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April 15, 2010

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

BELL BEND NUCLEAR POWER PLANT
RESPONSE TO RAI No. 79
BNP-2010-094 Docket No. 52-039

References:
1) M. Canova (NRC) to R. Sgarro (PPL Bell Bend, LLC), Bell Bend COLA – Request for Information No. 79 (RAI No. 79) – SEB1-2507, email dated February 17, 2010.

2) BNP-2010-085, R. Sgarro (PPL Bell Bend, LLC) to U.S. NRC Document Control Desk, "BBNPP Partial Response to RAI No. 79 and Request for Extension," dated March 19, 2010.

3) BNP-2009-400, T. Harpster (PPL Bell Bend, LLC) to U.S. NRC Document Control Desk, "BBNPP Schedule Update," dated December 8, 2009.

The purpose of this letter is to respond to portions of the request for additional information (RAI) identified in Reference 1. This RAI addresses Other Seismic Category I Structures as discussed in Chapter 3 of the Final Safety Analysis Report (FSAR) and submitted in Part 2 of the Bell Bend Nuclear Power Plant (BBNPP) Combined License Application (COLA).

PPL provided a partial response and request for extension in Reference 2. This submittal transmits the responses to some of the remaining RAI questions and identifies additional questions impacted by the relocation of the plant physical siting.

The enclosure provides our response to Questions 03.08.04-1, 2, 4, 5, 6, 13 subpart (a), 15, 16, 17, 19, 22 and 25, which include revised COLA content. The revision of the COLA is the only new regulatory commitment.

The response to Question 03.08.04-15 addresses the question elements, with the exception of the discussion of the values of loads, strains, and other effects considered in the analyses demonstrating compliance with AREVA Topical Report ANP-10264(NP), due to the variables associated with plant footprint relocation.

As the staff is aware, PPL is revising the footprint of the proposed BBNPP within the existing project boundary. This relocation may change site-specific characteristics, such as Ground Motion Response Spectra (GMRS), ground water elevations, and soil properties.

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NRO

The following additional questions from RAI No. 79 are also impacted by the relocation of the plant footprint:

03.08.04-13 Subpart (b)
03.08.04-15 Discussion of the values of loads, strains, and other effects considered in the analyses demonstrating compliance with AREVA Topical Report ANP-10264(NP).

This re-location will result in supplemental COLA information being submitted to the NRC, and will include information necessary to address these questions regarding FSAR Section 3.8. PPL is currently in the process of updating the schedule information previously provided to the staff in Reference 3, and will update the staff upon completion.

Should you have questions or need additional information, please contact the undersigned at 570.802.8102.

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

Executed on April 15, 2010

Respectfully,

Rocco R. Sgarro

RRS/kw

Enclosure: As stated

cc: (w/o Enclosures)

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Enclosure

Response to NRC Request for Additional Information No. 79
Questions 03.08.04-1, 2, 4, 5, 6, 13 subpart (a), 15, 16, 17, 19, 22 and 25
Bell Bend Nuclear Power Plant

RAI 79
Question 03.08.04-1:

For COL information item COL 3.8(5) in the BBNPP COL FSAR, Subsection 3.8.4.1.8,"Buried Conduit and Duct Banks" (SRP Section 3.8.4), the COL applicant needs to provide a description of Seismic Category I buried conduit and duct banks.

"Please describe how these embedded structures are designed and demonstrate that the structures are safe under all combinations of applicable design loads."

Response:
This response is also provided as the response to RAI 79 Question 03.08.04-17 subquestion b1.

As cited in BBNPP FSAR Section 3.8.4.1.8, there are no Seismic Category I buried conduits outside of Seismic Category I buried duct banks for BBNPP. The buried structures are limited to electrical duct banks.

Seismic Category I duct banks involve the installation of multiple polyvinyl chloride (PVC) or steel electrical conduits encased in reinforced concrete. Their design is discussed in BBNPP FSAR Section 3.7.3.12 and Section 3.8.4. The Governing Codes & References for the design of structural concrete for duct banks is in accordance with IEEE 628-2001, ASCE 4-98, and ACI 349-01.

Design Loads and Load Combinations conform to BBNPP FSAR Section 3.8.4.4.5 and follow BBNPP FSAR Table 3E-1, in which Load Cases 1, 3, and 4 are applicable. As a buried structure, these load cases are simplified as follows:

a) $U = 1.4D + 1.4F + 1.7H$

b) $U = D + F + H + E_{ss}$

c) $U = D + F + H + W_t$

The applicable loads, D, F, H, E_{ss}, and W_t are defined in BBNPP FSAR Table 3E-1. As noted in BBNPP FSAR Section 3.8.4.4.5, duct banks are buried with a sufficient depth to mitigate the effects of surcharge loads and tornado or turbine generated missiles.

The duct bank is analyzed as a structural beam on an elastic foundation meeting the design requirements as specified in BBNPP FSAR Section 3.8.4.5. The structural acceptance criteria are in BBNPP FSAR 3.8.4.5 and the AREVA NP Topical Report ANP-10264(NP).

COLA Impact:

The BBNPP FSAR will be revised as follows:

3.8.4.3.2 Loading Combinations

{The ~~following additional~~ factored load combinations, which apply for reinforced concrete design of the ESWEMS Pumphouse, are provided in~~:~~ Table 3E-1 and Table 3E-2, including ~~provide~~ the description of the loading combinations and the minimum required Factor-of-Safety for building stability, respectively.}

3.8.4.3.3 Loading Combination for Buried Concrete Duct Banks

{Loading combinations for design of buried concrete duct banks include:

$U = 1.4D + 1.4F + 1.7H$

$U = D + F + H + E_{ss}$

$U = D + F + H + W_t$

The loading conditions are defined in FSAR Table 3E-1.}

RAI No. 79
Question 03.08.04-2:

For COL information item COL 3.8(5) in the BBNPP COL FSAR, Subsection 3.8.4.1.8, "Buried Conduit and Duct Banks" (SRP Section 3.8.4), the applicant states in the second paragraph (Page 3-179), in part, that "No Seismic Category I buried conduits outside of Seismic Category I buried duct banks exist for BBNPP."

Are there any seismic category II buried conduits? If yes, identify these conduits

Response

There are no buried Seismic Category II conduits for BBNPP.

COLA Impact

The COLA will not be revised as a result of this response.

RAI 79
Question 03.08.04-4:

For COL information item COL 3.8 (6) in the BBNPP COL FSAR, Subsection 3.8.4.1.9,"Buried Pipe and Pipe Ducts" (SRP Section 3.8.4), the last paragraph (Page 3-180) states that "Buried piping is buried directly in the soil (i.e., without concrete encasement) unless detailed analysis indicates that additional protection is required."

