearing surface are prepared to document the subgrade conditions, identify areas needing additional rock removal, placement of dental concrete or grout or installation of rock bolts for slope integrity or prior to placing concrete or a mud mat for subgrade protection.

To read:

Geologic maps of the excavation sides and the bearing surface are prepared to document the subgrade conditions, identify features requiring additional exploration, and identify areas needing additional rock removal, placement of dental concrete or grout or installation of rock bolts for slope integrity or prior to placing concrete or a mud mat for subgrade protection.

2. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.12.3 will be revised from:

Weathered discontinuities which are encountered during excavation of the foundation are cleaned a minimum of two times their width or if the joint widens with depth cleaned downward farther until a wedging effect can be achieved with fill concrete.

The rock properties used for bearing capacity and settlement analyses described in Subsection 2.5.4.10 were conservatively chosen, and include a reduction factor to account for blast damage to the rock during excavation. However, the rock mass properties can be improved by implementing a program of grouting to fill cracks formed, discontinuities widened, or stabilize rock blocks slightly displaced during blasting.

To read:

Weathered discontinuities which are encountered during excavation of the foundation are cleaned down to a depth of a minimum of two times their width or, if the joint widens with depth, cleaned downward farther until a wedging effect can be achieved with fill concrete.

The rock properties used for bearing capacity and settlement analyses described in Subsection 2 .5.4.10 were conservatively chosen, and include a reduction factor to account for blast damage to the rock during excavation. However, the rock mass properties can be improved by implementing a program of grouting to fill cracks formed, discontinuities widened, or stabilize rock blocks slightly displaced during blasting.

A grouting program is used to treat slipped bedding planes, cracks, and joints. Cracks that open to the horizontal surface are blown with air to remove loose material and cement grout is then poured into the crack until the grout level reaches the surface through the full length of the crack. For cracks that open to a vertical surface or steeply sloping cuts, small pipes (¾ to 1½ inch diameter) are installed; one at the lowest portion of the crack and one at the highest portion. The area around the pipes and the remaining portion of the crack are dry packed before concrete placement. Grout is first pumped into the crack at a low pressure (~5 psi) until refusal, and then into the upper pipe at the same low pressure until refusal. Cracks that opened to both the horizontal surface and vertical cuts are grouted with a pressure of ~5 psi until refusal, through pipes installed in the vertical crack as previously mentioned and through pipes installed in the surface cracks after these cracks are blown clean of loose material and covered with concrete. In a few cases, angled holes may be drilled to intercept cracks at a certain depth; for these, pipes are installed and caulked into each hole. The surface exposure of the crack is dry packed and then covered with concrete. These holes are grouted to refusal using a pressure of ~5 psi. For these methods, the grout application uses a water cement mix ratio of between 2:1 and a 1:1 mix.

3. COLA Part 2, FSAR Chapter 2, Subsection 2.5.4.12.6 will be revised from:

Inspection and mapping of the completed excavations is accomplished through observation and examination by appropriately-qualified and trained project inspection personnel. Soundings, test holes, and similar measures are used to augment visual identification of areas needing repairs and to document that appropriate corrective measures have been completed. The quality assurance program in place during design, construction and operations phases is discussed in Section 17.5. Foundation improvement verification work will be conducted under that program. Milestones for implementation are not identified at this time because the construction planning has not yet been developed for this detailed activity.

To read:

Inspection and mapping of the completed excavations is accomplished through observation and examination by appropriately-qualified and trained project inspection personnel. Geophysical techniques such as Ground Penetrating Radar, electrical resistivity, and microgravity surveying are performed on the excavation base to check for indications of larger cavities, consistent with the capability of the techniques, or anomalies that are further explored using test holes or probes. Soundings, test holes, and similar measures are used to augment visual identification of areas needing repairs and to document that appropriate corrective measures have been completed. The quality assurance program in place during design, construction and operations phases is discussed in Section 17.5. Foundation improvement verification work will be conducted under that program. Milestones for implementation are not identified at this time because the construction planning has not yet been developed for this detailed activity.

ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-09

FSAR Section 2.5.4.1.3.3 also indicates that a grouting program was adopted for Units 1 and 2 which includes treatment for cracks developed during blasting. Please provide a detailed description of the blasting program and explain how you plan to ensure that unnecessary fracturing of the in-situ rock will not occur. Please explain why this program was not included as part of the ITAAC.

BLN RAI ID: 0989

BLN RESPONSE:

Blasting to loosen rock for removal is required for the reactor building foundations. No blasting program or plan can, as the RAI requests, ensure that unnecessary fracturing of the in-situ rock will not occur.

Blasting operations are typically performance-based, but do not include measureable criteria or referenced standards against which to judge satisfactory performance. Typical language for such a process includes "Only those excavation methods which will, to the greatest extent possible, minimize damage to the rock foundation shall be used:" and "All blasting before concrete placement shall be carefully done using the smallest practical charges to avoid opening joints, bedding planes or seams, or otherwise disturbing adjacent rock. Blasting to form a vertical face shall be done by pre-split or cushion blasting methods to produce a face without large irregularities." It is also common to include language such as "Procedures for blasting shall be modified as work progresses to minimize damage to rock left in place."

The blasting program does not satisfy the criteria for an ITAAC in FSAR Section 14.3. First, programs (as distinct from as-built conditions) are not an appropriate topic for ITAAC. This is especially true for the blasting program, which is not required by NRC regulations. Second, the as-left condition of the rocks following blasting is not a safety significant parameter. As discussed in the FSAR, if there are fractures in the rocks from blasting or nature conditions, those conditions can be remediated. Finally, because there is not an established criterion to measure when the end result of blasting is acceptable, creating an ITAAC is not feasible.

Regardless of what methods are used to remove the rock above the foundation level, provisions are included in the FSAR Subsections 2.5.4.5.5 and 2.5.4.12 that allow for observation and inspection of the foundation materials and application of judgment towards implementing repair/treatment of areas that may have damage from rock removal operations or may exhibit natural features needing remediation. Detailed geologic mapping is to be done to identify areas for treatment and to document conditions present.

Subsection 2.5.4.12 of the FSAR for Units 1 and 2 (FSAR Section 2.5.7, Reference 201) discusses the condition of the foundations at final grade. The condition is stated as "excellent." It is noted that loose rock, overhangs and rock heavily damage by blasting was removed. The use of rock hammers, bars, shovels and jet washing was reported as suitable to remove loose or splintered rock. The areas were mapped by geologists and photographed before concrete placement. Photographs reviewed by MACTEC and WLA personnel at the Bellefonte site show high quality foundation rock.

It is also noted that the analyses for foundation bearing capacity and settlement included consideration of potential for some surface damage to the rock from blasting. This was done by using a high value of the disturbance factor in the Hoek-Brown calculations of rock strength and deformation parameters as described in FSAR Subsection 2.5.4.2.3.3. These analyses are not intended as a method to allow accepting unsuitable conditions, instead the satisfactory results of the analyses using these reduced

parameter values provide confidence that minor fractures or damages do not create an unsafe foundation condition.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-10

FSAR Section 2.5.4.1.5 indicates that all safety-related facilities will be located on fresh or hard rock, or on fill concrete placed over fresh or hard rock. Please describe the criteria to be used for placement of fill concrete. Specifically, explain if the concrete is to have a shear wave velocity (stiffness and strength) equivalent to that of the hard rock. Is it to be placed in lifts so as not to adversely influence its in-situ velocities?

BLN RAI ID: 0990

BLN RESPONSE:

The term "Fill Concrete" as described in FSAR Subsection 2.5.4.1, applies to a controlled concrete mat that is placed between the prepared bearing surface extended into fresh, hard (sound) rock, and the base of the structural basement of the AP-1000 nuclear island structures. Fill concrete will provide a sound coupling between the basemat and sound rock for cases where excavation is required below the basemat elevation to reach the sound rock surface (below a weathered or dilated bedrock surface), or to remove possible blast-damaged or disturbed rock. The fill concrete will infill possible irregularities in the prepared sound rock surface, and form a stable and level surface for construction of the structural basemat. FSAR Figures 2.5-348a and 2.5-348b illustrate the expected application and lateral extent of fill concrete (labeled as "Leveling Concrete" in the figures). Prior to placement of the fill concrete, localized and limited dental concrete, as described in FSAR Subsection 2.5.4.12.3, will be placed, as required, to fill in any possible small rock voids or weathered discontinuities.

The specific criteria for the placement of fill concrete are developed during the final design phase for the AP1000 plant. For the BLN COLA, a description of fill concrete placement is presented in a generic sense (e.g., general location and geometry of fill concrete) in FSAR Figures 2.5-348a and 2.5-348b. This information is used to ensure consistency between the final fill concrete placement and assumptions incorporated in the COLA analyses. There are standard practice measures for placement and mix of fill concrete based on past and current applications for large structures and nuclear safety-class foundations bearing on rock, and generic requirements presented in the AP-1000 DCD. For example, fill concrete mix designs are in accordance with ACI 318-02 (DCD Chapter 2 Reference 1). Standard nuclear practice incorporates field observation to verify that the approved mixes are used, and to field test specimens that are used to verify required compressive strengths. Laboratory testing of fill concrete specimens ("test cylinders"), and/or field tests (e.g., SASW testing), are also performed to verify that the average design shear wave velocity of is obtained for the placed fill concrete. Placement methods include placement of fill concrete in layers of controlled thickness and slump, and lower layers are hardened by curing before the succeeding layers are placed.

There is no requirement for fill concrete to exhibit an "equivalent" Vs as the underlying hard rock. During development of the COL, discussions with the Technical Advisory Group members indicated that fill concrete should exhibit an installed shear wave velocity (Vs) in the range of 6,000 to 7,000 fps. This criterion provides little, if any, velocity impedance contrast of significance with the underlying bedrock. In fact, the field-measured Vs of the "Shaly or Argillaceous" middle Stone River Group units A and C generally are consistent with this velocity range, and have been specifically accounted for in the COLA dynamic profiles (e.g., FSAR Figures 2.5-353 through 2.5-356). Therefore, the fill concrete Vs range will be similar to the average Vs of the lower-velocity bedrock units underlying the Units Nos. 3 and 4 basemats. No specification has been developed for fill concrete Vs to reach the velocity of the harder middle Stones River Group units B, D, E, or F that exhibit average Vs in the range of about 10,000 fps. It

is not necessary, or reasonably achievable, for fill concrete to match the velocity of the harder (higher velocity) rock units. As discussed above, standard fill concrete placement methods used by the nuclear industry include placement in controlled and tested layers, and result in a relatively uniform concrete section that does not adversely impact the final Vs or form layers with potentially significant lower Vs.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Section 2.5.4.1.5 will be revised from:

The BLN investigation did not encounter adverse geologic conditions in the Units 3 and 4 safety-related foundation explorations that pose a stability or safety hazard. Major safety-related structures are founded on fresh, hard bedrock, or on fill concrete placed over fresh, hard bedrock.

To read:

The BLN investigation did not encounter adverse geologic conditions in the Units 3 and 4 safety-related foundation explorations that pose a stability or safety hazard. Major safety-related structures are founded on fresh, hard bedrock, or on fill concrete placed over fresh, hard bedrock. This fill concrete fills in irregularities or depressions in the rock to provide a level surface and uniform interface for the structural basemat foundation. The mix design and placement criteria will follow ACI 318-02 and standard industry practice to provide a uniform concrete section that exhibits in-place shear wave velocities consistent with the underlying bedrock. Standard nuclear practice incorporates field observation to verify that the approved mixes are used, and to field test specimens that are used to verify required compressive strengths.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-11

FSAR Section 2.5.4.5.2.1 indicates that analysis shows that factors of safety for the excavation slope are greater than 2.0 at a 1.5:1 inclination. Please describe the slope analysis method and provide details of the slope safety factor evaluation.

BLN RAI ID: 0991

BLN RESPONSE:

The slope stability analysis was performed using the Method of Slices – Simplified Bishop Method for circular slip surfaces, which is a limit equilibrium method where the shear stresses on an assumed failure plane are compared to the shear strength of the material (Reference 1). The Bishop Method is one of several methods in common use that model the slope cross-section using a series of vertical slices. The differences in the methods are mainly in the type of simplifying assumptions made in the analysis model regarding the shape of the failure surface and the forces acting on the sides of the vertical slices. In most cases, the more accurately modeled slice side forces yield higher factors of safety. The slope geometry and soil parameters are the same in all methods.

The Simplified Bishop method approximates the inter-slice forces by assuming that the sum of the vertical inter-slice forces on opposite sides of a slice is zero. It is estimated that the error for this assumption is about 1%, whereas the error in neglecting the horizontal and vertical inter-slice forces, as is done in the Ordinary Method of Slices, is about 15% (Reference 1). The Simplified Bishop method is accepted and commonly used because of the recognized greater accuracy.

Because the slope is only present during construction and is covered when the reactor excavation is backfilled, a total stress analysis was performed.

The soil parameters from the site characterization are presented in Tables 2.5-233 and 2.5.4-234 in the FSAR. For the total stress analysis, the total stress parameters from triaxial testing or the unconfined compressive strength test are used. Tests on site soils were also conducted during the site studies for Units 1 and 2 FSAR (FSAR Subsection 2.5.7, Reference 201). For purposes of the slope stability analysis, data from this previous work were reviewed as well. The cohesion parameters from the Units 1 and 2 FSAR tests showed a range of 260 to 3520 pounds per square foot (psf) with an average of 1496 psf. Considering all the data reviewed, a conservative value for cohesion of 500 psf was selected for use in the total stress analysis. A friction angle of 1.6°, the lower value for tests on FSAR Table 2.5-233, was selected.

