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Sent: Tuesday, October 14, 2008 4:32 PM
To: Christian Araguas; Williams, Dana M.
Cc: jprebula@bechtel.com; Prunty, Robert
Subject: FW: PDF Signed Letter ND-08-1577
Attachments: ND-08-1577 ESPA_Ref_Changes.pdf

Christian,

Here is our reference change letter. These changes will be incorporated in Revision 5 of the ESP, currently scheduled for January 15, 2009. The Document Control copy will be in FedEx tomorrow. Give me a call if you have any questions.

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Hearing Identifier: Vogtle_Public_EX
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Sent Date: 10/14/2008 4:31:39 PM
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SOUTHERN COMPANY
Energy to Serve Your World℠

OCT 14 2008

Docket Nos.: 52-011

ND-08-1577

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

Southern Nuclear Operating Company
Vogtle Early Site Permit Application Reference Changes

Ladies and Gentlemen:

In letter AR-08-0483, dated March 28, 2008, Southern Nuclear Operating Company (SNC) submitted Revision 4 of the Vogtle Early Site Permit (ESP) Application to the U.S. Nuclear Regulatory Commission (NRC). In addition, in letter AR-08-1171, dated August 14, 2008, SNC responded to NRC Request for Additional Information (RAI) Letter No. 11 and action items from the NRC seismic calculation audit at the Bechtel Frederick, MD office on June 19 and 20, 2008. In letter AR-08-1171, SNC provided proposed changes to ESP application Revision 4 text that will be incorporated into Revision 5 of the ESP application. As discussed with the NRC staff, this letter provides additional changes to the proposed Revision 5 text. The additional changes update references in selected subsections of Site Safety Analysis Report (SSAR) Section 2.5.4, "Stability of Subsurface Materials and Foundations," and in SSAR Appendix 2.5E, "AP1000 Vogtle Site Specific Seismic Evaluation Report." Enclosure 1 contains the proposed ESP application SSAR text changes that perform this action and Enclosure 2 contains an updated SSAR Appendix 2.5E. These proposed ESP application changes will be reflected in Revision 5 to ESP application.

If you have any questions regarding this letter, please contact Mr. Jim Davis at 205-992-7692.

Ms. M. M. Caston states she is Vice President and General Counsel of Southern Nuclear Operating Company, is authorized to execute this oath on behalf of Southern Nuclear Operating Company and to the best of her knowledge and belief, the facts set forth in this letter are true.

Respectfully submitted,

SOUTHERN NUCLEAR OPERATING COMPANY

Moanica M. Caston

Sworn to and subscribed before me this 14th *day of* October, *2008*

Notary Public: Glenda C. Spinks

My commission expires: 11/10/2010

MMC/BJS/dmw

Enclosures:

1. Vogtle Early Site Permit Application Proposed Reference Changes
2. Westinghouse Report SV0-1000-S2R-802, Revision 4, AP1000Vogtle Site Seismic Evaluation Report, October 2008

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Southern Nuclear Operating Company

ND-08-1577

Enclosure 1

Vogtle Early Site Permit Application

Proposed Reference Changes

NOTE: This enclosure consists of a four-page document.

Proposed 2.5.4 Design Reference Changes

2.5.4.5.3 Backfill Design

(4th paragraph)

The Phase I test pad program is complete and is documented in Appendix 2.5D. The objective of this program was to establish site-specific design properties for the backfill, including density, compaction, gradation, and shear wave velocity, and to show that the backfill will satisfy the AP1000 standard plant design ~~siting criteria~~ ~~**(WEC 2007)**~~. The test pad was constructed below grade, was 20 ft deep, and was 20 ft x 60 ft in plan area. The test pad was constructed in the switchyard borrow area using methods similar to those used to construct the backfill for VEGP Units 1 and 2. The placement and compaction of the backfill were monitored and tested. Results of the test pad program demonstrated that the siting criterion for shear wave velocity of 1,000 fps at the NI foundation depth was achieved with the backfill material within the 20 ft thickness of the test pad.

2.5.4.10.1 Bearing Capacity

All structures in the power block footprint will be founded on the structural backfill compacted to a minimum of 95% (ASTM D 1557) as presented in Section 2.5.4.5. The structural backfill will be about 90 ft thick in the power block area. The Nuclear Island will be founded at a depth of about 40 ft below grade (about 50 ft of structural backfill beneath the foundation). Other structures will be founded at an approximate depth of 4 ft below grade. The allowable static bearing capacity values are calculated with Terzaghi's bearing capacity equation. An internal angle of friction of 36° was used for the compacted backfill as developed from field and laboratory testing of borrow materials during the Phase I test pad program (Appendix 2.5D) and the COL investigation (Appendix 2.5C). The influence of the Blue Bluff Marl on the allowable bearing pressure was evaluated using procedures outlined by Vesic (1975). With a factor of safety of 3.0 **(ASCE 1994)**, site conditions provide an allowable bearing pressure of 34 ksf under static loading conditions for the Nuclear Island, which is greater than the ~~DCD~~ required~~ment of~~ 8.6 ksf **(WEC CCC-004)**. An internal friction angle of 34° was used to calculate the allowable bearing capacity values for foundations placed on compacted fills at depths of about 4 ft below finished grade as provided in Figure 2.5.4-13.

The allowable bearing capacity of the structural backfill under the Nuclear Island for dynamic loading conditions was evaluated using Terzaghi's bearing capacity equation for local shear (**Peck et al. 1974**) and Soubra's method with seismic bearing capacity factors **(Soubra 1999)** using Terzaghi's bearing capacity equation for general shear with an internal friction angle of 36°. To simulate the potential for higher edge pressures during dynamic loading, three foundation widths were considered (10, 25, and 50 ft) corresponding to 10, 25, and 50 percent of the width of the Nuclear Island basemat. The results from these two methods compared well, with Terzaghi's approach for local shear providing more conservative values. The computed average ultimate capacities for the three widths (10, 25, and 50 ft) were 89, 100, and 119 ksf, respectively. A width of 25 ft and a factor of safety of 2.25 **(ASCE 1994)** were used for site specific conditions providing an allowable bearing pressure greater than 42 ksf under dynamic loading conditions for the Nuclear Island. This value is greater than the required 35 ksf for dynamic bearing **(WEC SC2-065)** as ~~provided in the DCD as~~ well as the Vogtle site specific maximum dynamic demand (for the ESP soil profile as described in Appendix 2.5E) of 18 ksf.