This sentence is confusing. The detailed analysis should have been performed and the decision on whether or not to use concrete encasement made and recorded in the BBNPP COL FSAR. Describe the detailed analysis and where it has been applied.

Response:

Seismic Category I buried piping is placed directly in the soil without concrete encasement. The direct buried piping is designed with appropriate wall thicknesses and buried at sufficient depth to allow direct placement without concrete encasement.

COLA Impact:

The BBNPP FSAR Subsection 3.8.4.1.9, Buried Pipe and Pipe Ducts, will be revised as follows:

Buried piping is buried directly in the soil ~~(i.e.,~~ without concrete encasement~~) unless detailed analysis indicates that additional protection is required.~~ The depth of the cover is sufficient to provide protection against frost, surcharge effects, and tornado missiles. Appropriate bedding material is provided beneath the pipe. Soil surrounding the pipe is typically compacted structural backfill. ~~As an alternate, concrete may be used as discussed in Section 3.7.3.12.~~}

RAI 79
Question 03.08.04-5:

For COL information item COL 3.8(6) in the BBNPP COL FSAR, Subsection 3.8.4.1.9,"Buried Pipe and Pipe Ducts" (SRP Section 3.8.4) , the applicant states in the last sentence of the last paragraph (Page 3-180) that "As an alternate, concrete may be used as discussed in Section 3.7.3.12."

In BBNPP FSAR Section 3.7.3.12,"Buried Seismic Category I Piping, Conduits, and Tunnels", the second paragraph in Page 3-51 states that "The effects of bends and differential displacement at connections to buildings are evaluated using equations for beams on elastic foundations, and subsequently combined with the buried pipe axial stress."

The applicant is requested to provide information on the following:
(a) How was the foundation modulus for beams on elastic foundations calculated or otherwise obtained?
(b) Provide details on the concrete enclosures, including steel reinforcing.
(c) Procedures for the Design of Restrained Underground Piping of ASME B31.1 provide guidance for the thermal loading only. Where is the guidance for the seismic analysis provided in ASME B31.1?

Response:

(a) This response is also provided in response to RAI 79 Question 03.08.04-16 subquestion (a).

The buried pipe is modeled as a beam on an elastic foundation. The soil vertical spring constant of the supporting soil, which is also called the modulus of subgrade reaction of the soil, or foundation modulus, is calculated by using equations as follows:

$$k_y = \frac{G}{(1-\nu)}\beta_y\sqrt{BL}\,\eta_y$$

β_y is determined using the ratio of L/B or B/L (Reference Design Chart in S.Arya, M O'Neil, and G. Pincus, 1981 & R.V. Whitman, 1972) or other publication in soil mechanics.

where: $\eta_y = 1 + 0.6(1-\nu)\dfrac{H}{R_o}$

In which:

R_o = Equivalent radius of the foundation, $R_o = \sqrt{\dfrac{BL}{\pi}}$

B = Width of the foundation, such as diameter of the pipe or width of duct bank
L = Length of the supporting soil –maximum $10B$
H = Embedded depth of the buried utility

v = Soil Poisson ratio, 0.37 (BBNPP FSAR Table 2.5-52)
G = Soil shear modulus, 710 ksf and 5180 ksf for static and dynamic conditions, respectively (BBNPP FSAR Table 2.5-51 and Table 2.5.52)
β_y = Coefficient as a function of the ratio of L/B

(b) There are no concrete enclosures, such as pipe ducts or tunnels for buried utilities, in the BBNPP design.

(c) There is no ASME B31.1 piping in Seismic Category I buried piping. The seismic analysis of restrained segments for site-specific Seismic Category I piping is done in accordance with guidance provided in Section NC-3650 of the ASME Boiler & Pressure Vessel Code, Section III. The BBNPP FSAR will be revised to indicate the applicable ASME code.

COLA Impact:

The BBNPP FSAR will be revised as follows:

3.7.3.12 Buried Seismic Category I Piping, Conduits, and Tunnels

Seismic analysis of restrained segments of <u>site-specific</u> buried pipe utilizes guidance provided in ~~Appendix VII, Procedures for the Design of Restrained Underground Piping, of ASME B31.1 2004 (ASME, 2004)~~ <u>Section NC-3650 of Rules for Construction of Nuclear Facility Components, ASME Boiler & Pressure Vessel Code, Section III (ASME, 2004).</u>

3.7.3.15 References

~~**ASME, 2004.** Procedures for the Design of Restrained Underground Piping, Appendix VII, Power Piping, ASME B31.1-2004, American Society of Mechanical Engineers, 2004.~~

<u>**ASME, 2004.** Rules for Construction of Nuclear Facility Components, ASME Boiler and Pressure Vessel Code, Section III, American Society of Mechanical Engineers, 2004.</u>

RAI 79
Question 03.08.04-6:

For COL information item COL 3.8(6) in the BBNPP COL FSAR, Subsection 3.8.4.1.9, "Buried Pipe and Pipe Ducts" (SRP Section 3.8.4) , the applicant states in the last sentence of the last paragraph (Page 3-180) that: "As an alternate, concrete may be used as discussed in Section 3.7.3.12."

In BBNPP COL FSAR Section 3.7.3.12, "Buried Seismic Category I Piping, Conduits, and Tunnels", the third and fourth paragraphs in Page 3-51 state that "For long straight sections of buried pipe, maximum axial strain and curvature are calculated per equations contained in ASCE 4-98 (ASCE, 1986). These equations reflect seismic wave propagation and incorporate the material's modulus of elasticity to determine the corresponding maximum axial and bending stresses. The procedure combines stresses from compression, shear and surface waves by the square root of the sum of the squares (SRSS) method. Maximum stresses for each wave type are then combined using the SRSS method. Subsequently, seismic stresses are combined with stresses from other loading conditions, e.g., long-term surcharge loading.

For straight sections of buried pipe, the transfer of axial strain from the soil to the buried structure is limited by the frictional resistance developed. Consequently, axial stresses may be reduced by consideration of such slippage effects, as appropriate."

The applicant is requested to provide information for the following:
(a) How is the apparent wave velocity determined when using the ASCE 4-98 equations to calculate the maximum axial strain and curvature? What wave impinging angle is assumed?
(b) Described how the long-term surcharge loading is considered, including magnitude of these loads
(c) Provide information that shows how the friction force is calculated /estimated in the axial strain calculation considering the slippage effect? What assumptions were made regarding the wave field, wave impinging angle, and apparent wavelength in the calculation/estimation for the friction force including the wave field, wave impinging angle, and apparent wavelength? Provide rationales for these assumptions.