The maximum groundwater levels measured from monitoring wells were Elevation 605 feet for Unit 3 and Elevation 615 feet for Unit 4. These values are contained in Tables 2.5-243 and 2.5-244 in the FSAR. A water level of Elevation 618 feet, which represents the highest water level reported in Table 2.4.12-204, was used to reflect possible saturation of most of the slope for the slope stability analyses.

The height of the soil slope varies depending on site topography and depth to top of weathered rock. From a general review of the topographic contours shown on FSAR Figure 2.5-347 and the boring records contained in FSAR Chapter 2, Appendix 2AA for borings near the edges of the nuclear islands, a slope height of 17 feet was determined as a reasonable maximum for analysis. Using the soil parameters and water levels discussed above, stability analyses were performed for failure circle radii of 25 and 35 feet following the procedures described in NAVFAC DM-7 (Reference 2). The 35-foot radius represents a failure surface which begins behind the top of the slope and exits at the toe of the slope. The 25-foot radius represents a failure surface which begins on the slope face near the top of the slope and exits at the

toe. For the conservative total stress analyses, the computed Factors of Safety were 3.0 for the 25-foot radius and 2.1 for the 35-foot radius.

References

1. Fang, H-Y. "Stability of Earth Slopes," in Foundation Engineering Handbook, Winterkorn, H.F. and Fang, H-Y., Editors, Van Nostrand Reinhold, New York, 1975.
2. Department of the Navy, Naval Facilities Engineering Command, *Soil Mechanics Design Manual 7.1,* Chapter 7, 1982.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

1. COLA Part 2, FSAR. Chapter 2, Section 2.5.4.5.2.1 (first paragraph, last two sentences) will be revised from:

...The soil excavation is sloped at a 1.5 (Horizontal): 1 (Vertical) inclination, as illustrated in Figures 2.5-348a and 2.5-348b, so lateral support is not required. Analyses of the slope temporary soil slopes show factors of safety greater than 2.0.

To read:

...The soil excavation is sloped at a 1.5 (Horizontal): 1 (Vertical) inclination, as illustrated in Figures 2.5-348a and 2.5-348b, so lateral support is not required.

A slope stability analysis was performed for the 1.5:1 temporary slope illustrated on the referenced figures. The analysis was performed using the Method of Slices – Simplified Bishop Method (Circular Slip Surface), as described in NAVFAC Design Manual 7.1 (Reference 2.5-476). This method is a Limit Equilibrium method, where the shear stresses induced on an assumed failure plane are compared to the shear strength of the material.

A total stress analysis was conducted using conservatively-selected values of cohesion and angle of internal friction of 500 psf and 1.6°, respectively.

Maximum groundwater levels in monitoring wells in the soil in the area of Units 3 and 4, as shown on Table 2.5-243, were Elevation 605 feet and Elevation 615 feet, respectively. For the conservative total stress analysis, a water level at Elevation 618, the highest water level reported in Table 2.4.12-204, was used to reflect possible saturation of most of the slope.

The results of the conservative analyses indicate a minimum Factor of Safety for the 1.5:1 temporary cut slope of 2.1.

2. COLA Part 2, FSAR. Chapter 2, Section 2.5.7 will be revised to add new reference 476:

476. Department of the Navy, Naval Facilities Engineering Command, *Soil Mechanics Design Manua 7.1,* Chapter 7, 1982.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-12

FSAR Section 2.5.4.5.2.1 states that "kinematic analyses ... and an average assumed interface friction value of 35°, bedding plane failure is not a viable failure mode." Please describe your rationale for assuming a 35° interface friction value and explain why bedding failure is not a viable failure mode.

BLN RAI ID: 0992

BLN RESPONSE:

The selected friction angle of 35 degrees used in the kinematic analysis represents a conservatively-skewed average value derived from a data set of literature-reported test results for limestone (e.g. "limestone: Patton, 1966 (Reference 1); "Indiana Limestone" Mauer, 1965(Reference 2) ; and "Indian Limestone" and "Wolfcamp Limestone"; Goodman, 1989 (Reference 3)). Table 01 lists the literature-reported values that range between 33 and 54.5 degrees. These values include tests performed on a combination of wet and dry, and saw-cut and natural, rock core samples.

Table 01 - Determination of Friction Angle

Direct shear test data for Limestone rocks			Data compiled from Table 2.2 in Coulson (1970) & Table 3.3 in Goodman (1989)(Reference 4)						
Rock type	Surface	Moisture	Normal Pressure (psi)	Stage of Friction	Coefficient of Friction range		Phi Range		Source
Limestone	Sawed	Dry	4 to 75	initial	0.67	0.84	33.8	40	Patton (1966)
Limestone	Sawed	Dry	4 to 75	residual	0.65	0.81	33	39	Patton (1966)
Limestone	Sawed	Wet	4 to 75	initial	0.69	0.71	34.6	35.4	Patton (1966)
Limestone	Sawed	Wet	4 to 75	residual	0.64	0.73	33.5	37.6	Patton (1966)
Indiana Limestone	Fracture	Dry	1,000 to 4,000	residual	1.4	0.7	54.5	35	Mauer (1965)
Indiana Limestone	-	-	-	-	-	N/A	42	42	Goodman (1989)
Wolfcamp Limestone	-	-	-	-	-	N/A	34.8	34.8	Goodman (1989)
								35	Average Used in Calculation

The wet, saw-cut values reported by Patton (1966) tend to yield a conservative "base" friction angle value (Goodman, 1989), as the saw cut surfaces are relatively smooth and do not include typical surface roughness and asperities that tend to resist sliding and increase the frictional resistance. Natural bedding planes and joint surfaces (and mechanical breaks along those surfaces in exploratory rock core), are typically slightly rough to rough to the touch, and are wavy with low amplitude asperities. Surface

roughness was evaluated in recovered core from borings and noted by the BLN COLA rig geologists (FSAR Appendix 2BB). These observations indicate that joint surfaces obtained from middle Stones River group limestone in the BLN Units 3 and 4 foundation excavation elevations generally is between planar and very rough, and therefore contribute a significant increase in friction angle over laboratory saw-cut surfaces. Therefore, the use of the literature-reported basal friction values that include saw cut samples are believed to be conservative. Additionally, the rock mass observed in the historical construction photographs for Units 1 and 2 appeared to be free of wet seeps or zones. The incorporation of wet saw cut surface specimen results from the literature-reported values conservatively accounts for the possibility for encountering unexpected seepage zones during excavations for Units 3 and 4, and for saturation of the rock mass by major storm events.

The COLA exploration program, and review of Bellefonte Units 1 and 2 construction photographs, indicate that bedding plane surfaces are the primary rock mass discontinuity. Bedding strike and dip is quite uniform across the site, and strikes to the north, and dips eastward between about 15 and 17 degrees. In many cases, bedding planes recovered in rock core are bonded or tight. The Bellefonte Units Nos. 1 and 2 historical foundation construction photographs show bedding planes were stable in cuts made at all orientations in the foundation excavations. Bedding planes in the construction photographs appeared to be tight and stable, and no failure scars, partial failures, or bedding plane reinforcement (e.g. rock bolts) were observed in the photographs. Rather, the cuts appeared to be universally stable without support.

The low dip angle of bedding will be reflected in the daylighted rock exposures in the steep excavations that are made for the Units Nos. 3 and 4 plant foundations. The uniform eastward dip of bedding indicates that bedding planes will dip out of excavation cuts (potentially unstable geometry) along the west margins of the Units Nos. 3 and 4 foundation excavations, but will dip into, or transverse to, the excavation cuts (stable geometries) along the other excavation walls. The kinematic analyses demonstrates that the bedding dip inclinations of between 15 and 17 degrees are much lower than the assumed basal friction angle of 35 degrees, and therefore sliding along bedding planes that daylight out of the western excavation cuts is not a viable failure mode. The low angle of bedding inclination presents a stable condition resisting bedding plane sliding even if lower basal friction angles than assumed for the rock mass are considered. Therefore, the assignment of the 35 degree basal friction angle for bedding plane surfaces is not especially critical for the conclusion that bedding plane sliding is not a viable failure mode; a high factor of safety exists against bedding plane sliding even if somewhat lower basal friction angles are considered.

References

1. Goodman, R.E., 1989, Introduction to rock mechanics, 2nd Ed. John Wiley & Sons: New York, p.562.
2. Mauer, W. C., 1965, Shear failure of rock under compression. Journal of the Society of Petroleum Engineers, V. 5, No. 2, p. 167.
3. Patton, F. D., 1966, Multiple modes of shear failure in rock and related materials [Ph.D. Thesis]: University of Illinois, Department of Geology.
4. Coulson, J.H., 1970, The effects of surface roughness on the shear strength of joints in rocks [Ph.D. Thesis]: Urbana-Champaign, University of Illinois, p 312.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Section 2.5.4.5.2.1, last paragraph, will be revised from:

The rock is excavated unsupported, at an approximate 85° inclination from horizontal. Kinematic analyses using properties from the Hoek-Brown evaluations discussed in Subsection 2.5.4.2.3.3 and an average assumed interface friction value of 35°, bedding plane failure is not a viable failure mode. Movement of individual rock blocks is kinematically possible, but the number of frequency of such potential failures is believed to be low and could be addressed by localized excavation support, block removal or flattening the cut slope, all typical procedures for rock excavations. Based on the performance of the rock cut slopes during construction of Bellefonte Units 1 and 2, and current analysis discussed in Subsection 2.5.5, the slopes can perform satisfactorily at this inclination.

To read:

The foundation excavations are constructed at an approximate 85° inclination from horizontal and are generally unsupported. Kinematic analyses performed to evaluate the stability of proposed unsupported rock cuts incorporated data from borings in the excavation areas, and evaluation of the conditions and performance of similar rock excavations made in the same middle Stones Group units for the BLN Units 1 and 2. An average assumed interface friction value of 35°, based on careful review of rock core samples and typical literature-reported values for similar rock, was used in the kinematic analysis for the bedding plane surfaces which are by far the dominant rock mass structural feature. Bedding plane surfaces dip between about 15 and 17 degrees, and potentially daylight in excavation walls along the west margin of the construction area. The assumed interface friction value for bedding plane (and other minor, secondary joints) represents an average value of reported residual tests of limestone performed on wet, saw-cut rock samples. Kinematic analysis demonstrates that bedding plane failure is not a viable failure mode, because the gentle bedding dip inclination is much lower than the frictional resistance along the bedding plane surfaces (e.g. bedding dip of 15 to 17 degrees versus rock mass estimated friction angle of 35 degrees), effectively resisting the potential for sliding along these surfaces. Movement of small individual rock blocks loosened by excavation/blasting is possible, but the number, size, and frequency for such potential failures are low and addressed by localized excavation support ("spot bolts"), block removal, or flattening the cut slope. These measures are typical procedures for rock excavations. Based on the performance of the rock cut slopes during construction of Bellefonte Units 1 and 2, and current analysis discussed in Subsection 2.5.5, the slopes can perform satisfactorily in unsupported, 85 degree cut slopes in the fresh rock.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-13

FSAR Section 2.5.4.5.4 states that the space between the edge of the concrete basemat for the nuclear islands and the rock excavation will be filled in with backfill material consisting of lean concrete. Please specify the strength of the concrete to be used for lean concrete mixtures.

BLN RAI ID: 0993

BLN RESPONSE:

In FSAR Subsection 2.5.4.10.1, the required strength for lean, non-structural concrete is given as 17.4 MPa (2,500 pounds per square inch), a value that is provided in FSAR Subsection 2.5.7, Reference 458. The same requirement is to be used for the backfill between the rock excavation and the concrete basemat for the nuclear islands. The FSAR text will be modified as shown below in a future revision of the BLN COLA.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Section 2.5.4.5.4, second paragraph will be revised from:

In the space between the edge of the concrete basemat for the nuclear islands and the rock excavation, backfill material consists of lean concrete. In the space between the foundation walls and the soil excavation, the material to be used as backfill consists of Class I soils or soils with lower percentage of fines and lower plasticity. The geotechnical properties of Class I soils were discussed in Subsection 2.5.4.5.3.2.

To read:

In the space between the edge of the concrete basemat for the nuclear islands and the rock excavation, backfill material consists of lean, nonstructural concrete. The concrete has a specified compressive strength of 17.4 MPa (2,500 pounds per square inch) (Reference 458). In the space between the foundation walls and the soil excavation, the material to be used as backfill consists of Class I soils or soils with lower percentage of fines and lower plasticity. The geotechnical properties of Class I soils were discussed in Subsection 2.5.4.5.3.2.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-14

FSAR Section 2.5.4.5.4 states that backfill soil will be placed adjacent to the exterior walls of the nuclear islands. Please provide an evaluation of compaction-induced additional loads acting on the exterior walls of nuclear islands. Please explain why this is not included as part of the ITAAC to confirm that the in-situ properties of the backfill material are acceptable after compaction.

BLN RAI ID: 0994

BLN RESPONSE:

Compaction of backfill in small, confined areas, such as between the nuclear island foundation walls and soil/rock cuts, is referenced in two locations in the FSAR. In FSAR Subsection 2.5.4.5.4.1.1, the suitability of smaller pieces of compaction equipment in confined areas "due to maneuverability and the lower pressures that would be imparted on the adjacent wall" is discussed. In FSAR Subsection 2.5.4.10.5, the 4th bullet item states that the lateral earth pressures were developed based on use of light, hand-guided compaction equipment to compact the soil within 5 ft. of the Nuclear Island walls to avoid compaction-induced soil stresses against the wall.