The bearing capacity of the structural backfill was also evaluated in terms of the ratio of the ultimate bearing capacity against the structure demand. This capacity over demand (C/D) ratio provides an alternative measure of the margin of safety against bearing failure. These C/D ratios were evaluated for the static and dynamic demand conditions as provided by Westinghouse **(WEC CCC-004 and WEC SC2-065)**~~in the DCD~~, as well as the maximum dynamic demand from the Vogtle site specific seismic evaluation (Appendix 2.5E). The results are given below;

Condition	**~~DCD~~ Static**	**~~DCD~~ Dynamic**	**Site-Specific Dynamic**
Ultimate Capacity (C), ksf	102	100(1)	100(1)
Demand (D), ksf	8.6(4)	35(3)	18(2)
C/D	11.9	2.9	5.6

(1) Based on a reduced foundation width of 25 feet to account for higher edge pressures during a seismic event.
(2) Based on analysis using ESP profile in Appendix 2.5E
(3) APP-1000-S2C-065, Rev. 0, *Nuclear Island Stick Model Analysis at Soil Sites*
(4) APP-1000-CCC-004, Rev.0, *Nuclear Island - Stability Evaluation*

The C/D ratios are higher than those typically utilized for standard practice. While these results do not take into account settlement of the structures, the significant margin suggests that settlements will be minimal and within the design requirements **(WEC SC2-065)**~~of the DCD~~. A further discussion of settlement is provided in Section 2.5.4.10.2.

2.5.4.11 Design Criteria

Applicable geotechnical-related design criteria are ~~provided in the AP1000 DCD~~ ~~**(WEC 2007)**~~ ~~and are~~ discussed in various sections of the SSAR~~.~~ ~~The criteria~~and are summarized below~~ are considered geotechnical-related criteria~~.

Section 2.5.4.8 specifies that the acceptable factor of safety against liquefaction of site soils should be ≥ 1.1 in accordance with Regulatory Guide 1.198.

Bearing capacity criteria are presented in Section 2.5.4.10. A minimum factor of safety of 3 is used when applying bearing capacity equations. This factor of safety is also applied against breakout failure due to uplift forces on buried piping. For soils, this factor of safety can be reduced to 2.25 when dynamic or transient loading conditions apply. **(ASCE 1994)**

Section 2.5.5.2 specifies that the minimum acceptable long-term static factor of safety against slope stability failure is 1.5~~. Section 2.5.5.3 specifies~~ and that the minimum acceptable long-term seismic factor of safety against slope stability failure is 1.1 **(USACE 2003)**.

Appendix 2.5E describes the site-specific analyses that have been performed to show the acceptability of the AP1000 plant at the Vogtle site.

Section 2.5.4 References

(selected)

(ASCE 1994) American Society of Civil Engineers, *Bearing Capacity of Soils*, Technical Engineering and Design Guide, 1994.

(USACE 2003) U. S. Army Corps of Engineers, *Engineering and Design - Slope Stability,* EM 1110-2-1902, Office of the Chief of Engineers, Dept. of the Army, 2003.

(WEC 2007) deleted
~~"AP 1000 Design Control Document, Revision 16," Westinghouse Electric Company LLC., Pittsburgh, PA, May 2007.~~

(WEC SC2-065) APP-1000-S2C-065, Rev. 0, *Nuclear Island Stick Model Analysis at Soil Sites*

(WEC CCC-004) APP-1000-CCC-004, Rev.0, *Nuclear Island - Stability Evaluation*

2.5.5.2 New Slopes

(3rd paragraph)

The proposed permanent non-safety-related slopes will be analyzed for dynamic and static conditions during the design stage. The minimum acceptable factors of safety against stability failure of permanent slopes are 1.5 for long-term static conditions and 1.1 for long-term seismic conditions **(USACE 2003)**. The construction excavation cut slopes will be analyzed for static conditions during the design stage. The minimum acceptable factor of safety against stability failure of excavation slopes is 1.3, based on what was used for Units 1 and 2. These analyses will be performed to ensure that these slopes will not pose a hazard to the public. Such analyses are not part of the ESP SSAR.

Southern Nuclear Operating Company

ND-08-1577

Enclosure 2

Westinghouse Report SV0-1000-S2R-802

Revision 4

AP1000 Vogtle Site Seismic Evaluation Report

October 2008

NOTE: This enclosure consists of a 92-page document.

SV0-1000-S2R-802
Revision 4

October 2008

AP1000
Vogtle Site Specific Seismic Evaluation Report

Westinghouse Electric Company LLC
Nuclear Power Plants
Post Office Box 355
Pittsburgh, PA 15230-0355

Record of Revisions

Rev	Date	Revision Description[1]
0	See EDMS	Original Issue
1	3/20/08	Comments addressed from Southern Company, Bechtel, and Shaw.
2	3/24/08	Changed note on Figure 3-3 from "Note: DRS = GMRS" to "Note: DRS = FIRS".
3	8/12/08	Revised section 2.0, 3.0, 6.0, 7.0, Table 5.1-2 and Figure 5.1-1 to 5.1-36. Add Section 5.3 and Appendix A.
4	10/2/08	Eliminated reference to DCD and replaced Reference 2

Note (1) Significant changes are briefly described in this table. In the rest of the report, each row that has changed is marked using a revision bar in the margin of the page. This approach satisfies the change identification requirements in WP 4.5 Section 7.4.

Table of Contents

List of Tables

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1.0 Introduction

This report describes the site-specific analyses that have been performed to show the acceptability of the AP1000 plant at the Vogtle site. The site specific seismic analyses were performed to address the following:

- Parametric analyses to identify the importance of the different geotechnical variations at the site that could affect the nuclear island dynamic response and settlement.
- Analyses of nuclear island, turbine building, and annex building (structure to structure interaction) to confirm surface input against design basis of annex building (Seismic Category II).
- Seismic analyses of the AP1000 nuclear island (NI) using Vogtle site specific soil and site specific SSE seismic input to confirm that the AP1000 NI seismic response is less severe than the design basis seismic response.
- Demonstration that the NI site-specific stability factors of safety are within the limits established by the NRC using a sliding friction coefficient of 0.45.
- Provision of dynamic bearing pressure loads.
- Settlement analyses of the nuclear island to show that the differential settlement at Vogtle is less than those used for the AP1000 design and to establish the parameters for the settlement monitoring program to be used during construction.

1.1 Acronyms

ASB = Auxiliary and Shield Building
BE = Best Estimate
CIS = Containment Internal Structures
CSDRS = Certified Seismic Design Response Spectra
DCD = Design Control Document
DRS = Design Response Spectra
EL (El.) = Elevation (unless otherwise noted all EL are generic AP1000 EL where grade is at EL 100')
ESP = Early Site Permit
EW = East West
FIRS = Foundation Input Response Spectra
GMRS = Ground Motion Response Spectra
LB = Lower Bound
NI = Nuclear Island
NS = North South
SEN = Sensitivity
SSI = Soil Structure Interaction
SCV = Steel Containment Vessel
UB = Upper Bound
ZPA = Zero Period Acceleration

2.0 Vogtle Site Characteristics

The Vogtle GMRS, geotechnical conditions, and ground material have differences from the design analyses performed for the AP1000 seismic analyses (Reference 2) that site specific analyses must be performed for the Vogtle site. The differences between the Vogtle GMRS and the AP1000 CSDRS, that in part require these site-specific seismic analyses, are presented in section 3.0. The results of the Vogtle ESP soil investigation and the resulting site response calculations are used to determine the AP1000 Nuclear Island site-specific responses presented in this report unless otherwise noted.