Response:

(a1) Apparent Wave Velocity: In order to determine maximum axial strain and curvature on the buried utilities, Equations 3.5-1 and 3.5-3 of ASCE 4-98 use apparent wave velocity, "c." ASCE 4-98 does not specify how to calculate the apparent wave velocities for different seismic phases. However, the code commentary, Part C3.5.2.1, provides a commentary on estimating the apparent velocity to determine the maximum axial strain and curvature. The commentary refers specifically to Equations 3.5-1 and 3.5-2 for estimating maximum axial strain, but also applies to Equation 3.5-3. The code commentary states in part:

> "The major difficulty in applying Equations 3.5-1 and 3.5-2 is in estimating the appropriate wave types and the apparent propagation velocities, "c," which are related to the compressional, shear, and Rayleigh-wave velocities. The peak ground acceleration and peak ground velocity are composed of a mixture of these wave types, and their apparent wave velocities are a function of their travel path through the deeper and higher-velocity material."

This statement indicates the apparent wave velocity includes deeper and higher-velocity material than the material at the elevation of the buried utilities for the BBNPP.

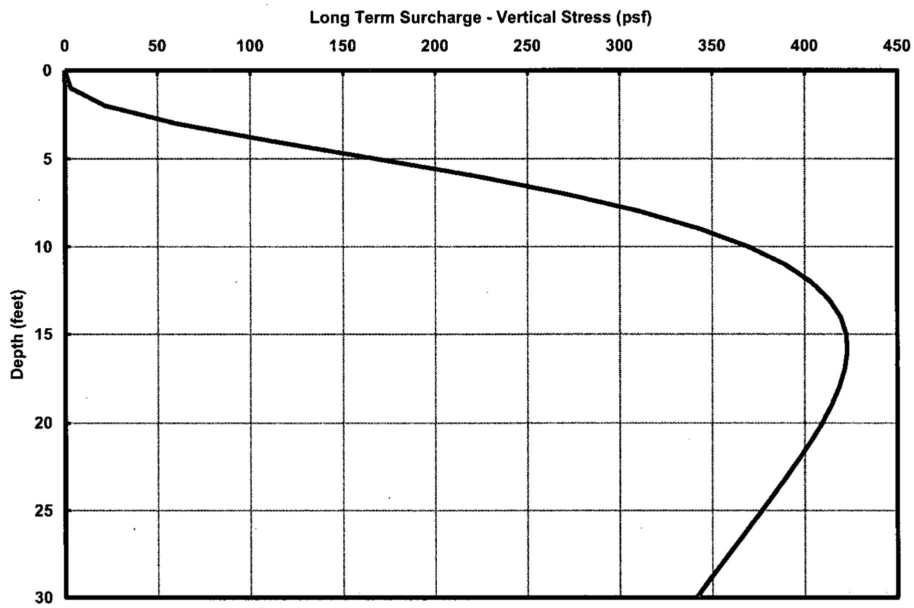
Based on BBNPP FSAR Section 2.5.2.5.1.6, the apparent wave velocities for shear waves (S-wave) are considered for the BBNPP site since "vertically-propagating shear waves are the dominant contributor to site response". The BBNPP FSAR does not address surface waves.

Section 3.10.3.1 of AREVA NP Topical Report ANP-10264(NP) defines the apparent wave velocity, "c", and the maximum ground velocity, "V", consistent with ASCE 4-98.

BBNPP FSAR Table 2.5-52 presented seismic velocities (V_s and V_p) for Glacial Overburden 1 - where the buried utilities are embedded - equal to 1150 feet per second (fps) and 2550 fps for shear waves and compressional waves, respectively. However, the seismic velocities presented in BBNPP FSAR Table 2.5-52 (V_s and V_p) are not apparent wave velocities (c_s and c_p). As quoted above, the ASCE 4-98 commentary notes that "c" is a function of deeper and higher-velocity material. Hence, it is not solely dependent on the seismic velocity in the surficial layer that contains the buried utility. The code commentary, Part C3.5.2.1, also states that "The use of wave velocities less than 3,000 feet per second (fps) (900 m/s) would generally be inappropriate". Therefore, a conservative apparent wave velocity of 3,000 fps is used for a shear wave. The apparent wave velocity for a compressional wave is estimated as 5,250 fps. This value is based on the conservative apparent wave velocity for shear waves of 3,000 fps recommended in ASCE 4-98, and the measured minimum ratio of 1.76 from V_p to V_s values presented in BBNPP FSAR Table 2.5-52. These apparent wave velocities are used to determine maximum axial strain and curvature for buried utilities.

(a2) Impinging Angle: ASCE 4-98 addresses impinging angle using the wave-velocity coefficients for different seismic waves (ASCE 4-98 Table 3.5-1). The wave-velocity coefficients maximize the axial strain and curvature by assuming the critical angle of incidence between the longitudinal axis of the buried utility and the seismic wave. An angle of 45 degrees produces the maximum axial strain due to shear-wave propagation and is factored into the wave-velocity coefficient of 2.0 presented in ASCE 4-98.

(b) Long Term Surcharge: Long term surcharge from shallow foundations adjacent to buried pipes is calculated using Boussinesq's method or Westergaard's method of stress distribution, and/or Slope Method. Applicable Poisson's ratios, which ranged from 0.35 to 0.40 per BBNPP FSAR Table 2.5-5, excluding the value for concrete fill, which is not used at BBNPP, are included in the Westergaard's equation. For example, given a 10 foot wide strip foundation at the ground surface, with a contact pressure of 2,000 pounds per square foot (psf), and the point for which the surcharge load is calculated being 5 feet outside the edge of the strip footing, long term surcharge is calculated based on the methodology in Poulos and Davis (1974), as illustrated below:



(c1) Friction Force: The maximum friction force per unit length of pipe is in accordance with "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems", prepared by the Committee on Gas and Liquid Fuel Lifelines of the ASCE Technical Council on Lifeline Earthquake Engineering, 1984. The axial soil frictional resistance along the pipe is a bilinear spring. For example:

In Clays:

$$tu = \pi D \alpha Su$$

where:

tu = ultimate axial soil frictional load per unit pipe length
D = pipe outside diameter
α = adhesion coefficient based on pile shaft load transfer theory.
Su = clay undrained shear strength

In Sands:

$$tu = ((\pi D)/2)\gamma H(1 + K_o)$$

where:

tu = ultimate axial soil frictional load per unit pipe length
D = pipe outside diameter
H = depth from ground surface to pipeline center
K_o = coefficient of soil pressure at rest
γ = effective soil unit weight

Generally, the basis for determining the maximum friction force per unit length of pipe comes from similar considerations for unit skin friction, or shaft resistance, for piles. For cohesive soils, undrained loading is assumed, and the alpha method used to estimate the friction force based on the undrained soil shear strength. For granular materials, assuming drained loadings, the maximum friction force will be developed based on overburden stresses, the coefficient of earth pressure at rest, and the interface friction angle between soil and pipe.