As requested in the RAI, compaction-induced loads acting on the exterior walls of the nuclear islands were evaluated. The added loads caused by compaction are a function of the weight of the compactor, the type (static or vibratory), the distance it is operated from the wall and the soil types. A method for evaluating the lateral earth pressures on a wall due to the earth and the compaction equipment is given in Figure 13 of NAVFAC DM 7.2 (Reference 1). A copy of the referenced figure is included in Attachment 02.05.04-14A. Another method for calculating the lateral earth pressures to incorporate stresses from compaction is presented in a paper by Duncan et al. (Reference 2). A comparison of both methods found that the NAVFAC method yields slightly higher pressures for the same distance from the wall, so the NAVFAC method was used for the evaluation.

Compactors typically used near walls include:

- Self-propelled single-drum vibratory rollers;
- Walk-behind vibratory rollers;
- Self-propelled double-drum vibratory rollers;
- Pneumatic-tired rollers; and
- Vibratory plate compactors.

FSAR Subsection 2.5.4.5.4 discusses the backfill material for use against the nuclear island basement walls and recommends a granular type of material. For this type of material, the appropriate type of compaction equipment is the self-propelled single-drum vibratory roller. Therefore, only this type of compaction equipment was considered in the evaluation.

A summary of typical self-propelled drum compactors is contained in Reference 2. Review of the listed compactors shows the one producing the greatest total force is a Dynapac Model CA30 with a total force of 70.6 kips, a drum width of 84 inches and a wheelbase of 113 inches. Using the NAVFAC methodology, stresses for this compactor operating at distances from the wall varying from 1 foot to 5 feet were compared to stresses calculated at the same depth using the static at-rest earth pressure calculated from FSAR Figure 2.5-360, assuming no water table is present. The comparison showed that keeping the compactor at least five feet away from the wall, as is stated in FSAR Subsection 2.5.4.10.5,

will keep stresses on the wall due to the compaction equipment at a negligible level. Therefore, no adjustment to the recommended lateral earth pressure for static, at-rest conditions is needed. FSAR Figure 2.5-360 does not need revision in response to this RAI.

The requirements for compaction control are designed so that the compacted soil as controlled by its percentage compaction and moisture content achieves the engineering properties that are attributed to the compacted fill in the FSAR. With the properties of the compacted fill met, the earth pressure recommendations given in the FSAR are also valid. The properties of the compacted fill are not dependent on the type or method of compaction used to meet the requirements. Adding an ITAAC for types of compaction equipment is not necessary and would be inconsistent with the criteria for an ITAAC in FSAR Section 14.3. As discussed in that section, ITAAC are intended to verify the as-built configuration and performance characteristics of structures, systems, and components, rather than the equipment or process used for installation or construction.

References

1. Department of the Navy, Naval Facilities Engineering Command, *Soil Mechanics Design Manual 7.2,* Chapter 1, 1982.
2. Duncan, J.M., G.W. Williams, A.L. Sehn, and R.B. Seed, *Estimation Earth Pressures Due to Compaction,* Journal of Geotechnical Engineering, Vol. 117, No. 12, 1991.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-14A

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-15

FSAR Section 2.5.4.6.3 describes the issues related to previous construction dewatering, related to Units 1 and 2. Please provide a detailed dewatering plan for the COLA site. Please explain why the water level reduction effort is slight and provide evidence that the reduction in the perched water will have a minimum impact on the settlement of the adjacent ground during the construction and post-construction periods.

BLN RAI ID: 0995

BLN RESPONSE:

Dewatering Plan

The groundwater system is described in FSAR Subsection 2.4.12. Groundwater at the BLN site occurs in bedrock openings along fractures and bedding planes and in pore spaces in the material above the bedrock. The bedrock is a poor water-bearing material. The zone of material above the bedrock, consisting of gravel, boulders and weathered shale in a silty clay residuum matrix has been designated as the epikarst aquifer. FSAR Subsection 2.4.12.1.2 characterizes the groundwater at the site as flowing through the soil overburden and the epikarst between the soil and bedrock. The groundwater flow is subject to three-dimensional controls by horizontal, vertical and inclined fractures, joints and bedding planes resulting in a non-uniform occurrence across the site and an inconsistent presence in monitoring wells. Groundwater in the epikarst may not be present during all seasons.

Because of the groundwater system as described in FSAR Subsection 2.4.12.1, inflows of water into an excavation are not predictable as to location, rate or quantity. Locally, areas of epikarst that are composed of more gravel-size material will serve as reservoirs of water that will quickly drain into an excavation for a short time, until the stored water is depleted. Because the overall epikarst is clayey in nature, the rate of water recharge into a more gravelly zone may be very slow. So, once the initial stored water is drained, future flow will be controlled by the low permeability surrounding materials. Below the bedrock level, rock cores had high values of Rock Quality Designation (RQD) which is a measure of presence of separated joints and fractures in the ground. Attempts to conduct in-situ water inflow testing through pressure tests generally resulted in minimal or no water intake.

The soil and rock conditions at the Units 3 and 4 sites are similar to those at Units 1 and 2. The same geologic formations are present. It is beneficial to use the observations from Units 1 and 2 excavations as reported in the Units 1 and 2 FSAR Subsections 2.5.4.5.2 and 2.5.4.6 (FSAR Subsection 2.5.7, Reference 201) as a guide to the water conditions that may be expected for Units 3 and 4. As reported in these referenced sections, groundwater inflow into the excavation was very slow, and dewatering of the powerhouse excavation was accomplished using sumps at low points in the excavation. It was also noted that early excavations encountered groundwater, but that as the excavation progressed, the general groundwater level lowered at about the same rate as the excavation due to the pumping from sumps.

Because the soils and rock have low capability for water flow, dewatering by wells or control of inflow by cutoff walls is not needed. Engineering experience both at the Units 1 and 2 excavations and at many other excavations in rock at other construction sites shows that water inflows may be controlled using collector ditching, local sumps and pumping.

The basic elements of the dewatering plan include:

1. For the upper portion of the excavations that are in soil, provide a perimeter drainage ditch at the base of the soil slope, on the bench that will be formed at the top of rock as shown on the attached Figure 02.05.04-15A-1
2. At points to be determined based on construction sequence, provide sump pits to remove water from the perimeter ditch using pumps. Direct pumped water to a release point outside the construction area such that it will not flow back into the excavation.
3. For excavations below the bedrock level, use local ditching and sump pits as needed to intercept seepage from exposed bedrock face joints, fractures and bedding planes. Maintain the excavation so the perimeter is slightly lower than the center to promote collection of rock face seepage for removal. Remove water at sump pits using pumps. Direct pumped water to a release point outside the construction area such that it will not flow back into the excavation.

It is not feasible at this time to set out a detailed plan showing locations for specific sumps or ditches prior to the excavation work because where they will be needed will not be known until work is underway and the locations of water-bearing joints or fractures are identified. The sequence of the excavation will also be a factor in placing water control features, and the sequence is not known at this level of planning.

Impacts of Dewatering

As described in FSAR Subsection 2.4.12, the soils and rock have poor water-bearing characteristics. The experience with dewatering during the Units 1 and 2 construction is applicable to the Units 3 and 4 excavations because the rock and soil conditions at Units 3 and 4 are essentially the same as at Units 1 and 2. These reasons formed the basis for the statement that water level reduction efforts are slight. The term "slight" is intended to mean no construction of slurry cutoff walls is needed, nor is any installation of continuously pumped deep wells needed.

With regard to the request to show that reduction of water levels in the perched zone will have minimal effect on settlement of the adjacent ground during construction and post-construction. It is noted that in the response made to NRC RAI 02.04.12-6b, the term "perched water" was removed and the use of the term "epikarst aquifer" was substituted.

The presence of water in the epikarst aquifer is subject to seasonal fluctuations. Monitoring well and boring records summarized on FSAR Tables 2.5-241, 2.5-242 and 2.5-243 show water levels are below to slightly above the rock level. FSAR Table 2.5-243 shows the difference between the highest water level and the top of bedrock level and between the lowest water level and the top of bedrock level. This difference varies from 2.7 feet below to 10.7 feet above the top of bedrock. In the wells themselves, the seasonal fluctuation varies widely from less than a foot to about 6 feet. Soils in the epikarst aquifer have thus been subjected to changing stresses due to the natural fluctuations, and have already experienced settlement due to such fluctuations. Indeed, occurrence of drought conditions over past years would most likely have reduced the epikarst aquifer to a dry condition for some period of time.

Also, the soils in the epikarst zone are overconsolidated (FSAR Subsection 2.5.4.2.2.4). Even if water levels in the epikarst were to fall to new lows, there is only a small thickness of soil that would be subjected to increased stresses. As a bounding condition, the increased stresses from a water level change of 10.7 feet, the maximum difference between highest water level and the top of bedrock shown on FSAR Table 2.5-243, was used with the highest recompression ratio from laboratory consolidation tests as shown on FSAR Table 2.5-235 to calculate the settlement. The result is about ¼ inch, an amount that is not considered significant with respect to ground levels adjacent to the excavation.

After construction is completed, no permanent dewatering is planned. Thus, water levels in the ground will return to levels controlled by rainfall and infiltration.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR Chapter 2, Figures 2.5-348a and 2.5-348b will be revised as part of the response to NRC RAI No. 02.05.04-06.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-15A

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-16

Please include in FSAR Section 2.5.4.7.3 a reference to the resonant column torsional shear (RCTS) results (FSAR Table 2.5-245).

BLN RAI ID: 0996

BLN RESPONSE:

The text of FSAR Subsection 2.5.4.7.3 alludes to the RCTS results but does not reference Tables 2.5-245 and 2.5-246. This was an oversight and the table references will be added in a future revision of the BLN COLA.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Subsection 2.5.4.7.3 will be revised from:

The following laboratory testing technique was used to measure dynamic soil properties:

- Resonant Column/ Torsional Shear (RCTS) testing of shear modulus and damping of six undisturbed samples of native residual soil.

To read:

The following laboratory testing technique was used to measure dynamic soil properties:

- Resonant Column/ Torsional Shear (RCTS) testing of shear modulus and damping of six undisturbed samples of native residual soil.

The results of these tests are provided in Tables 2.5-245 and 2.5-246

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-17

FSAR Section 2.5.4.8 states that the clayey and stiff nature of the native residual soil and fill exhibit a low susceptibility to liquefaction. You performed a liquefaction screening assessment in conformance with Regulatory Guide 1.198 to demonstrate the low liquefaction hazard associated with the residual soils and fill beneath the Units 3 and 4 power blocks. RG 1.198 Section C indicates that the initial phase of a site characterization program includes borings with SPT or CPT tests to determine the penetration resistance and soil characteristics for measuring classification and water content. RG 1.198 Section 1.2 provides guidance in adjustment of N values for evaluation of the liquefaction potential. Please explain the reason for not using the SPT data obtained from the soil investigations to perform the liquefaction potential assessment at the Units 3 and 4 site. In addition, provide the adjusted N data from the SPT test for the Bellefonte Nuclear (BLN) site to justify that the liquefaction threshold criteria is satisfied from the BLN site data, including the Category I structures and other non safety related construction sites. Please describe if the shear wave velocity was used as a threshold to evaluate the liquefaction potential at the BLN site.

BLN RAI ID: 0997

BLN RESPONSE:

Standard Penetration Test (SPT) data was collected in geotechnical borings during the soils investigation in conformance with Regulatory Guide 1.198. The use of SPT data is applicable in granular soils with generally low percentage of fines (material passing the No. 200 sieve). Empirical correlation methods using the SPT-based approach of Youd et al (Reference 1) are specifically restricted to non-cohesive granular soils, and no industry-accepted SPT data base has been developed for cohesive soils, such as which comprise the soil materials at the Bellefonte site. The majority of soil overlying bedrock at the site is not granular, but is fine-grained residual clay and silt formed by in-place weathering processes acting on rocks similar to the underlying limestone bedrock. Assessment of liquefaction potential of these fine-grained soils is done in practice using an alternative screening analysis as allowed under the guidance of Regulatory Guide 1.198. Two analysis approaches were used. The first was based on Youd (Reference 2) and Youd and Perkins (Reference 3), and takes into account geologic conditions such as age and the evidence of paleoliquefaction (described in FSAR Subsection 2.5.4.8.2). The second analysis was a screening textural analysis based on Seed (Reference 4) that evaluates the fine-grained nature of the soil, as a measure of the percentage of fines and the Atterberg limits (described in FSAR Subsections 2.5.4.8.3). The results of both analyses form the basis for the conclusion that there is a low susceptibility or potential for liquefaction of onsite or fill soils.

As discussed above, using SPT data to evaluate the liquefaction threshold criteria is not appropriate for the fine-grained soils at the site. Two independent screening analyses were performed that reached the same conclusion – very low potential susceptibility or potential for liquefaction. Therefore providing the adjusted N-value data from the SPT tests is not necessary.

Shear wave velocity, as measured from seismic CPT's and P-S Suspension logging surveys, was not used as a threshold to evaluate the liquefaction potential at the BLN site. Similar to the SPT-based methods, empirical shear wave-based liquefaction analyses methods are restricted to non-cohesive granular soils, and are not applicable to cohesive soils. Therefore, it is not valid to use shear wave velocity to assess the liquefaction susceptibility for the Bellefonte soils.

References

1. Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H. (2001). *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(10), 817-833.
2. Youd, T.L., *Mapping of Earthquake-Induced Liquefaction for Sesimic Zonation*, Fourth International Conference on Seismic Zonation, 1991.
3. Youd, T.L., and D.M. Perkins, *Mapping Liquefaction-Induced Ground Failure Potential*, Journal of the Geotechnical Engineering Division, April 1978.
4. Seed, R. B., K.O. Cetin, R.E.S. Moss, A. M. Kammerer, J. Wu, J. M. Pestana, M. Riemer, R.B. Sancio, J.D. Bray, R.E. Kayen, and A. Faris, *Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework*, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, 2003.

