Progress Energy

Serial: NPD-NRC-2009-104 10 CFR 52.79
June 9, 2009

U.S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555-0001

LEVY COUNTY NUCLEAR POWER PLANT, UNITS 1 AND 2
DOCKET NOS. 52-029 AND 52-030
RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER NO. 030 RELATED TO STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

Reference: Letter from Brian C. Anderson (NRC) to Garry Miller (PEF), dated May 8, 2009, "Request for Additional Information Letter No. 030 Related to SRP Section 2.5.4 for the Levy County Nuclear Plant Units 1 and 2 Combined License Application"

Ladies and Gentlemen:

Progress Energy Florida, Inc. (PEF) hereby submits our response to the Nuclear Regulatory Commission's (NRC) request for additional information provided in the referenced letter.

A partial response to the NRC request is addressed in the enclosure. The enclosure also identifies changes that will be made in a future revision of the Levy County Nuclear Power Plant Units 1 and 2 application.

If you have any further questions, or need additional information, please contact Bob Kitchen at (919) 546-6992, or me at (919) 546-6107.

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

Executed on June 9, 2009.

Sincerely,

Garry D. Miller
General Manager
Nuclear Plant Development

Enclosure

cc: U.S. NRC Region II, Regional Administrator
Mr. Brian Anderson, U.S. NRC Project Manager

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NRO

Levy Nuclear Power Plant Units 1 and 2
Response to NRC Request for Additional Information Letter No. 030 Related to SRP Section 2.5.4 for the Combined License Application, dated May 8, 2009

NRC RAI #	Progress Energy RAI #	Progress Energy Response
02.05.04-15	L-0198	Future submittal
02.05.04-16	L-0199	Response enclosed – see following pages
02.05.04-17	L-0200	Response enclosed – see following pages
02.05.04-18	L-0201	Response enclosed – see following pages
02.05.04-19	L-0202	Future submittal
02.05.04-20	L-0204	Response enclosed – see following pages
02.05.04-21	L-0205	Response enclosed – see following pages
02.05.04-22	L-0206	Future submittal

NRC Letter No.: LNP-RAI-LTR-030

NRC Letter Date: May 8, 2009

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-16

Text of NRC RAI:

FSAR Section 2.5.4.7, Response of Soil and Rock to Dynamic Loading, states that dynamic tests were not performed as part of the site investigation because of the low seismic environment, foundation configuration and "no site specific soil structure interaction analysis for safety class structures is required".

Please clarify or justify the statement that "no site specific soil structure interaction analysis for safety class structures is required".

PGN RAI ID #: L-0199

PGN Response to NRC RAI:
AP1000 DCD, Revision 17, Tier 2 Section 2.5.2.1 stipulates that site specific soil structure interaction analysis may be performed in lieu of the following:

1. The free field peak ground acceleration at the finished grade level is less than or equal to a 0.30g SSE.
   - *The LNP PGA is described in LNP FSAR Table 2.0-201 presents that the peak ground acceleration is 0.069g horizontal and 0.051g vertical.*

2. The site-specific ground motion response spectra (GMRS) at the finished grade level in the free-field are less than or equal to the AP1000 certified seismic design response spectra (CSDRS).
   - *LNP FSAR Figure 2.5.2-296 shows that the horizontal and vertical GMRS are bounded by the CSDRS with considerable margin. These spectra were developed on the uppermost in-situ competent material using performance-based procedures in accordance with RG 1.208 as described in DC/COL-ISG-1. As discussed in FSAR 2.5.2.5.1.1, the first site layer, S1, consists of approximately 1.8 m (6 ft.) of Quaternary sands. This layer is relatively loose, and as discussed further in FSAR 2.5.2.5, is not considered part of the site GMRS profile.*
   - *Development of GMRS is being revisited for the response to LNP RAI Letter 46.*

3. In lieu of (1) and (2) above, for a site where the nuclear island is founded on hard rock with shear wave velocity greater than 8,000 feet per second, the site-specific GMRS may be defined at the foundation level and shown to be less than or equal to the CSDRS.