The friction force per unit length of pipe is as follows:

$tu = \pi D p C_f$

where:

tu = ultimate axial soil frictional load per unit pipe length
D = pipe outside diameter
p = average radial soil pressure on pipe (calculated by $p = 0.5\ (1+K_o)\ \gamma H$)
K_o = coefficient of soil pressure at rest
γ = effective soil unit weight
H = depth from ground surface to pipeline center
C_f = coefficient of friction for smooth pipe embedded in soil

BBNPP FSAR Table 2.5-57 specifies K_o = 0.43 for Glacial Overburden and Granular Fill/Backfill; and γ= 140 pounds per cubic foot (pcf) per BBNPP FSAR Subsection 2.5.4.10.2.1.1 for the unit weight of backfill. The coefficient of friction for a smooth pipe embedded in soil ranges from 0.3 to 0.5. An average coefficient of friction of 0.4 on buried utilities is assumed.

(c2) Slippage Effect: : The upper bound of maximum axial strain when considering potential slippage between the soil and the pipe is given by Equation 3.5-1 of ASCE 4-98. In "Seismic Response of Buried Pipes and Structural Components", a report prepared by the Committee on Seismic Analysis of the ASCE Structural Division Committee on Nuclear Structures and Materials, the following statement is made:

> "[T]he upper bound axial force for a straight pipe is equal to the friction force per unit length times one quarter of the wave length (Sakurai and Takahashi, 1969)".

This statement acknowledges that the maximum force transferred by friction from the soil to the pipe occurs when ground strain is maximized. The maximum ground strain that can develop occurs over one-quarter wavelength of the predominant sinusoidal seismic wave. ASCE 4-98 has used this one-quarter wavelength criterion in Equation 3.5-2. As a result of slippage between the soil and buried utility, the calculated maximum axial strain on a buried utility is the lesser of ASCE 4-98, Equations 3.5-1 or 3.5-2, as included in the code commentary. In general, the maximum strain per Equation 3.5-2 is less than those calculated using Equation 3.5-1. The BBNPP FSAR 3.7.3.12 states:

> "Consequently, axial stresses may be reduced by consideration of such slippage effects, as appropriate".

The AREVA NP Topical Report ANP-10264(NP) includes ASCE 4-98, Equations 3.5-1 and 3.5-3, respectively, with the same criterion.

(c3) Assumptions regarding the wave field and wave impinging angle are described in the response in Item (a1) and (a2).

(c4) Apparent Wavelength: The wavelength, which is used in Equation 3.5-2 of ASCE 4-98, is the apparent wavelength of the dominant seismic wave associated with peak ground velocity. BBNPP FSAR Section 2.5.2.5.1.6 states the dominant seismic wave is the shear wave. Its wavelength is estimated using the following relationship: $\lambda_w = \dfrac{c}{f}$

Where:
c = apparent wave velocity of the dominant seismic wave
f = frequency in Hz at which the peak ground velocity occurs

COLA Impact:
The BBNPP FSAR will not be revised as a result of this response.

RAI 79
Question 03.08.04-13:

For COL information items COL 3.8(8) through 3.8(10) in the BBNPP COL FSAR, Subsection 3.8.4.4.5, "Buried Conduit and Duct Banks, and Buried Pipe Ducts" (SRP Section 3.8.4), the fourth paragraph (BBNPP COL FSAR Page 3-184) states that "Soil overburden pressures on buried duct banks typically do not induce significant bending or shear effects, because the soil cover and elastic support below the duct bank are considered effective and uniform over the entire length of the buried duct bank. When this is not the case, vertical soil overburden pressure is determined by the Boussinesq method."

The applicant is requested to provide the following information:

a) At the BBNPP site, are there any locations of buried utilities such that the vertical soil overburden pressure needs to be computed by the Boussinesq method? Describe what level of non-uniformity of the soil support makes use of the Boussinesq analysis necessary.
b) Describe how the Boussinesq method is applied to the soil conditions for the BBNPP.

Explain the meaning of "effective and uniform" in "the soil cover and elastic support below the duct bank are considered effective and uniform over the entire length of the buried bank." How is it determined that the soil cover and elastic support are "effective and uniform"?

Response:

a) At the BBNPP site, there are no locations of buried utilities such that the vertical soil overburden pressure needs to be computed using the Boussinesq method. The buried duct bank is placed in the trench before backfill with soil cover. The soil below the duct bank acts as an elastic foundation, which provides continuous support for the duct bank. The electrical duct bank has relatively wide and shallow dimensions. As a result, the soil cover and soil below the duct bank are considered uniform and effective over the entire length and width of the duct bank. This assumption facilitates the evaluation of vertical soil overburden pressure acting on the buried duct bank. The pressure is determined in accordance with Section 3.10.1.3 of the AREVA Topical Report ANP-10264(NP).

b) The response to this part of the RAI is impacted by the relocation of the BBNPP footprint.

COLA Impact:

The BBNPP FSAR will not be revised as a result of this response:

RAI 79
Question 03.08.04-15:

For COL information items COL 3.8(8) through 3.8 (10) in the BBNPP COL FSAR, Subsection 3.8.4.4.5,"Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts" (SRP Section 3.8.4) , the 2nd paragraph (Page 3-183), states: "The design of Seismic Category I, buried electrical duct banks and buried Essential Service Water pipes (hereafter in this section referred to as buried duct banks and buried pipe) has been confirmed to meet the requirements specified in Section 3.8.4.4.5 and the AREVA NP Topical Report ANP-10264(NP) and demonstrates sufficient strength to accommodate:

• Strains imposed by seismic ground motion.

• Static surface surcharge loads due to vehicular loads (AASHTO HS-20 (AASHTO, 2002)) truck loading, minimum, or other vehicular loads, (including during construction) on designated haul routes.

• Static surface surcharge loads during construction activities, e.g., for equipment laydown or material laydown.

• Tornado missiles and, within their zone of influence, turbine generated missiles.

• Ground water effects."