This response is PLANT-SPECIFIC

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-18

FSAR Section 2.5.4.10.1 states that the bearing capacity was evaluated for each Unit using two independent methods. Method 1 uses the ultimate bearing capacity of the Terzaghi approach based on the strength of the rock mass. Due to the finite dimension of the Bellefonte Nuclear (BLN) island designs, the Terzaghi equation originally developed for infinite base needs to be modified to incorporate the correction factor for parameters Nc and Nγ to take into account the footing finite geometry configuration, such as rectangular or circular, etc. Furthermore, due to the non-symmetrical configuration of the footing of the nuclear island designs, the consequences of the eccentric loading need to be considered during the bearing capacity evaluation. Please explain whether the geometric correction factors were incorporated into your use of the Method 1 approach for the bearing capacity evaluation. If not, please update the bearing capacity values for the Method 1 approach with the correction factors. Please explain whether the effect of the eccentricity of the loading applied to the footing was considered for the bearing capacity investigation. If not, please update the bearing capacity analysis with the eccentric loading consideration.

BLN RAI ID: 0998

BLN RESPONSE:

Geotechnical engineering experience has been that settlement rather than bearing capacity is the controlling factor with regard to performance of foundations on rock. The initial bearing capacity calculations reported in the FSAR were based on an equivalent area mat with dimensions of 127 by 256 feet to represent the reactor mat. The resulting allowable bearing capacity far exceeded that required in DCD Table 2-1, and further refinement was not done. In response to this RAI, we have checked the initial calculations by updating the ultimate bearing capacity values calculated using the Terzaghi approach (FSAR Section 2.5.7, Reference 456). The Terzaghi equation is based on length to width (L/B) ratios greater than 10. For L/B ratios less than 10, shape correction factors are applied to the corresponding bearing capacity factors. Correction factors are provided in Table 6-1 of EM 1110-1-2908 (FSAR Section 2.5.7, Reference 456; copy of table attached as Attachment 02.05.04-18A) Because the value of cohesion for the rock was taken as 0, only the correction factor for the Nγ term was used in the current calculations. For the equivalent area mat dimensions, the L/B is 2 which has a corresponding correction factor of 0.9.

Applying the shape correction factor for Nγ results in a static ultimate bearing pressure of 692,000 pounds per square foot (psf), resulting in a factor of safety of 80 with respect to the DCD Table 2-1 required value of 8,600 psf.

The eccentric loading referred to in the RAI was for the condition where the reactor mat is underlain by both argillaceous limestone and micritic limestone. In making the bearing capacity calculations originally, the lower bound characteristics of the argillaceous limestone, the relatively weaker rock unit, as shown in FSAR Table 2.5-236, were used for conservatism. Thus, the values presented in the FSAR provide an assessment for the lowest strength conditions found in the characterization.

The indicated FSAR changes will be made in a future revision of the BLN COLA.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

COLA Part 2, FSAR. Chapter 2, Section 2.5.4.10.1; first paragraph, first and second bullet will be revised from:

Using the lower bound rock properties, both methods show bearing capacities well above the requirements in DCD Table 2-1 (8600 pounds per square foot [psf] for static and 35000 psf for dynamic). The calculated bearing capacities under both static and dynamic conditions are:

- Method 1; 251,000 psf, and
- Method 2; 236,000 psf.

To read:

Using the lower bound rock properties for argillaceous limestone as shown in Table 2.5-236, both methods show bearing capacities well above the requirements in DCD Table 2-1 (8600 pounds per square foot [psf] for static and 35000 psf for dynamic). The calculated ultimate bearing capacities for Method 1 and allowable for Method 2 are:

- Method 1; 692,000 psf , and
- Method 2; 236,000 psf. This method provides an allowable bearing pressure based on rock properties only, not the methods by which loading is applied. It is therefore applicable to both static and dynamic loading.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-18A

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-19

FSAR Section 2.5.4.10.4.1 states that estimates of post-construction settlement were calculated separately for Units 3 and 4 based on the theory of elasticity. Settlements were estimated by three methods. The maximum estimated settlement is 0.18 inches beneath Unit 3 and 0.2 inches beneath Unit 4. Please provide more details on the settlement calculations from each method, i.e. methodology, assumptions, use of settlement-time curve, etc... Please provide a comparison of the results obtained from each method.

BLN RAI ID: 0999

BLN RESPONSE:

Settlements were estimated by three methods with the results shown in parentheses;

- use of the Boussinesq Equation (Unit 3 0.10 in; Unit 4 0.10 in)
- use of the Corps of Engineers Equation (Unit 3 0.18 in; Unit 4 0.20 in)
- use of the Steinbrenner Equation (Unit 3 0.11 in; Unit 4 0.12 in)

A more detailed description of each of the methods used is presented in the following paragraphs.

Certain assumptions are common to the three methods.

1. The structures evaluated have foundations bearing directly on rock or bearing on a depth of fill concrete in turn resting on rock. Settlements are the result of elastic compression of the rock; time settlement curves are not applicable.
2. The irregularly-shaped reactor mat was subdivided into 5 rectangular portions for purposes of estimating settlement (Attachment 02.05.04-19A).
3. The settlement methods listed above evaluate settlement by dividing the subsurface into layers with discrete elastic modulus values. The change in stress at the midpoint of a layer is calculated using elastic theory for loads applied to a semi-infinite half-space. The compression of each layer is computed as the result of dividing the applied stress increment by the elastic modulus to obtain an incremental strain, then multiplying the incremental strain by the layer thickness. The layer results are summed to obtain a total settlement.
4. The rock modulus values used were the reduced modulus values obtained from the Hoek-Brown analysis as discussed in FSAR Subsection 2.5.4.2.3.3 and shown in Table 2.5-236. The lower bound modulus value for the argillaceous limestone, the weaker of the two rock types, was used to calculate settlements. The rock modulus was applied as if rock were continuous, even though there may be instances where some rock is expected to be removed and replaced with fill concrete. As stated in FSAR Subsection 2.5.4.10.4.1, the reduced modulus values of the in-situ rock are lower than that of the fill concrete. This results in additional conservatism for the settlement estimate since the rock modulus values are used in place of fill concrete modulus values.

Variations among the three methods relate to the approach used to define the layers, obtain the layer modulus and the stress in the layer. Settlement was evaluated at five locations with each method.

A more detailed description of each of the methods used is presented in the following paragraphs.

Boussinesq Equation and Constrained Modulus

The stress distribution beneath foundations bearing on layered systems is typically assumed to be the same as that for a non-layered homogeneous halfspace. The formula typically used for these computations is the Boussinesq formula (FSAR Section 2.5.7, Reference 459). (The Boussinesq equation is also used to determine the depth of influence, H, for use in the EM 1110-1-2908 (FSAR Section 2.5.7, Reference 456) equation per the criterion in Figure 5-2 of that reference).

The settlement of any location is determined by computing the stress induced in the individual layers of the foundation material and dividing the stress by the constrained modulus to obtain the strain in each layer. The layer thickness is multiplied by the strain to compute the layer compression, and the layer compressions are summed to compute the total settlement.

The vertical stress induced by a rectangular loaded area may be computed at an arbitrary location, as shown in Attachment 02.05.04-19A, which has coordinates x, y with respect to the center of the loaded area. The x is parallel to the dimension L and y is parallel to the dimension B of the loaded area. The formula is found in Li (FSAR Section 2.5.7, Reference 459).

The Boussinesq equation in the form by Li is quite lengthy and is separated into eight individual components which are then added to combine into one equation below.

For stress increase at depth z beneath any point x,y inside or outside of the loaded area:

$$T_1 = \arctan\left[\frac{(x+a)\cdot(y+b)}{z\cdot\sqrt{(x+a)^2+(y+b)^2+z^2}}\right]$$

$$T_2 = \arctan\left[\frac{(x+a)\cdot(y-b)}{z\cdot\sqrt{(x+a)^2+(y-b)^2+z^2}}\right]$$

$$T_3 = \arctan\left[\frac{(x-a)\cdot(y-b)}{z\cdot\sqrt{(x-a)^2+(y-b)^2+z^2}}\right]$$

$$T_4 = \arctan\left[\frac{(x-a)\cdot(y+b)}{z\cdot\sqrt{(x-a)^2+(y+b)^2+z^2}}\right]$$

$$T_5 = \left[\frac{z\cdot(x+a)\cdot(y+b)\cdot\left[(x+a)^2+(y+b)^2+2\cdot z^2\right]}{\left[(x+a)^2+z^2\right]\cdot\left[(y+b)^2+z^2\right]\cdot\sqrt{(x+a)^2+(y+b)^2+z^2}}\right]$$

$$T_6 = \left[\frac{z\cdot(x+a)\cdot(y-b)\cdot\left[(x+a)^2+(y-b)^2+2\cdot z^2\right]}{\left[(x+a)^2+z^2\right]\cdot\left[(y-b)^2+z^2\right]\cdot\sqrt{(x+a)^2+(y-b)^2+z^2}}\right]$$

$$T_7 = \left[\frac{z\cdot(x-a)\cdot(y-b)\cdot\left[(x-a)^2+(y-b)^2+2\cdot z^2\right]}{\left[(x-a)^2+z^2\right]\cdot\left[(y-b)^2+z^2\right]\cdot\sqrt{(x-a)^2+(y-b)^2+z^2}}\right]$$

$$T_8 = \left[\frac{z \cdot (x-a) \cdot (y+b) \cdot \left[(x-a)^2 + (y+b)^2 + 2 \cdot z^2 \right]}{\left[(x-a)^2 + z^2 \right] \cdot \left[(y+b)^2 + z^2 \right] \cdot \sqrt{(x-a)^2 + (y+b)^2 + z^2}} \right]$$

$$\Delta\sigma = \frac{q}{2 \cdot \pi} \cdot (T_1 - T_2 + T_3 - T_4 + T_5 - T_6 + T_7 - T_8)$$

To determine the average vertical strain in the layer, the stress increment is divided by the constrained modulus, M, for the layer. To calculate the settlement, the strain is multiplied by the layer thickness, and the results for the layers considered are summed to obtain the total settlement (ΔH) at the location being analyzed. The equations are given below.

$$M_i = E_i \cdot \left[\frac{1 - v_i}{(1 + v_i) \cdot (1 - 2v_i)} \right]$$

$$\Delta h_i = \frac{\Delta\sigma_i}{M_i} \cdot h_i$$

$$\Delta H = \sum_{i=1}^{n} \Delta h_i$$

The use of the constrained modulus implies that the size of the loaded area is large compared to the thickness of the layer. This is the case for the Nuclear Islands which are at least 91 ft. wide as compared to the maximum layer thickness used of about 10 ft. Vertical movements due to lateral deformations are not included when the constrained modulus is used. The Boussinesq equation does not provide a direct means of estimating the settlement if the foundation is rigid. The maximum depth of computation for settlement estimates using the Boussinesq equation is generally taken as the depth where the stress increase beneath the center of the foundation is $0.10 * \gamma * z$. This depth for the static loading of 8600 psf and the rock buoyant unit weight is H = 210 ft. (elevation 379 ft.). This determination of the depth of influence is consistent with Regulatory Guide 1.132 (Rev. 2) and is used with the Steinbrenner equation (unless a sharp modulus increase occurs) as well as with the Boussinesq equation.

Steinbrenner Equation

A widely used formula for computing the settlement of a rectangular shaped foundation area is the equation from the theory of elasticity contained in Bowles 1988 and 1996, (FSAR Subsection 2.5.7, References 460 and 461) as Equation 5-16:

$$\Delta H = q_o \cdot B' \cdot \frac{(1 - v^2)}{E_s} \cdot \left[I_1 + \left(\frac{1 - 2v}{1 - v} \right) \cdot I_2 \right] \cdot I_F$$

Where:

ΔH	=	settlement of corner of area B' x L'
q_o	=	intensity of contact pressure
B'	=	least lateral dimension of contributing base area

I_1, I_2 = influence factors
I_F = embedment factor from Figure 5-7 of Bowles 1988
E_s = soil or rock average modulus of elasticity within depth of influence, H
ν = Poisson's Ratio
H = soil or rock thickness below base area and within depth of influence

To compute the influence factors I_1 and I_2 in the above equation, the following definitions apply:

Center of Foundation

$$B' = \frac{B}{2} \qquad L' = \frac{L}{2}$$

$$M = \frac{L'}{B'} \qquad N = \frac{H}{B'}$$

Corner of Foundation

$$B' = B \qquad L' = L$$

$$M = \frac{L}{B'} \qquad N = \frac{H}{B}$$

$$W = \frac{\left(1+\sqrt{M^2+1}\right)\cdot\sqrt{M^2+N^2}}{M\cdot\left(1+\sqrt{M^2+N^2+1}\right)} \qquad X = \frac{\left(M+\sqrt{M^2+1}\right)\cdot\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}} \qquad Y = \frac{M}{N\cdot\left(\sqrt{M^2+N^2+1}\right)}$$

$$I_1 = \frac{1}{\pi}\cdot\left(M\cdot\ln(W)+\ln(X)\right) \qquad I_2 = \frac{N}{2\cdot\pi}\cdot\arctan(Y)$$

Bowles (FSAR Section 2.5.7, Reference 460) notes that his Equation 5-16 may be written in a format combining the terms inside the brackets into an influence factor called I_S. I_S is known as the Steinbrenner influence factor which accounts for the depth, H, of compressible material beneath the foundation, and is calculated as follows:

$$I_s = I_1 + \left(\frac{1-2\cdot\nu}{1-\nu}\right)\cdot I_2$$

The settlement using the Steinbrenner equation is calculated as:

$$\Delta H = q_o \cdot B'\cdot\frac{\left(1-\nu^2\right)}{E_s}\cdot I_s \cdot I_f \cdot m$$

Where:

ΔH = settlement of corner of area B' x L'
q_o = intensity of contact pressure
B' = least lateral dimension of contributing base area
I_S = Steinbrenner influence factor
I_F = embedment factor from Figure 5-7 of Bowles (FSAR Section 2.5.7 Reference 460)
E_s = soil or rock average modulus of elasticity within depth of influence, H
ν = Poisson's Ratio
H = soil or rock thickness below base area to depth of influence, H
m = number of corners of equal dimension rectangles contributing to settlement; otherwise add unequal areas individually

The average modulus value is computed as shown below:

$$E_{s(avg)} = \frac{\sum_{i=1}^{n} h_i}{\sum_{i=1}^{n} \frac{h_i}{E_i}}$$

Corps of Engineers Equation

EM 1110-1-2908 (FSAR Section 2.5.7, Reference 456) computes the settlement of the center a rectangular- shaped flexible foundation area using the equation from the theory of elasticity as contained in the Corps Equation 5-2:

$$\delta_a = \frac{1.12 \cdot q \cdot B \cdot \left(1 - \mu^2\right) \cdot \sqrt{L/B}}{E_d}$$

In this equation, μ is the Poisson's ratio. The Poisson's ratio is the average value taken from the results of laboratory stress-strain tests on rock core specimens from the site as described in FSAR Subsection 2.5.4.2.3.1.2.