The plant specific evaluations are based on 2D SASSI analyses as discussed in section 4. Comparisons of the site specific response spectra to the AP1000 SSI envelop response spectra at six key locations are provided in section 5. These 2D SASSI site-specific results were used to calculate inertia loads for stability, section 6, and bearing, section 7.

The results of these response spectra comparisons and the resulting stability evaluations and bearing pressures demonstrate that the AP1000 plant designed for the CSDRS is acceptable for the Vogtle site.

The Vogtle Electric Generating Plant (VEGP) site is located near Waynesboro, Georgia in Eastern Burke County. Two units already exist and two more will be added. The results of the ESP site investigations are the baseline for this report. Subsurface materials at the VEGP site were placed into generalized groups, which included:

a. Upper Sand Stratum (Barnwell Group)

- Very loose to very dense sands
- Average thickness of about 90 ft
- Vogtle ground water elevation at 165 ft (55-60 ft below grade)

b. Blue Bluff Marl (Lisbon Formation)

- Very hard, slightly sandy, cemented, calcareous silt/clay
- Average thickness of 76 ft

c. Lower Sand Stratum (coastal plain deposits)

- Dense sands
- Thickness of 900 ft

d. Dunbarton Basin Bedrock

- Triassic sandstone
- 1,049 ft below grade at B-1003

e. Paleozoic Crystalline Rock

- High shear-wave velocity

- The Pen Branch fault is the boundary of the Triassic Basin and Paleozoic basement rocks.

The Upper Sand Stratum was removed before construction of Units 1 and 2 and will be removed for Units 3 and 4 because it has highly variable density along the depth and from borehole to borehole. Also, a porous material was encountered at the bottom of the Barnwell Group/top of Blue Bluff Marl that caused drilling fluid losses.

This soil was removed and replaced with compacted granular fill for the construction of the existing units. The materials above the Blue Bluff Marl in the area of the Units 3 and 4 nuclear islands are assumed for the purposes of these analyses to consist of compacted granular fill as specified in Revision 3 of the ESP SSAR.

3.0 Vogtle Site Seismic Input

The AP1000 Certified Seismic Design Response Spectra (CSDRS) has peak ground accelerations for the safe shutdown earthquake equal to 0.30g for the AP1000 design. The vertical peak ground acceleration is conservatively assumed to equal the horizontal value of 0.30g. These seismic response spectra are shown in Figures 3-1 and 3-2. These response spectra are based on Regulatory Guide (RG) 1.60 (Reference 1) with an additional control point specified at 25 Hz. The spectral amplitude at 25 Hz is 30 percent higher than the Regulatory Guide 1.60 spectral amplitude. The AP1000 CSDRS are applied at the foundation level in the free field at hard rock sites, and at the finished grade for the other soil generic conditions.

For the Vogtle site, foundation input response spectra (FIRS) and the associated response spectra compatible time histories were generated at the depth of 40 ft below plant level (Vogtle plant level elevation 220 ft) consistent with the same site response calculation of the full soil profile that was used to generate the Ground Motion Response Spectra (GMRS) at grade. Using the FIRS motion, three sets of “in-column" time histories corresponding to the upper, mean, and lower bound soil properties were developed and used in the respective upper, mean, and lower bound SSI analysis. Computation of FIRS and "in-column" time histories are fully consistent with its application for SSI analysis.

The soil properties and soil amplification analysis used to develop the design motion at the ground surface were used to obtain Vogtle FIRS at the depth of 40 ft as a full soil profile outcrop motion.

Three time histories, two in horizontal direction (H1, H2) and one in vertical direction (Vt), were generated to match the FIRS at 40 ft. The strain-compatible soil properties from the full soil column analyses were extracted and compared with the velocity profiles that correspond to the variation of shear modulus with a factor of 1.5. The wider range of the two sets was shown to be for the variation of G with a factor of 1.5. The three profiles were subsequently used in the soil column analyses using the input motion time histories (H1, H2, Vt) to obtain “within” time histories at the depth of 40 ft for SSI analyses. The “within” time histories are applied as control motions in the SSI analysis and were input at the depth of 40 ft in the free-field site model.

Figure 3-3 shows the seismic response spectra associated with the three outcrop time histories components compared to the outcrop FIRS at 40 ft depth.

Each of the two horizontal input motions (H1 and H2) was used for site response analyses of the three strain compatible S-wave soil profiles: lower bound (LB), best estimate (BE) and upper bound (UB). These analyses provided six sets of acceleration time histories of the "within" soil-column motion at depth of 40 ft that are to be used as input for the SSI analysis. Similarly, for vertical motion, three "within" time histories were obtained at the depth of 40 ft using the outcrop vertical time history at the depth of 40 ft and each of three soil profiles (LB, BE, UB). Development of vertical "within" time histories is fully consistent with its application for SSI analysis.

The Vogtle GMRS which is the site-specific safe shutdown earthquake (SSE) is defined at the ground surface. The Vogtle foundation input response spectra are at an outcrop located at the 40' depth. These Vogtle response spectra are compared to the AP1000 SSE design response spectra that are also referred to as the AP1000 certified seismic design response spectra. The CSDRS also represents the AP1000 FIRS. This is because: (1) the CSDRS at a hard rock site is essentially the same at the grade level and at the foundation level; and (2) the CSDRS envelopes the in-column motions of the other generic soil conditions. The comparisons are shown in Figures 3-4 and 3-5. As seen from this comparison there are exceedances above the CSDRS; therefore, a plant specific seismic evaluation is performed to demonstrate that the AP1000 plant designed for the CSDRS is acceptable for the Vogtle site.

The surface response motion using the FIRS input motion at the depth of 40 ft for the LB, BE, and UB profiles are compared with the GMRS at the ground surface level in Figures 3-6 through 3-8. The GMRS at the ground surface level is computed in the same calculation that provided outcrop motion at 40' depth and includes use of fully randomized soil profiles and soil properties. The comparison shows that the free-field motion within the embedment depth of 40 ft and based on three soil profiles is adequately captured.

The site specific SSI analysis at Vogtle is based on the site specific soil profile and the site specific FIRS developed for the site. The development of the FIRS is fully consistent with its application for SSI analysis. The comparison presented in Figures 3-6 through 3-8 is to illustrate that use of FIRS along with the UP, BE and LB profiles results in a surface motion in the SSI free-field model that envelops the GMRS. This study confirms that the foundation motion as well as the free field motion along the embedment depth of the NI is adequately and conservatively modeled in the SSI analysis.