The applicant is requested to provide a description of the analyses and evaluations performed to support the conclusion that the design of the BBNNP buried utilities meets AREVA Topical Report ANP-10264NP and demonstrates sufficient strength to accommodate the loads and other effects listed above. Include in this discussion the values of loads, strains, and other effects considered in the analyses. The discussion should also describe measures taken to mitigate potential damaging effects of ground water entering the buried utility enclosures.

Response:

The discussion of values of loads, strains, and other effects considered in the analyses demonstrating compliance with AREVA Topical Report ANP-10264(NP) is impacted by the relocation of the BBNPP footprint. The remainder of the response is provided, below:

- <u>Strains Imposed by Seismic Ground Motion:</u>

Axial strains on buried duct banks are estimated by ASCE 4-98, Equation 3.5-1 or 3.5-2, whichever is less, per Commentary C.5.2.1 in the code.

Axial strain on buried pipe is addressed in AREVA Topical Report ANP-10264(NP). The equation is consistent with ASCE 4-98 Equation 3.5-1.

- Static Surface Surcharges:

Section 3.10.1.4, "Surface Loads" in the AREVA Topical Report ANP-10264(NP) provides the method for evaluating pressure transmitted to the buried pipe and electrical duct banks under live loads, such as those imposed by trucks or rail. AREVA Topical Report Table 3-6 lists values of surface loads transmitted to the pipe due to vehicular loads (AASHTO HS-20 (AASHTO, 2002)) and Railway (E80), including impact factors based on AREVA Topical Report Table 3.5. The tabulated values are applied to construction equipment or other construction conditions, as appropriate.

- Tornado Missiles and, within their zone of influence, Turbine Generated Missiles.

In accordance with Section 6.3.6, "Tornado (W_T) Loads", of the AREVA Topical Report ANP-10264(NP), the W_T load only applies to an exposed piping. In accordance with the U.S. EPR FSAR Section 3.5.2 safety-related buried utilities are protected from external generated missiles. Section 3.5.2 states:

> "Safety-related pipes and cables routed outside of missile-protected structures are buried a sufficient depth to provide protection for these items from missile impact."

The externally generated missiles for which the U.S. EPR is designed are addressed in the U.S. EPR FSAR Section 3.5.1. The depth of missile penetration though soil is evaluated, using formulas in the U.S. EPR FSAR Section 3.5.3.1.1. Alternatively, its evaluation may be based on Young's Method per "Young W.C., Depth Prediction for Earth-Penetrating Projectiles", by ASCE Journal of Solid Mechanics and Foundations Division, May 1969. The design soil cover to preclude missile impact on buried structures is conservatively determined for exceeding the predicted penetrating soil depth by a minimum of 20 percent. Lastly, buried concrete missile barriers with a minimum wall thickness of 18.2 inches based on concrete strength of 3,000 psi is considered adequate, ignoring the soil protection – Refer to the U.S. EPR FSAR Table 3.5-2.

- Ground Water Effect:

For utilities buried below the groundwater table, vertical force due to buoyancy is evaluated in accordance with Section 3.10.1.5, "Buoyancy Force" of the AREVA Topical Report ANP-10264(NP). Since safety-related utilities are buried with sufficient soil protection from the design external generated missiles, such deep cover mitigates flotation effects. As delineated in "Buried Pipe Design" by A.P. Moser, McGraw-Hill, Third Edition, the buoyant force of the utility cannot exceed its weight and the effective weight of the soil wedge, or anchorages would be designed to resist flotation. The minimum required Factor-of-Safety (FS) against flotation is taken at 1.5 for normal loading conditions, 1.3 under severe environmental conditions, and 1.1 under construction conditions. Potential intrusion of ground water into duct banks is mitigated using a waterproofing system and sloping the duct bank toward manholes, as necessary.

COLA Impact:

The BBNPP FSAR will be revised as follows:

3.8.4.4.5 Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts

The design of Seismic Category I~~,~~ buried electrical duct banks and buried ~~Essential Service Water~~ safety-related pipes (~~hereafter in this section~~ referred to as buried duct banks and buried pipe in this section) ~~has been confirmed to~~ meet the requirements specified in Section 3.8.4.4.5 and the AREVA NP Topical Report ANP-10264(NP) and demonstrates sufficient strength to accommodate:

RAI 79
Question 03.08.04-16:

For COL information items COL 3.8 (8) through 3.8 (10) in the BBNPP COL FSAR, Subsection 3.8.4.4.5,"Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts" (SRP Section 3.8.4) , the ninth paragraph (Page 3-184) states: "Bending stresses in buried pipe due to surcharge loading are determined via manual calculations, treating the flexible pipe as a beam on an elastic foundation. Resulting stresses are combined with operational stresses, as appropriate."

The applicant is requested to provide the following information:
(a) Describe the manual calculations performed, including assumptions made, and soil properties and spring constants used.
(b) Explain how the bending stresses are calculated?
(c) What is the modulus of subgrade reaction used for the analysis? Provide justification for using that value.
(d) Is the soil overburden included in the analysis?

Response:

(a) The buried pipe is modeled as a beam on an elastic foundation. The soil vertical spring constant of the supporting soil, which is also called as the modulus of subgrade reaction of the soil, or foundation modulus, is calculated by using equations as follows:

$$k_y = \frac{G}{(1-\nu)}\beta_y\sqrt{BL}\,\eta_y$$

β_y is determined using the ratio of B:L (Reference Design Chart in S.Arya, M O'Neil, and G. Pincus, 1981 & R.V. Whitman, 1972) or other publication in soil mechanics.

where: $\eta_y = 1 + 0.6(1-\nu)\dfrac{H}{R_o}$

In which:

R_o = Equivalent radius of the foundation, $R_o = \sqrt{\dfrac{BL}{\pi}}$

B = Width of the foundation, such as diameter of the pipe or width of duct bank
L = Length of the supporting soil –maximum 10B
H = Embedded depth of the buried utility
ν = Soil Poisson ratio
G = Soil shear modulus
β_y = Coefficient as a function of the ratio of L/B

(b) Bending stress on the buried pipe due to pressure loadings, such as pipe dead weight, soil overburden, and surface loads uniformly applied on the buried pipe, is calculated using the

Winkler Model of a beam on an elastic foundation, where the behavior of the soil is simplified by means of fictitious springs placed continuously underneath the pipe. The modulus of subgrade reaction of the soil, or foundation modulus, is as calculated above. Although the value of the foundation modulus can be varied and unique for a given type of soil, the above equation provides classical values, which have been used in many references for structural analysis.