- *This is not applicable, since the LNP site meets (1) and (2) above.*

4. In lieu of (1) and (2) above, for a site where the nuclear island is founded on hard rock with shear wave velocity greater than 8,000 feet per second, the site-specific peak ground acceleration and spectra may be developed at the top of the competent rock and shown at the foundation level to be less than or equal to those given in DCD Figures 3I.1-1 and 3I.1-2 over the entire frequency range.
   - *This is not applicable, since the LNP site meets (1) and (2) above.*

5. Foundation material layers are approximately horizontal (dip less than 20 degrees), and the minimum estimate of the low strain shear wave velocity of the soil below the foundation of the nuclear island is greater than or equal to 1000 feet per second.
   - *LNP FSAR Table 2.5.4.4-202 indicates that the dip at LNP 1 and LNP 2 are significantly less than 20 degrees. LNP FSAR Section 2.5.4.4.2 indicates that Vs is greater than 1000 fps for all subsurface layers.*

6. For sites where the nuclear island is founded on soil, the minimum estimate of the strain-compatible soil shear modulus and hysteretic damping is compared to the values used in the AP1000 generic analyses. Properties of soil layers within a depth of 120 feet below finished grade are compared to those in the generic soil site analyses (soft soil, soft-to-medium soil, and upper bound soft-to-medium soil). The shear wave velocity should generally increase with depth. The average low strain shear wave velocity in any layer should not be less than 80 percent of the average shear wave velocity in any layer at higher elevation.
   - *This is not applicable, since the LNP nuclear islands are not founded on soil (LNP FSAR Section 2.5.4.5).*

Meeting the DCD requirements of (1) through (6) above, a site-specific soil structure interaction analysis need not be performed, as described in AP1000 DCD, Revision 17, Tier 2 Section 2.5.2.1.

Additionally, as described in the response to RAI 02.05.04-2, the geologic and stratigraphic features at depths less than 120 feet below grade can be correlated from one boring location to the next with relatively smooth variations in thicknesses or properties of the geologic units; and, to a depth of 120 feet below finished grade, the site has less than 20 percent variation in the shear wave velocity from the average velocity in any layer.

Reference:

Westinghouse Electric Company, AP1000 Design Control Document, Revision 17, September 2008.

Associated LNP COL Application Revisions:

No COLA revisions have been identified associated with this response.

Attachments/Enclosures:
None

NRC Letter No.: LNP-RAI-LTR-030

NRC Letter Date: May 8, 2009

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-17

Text of NRC RAI:

Section 2.5.4.7, "Response of Soil and Rock to Dynamic Loading" states that the materials below the bottom of the nuclear island to an elevation of -99 ft NAVD88 will be improved, and the existing karst features should be eliminated. It does not appear from the geotechnical data presented that sufficient information is known about the distribution of voids to the degree that karst features can be targeted and eliminated.

Please clarify the statement and discuss any plans for additional exploration that you will implement to identify karst features to target during the grouting phase. Please describe how you will assess whether or not the karst features have been eliminated.

PGN RAI ID #: L-0200

PGN Response to NRC RAI:

Additional exploration pertaining to targeted karst identification is not planned during the production grouting phase of LNP construction. Improvements made to the subsurface during this effort have been conservatively not considered in project analyses.

The statement that refers to the elimination of existing karst features will be deleted.

Associated LNP COL Application Revisions:

The following change will be made to the LNP COLA in a future revision:

The following text will be deleted from FSAR Section 2.5.4.7:

> Within the improved zone, the existing karst features if any should be eliminated.

Attachments/Enclosures:

None

NRC Letter No.: LNP-RAI-LTR-030

NRC Letter Date: May 8, 2009

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-18

Text of NRC RAI:

FSAR Section 2.5.4.8.5, "Results of Liquefaction Analysis" states that Tables 2.5.4.8-202A and 202B contain the Factor of Safety against liquefaction. The table does state that these materials are liquefiable, but does not provide actual factors of safety as stated.

Please correct this apparent inconsistency.

PGN RAI ID #: L-0201

PGN Response to NRC RAI:

FSAR Tables 2.5.4.8-202A and 202B do not contain calculated factors of safety associated with liquefaction. The phrase that refers to calculated factors of safety associated with liquefaction will be deleted.

Associated LNP COL Application Revisions:

The following change will be made to the LNP COLA in a future revision:

The first sentence of the third paragraph of FSAR Section 2.5.4.8.5 will be revised in a future amendment from:

> The Borings, Soil Types and SPT Values used for Liquefaction Analysis are summarized in Tables 2.5.4.8-202A and 2.5.4.8-202B along with the FS for each.

to:

> The Borings, Soil Types and SPT Values used for Liquefaction Analysis are summarized in Tables 2.5.4.8-202A and 2.5.4.8-202B.