These spectra and time histories presented in this section are all based on the ESP soil properties. The site response was completely recalculated assuming a different higher shear wave velocity profile of the backfill. This resulted in a new set of FIRS; time histories; and lower bound, best estimate, and upper bound strain compatible soil profiles. These were developed for a sensitivity analysis to determine the sensitivity of the Vogtle AP1000 NI seismic response to a very wide range of shear wave velocity profiles of the backfill. This was done in part since the final backfill is not in place. The results of this sensitivity study (SEN) are provided in Sections 5.0, 6.0, and 7.0.

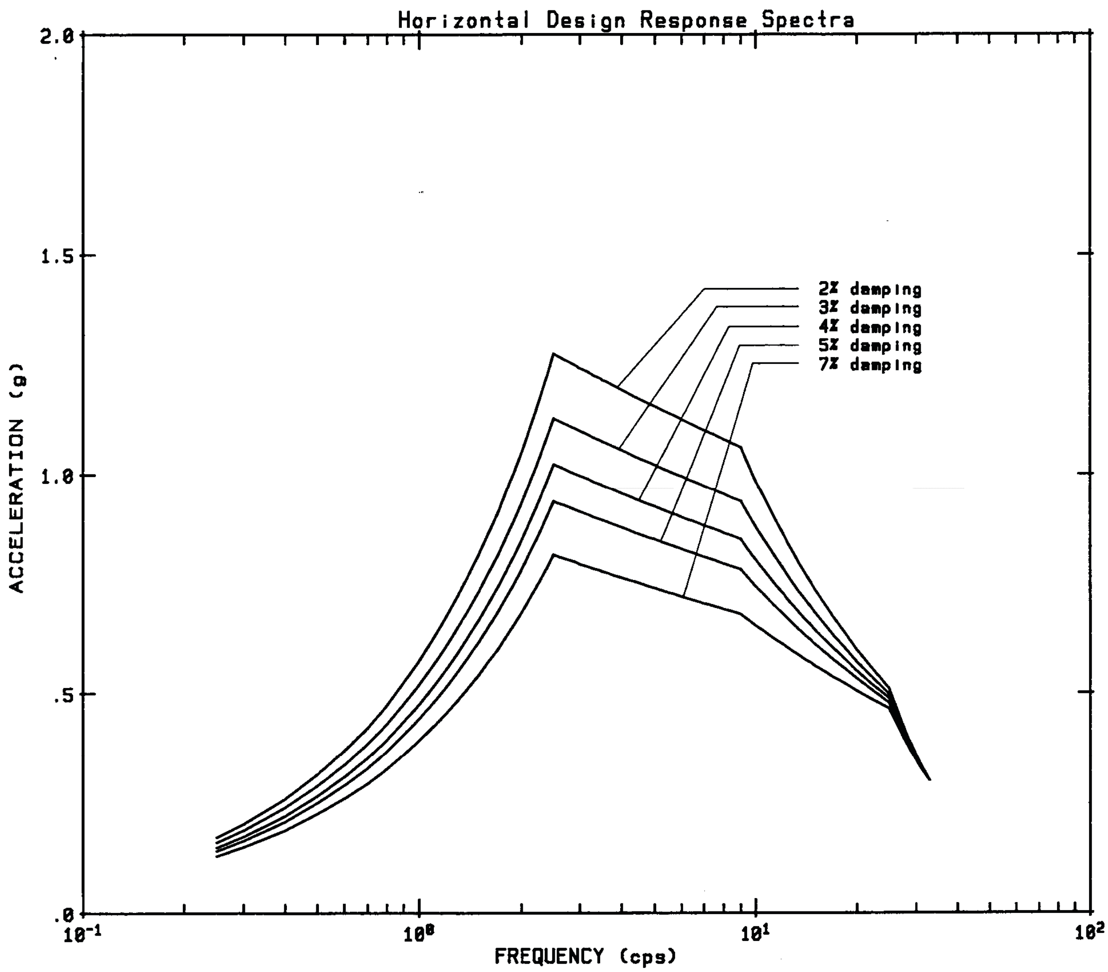


Figure 3-1 – AP1000 Horizontal Design Response Spectra for Safe Shutdown Earthquake