The buried pipe is idealized as a beam with distributed load over its length and supported by a continuous soil medium. The method of evaluation uses the principal of superposition of loaded beam-on-springs plus reaction from springs on a fixed support. The Winkler model presumed a linear force-deflection relationship and the contact between the beam and foundation. The governing equations for a uniformly loaded beam-on-Winkler foundation are as follows:

$$q = EI\frac{d^4w}{dx^4} + K_y v$$

in which: w = deflection of the beam
v = soil Poisson Ratio
Ky = modulus of subgrade reaction
$\frac{d^4w}{dx^4}$ =4th order differential equation

By introducing a parameter, $\beta = (\frac{K_y}{4EI})^{0.25}$

Where: E = Young's modulus of pipe
I = moment of inertia of pipe

Solutions of the governing equation, such as deflection and corresponding bending moment and shear, can be found by applying boundary conditions. Many text books provide tabulated values for derivative factors, which are needed for solutions, such as "Advanced Soil Mechanics of Materials" by Boresi, Arthur P., and Schmidt, Richard, J., 2003. When the bending moment is known, then the pipe bending stress can be calculated.

(c) Modulus of subgrade reaction, K_y is calculated using above equations, in which needed parameters for evaluation have been defined.

(d) Soil overburden is included in the analysis as one of the dead weight per Section 3.10.1.2 of the AREVA Topical Report ANP-10264(NP). It is evaluated in accordance with Section 3.10.1.3 of the report.

COLA Impact:

The BBNPP FSAR will not be revised as a result of this response.

RAI 79
Question 03.08.04-17:

For COL information items COL 3.8 (8) through 3.8(10) in the BBNPP COL FSAR, Subsection 3.8.4.4.5,"Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts" (SRP Section 3.8.4) , the fourth paragraph (Page 3-184) states: "The seismic design of buried duct banks and buried pipe is discussed in Section 3.7.3. Other loads are addressed in this section, but are combined with seismic effects of the aforementioned section."

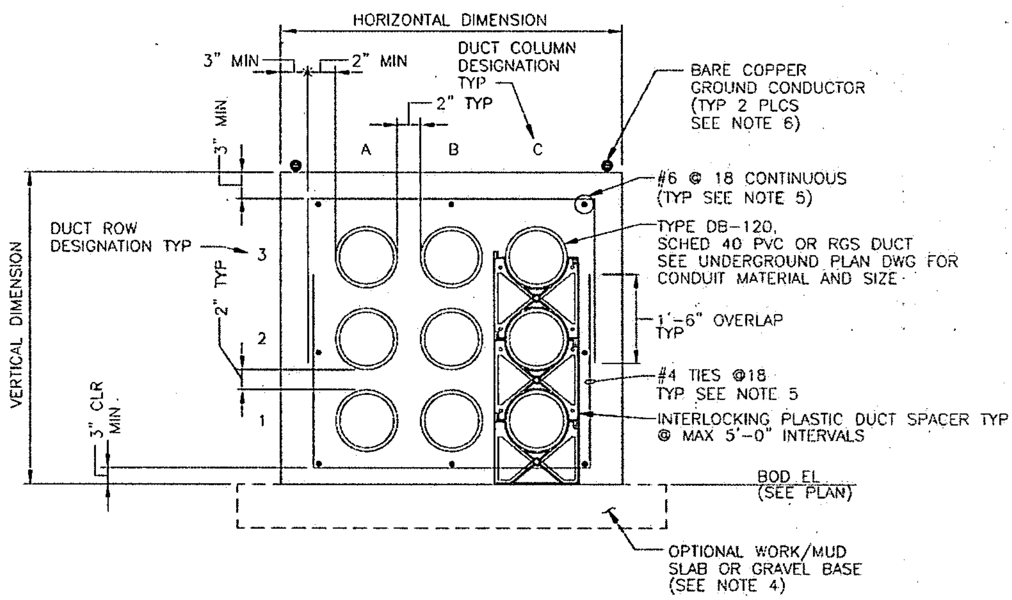
The applicant is requested to provide the following information:

(a) Is the cross section of the buried duct bank a rectangular shape?
(b) Provide details describing the design of the concrete duct bank enclosures.

The material presented in BBNPP FSAR Section 3.7.3 did not discuss whether racking of the cross section due to the SSE was considered and how the design accommodates this racking. Provide a technical rationale for not considering this effect.

Response:

(a) As shown in the PPL response NRC RAI 79, Question 03.08.04-3, in letter BNP-2010-085, dated March 19, 2010 (ML100830214), Item (a), the typical cross-section of the buried duct bank is a rectangular shape. The encasement cross section, including its reinforcement is shown below:



TYPICAL CONCRETE ENCASED
DUCT BANK CONSTRUCTION

(b, part 1) This response was also provided as the response to RAI 79 Question 03.08.04-1.

As cited in BBNPP FSAR Section 3.8.4.1.8, there are no Seismic Category I buried conduits outside of Seismic Category I buried duct banks for BBNPP. The buried structures are limited to electrical duct banks.

Seismic Category I duct banks involve the installation of multiple polyvinyl chloride (PVC) or steel electrical conduits encased in reinforced concrete. Their design is discussed in BBNPP FSAR Section 3.7.3.12 and Section 3.8.4. The Governing Codes & References for the design of structural concrete for duct banks is in accordance with IEEE 628-2001, ASCE 4-98, and ACI 349-01.

Design Loads and Load Combinations conform to BBNPP FSAR Section 3.8.4.4.5 and follow BBNPP FSAR Table 3E-1, in which Load Cases 1, 3, and 4 are applicable. As a buried structure, these load cases are simplified as follows:

a) $U = 1.4D + 1.4F + 1.7H$

b) $U = D + F + H + E_{ss}$

c) $U = D + F + H + W_t$

The applicable loads, D, F, H, E_{ss}, and W_t are defined in BBNPP FSAR Table 3E-1. As noted in BBNPP FSAR Section 3.8.4.4.5, duct banks are buried with a sufficient depth to mitigate the effects of surcharge loads and tornado or turbine generated missiles.

The duct bank is analyzed as a structural beam on an elastic foundation meeting the design requirements as specified in BBNPP FSAR Section 3.8.4.5. The structural acceptance criteria are in BBNPP FSAR 3.8.4.5 and the AREVA NP Topical Report ANP-10264(NP).