Attachments/Enclosures:

None

NRC Letter No.: LNP-RAI-LTR-030

NRC Letter Date: May 8, 2009

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-20

Text of NRC RAI:

In the supplemental materials identified as paragraph IV., "Permeation Grouting Discussion", it was noted that the uplift analysis indicated sufficient reduction of shear stresses in the grouted rock, and the computed factor of safety exceeded 1.5.

Please provide a sample calculation of your uplift analysis with figures showing assumptions.

PGN RAI ID #: L-0204

PGN Response to NRC RAI:

The stability of the dewatered base was checked for local failure (by piping or boiling) and general failure of the rock below the excavation level (by heave or uplift). The local reduction of effective stress due to upward seepage was checked at the exit point where the minimum seepage path length existed.

A 15-foot wide block of heave zone was considered, adjoining to the diaphragm wall as shown below on Figure RAI 02.05.04-20-1. This width represents half of the diaphragm wall penetration depth of 30 feet.

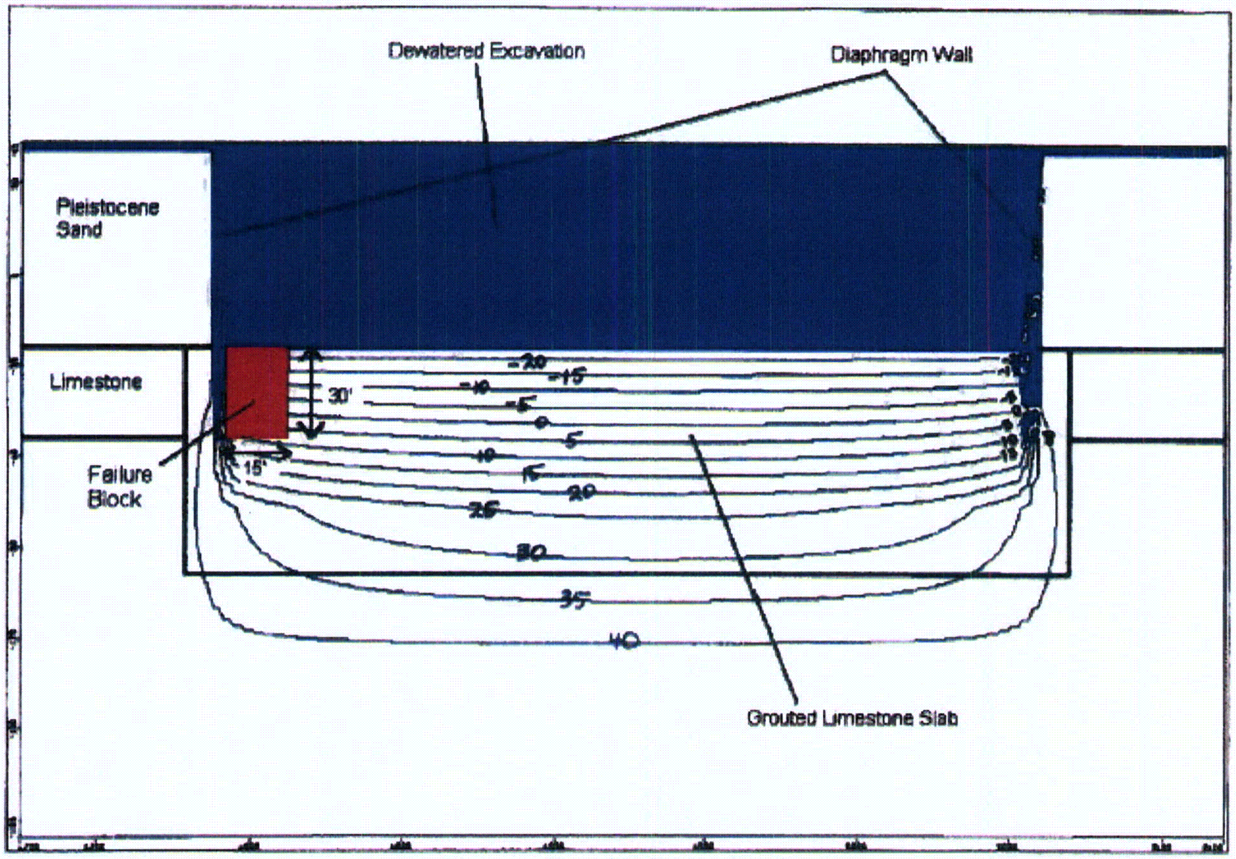


FIGURE RAI 02.05.04-20-1
FAILURE BLOCK FOR UPLIFT ANALYSIS

The hydraulic heads at the bottom and the top of the block are 10 ft amsl and -24 ft amsl, respectively. The average value of head loss in the block is:

$$10 - (-24) = 34 \text{ feet}$$

The average hydraulic gradient (i_{av}) is the average head loss in the block divided by the length of the flow path (the height of the soil block):

$$i_{av} = 34/30 = 1.13$$

The factor of safety (FS) against heaving in the rock (Das, 2002) is:

$$FS = F/U$$

Where:

F = submerged unit weight of the grouted limestone block + shear force acting on the block

U = uplift force caused by seepage on the same volume of rock.

For per unit length of the excavation:

F = $((\gamma_{sat} - \gamma_w) \times D \times D/2) + (C \times D)$ (neglecting shear force due to frictional component of the rock - conservative)

U = Volume of block x $i_{av} \times \gamma_{w,}$

Where:

γ_{sat} = saturated unit weight of rock (pcf)
$\gamma_{w,}$ = unit weight of water (pcf)
C = cohesion component of the rock (psi)
D = penetration depth of diaphragm wall (ft)

For the top bedrock layer of LNP 2:
Unit weight (γ_{sat}) = 134 pcf
Cohesion (C) = 24 psi
Friction angle (Ø) = 24°

For the top bedrock layer of LNP 1:
Unit weight (γ_{sat}) = 138 pcf
Cohesion (C) = 37psi
Friction angle (Ø) = 24°

Since rock density and cohesion values are smaller for LNP 2, the following sample calculation relates to the LNP 2 site.

Neglecting improvement in bedrock properties due to grouting, cohesion, frictional forces along the diaphragm wall and prism interface, and the frictional component of the shear strength along the rock and prism interface, the following conservative values were considered for checking the stability of the bedrock due to dewatering:

Unit Weight (γ_{sat}) = 134 pcf
Cohesion (C) = 24 psi = 24 x 144 psf = 3,456 psf
F = (134-62.4) x 30 x 15 + 3456 x 30 = 135,900 lbs/ft
U = 30 x 15 x 1.13 x 62.4 = 31,730 lb/ft
FS = 135,900/31,730 = 4.3 > 1.5

Thus, the stability of the excavated bedrock is considered to be sufficiently safe against heaving or uplift.

Associated LNP COL Application Revisions:

No COLA revisions have been identified associated with this response.

Attachments/Enclosures:

None

NRC Letter No.: LNP-RAI-LTR-030

NRC Letter Date: May 8, 2009

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.05.04-21

Text of NRC RAI:

FSAR Section 2.5.4.10.4 indicates that the lateral earthquake load includes seismic lateral earth pressures for at-rest conditions and hydrodynamic water thrust. You state that the seismic at-rest pressure is calculated from the Woods' method and hydrodynamic pressure is calculated from the Westergaard method.

Please provide sample calculations of each. Please provide figures that aid in identifying parameters used in the equations.

PGN RAI ID #: L-0205

PGN Response to NRC RAI:

Seismic At-Rest Pressure

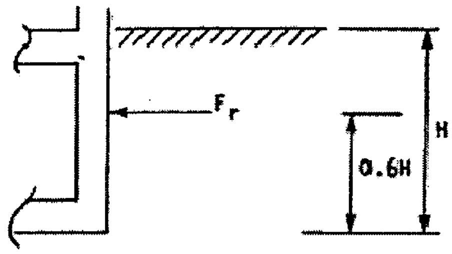
Seismic at-rest pressure was calculated from the Woods method. The Mononobe-Okabe approach is not applicable since the diaphragm wall does not move sufficiently to mobilize the shear strength of the soil.

Seismic induced earth pressure was considered to be an inverse triangular shape, simplified from ASCE-98's parabolic shape. This simplified shape induced higher pressures at the upper portion of the wall and lower pressures at the lower portion of the wall. Thus, the wall exhibits the same dynamic thrust with a larger seismic-induced moment.

Poisson's ratio for the undifferentiated sediments was considered to be 0.3, conservatively estimated compared to existing soils, concrete fill, or reinforced concrete. Thus, from ASCE 4-98, Cv is 0.94 (see Figure RAI 02.05.04-21-1 below).