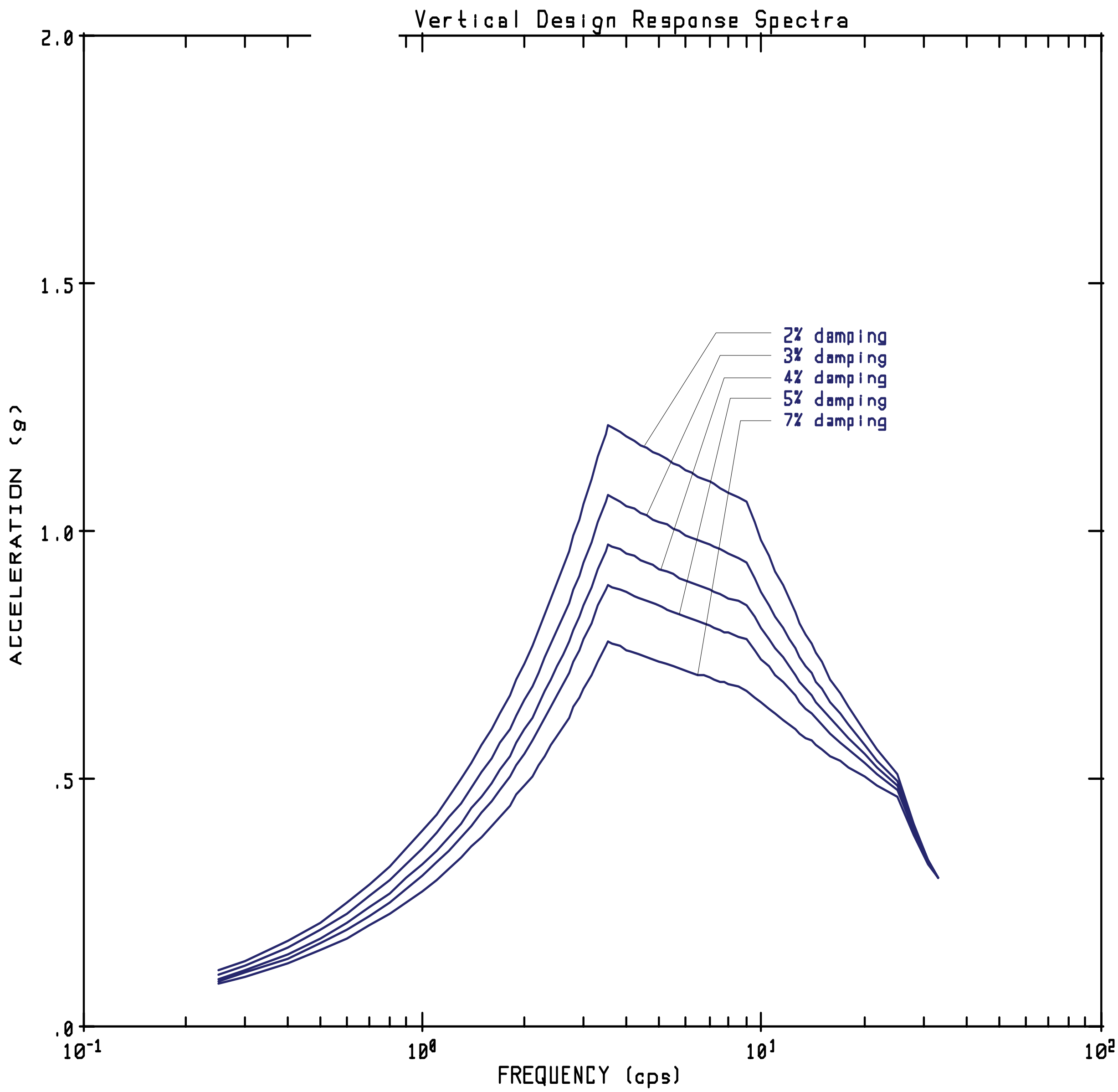
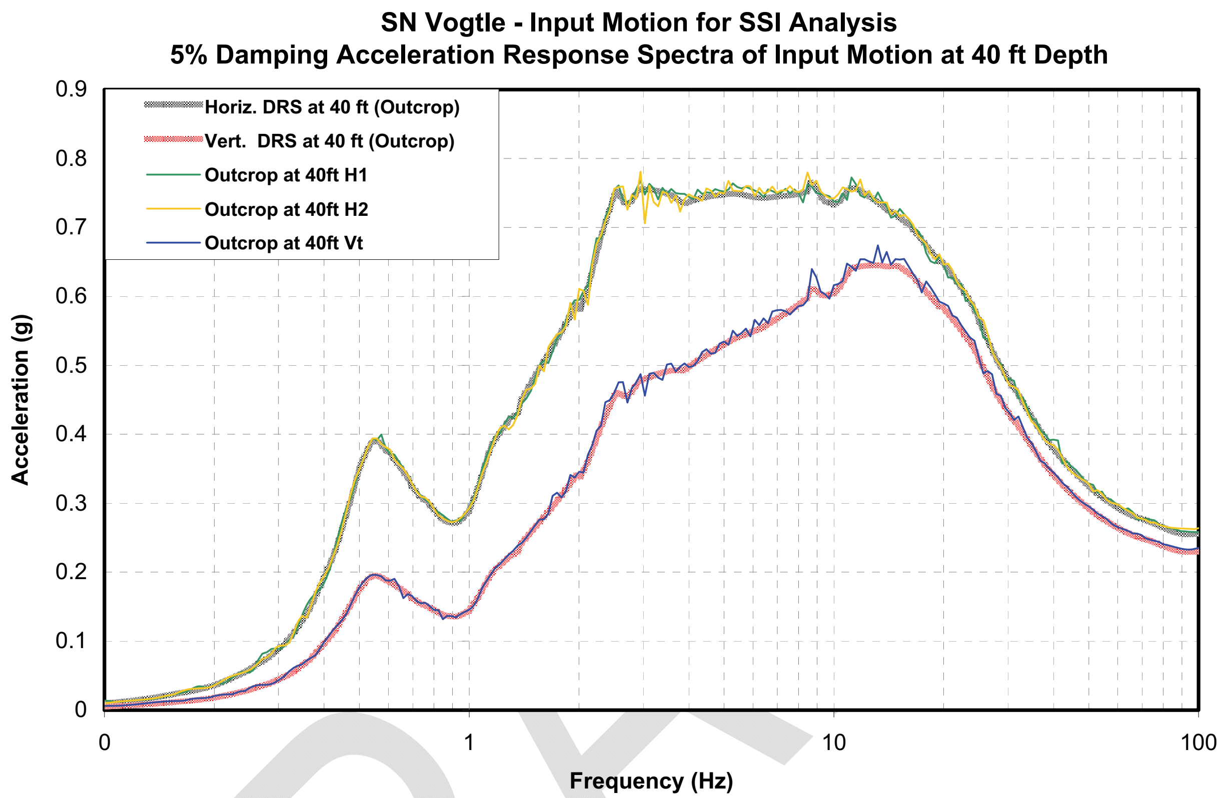


Figure 3-2 – AP1000 Vertical Design Response Spectra Safe Shutdown Earthquake



Note: DRS = FIRS

Figure 3-3 - Acceleration Response Spectra – Input Outcrop Motion at 40 ft Depth

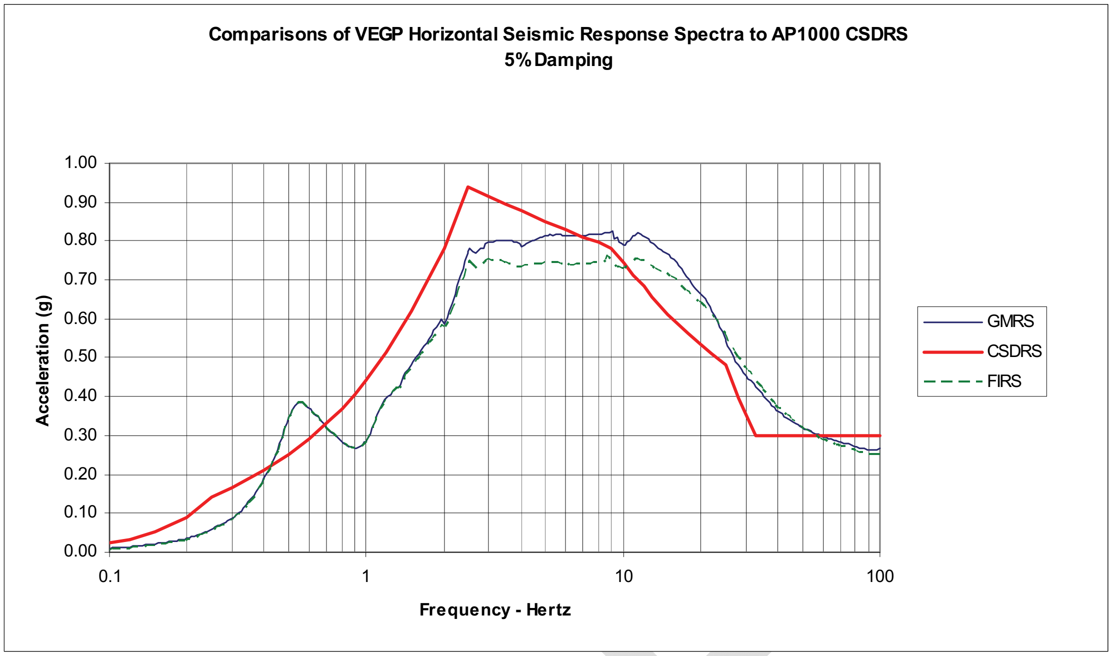


Figure 3-4 – Comparison of AP1000 Horizontal CSDRS to Vogtle 40' Outcrop FIRS and GMRS