(b, part 2) Evaluation identified that no significant racking of the duct bank cross section occurs during SSE loading conditions, based on the following:

The buried duct bank is placed on the bottom of the trench before backfilling the trench with soil cover. The soil below the duct bank acts as elastic foundation, providing continual support for the duct bank. The electrical duct bank is relatively wide and shallow in dimensions. As a result, the soil cover or overburden is considered uniform and effective over the entire width and length of the duct bank. During a Safe-Shutdown Earthquake (SSE) event, vertical inertia forces act on the beam at center of the duct bank. However, if a large force is applied off center of the beam, then racking will occur. Should significant torsion occur, it could cause the duct bank to twist and crack. If this were to happen, the duct bank may not retain original the rectangular shape and become a more skewed parallelogram.

- In the "Design of Reinforced Concrete" by Jack C. McCormac, Harper & Row Publishers, 1978, torsional stress will add to the stresses caused by shear on one side and subtract from them on the other. For a member with solid cross section, such as rectangular duct bank, if the torsional stress is less than $1.5\sqrt{f'_c}$, it will not appreciably reduce either the shear or the flexural strengths of reinforced concrete members. The stress value is roughly equal to about one fourth of the torsional strength of the member without concrete reinforcement. Hence, limit of torsional moment, $T_u = 0.5\phi\sqrt{f'_c}\sum x^2 y$, where x= short size of duct bank; y =long side of duct bank, f'_c = concrete compressive strength; and ϕ=0.85. Given: f'_c = 4,000 psi, the limits of torsional moments are calculated and tabulated in Table 1, below for the following representative duct bank sections.

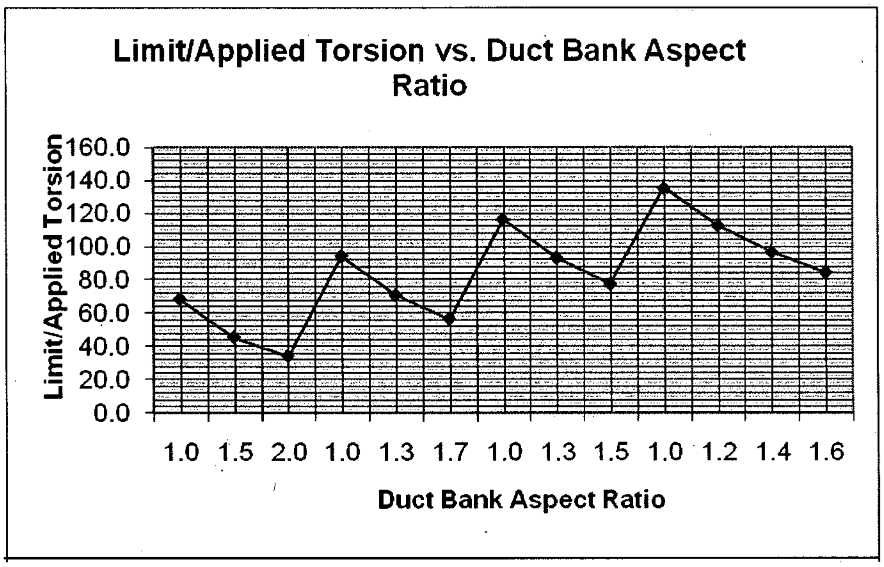
- In reference to BBNPP FSAR Figure 3.7-152, the peak ground vertical acceleration is about 0.33g, or 1.33g, including gravity. Soil density is 140 pcf (BBNPP FSAR Section 2.5.4.10.2.1.1) and concrete density is 150 pcf. The pressure load is determined in accordance with Section 3.10.1.3 of the AREVA Topical Report ANP-10264(NP). At a 10 ft buried depth and conservatively applying a 5% accidental eccentricity of the SSE loadings that are the same as for building design (Ref. NUREG-0800, Standard Review Plan 3.7.2, Acceptance Criterion 11), torsional moments applied on the duct beam, including the ratios between the limit torsional moments (at which the torsion has no effect on the duct bank section) and applied torsional moments due to SSE loadings are provided in Table 1, and graphically illustrated in Chart A:

Table 1

Duct Bank Height, x (in)	24	24	24	36	36	36	48	48	48	60	60	60	60
Duct Bank Width, y (in)	24	36	48	36	48	60	48	60	72	60	72	84	96
Limit Tors. Mom.,Tu (ft.kip)	31	46	62	105	139	174	248	310	372	484	581	677	774
Overburden Weight (kips)	2.8	4.2	5.6	4.2	5.6	7.0	5.6	7.0	8.4	7.0	8.4	9.8	11.2
Self Weight (kips)	0.6	0.9	1.2	1.4	1.8	2.3	2.4	3.0	3.6	3.8	4.5	5.3	6.0
Total Weight (kips)	3.4	5.1	6.8	5.6	7.4	9.3	8.0	10.0	12.0	10.8	12.9	15.1	17.2
Vertical Acceleration (g)	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
SSE Load (kips)	4.5	6.8	9.0	7.4	9.8	12.3	10.6	13.3	16.0	14.3	17.2	20.0	22.9
Tors. Eccentricity (ft)	0.1	0.2	0.2	0.2	0.2	0.3	0.2	0.3	0.3	0.3	0.3	0.4	0.4
Applied Tors. Mom. (ft.kip)	0.45	1.02	1.81	1.11	1.97	3.08	2.13	3.33	4.79	3.57	5.15	7.01	9.15
Ratio: Limit/Applied Tors.	**68.5**	**45.7**	**34.2**	**94.4**	**70.8**	**56.6**	**116.4**	**93.1**	**77.6**	**135.4**	**112.8**	**96.7**	**84.6**

Ratio: Width/Height	1.0	1.5	2.0	1.0	1.3	1.7	1.0	1.3	1.5	1.0	1.2	1.4	1.6
Ratio: Limit/Applied	68.5	45.7	34.2	94.4	70.8	56.6	116.4	93.1	77.6	135.4	112.8	96.7	84.6

Chart A



The cited conservatisms include:

- Not taking credit for the continuous support from surrounding soil and treating the duct bank as a structural beam subjected to an applied torsion;
- Inertia loadings due to overburden and self weight of the duct bank are applied at 5% accidental eccentricity in order to cause maximum torsional moments.

The evaluation demonstrates a minimum ratio between the torsional limit and the applied torsion to be 34.2 at the aspect ratio of 2. This value represents a significant design margin against the conservatively applied torsion and potential racking effects. It is concluded that there will be no significant racking effect during an SSE loading condition.