EM 1110-1-2908 recommends an expression for the weighted average modulus (E_{dw}) to use in Equation 5-2. However, the reduced deformation modulus for the argillaceous limestone produced by the Hoek-Brown method and shown in FSAR Table 2.5-236 was used for the calculations made because this deformation modulus was calculated specifically for the BLN site and is considered to be more reflective of site conditions..

The settlement of the center of the foundation area is computed by substituting E_{rm} and the q, B and L into Equation 5-2 of EM 1110-1-2908. To estimate settlement at locations on the foundation other than the center, charts contained in EM 1110-1-2908 are used. For locations other than the center, the procedure is to multiply the settlement from the preceding equation by the appropriate reduction factors. The settlement of a rigid foundation may also be estimated from the results and charts in EM-1110-1-2908. Only settlements of flexible foundations were computed.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been identified associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-19A

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-20

FSAR Section, 2.5.4.10.4.2 states that the nuclear island meets the criteria in DCD Table 2-1 for a uniform site, and that, therefore, differential settlement is not a factor. Please explain how and why these nuclear islands meet the uniformity criteria despite the fact that geologic units C and D, with a significant shear wave velocity difference, underlie the Unit 3 nuclear island.

BLN RAI ID: 1000

BLN RESPONSE:

The DCD indicates the AP1000 is designed for a site where "...the foundation conditions do not have extreme variation within the nuclear island footprint." The uniformity criteria are described in DCD Subsection 2.5.4.5.3, Site Foundation Material Evaluation Criteria. There are three criteria that are to be applied to the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint to demonstrate site uniformity:

1. The depth to soil/rock layer interfaces in the 120-foot depth profile should deviate no more than 5 percent from the average interface depth. If a deviation greater than 5 percent occurs, then the profile should be modified by adding additional layers/interfaces, or additional borings (to provide better resolution of the average layer interface depth(s)).
2. For layers exhibiting low strain shear wave velocity (Vs) greater than or equal to 2,500 feet per second (fps), the layer should have approximately uniform thicknesses and should have a dip no greater than 20 degrees, and the Vs within any layer should not vary from the average by more than 20 percent.
3. For a layer with a low strain Vs less than 2,500 fps, the layer should have approximately uniform thickness and should have a dip no greater than 20 degrees, and the Vs within the layer should not vary from the average by more than 10 percent.

With respect to application of the DCD uniformity criteria for evaluation of differential settlement, Criteria 1 and 2, above, apply. As shown on FSAR Figure 2.5-339, the Units 3 and 4 nuclear island basemats are supported on fresh/slightly weathered middle Stones River Group limestone bedrock that exhibits a low strain Vs greater than 2,500 fps. This is true for both the middle Stones River Group subunits C (argillaceous and silty limestone) and D (limestone) that form the upper rock units under Unit 3 (and Unit 4).

Relative to Criterion 1, the variation in depth to the interface between subunits C and D below the nuclear island basemats exceed 5 percent of the average depth to the interface. Velocity profiles shown in FSAR Figures 2.5-353 through 2.5-356 are developed at different locations and specifically account for the variation in the depth to the C-D interface. These varied velocity profiles were considered in development of the ground motion response analyses, satisfying the DCD Criterion 1 requirement where the velocity layer/interfaces exceed the 5 percent depth criteria.

Relative to Criterion 2, the interface between the middle Stones River Group limestone bedrock subunits C and D (and other middle Stones River Group subunits) exhibits an average dip of 17 degrees, less than the 20 degree maximum dip criteria.

In addition to meeting the DCD criteria, both subunits C and D are stiff rock strata that exhibit bearing capacity substantially greater than the DCD requirement, and estimated settlement well below DCD thresholds. As discussed in FSAR Subsection 2.5.4.7.4, variability in rock properties between the

limestone (subunit D) and argillaceous limestone (subunit C) are not deemed significant because the strength and modulii of the relatively weaker argillaceous beds are still well above requirements for foundation bearing capacity, settlement, etc. In addition to specifically evaluating differences in basic properties of the various rock subunits, foundation analyses incorporated consideration of in situ rock mass structures, such as joints, using core boring, borehole testing (e.g., Goodman Jack (FSAR Subsection 2.5.7, Reference 438) and the Hoek Brown criterion (FSAR Subsection 2.5.7, Reference 442)) to factor the general "weakening" and "softening" effect of discontinuities.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

No COLA revisions have been associated with this response.

ASSOCIATED ATTACHMENTS/ENCLOSURES:

None

NRC Letter Dated: August 5, 2008

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-21

FSAR Section 2.5.4.10.5 states that the earth pressure coefficients for the at-rest and passive conditions determined using the methods described in Reference 462 are illustrated in FSAR Figures 2.5-360 and 2.5-361. FSAR Figure 2.5-360 shows the distribution of earth pressures from the soil backfill (at-rest condition), and, below the water table, the additional pressure caused by the hydrostatic pressure. FSAR Figure 2.5-361 shows the soil passive pressure distribution. No hydrostatic pressure is included in the passive pressure because water has no shear strength and provides no additional passive resistance. Since the hydrostatic associated pressure also contributes to the lateral pressure applied to the nuclear island walls, excluding the hydrostatic pressure from the passive, at-rest, or active earth pressure evaluation is not justified. Please include the hydrostatic pressure into passive or at-rest earth pressure evaluation and revise the contents of FSAR Figs. 2.5-360 and 2.5-361, accordingly.

BLN RAI ID: 1001

BLN RESPONSE:

Discussions with NRC technical staff have clarified that inclusion of the hydrostatic pressure in the at-rest and passive earth pressure diagrams is appropriate for the nuclear island basement walls. The FSAR text and Figures 2.5.4-360 and 2.5.4-361 will be revised as described below in a future revision of the COLA.

This response is PLANT-SPECIFIC.

ASSOCIATED BLN COL APPLICATION REVISIONS:

1. COLA Part 2, FSAR. Chapter 2, Subsection 2.5.4.10.5, last paragraph will be revised from:

Earth pressure coefficients for the at-rest and passive conditions determined using the methods described in Reference 462 are illustrated in Figures 2.5-360 and 2.5-361. Figure 2.5-360 shows the distribution of earth pressure from the soil backfill (at-rest condition), and, below the water table, the additional pressure caused by hydrostatic pressure. Figure 2.5-361 shows the soil passive pressure distribution. No hydrostatic pressure is included in the passive pressure because water has no shear strength and provides no additional passive resistance.

To read:

Earth pressure coefficients for the at-rest and passive conditions determined using the methods described in Reference 462 are illustrated in Figures 2.5-360 and 2.5-361. Figure 2.5-360 shows the distribution of earth pressure from the soil backfill (at-rest condition), and, below the water table, the additional pressure caused by hydrostatic pressure. Figure 2.5-361 shows the soil passive earth pressure distribution, including the additional pressure caused by hydrostatic pressure below the water table.

2. COLA Part 2, FSAR Chapter 2, Figure 2.5.4-360 will be revised as indicated in Attachment 02.05.04-21A.

3. COLA Part 2, FSAR Chapter 2, Figure 2.5.4-361 will be revised as indicated in Attachment 02.05.04-21B.

Enclosure
TVA letter dated September 19, 2008
RAI Responses

ASSOCIATED ATTACHMENTS/ENCLOSURES:

Attachment 02.05.04-21AAttachment 02.05.04-21B

Attachment 02.05.04-05A

(5 pages, including this cover)

Figure 1

Vertical Limits of Excavation, Bellefonte Nuclear Plant, Unit 3

(From FSAR Figure 2.5-348a)

Figure 2

Shear Wave Velocity Stratigraphic Column

(from FSAR Figure 2.5-299)

Figure 3

Calculation Geometry, Input Parameters, and Results for Three SH-Wave Incident Angles

Figure 4

Divergence Angle, ΔJ_e, as a Function of SH-Wave Plane-Wave Incident Angle
for the Reactor Unit #3 Fill Concrete and Basemat

Figure 1

Vertical limits of excavation, Bellefonte Nuclear Plant, Unit 3 (from FSAR Figure 2.5-348a)

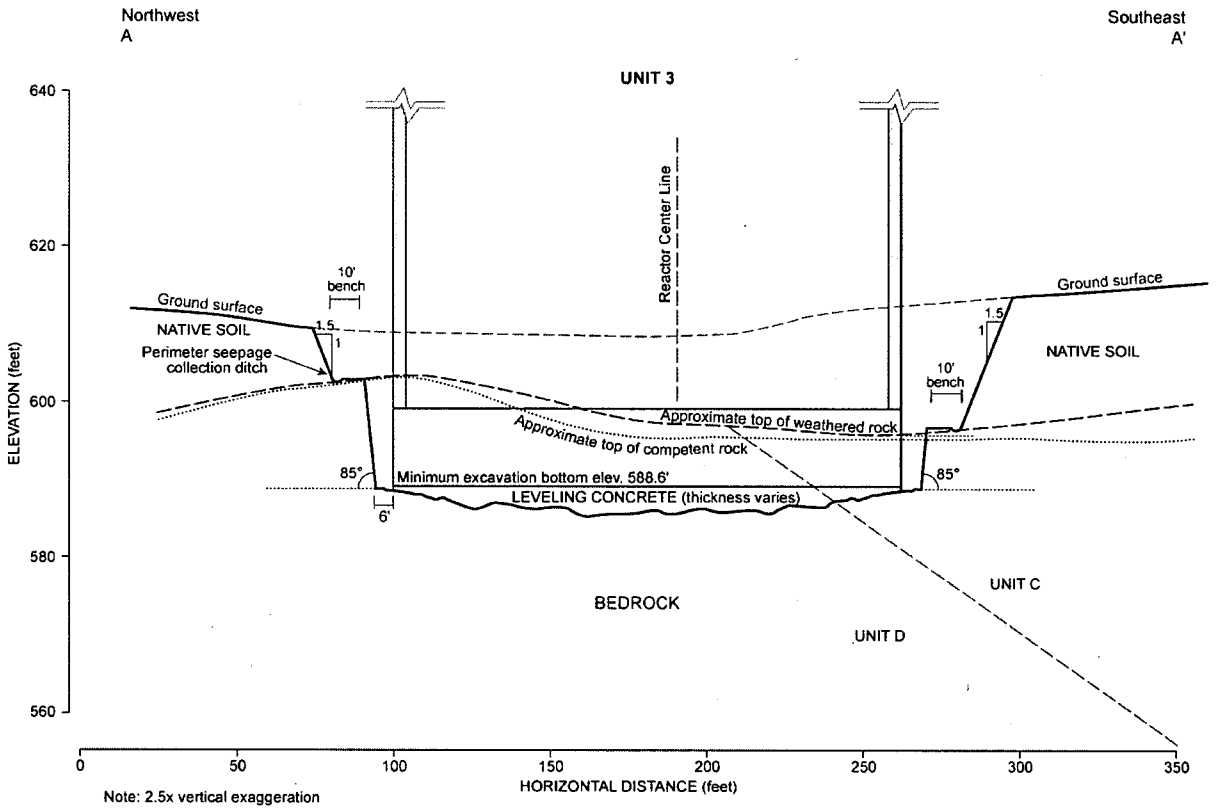


Figure 2

Shear Wave Velocity Stratigraphic Column (from FSAR Figure 2.5-299)