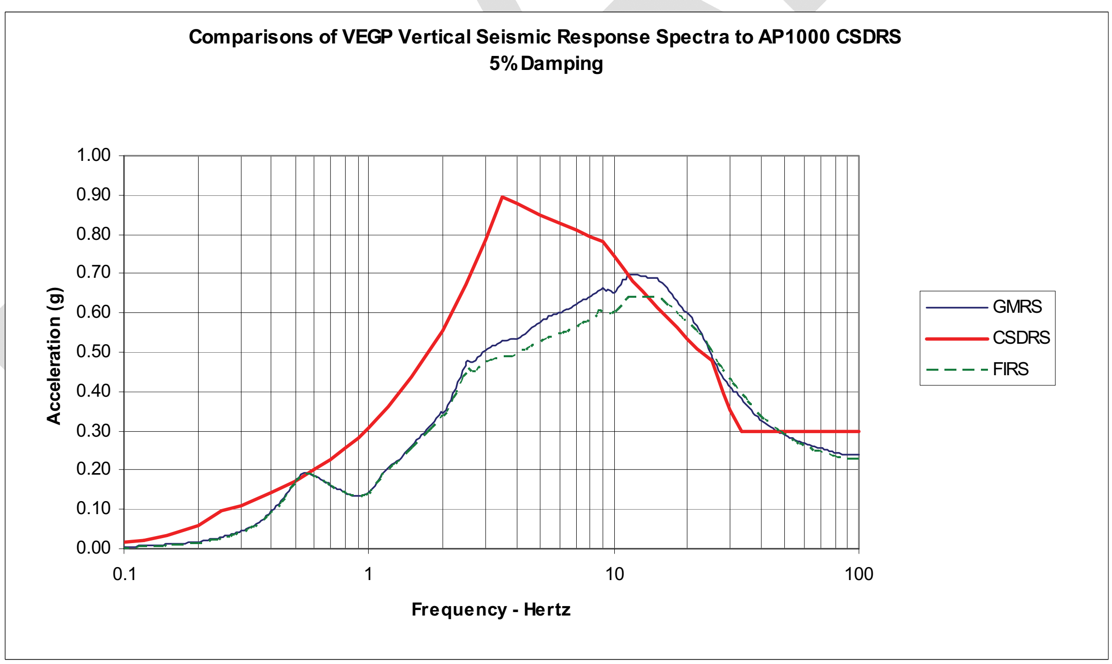
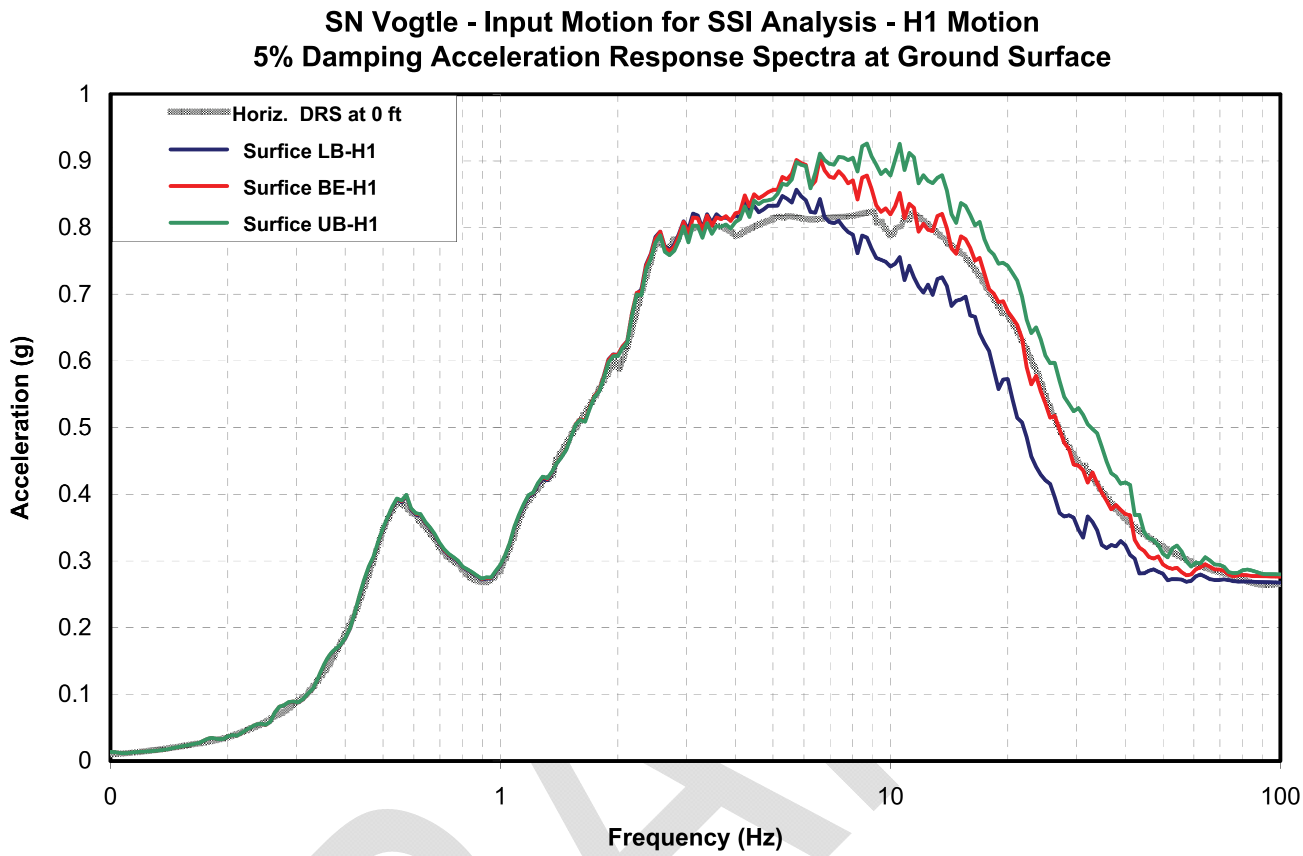
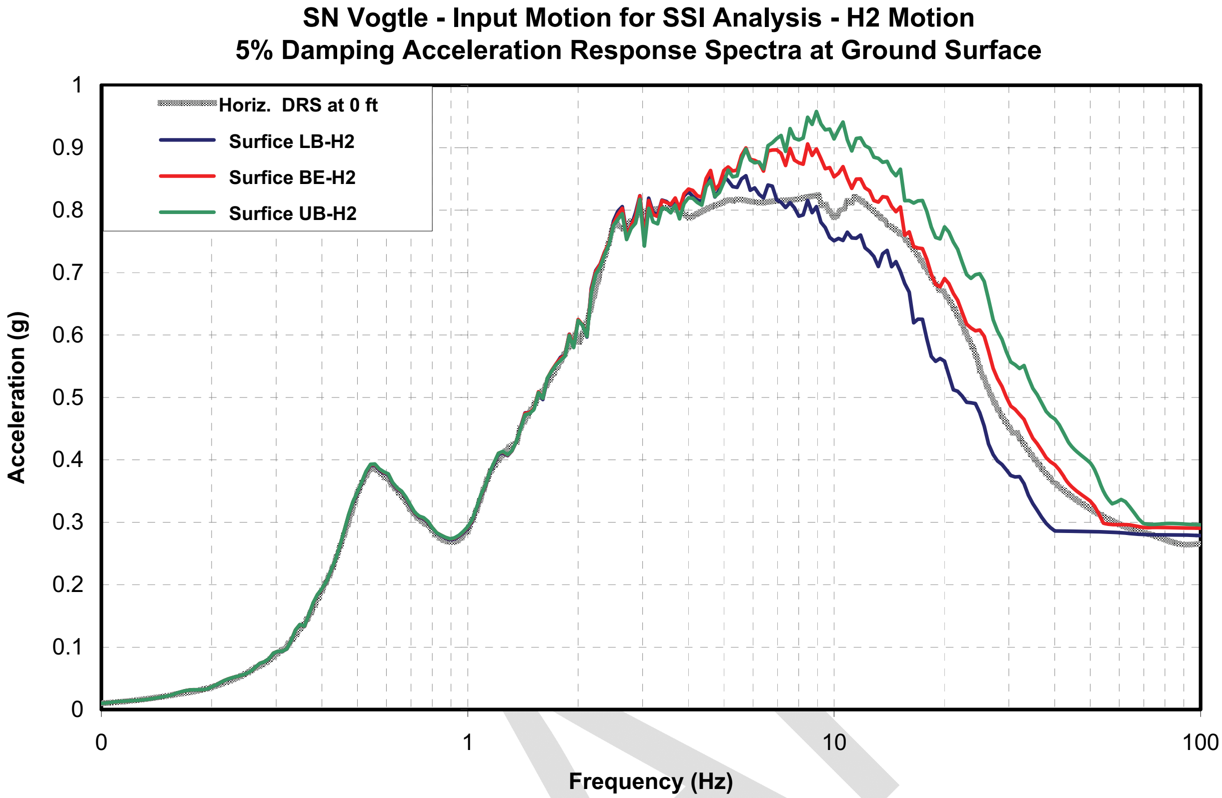