COLA Impact:

The BBNPP FSAR will be revised as follows:

3.8.4.3.2 Loading Combinations

{The ~~following additional~~ factored load combinations, which apply for reinforced concrete design of the ESWEMS Pumphouse, are provided in~~:~~ Table 3E-1 and Table 3E-2, including ~~provide~~ the description of the loading combinations and the minimum required Factor-of-Safety for building stability, respectively.}

3.8.4.3.3 Loading Combination for Buried Concrete Duct Banks

{Loading combinations for design of buried concrete duct banks include:

$U = 1.4D + 1.4F + 1.7H$

$U = D + F + H + E_{ss}$

$U = D + F + H + W_t$

The loading conditions are defined in FSAR Table 3E-1.}

RAI 79
Question 03.08.04-19:

For supplemental information item SUP 3.8 (2) in the BBNPP COL FSAR, Subsection 3.8.4.4.7, "ESWEMS Pumphouse and ESWEMS Retention Pond" (SRP Section 3.8.4), the applicant states "Local stress analyses was used to evaluate slabs and walls to resist external hazards (such as, tornado generated missile impact and water wave induced forces)."

The applicant is requested to describe the local stress analyses conducted, including assumptions made and material properties used.

Response:

Local stress analyses were performed for the ESWEMS Pumphouse pumpwell wall facing the ESWEMS Retention Pond, which is subject to tornado generated missile impact loads, as well as the hydrostatic pressure resulting from the probable maximum flood (PMF) plus water wave run-up and wind set-up waves. The other external walls and slabs of the ESWEMS Pumphouse are constructed from similar materials and have the same dimensions or larger; therefore, the pumpwell wall is the bounding critical case. The local stress analysis for the wall is based on maximum applied pressure of 590 pounds per square foot (psf) due to a rising wave of 4.4 ft, as indicated in BBNPP FSAR Section 3E-4. The calculated local stress is 267 pounds per square inch (psi). The ESWEMS Pumphouse is designed to withstand the tornado missile spectrum identified in Regulatory Guide 1.76 (NRC, 2007), as described in BBNPP FSAR Chapter 3.5.1.4. The ESWEMS Pumphouse is designed to the same tornado-generated missile impact resistance for both penetration and structural response as that identified as needed for protection in the U.S EPR FSAR, Table 3.5-2.

The materials for construction of the ESWEMS Pumphouse are noted in BBNPP FSAR Section 3.8.4.1, "Materials". The ESWEMS Pumphouse concrete is mix Class I with a 28-day compressive strength of 5,000 psi. The Pumphouse reinforcement is deformed billet steel conforming to ASTM A615, Grade 60, with a minimum yield strength of 60 ksi.

COLA Impact:

The BBNPP FSAR will not be revised as a result of this response.

RAI 79
Question 03.08.04-22:

For COL information item COL 3.8 (11) in the BBNPP COL FSAR, Subsection 3.8.4.5, "Structural Acceptance Criteria" (SRP Section 3.8.4), the applicant states in the 4th paragraph (Page 3-186), in part: "When allowable stresses are exceeded, joints are added as required to increase flexibility and hence, to mitigate member stresses."

The applicant is requested to provide the following information:

(a) Describe the nature of the joints that are added to alleviate excessive stresses and to describe the process for determining their location.
(b) Describe the design of these added joints, and explain how their design provides adequate resistance to SSE, and other loads.

Response:

(a) When slippage occurs in a short and straight buried pipe, whose length is less than ¼ the seismic apparent wavelength, Equation 3.5-2 of ASCE 4-98 results in less axial strains on the pipe. Therefore, when calculated allowable stresses are exceeded, the pipe should be rerouted, using multiple short legs connected by elbows. The elbows are considered as "added joints' to increase the pipe flexibility and hence reduce pipe stresses. Locations will be determined during detailed design.

(b) The effect of forces on bends, intersections, and anchor points are evaluated in accordance with ASCE 4-98, Part 3.5.2.2, including anchor point movement, which is evaluated in accordance with ASCE 4-98, Part 3.5.2.3. A report on "Seismic Response of Buried Pipes and Structural Components", prepared by the Committee on Seismic Analysis of the ASCE Structural Division Committee on Nuclear Structures and Materials, has an illustrated example in Appendix 3 of the report, including the process and methodology to evaluate pipe strains and stresses based on multiple short leg runs with flexible and/or inflexible elbows to reduce the pipe stresses.

COLA Impact:

The BBNPP FSAR will be revised as follows:

3.8.4.5 Structural Acceptance Criteria

{Acceptance criteria for the buried ~~Essential Service Water System~~ safety-related pipes are identical to those of non-buried pipe. The effect of forces on bends, intersections, and anchor points on site-specific safety-related pipes are evaluated in accordance with ASCE 4-98 (ASCE, 2000), including anchor point movement. Member stresses are maintained lower than allowable stresses. When allowable stresses are exceeded, pipe is rerouted, using multiple short legs connecting by elbows. ~~joints are added as required to increase flexibility and hence, to mitigate member stresses.~~ The elbows are considered as "added joints' to increase the pipe flexibility, reducing pipe stresses.

RAI 79
Question 03.08.04-25:

In BBNPP COL FSAR, Section 3E.4, "ESWEMS PUMPHOUSE AND ESWEMS RETENTION POND" (SRP Section 3.8.4), the first and second paragraphs under the title of "Design and Stress Analysis Using GT STRUDL Response Spectrum Method in Page 3E-5 states that "SSE accelerations are applied to dead load, equipment load (e.g., electrical/HVAC equipment, mechanical pumps, etc), 25 percent of the design live load, and minimum 75 percent of the design snow load.

The hydrostatic and hydrodynamic probable maximum precipitation (PMP) pressures are applied to walls and slabs of the ESWEMS pumpwells structural finite element model and consist of ..."

The applicant is requested to provide information for the following:
(a) How is the SSE load specified in the analysis, time histories or response spectra? Is the effect of SSI included?
(b) How is the hydrodynamic probable maximum precipitation (PMP) pressure calculated? How is it modeled in the response spectrum analysis?

Response:

(a) The Response Spectrum Method applies the inertia loads in the three directions of earthquake motion. The dynamic analysis did not include water pressure and dynamic wave effect because they were accounted for in a separate local stress evaluation. The 3 D FEM Model is based on fixed supports; as such, the effect of Soil Structure Interaction (SSI) is not included. Even though the response spectrum analysis does not include SSI, the analysis was compared with a model that includes the SSI effects in BBNPP FSAR Section 3.7. The result analysis from the model used for the response spectrum analysis was considered conservative.

(b) As cited above, PMP pressure was not modeled in the Response Spectrum analysis. The maximum pressure due to PMP was taken at 590 psf (28 kPa) (Ref. BBNPP FSAR Section 3E.4), as a static pressure for the local stress calculation on the critical water-facing pump well wall.

COLA Impact:

The BBNPP FSAR will not be revised as a result of this response.