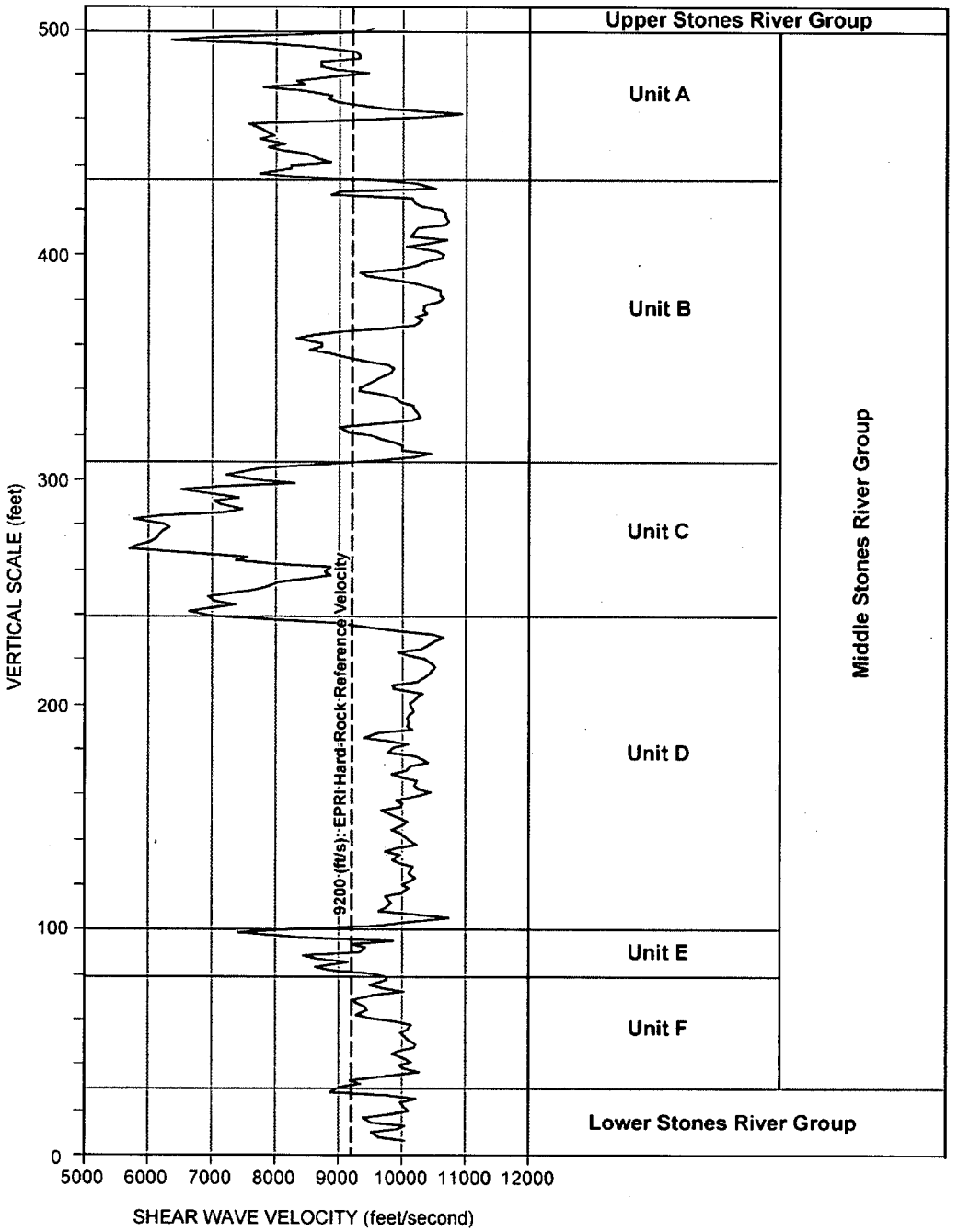


Figure 3

Calculation Geometry, Input Parameters, and Results for Three SH-Wave Incident Angles

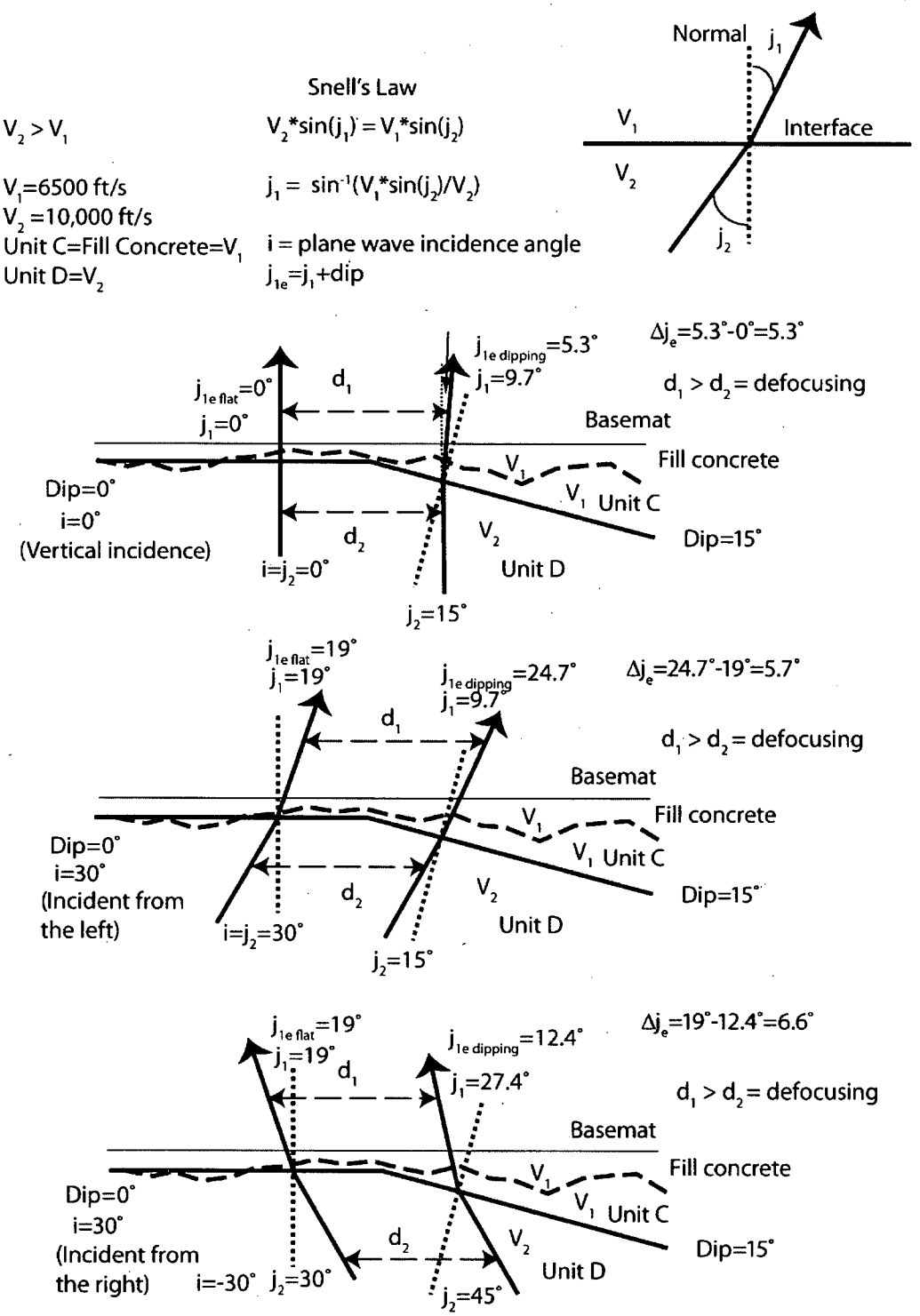
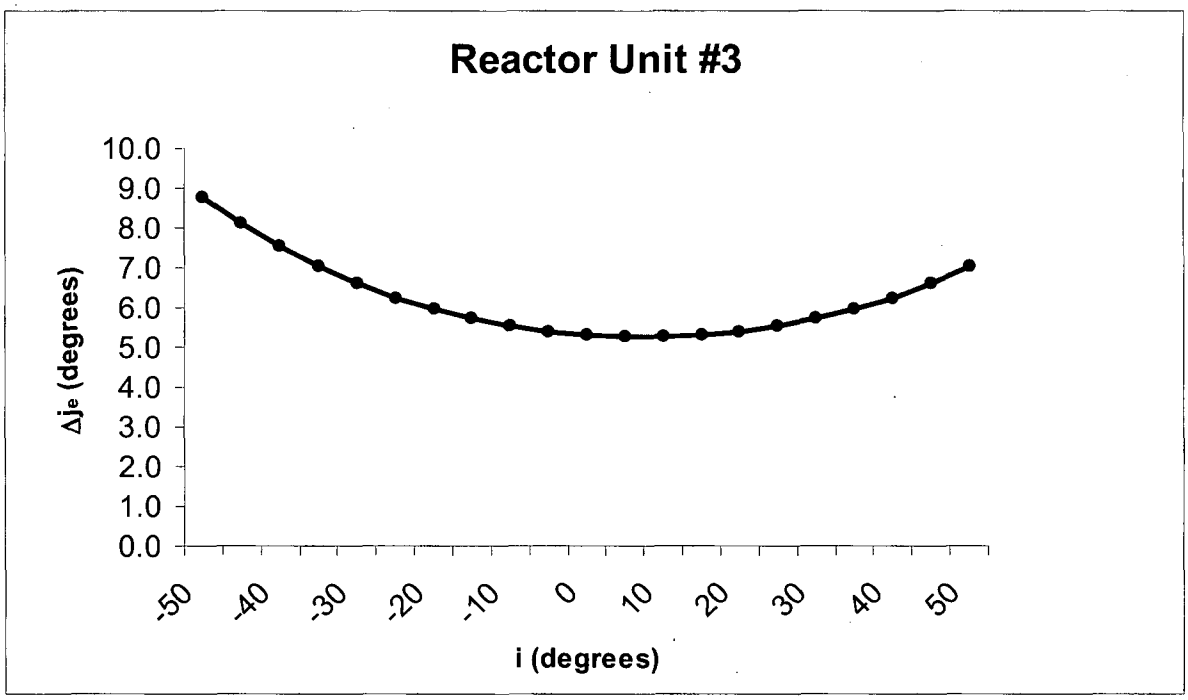


Figure 4

Divergence Angle, ΔJ_e, as a Function of SH-Wave Plane-Wave Incident Angle for the Reactor Unit #3 Fill Concrete and Basemat



Attachment 02.05.04-06A

(4 pages, including this cover sheet)

Revised Figure 2.5-314, Rev. 1

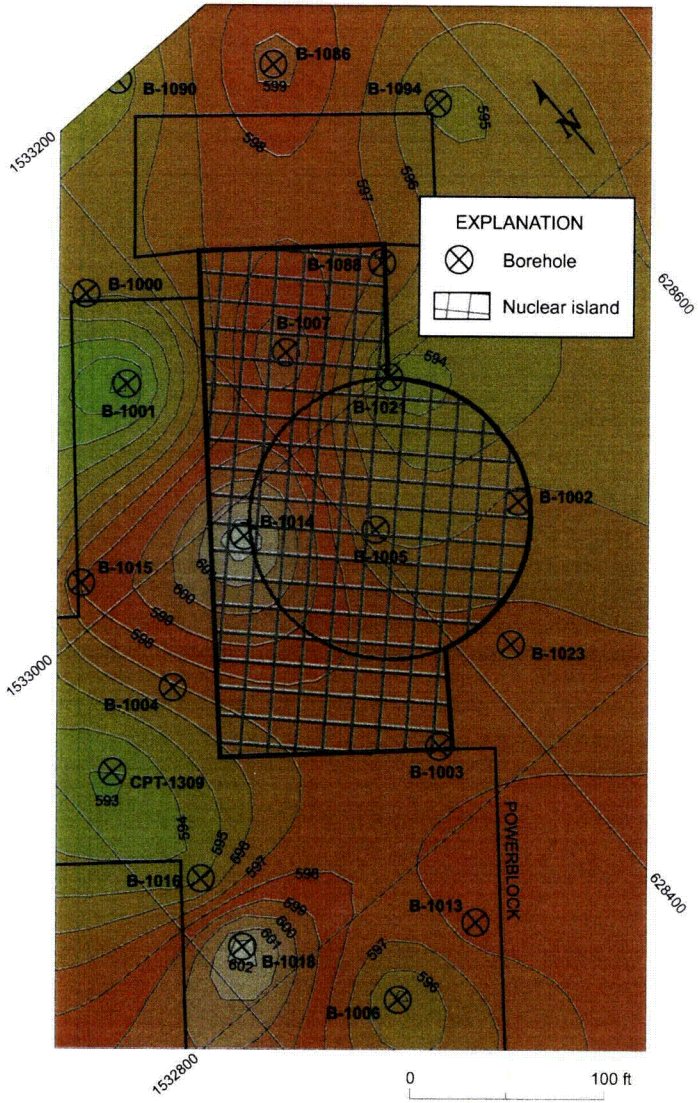
Elevation Contour Maps of Top of Competent Rock from Borehole Data