Figure 3-5 – Comparison of AP1000 Vertical CSDRS to Vogtle 40' Outcrop FIRS and GMRS



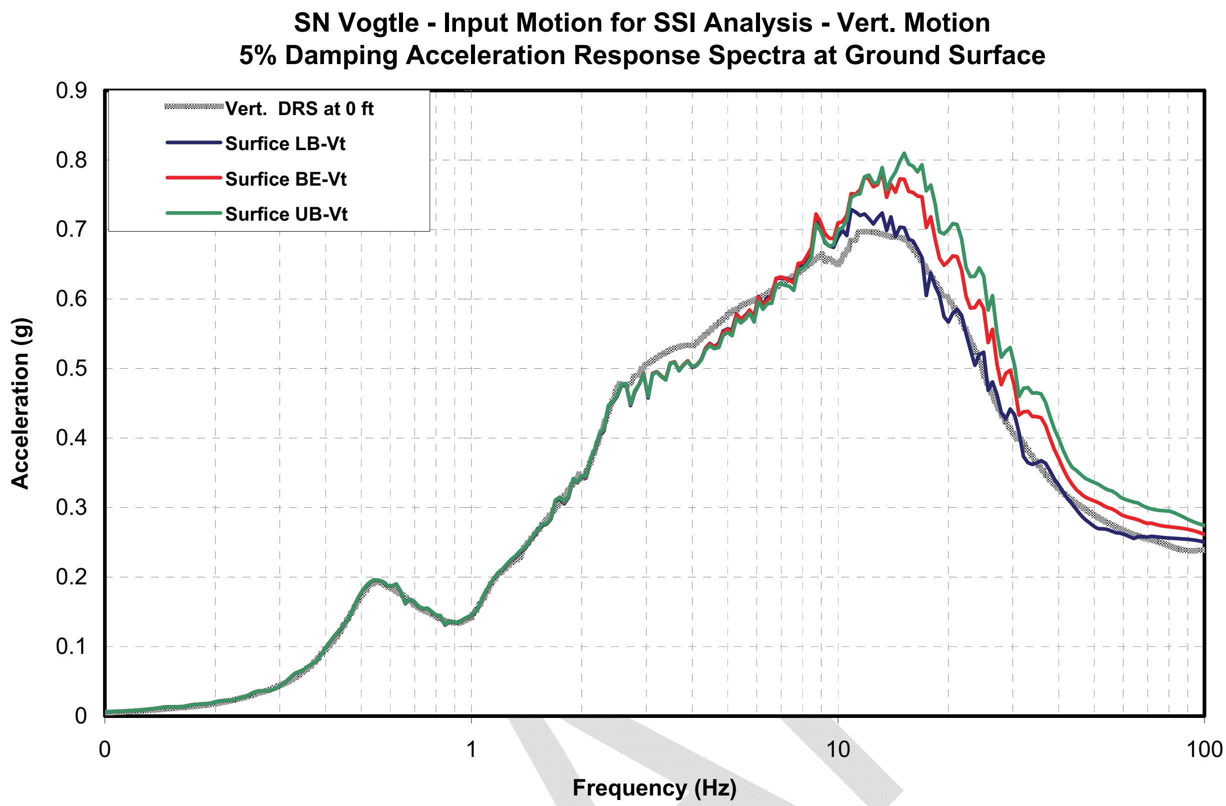
Note: DRS = GMRS

Figure 3-6 - Acceleration Response Spectra – Horizontal H1 Motions at Ground Surface



Note: DRS = GMRS

Figure 3-7 - Acceleration Response Spectra – Horizontal H2 Motions at Ground Surface



Note: DRS = GMRS

Figure 3-8 - Acceleration Response Spectra – Vertical Motions at Ground Surface

4.0 Seismic Models

The AP1000 nuclear island (NI) consists of three distinct Seismic Category I structures. The three building structures that make up the nuclear island are the coupled auxiliary and shield building (ASB), the steel containment vessel (SCV), and the containment internal structures (CIS).

The nuclear island structures, including the SCV, the CIS, and the ASB are founded on a common basemat. The nuclear island is embedded approximately forty feet below an assumed plant grade (for modeling purposes) located at Elevation 100'-0". Thus, the bottom of the basemat is located at Elevation 60'-0". See Figure 5-2 for Vogtle site elevations.

The steel containment vessel is a freestanding cylindrical steel structure with elliptical upper and lower heads. It is surrounded by the reinforced concrete shield building. The inside diameter and height are equal to 130' and 215'-4", respectively. The top of containment is at Elevation 281'-10".