$F_r = \alpha_h C_\upsilon \gamma H^2$

$M_r = \alpha_h D_\upsilon \gamma H^3$



F_r	=	resultant force associated with dynamic soil pressure distribution shown in Fig. 3.5-1
M_r	=	resultant overturning moment about base of retaining structure for pressure distribution in Fig. 3.5-1
α_h	=	horizontal earthquake acceleration (g)
γ	=	soil unit weight
H	=	embedment height
υ	=	Poisson's ratio
C_υ, D_υ	=	coefficients as a function of Poisson's ratio

υ	C_υ	D_υ
0.5	1.13	0.67
0.4	1.04	0.63
0.3	0.94	0.56
0.2	0.87	0.52

FIGURE 3.5-2. Resultant Force and Overturning Moment for Elastic Solution Dynamic Soil Pressures

FIGURE RAI 02.05.04-21-1

DYNAMIC SOIL PRESSURE RESULTANT FORCE (ASCE 4-98)

Alternately, Kramer (1996) provides a comparable "thrust factor" based on Poisson's ratio, as shown on Figure RAI 02.05.04-21-2 below:

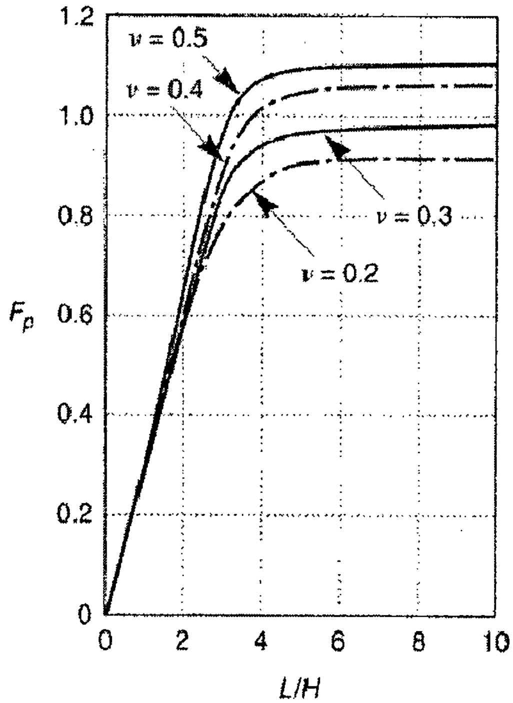


Figure 11.17 Dimensionless thrust factor for various geometries and soil Poisson's ratio values. After Wood (1973).

FIGURE RAI 02.05.04-21-2
DIMENSIONLESS THRUST FACTOR (KRAMER, 1996)

Based on Figure RAI 02.05.04-21-2, the thrust factor is approximately 0.98, considering that the width of the soil is much larger than the thickness of the soil.

The higher thrust factor (0.98 from Kramer, instead of 0.94 from ASCE) is conservatively considered.

$\Delta P_{eq} = \gamma_b \times H^2 \times a_h/g \times F_p$

Where:

γ_b = 62.7 pcf (soil effective unit weight)
a_h = 0.1 g (horizontal acceleration)
F_p = 0.98 (thrust factor)
H = 40 feet (depth of soil adjacent to nuclear island)

Regarding the horizontal acceleration (a_h), the frequencies of seismic motions are considered to be less than half of the fundamental frequency of the backfill; i.e. dynamic amplification is neglected.

Thus, the seismic induced lateral load is 9.83 kips per foot.

Considering the inverse triangular distribution of lateral seismic earth pressure, the maximum pressure is calculated as:

$P_{max} = 2 \times \Delta P_{eq} / H$

Thus, the maximum pressure is conservatively 0.5 ksf.

The pressure (P_{ELE}) at an elevation ELE is then:

$P_{ELE} = (ELE - 11 \text{ feet}) / H \times P_{max}$

Thus, the seismic earth pressure diagram is developed as shown below on Figure RAI 02.05.04-21-3:

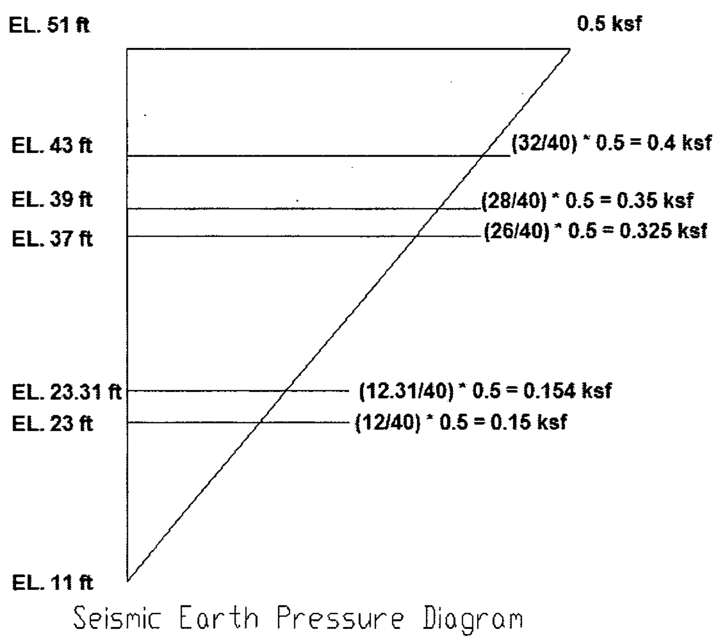


FIGURE RAI 02.05.04-21-3
SEISMIC EARTH PRESSURE DIAGRAM

Hydrodynamic Water Thrust

Hydrodynamic pressure was calculated using Westergaard's equation (Kramer, 1996):

$$P_w = \frac{7}{8} \cdot \frac{a_h}{g} \gamma_w \cdot \sqrt{Z \cdot D}$$

Where:

γ_w = 62.4 pcf
a_h = 0.1 g (horizontal acceleration)
Z = Depth of water of interest
D = Total depth of free pore water

Conservatively considering the free pore water condition, at an example elevation of 11 feet:

$$Z = 43 \cdot \text{feet} - 11 \cdot \text{feet} = 32 \cdot \text{feet}$$

$$P_w = \frac{7}{8} \times 0.1 \times 62.4 \cdot \sqrt{32 \cdot \text{feet} \cdot 32 \cdot \text{feet}} = 174.72 \cdot \text{psf} = 0.175 \cdot \text{ksf}$$

Thus, the hydrodynamic pressure diagram is developed as shown below on Figure RAI 02.05.04-21-4:

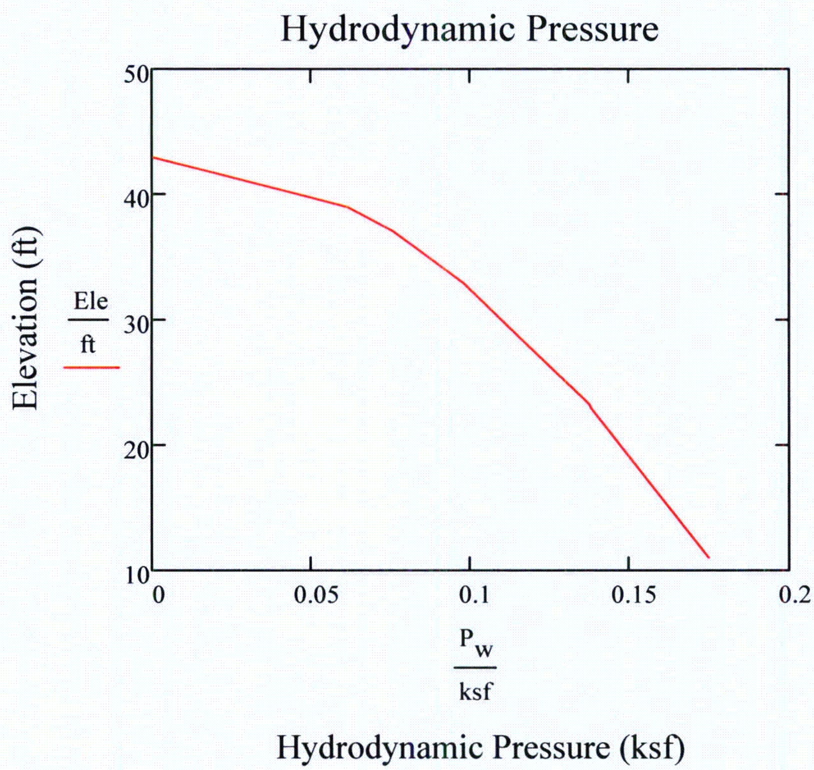


FIGURE RAI 02.05.04-21-4
HYDRODYNAMIC PRESSURE DIAGRAM

Associated LNP COL Application Revisions:

No COLA revisions have been identified associated with this response.

Attachments/Enclosures:

None