Revised Figure 2.5.4-348a, Rev. 1

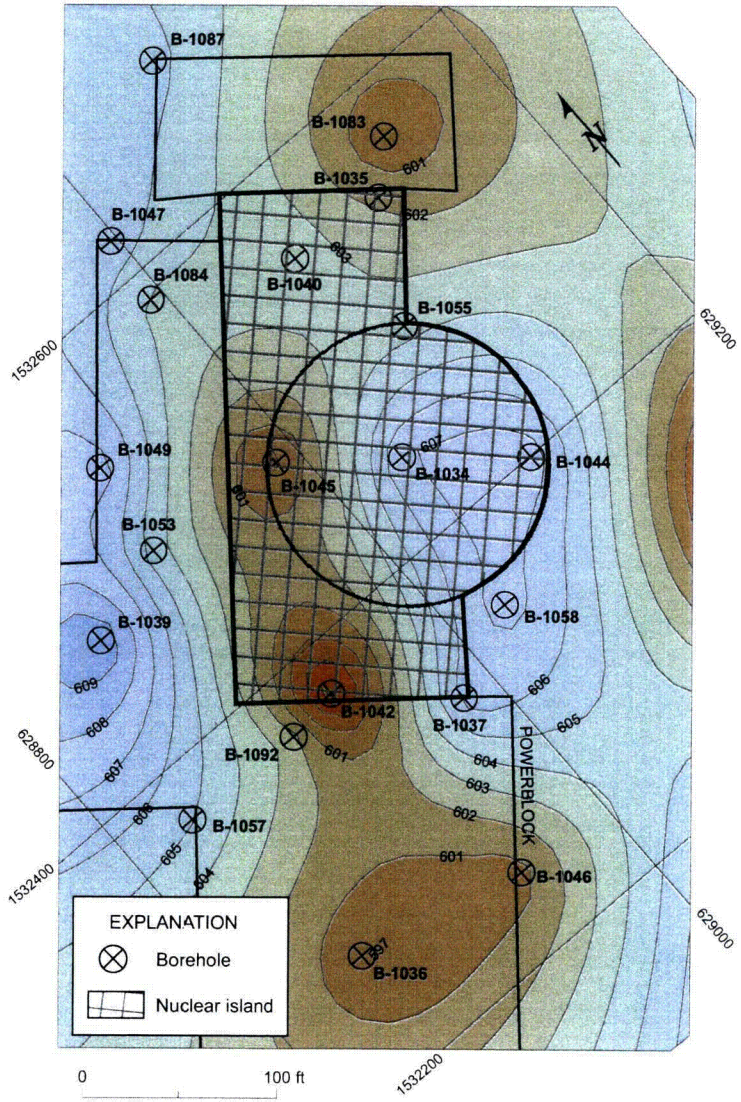
Vertical Limits of Excavation, Unit 3

Revised Figure 2.5.4-348b, Rev. 1

Vertical Limits of Excavation, Unit 4



UNIT 3



UNIT 4

FIGURE 2.5-314
Elevation Contour Map of Top of Competent Rock – Units 3 & 4

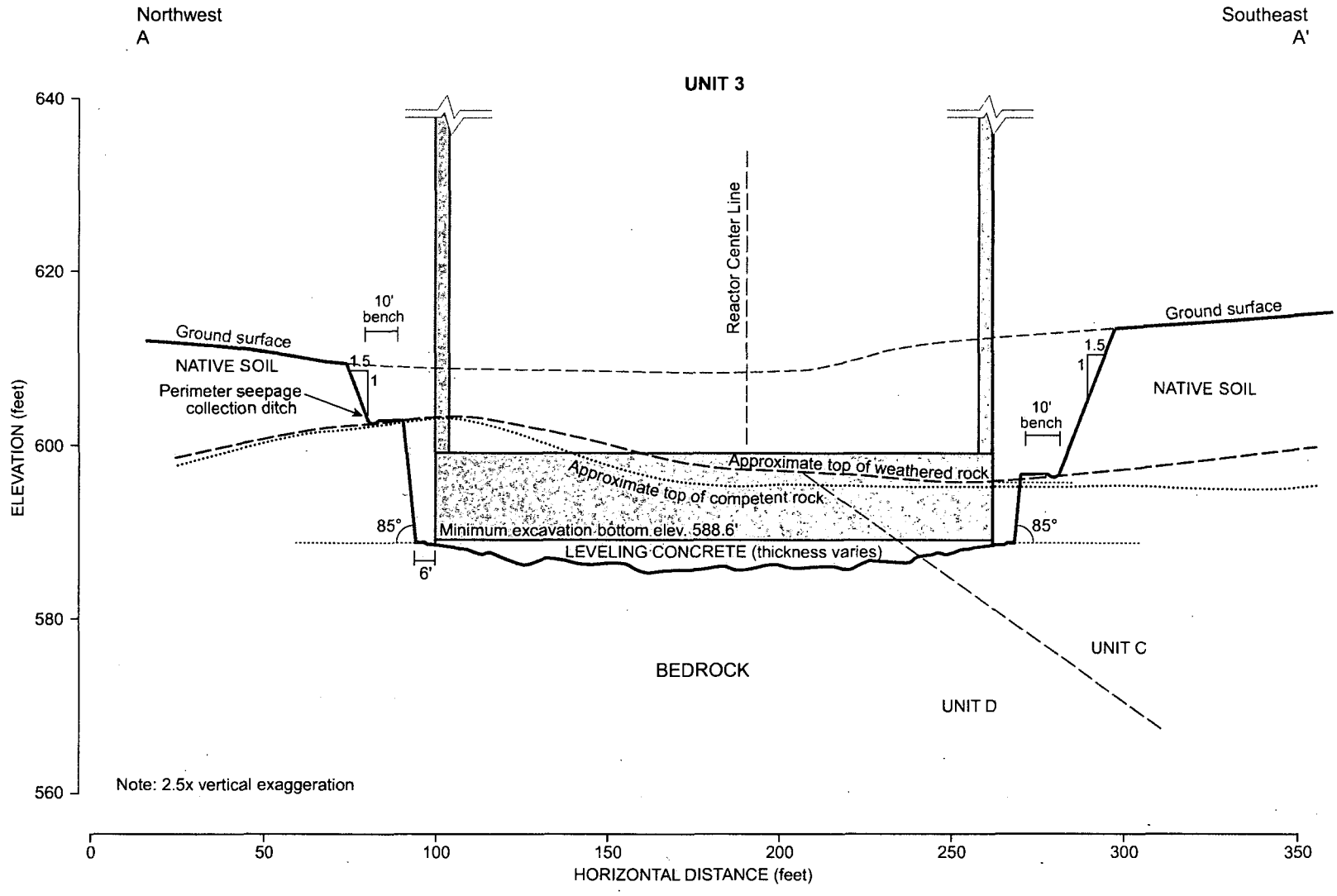


FIGURE 2.5-348a
Vertical Limits of Excavation, Unit 3

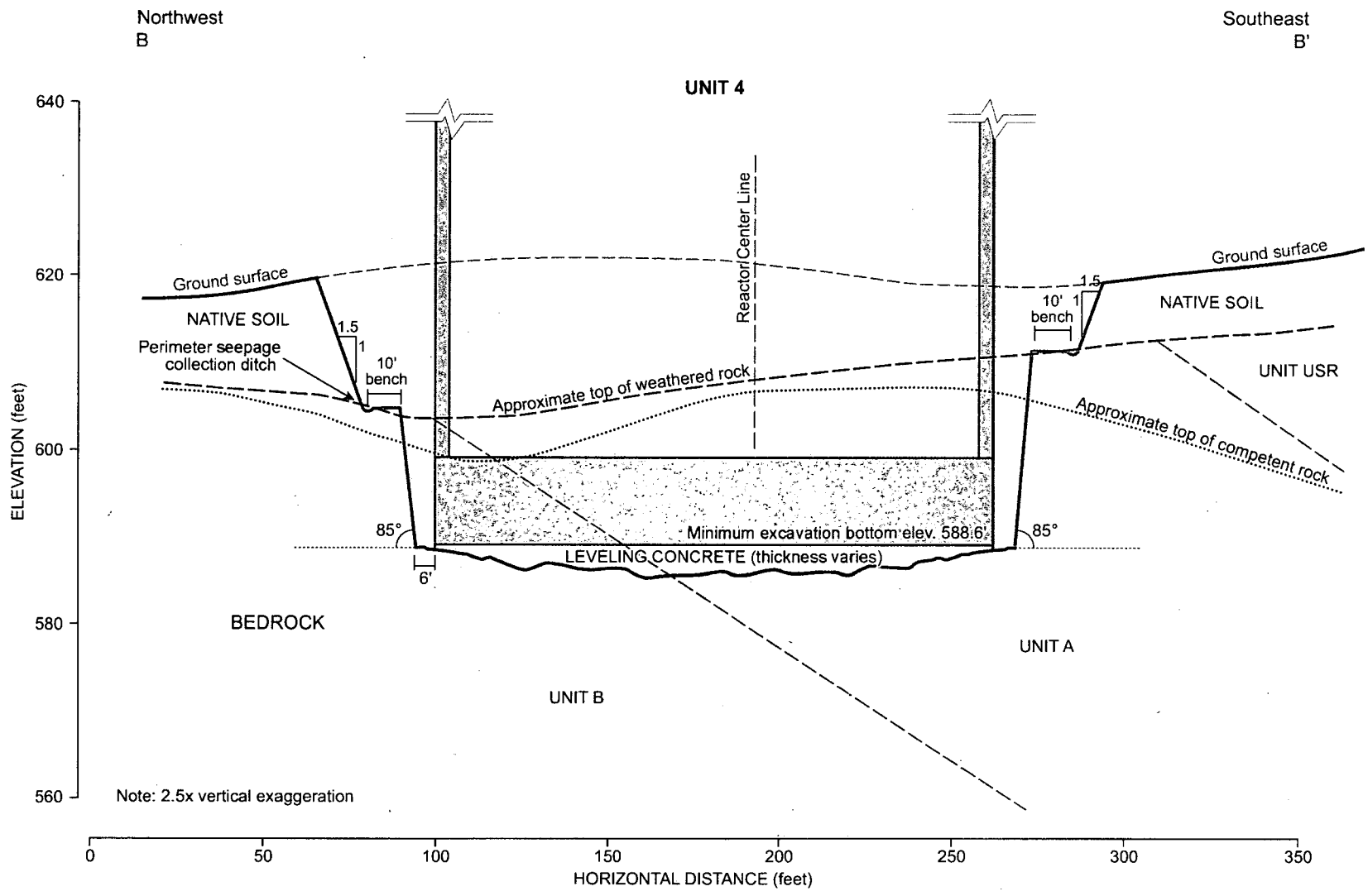


FIGURE 2.5-348b
Vertical Limits of Excavation, Unit 4

Attachment 02.05.04-14A

Figure 13 taken from Reference 1

Horizontal Pressure on Walls from Compaction Effort

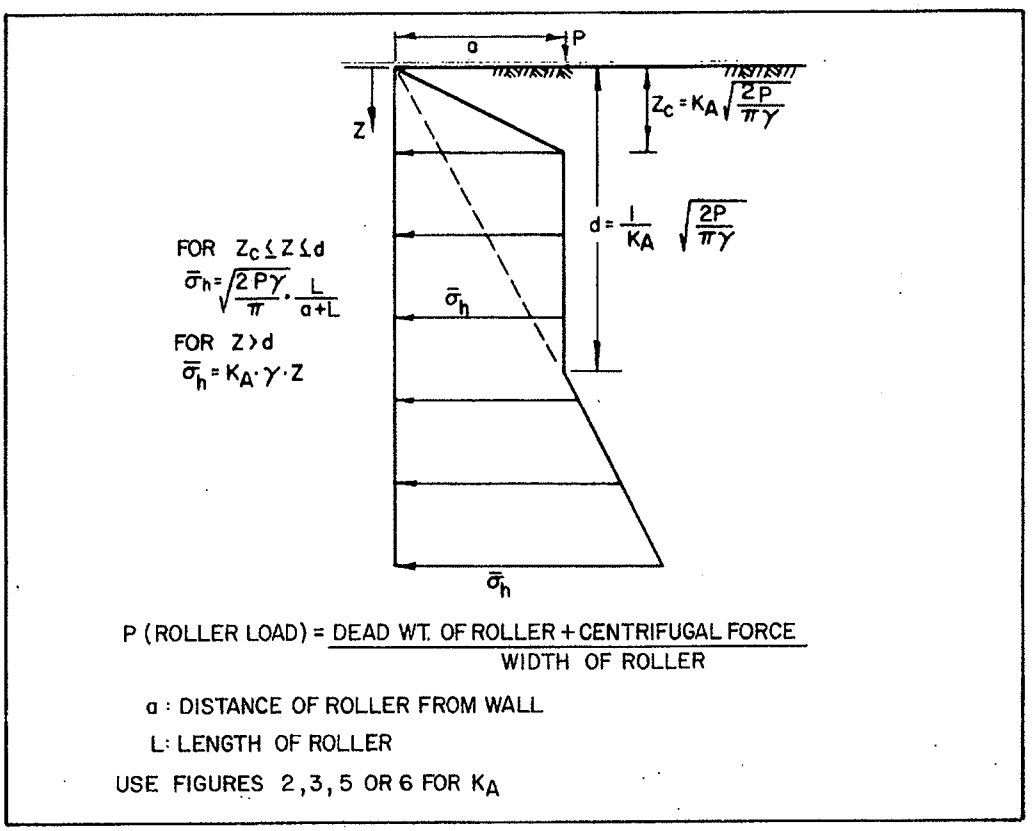


FIGURE 13
Horizontal Pressure on Walls from Compaction Effort

Source: Reference 1.

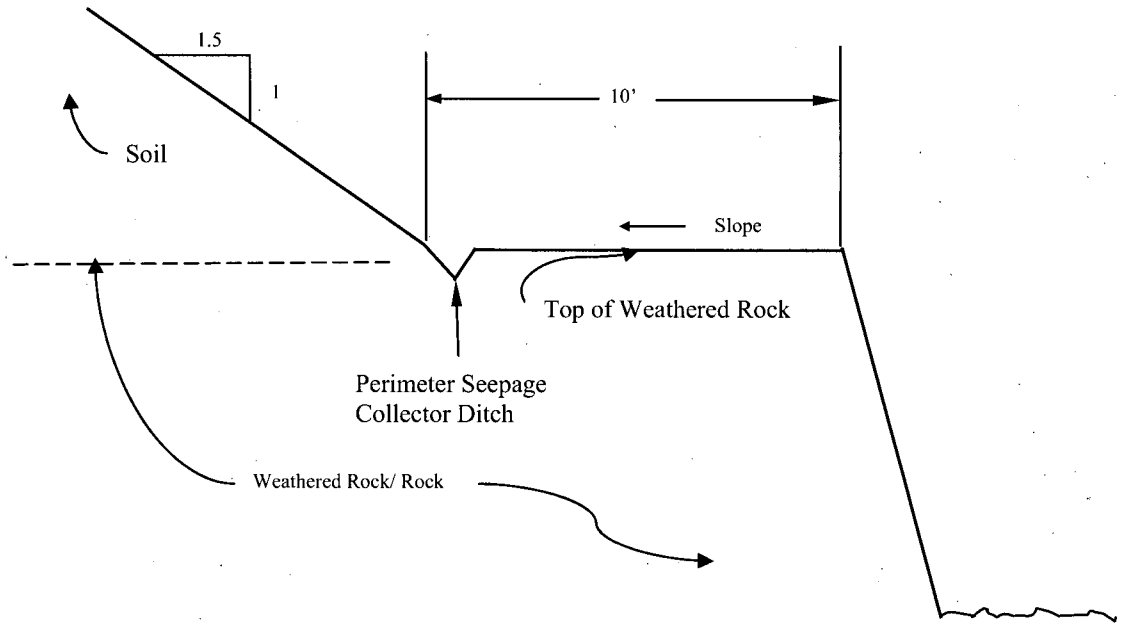
Note: For use with at-rest earth pressures, the value of the parameter K_A is taken as the value for K_O, the coefficient of at-rest earth pressure, from FSAR Figure 2.5-360.

Attachment 02.05.04-15A

Figure 02.05.04-15A-1

Sketch Showing Typical Perimeter Drainage Ditch for Soil Slope

(Not to scale).



Attachment 02.05.04-18A

Table 6-1 from EM 1110-1-2908

All terms are as previously defined.

c. Local shear failure. Local shear failure represents a special case where failure surfaces start to develop but do not propagate to the surface as illustrated in Figure 6-1a. In this respect, the depth of embedment contributes little to the total bearing capacity stability. An expression for the ultimate bearing capacity applicable to localized shear failure can be written as:

$$q_{ult} = cN_c + 0.5\gamma BN_\gamma \tag{6-4}$$

All terms are as previously defined.

d. Correction factors. Equations 6-1, 6-3, and 6-4 are applicable to long continuous foundations with length to width ratios (L/B) greater than ten. Table 6-1 provides correction factors for circular and square foundations, as well as rectangular foundations with L/B ratios less than ten. The ultimate bearing capacity is estimated from the appropriate equation by multiplying the correction factor by the value of the corresponding bearing capacity factor.

Table 6-1
Correction factors (after Sowers 1979)

Foundation Shape	C_c N_c Correction	$C\gamma$ N_γ Correction
Circular	1.2	0.70
Square	1.25	0.85
Rectangular		
L/B = 2	1.12	0.90
L/B = 5	1.05	0.95
L/B = 10	1.00	1.00

Correction factors for rectangular foundations with L/B ratios other than 2 or 5 can be estimated by linear interpolation.

e. Compressive failure. Figure 6-1c illustrates a case characterized by poorly constrained columns of intact rock. The failure mode in this case is similar to unconfined compression failure. The ultimate bearing capacity may be estimated from Equation 6-5.

$$q_{qult} = 2\ c\ \tan\ (45 + \phi/2) \tag{6-5}$$

All parameters are as previously defined.

f. Splitting failure. For widely spaced and vertically oriented discontinuities, failure generally initiates by splitting beneath the foundation as illustrated in Figure 6-1e. In such cases Bishnoi (1968) suggested the following solutions for the ultimate bearing capacity:

For circular foundations

$$q_{ult} = JcN_{cr} \tag{6-6a}$$

For square foundations

$$q = 0.85JcN_{cr} \tag{6-6b}$$

For continuous strip foundations for L/B ≤ 32

$$q_{ult} = JcN_{cr}/(2.2 + 0.18\ L/B) \tag{6-6c}$$

where

J = correction factor dependent upon thickness of the foundation rock and width of foundation.

L = length of the foundation

The bearing capacity factor N_{cr} is given by:

$$N_{cr} = \frac{2N_\phi^2}{1+N_\phi}\ (\cot\phi)\ (S/B)\ \left(1 - \frac{1}{N_\phi}\right) - N\phi\ (\cot\phi) + 2N\phi^{1/2} \tag{6-6d}$$

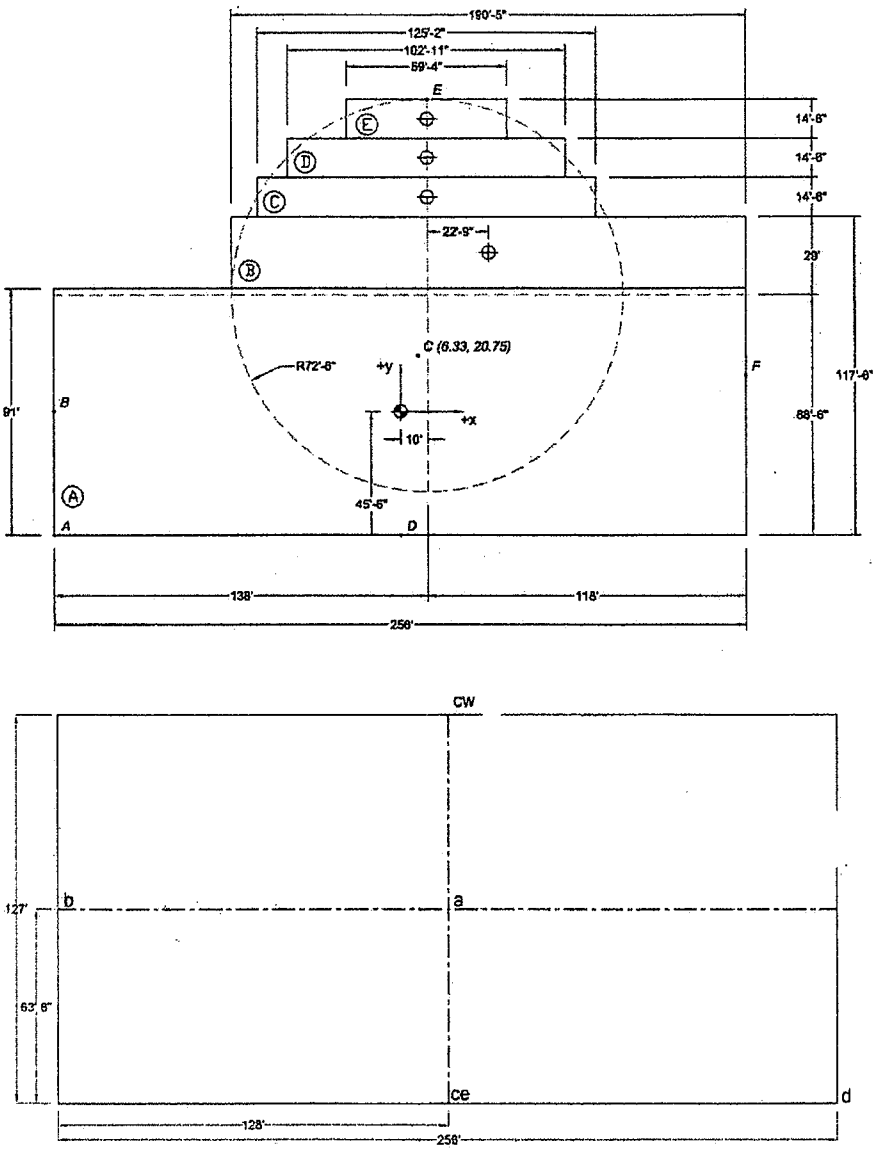
All other terms are as previously defined. Graphical solutions for the correction factor (J) and the bearing capacity factor (N_{cr}) are provided in Figures 6-2 and 6-3, respectively.

g. Input parameters. The bearing capacity equations discussed above were developed from considerations of the Mohr-Columb failure criteria. In this respect, material property input parameters are limited to two parameters; the cohesion intercept (c) and the angle of internal friction (ϕ). Guidance for selecting design shear strength parameters is provided in Chapter 4. However, since rock masses generally provide generous margins of safety against bearing capacity failure, it is recommended that

ATTACHMENT 02.05.04-19A

Figure 02.05.04-19A-1

Rectangles for Settlement Calculation

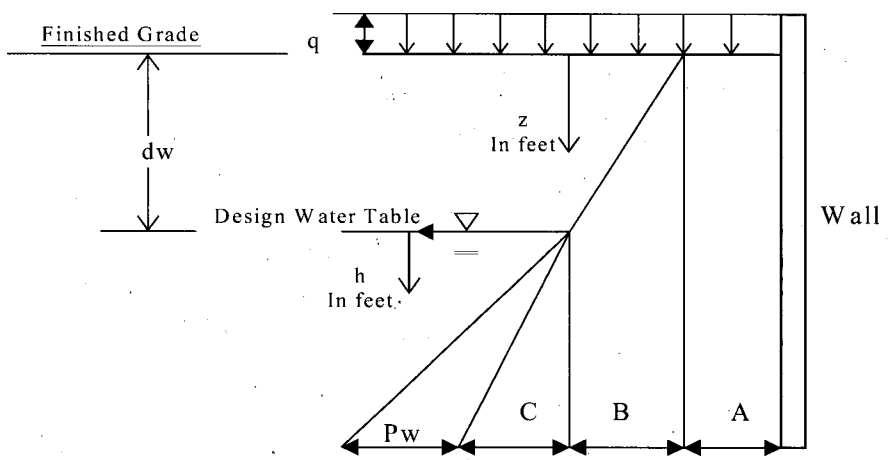


Attachment 02.05.04-21A

Revised Figure 2.5.4-360

Static Lateral At-Rest Earth Pressures, on 1-Ft.-Wide Vertical Strip, on Nuclear Island Below-Grade Walls

At-rest Earth Pressure on 1 foot wide vertical strip



A = 0.81 (q) = Effect of uniform full coverage surface surcharge = 203 for example using surcharge of 250 psf

B = 106.9 (z) =Earth pressure at rest above water table

C = 56.4 (h) = Earth pressure at rest increment below water table

Pw = 62.4 (h) = Hydrostatic pressure increment

H = A + B = Static lateral earth pressure above water table ($z \leq dw$)

H = 203 + 106.9 (dw) + 56.4 (z – dw) + 62.4 (h) = Static lateral earth pressure below water table ($z > dw$); h = z-dw.

Conditions on information:

- Units of pressure = lbs/ft²
- A surcharge value, q, of 250 lbs/ft² is used as an example only; actual value must be provided by designer.

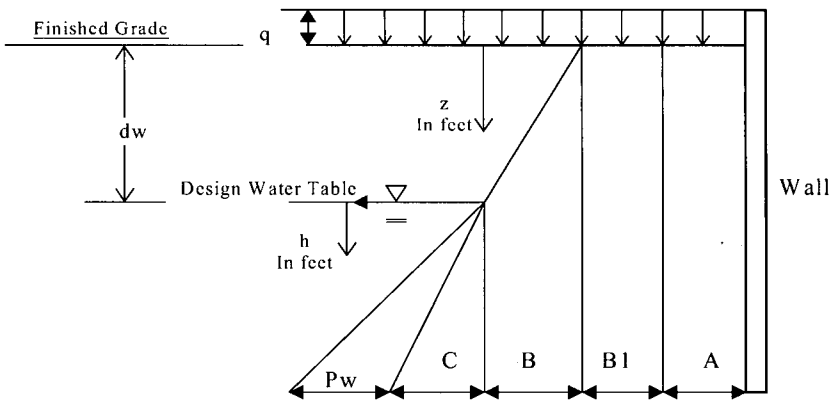
- Backfill of borrow soil meeting Class I properties as defined in FSAR, 1986 compacted to 95% MDD by ASTM D698
- No heavy compaction equipment used within 5 ft. of wall
- $\gamma s = 132$ lbs/ft^3 = saturated unit weight of backfill above water table
- $\gamma = 69.6$ lbs/ft^3 = submerged soil density
- $\phi cu = 11$ deg = angle of internal friction of soil (95% Maximum dry density at 2% above optimum moisture; total stress)
- $\upsilon = 0.49$= Poisson's ratio of soil based on seismic conditions
- $K0 = 0.81$ = At-rest earth pressure coefficient of soil
- Plane strain conditions (corner adjustment factors not included)
- Dynamic soil pressure not included

Attachment 02.05.04-21B

Revised Figure 2.5.4-361

Static Lateral Passive Earth Pressures, on 1-Ft.-Wide Vertical Strip,
on Nuclear Island Below-Grade Walls

Passive Earth Pressure on 1 foot wide vertical strip



A =

1.47 (q) = Effect of uniform full coverage surface surcharge = 367.5 for example surcharge of 250 psf

B1 = 3346 psf = Passive earth pressure at ground surface due to soil cohesion

B = 194 (z) = Passive earth pressure above water table

C = 102.3 (h) = Passive earth pressure increment below water table

Pw = 62.4 (h) = Hydrostatic pressure increment

Pp = A + B1 + B = Passive lateral earth pressure above water table ($z \leq dw$)

Pp = A + B1 + 194 (dw) + 102.3 (h) + 62.4 (h) = Passive lateral earth pressure below water table, ($z > dw$); h = z-dw

Conditions on information:

- Units of pressure = lbs/ft^2
- A surcharge value, q, of 250 lbs/ft^2 is used as an example only; actual value must be provided by designer.

- Backfill of borrow soil meeting Class I properties as defined in FSAR, 1986 compacted to 95% MDD by ASTM D698
- No factors included
- $\gamma s = 132$ lbs/ft^3 = saturated unit weight of backfill above water table based on 95% Maximum dry density at 2% above optimum moisture
- $\gamma = 69.6$ lbs/ft^3 = submerged soil density
- $\phi cu = 11$ deg = angle of internal friction of soil (95% Maximum dry density at 2% above optimum moisture; total stress)
- Kp = 1.47 = Coefficient of passive earth pressure due to Ø (Rankine equation)
- Ccu = 1380 psf = shear strength intercept of soil (total stress, saturated CU test)
- Plane strain conditions (corner adjustment factors not included)
- Dynamic soil pressure not included