The containment internal structures are designed using reinforced concrete and structural steel. At the lower elevations conventional concrete and reinforcing steel are used, except that permanent steel forms are used in some areas in lieu of removable forms based on constructability considerations. These modules are structural elements built up with welded structural shapes and plates. Concrete is used where required for shielding, but reinforcing steel in the form of bars is not normally used.

The shield building is an enhanced cylindrical reinforced concrete structure which includes the open annulus area surrounding the containment vessel. It has a conical roof structure which supports the containment air cooling diffuser and the Passive Containment Cooling System (PCS) water storage tank.

The auxiliary building is a reinforced concrete structure. Structural modules, similar to those used in the containment internal structures, are used in the southern portion of the auxiliary building. It essentially wraps approximately 50 percent of the circumference of the shield building. The floor slabs and the structural walls of the auxiliary building are structurally connected to the cylindrical section of the shield building. The auxiliary building includes the fuel handling area located south of the shield building.

The AP1000 NI structural models used to analyze the Vogtle site were modified from the models used during the hard rock licensing (Design Control Document, Revision 15). The seismic analyses performed for the Vogtle site includes site-specific soil properties and embedment effects and uses the current seismic models that represent the latest AP1000 NI structural configuration (Reference 2). The shield building design has been enhanced to mitigate the effects of aircraft impact. Shown in Table 4.0-1 is a comparison of the base seismic reactions at Elevation 60.5' (AP1000 generic elevation of the bottom of the NI foundation) for the 2D hard rock case with the enhanced shield building to the DCD Rev. 15 configuration. The vertical seismic reactions are combined (+/-) with the dead weight (DW). As seen from this comparison, larger seismic reactions are obtained using the current NI structural configuration (the enhanced shield building) to those associated with the NI structural configuration for the AP1000 design during the hard rock licensing (DCD Rev. 15). Therefore, it is concluded that using NI structural models that includes the enhanced shield building for the Vogtle site-specific seismic assessments would bound the foundations loads from the same Vogtle site-specific seismic assessment using the DCD Rev. 15 NI structural configuration.

Table 4.0-1 – Seismic Reactions at the Bottom of the Basemat (Elevation 60.5')

Seismic Reaction	2D Hard Rock Enhanced Shield Building	2D Hard Rock (DCD 15)
Shear NS	123.75	99.81
Shear EW	112.31	93.52
DW + Vertical	385.1	382.2
DW - Vertical	187.6	179.2
Moment about Line I	13,011	12,639
Moment about SBW side	14,034	13,644
Moment about Line 11	17,506	14,791
Moment about Line 1	17,607	14,903

The Vogtle site-specific seismic analyses are performed using the same 2D stick models of the Nuclear Island used to obtain the 2D Certified Design AP1000 broaden envelop response spectra at the key locations (Reference 2).

4.1 2D Models

The 2D models of the Nuclear Island are stick models of the Auxiliary Shield Building (ASB), the Steel Containment Vessel (SCV), and the Containment Internal Structure (CIS). The concrete structures are modeled with linear elastic uncracked properties. However, the modulus of elasticity is reduced to 80% of its value to reduce stiffness to reflect the observed behavior of concrete when stresses do not result in significant cracking as recommended in Table 6.5 of FEMA 356.

The 2D models of the Nuclear Island are considered in conjunction with their foundation and supporting media to form a soil-structure interaction model. The 2D models provide good representation of the important modes of the structure and seismic interaction between the nuclear island structures. The SASSI model with adjacent soil layers to the Nuclear Island basemat is shown in Figure 4.1-1. It is noted that in this figure the different sticks for the ASB, SCV, and CIS are collocated, and therefore, appear as one stick even if there are three sticks present. The soil adjacent to the foundation is modeled by eight layers as shown. The horizontal soil element spacing is approximately 5 feet. Spring elements are used to connect the foundation to these adjacent soil layers. The springs transfer the compression between the structure and the soil. The soil beneath the foundation is modeled using 81 elements to a depth of 1050 feet. Three ESP soil profiles are used as shown in Figure 4.1-2. Two lines show in each figure; one is the ESP soil profile and the other is SASSI input soil profile data. The maximum sizes of the soil layers for the different depths are shown. The three soil profiles are lower bound, best estimate, and upper bound. The ESP profiles are discussed in Section 3.0.

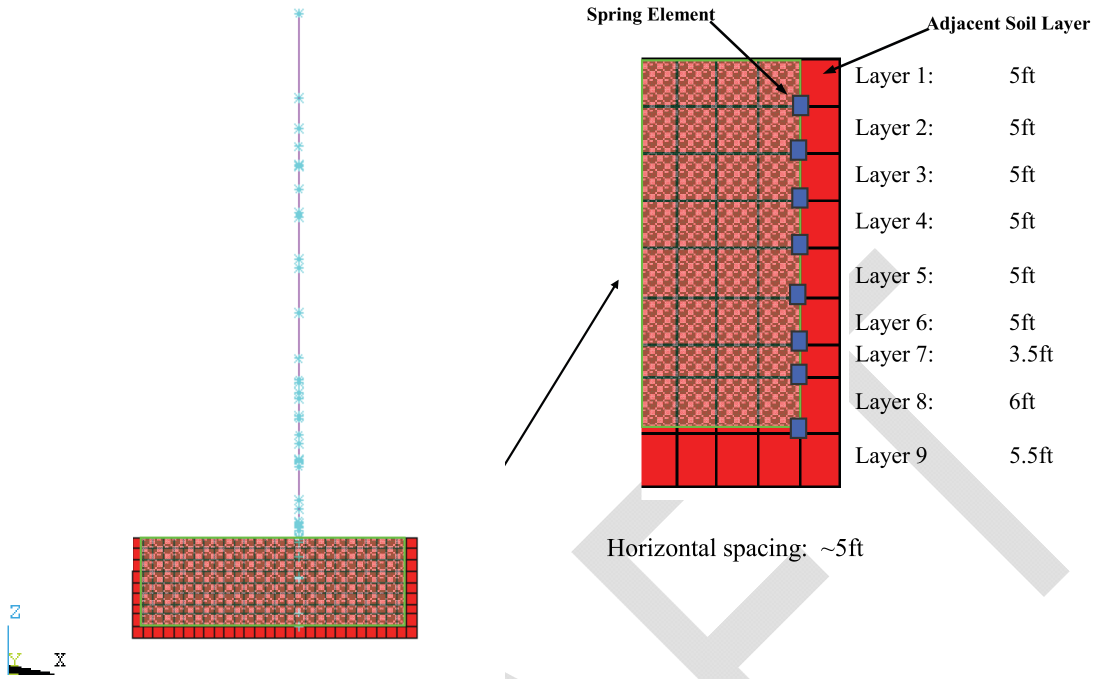
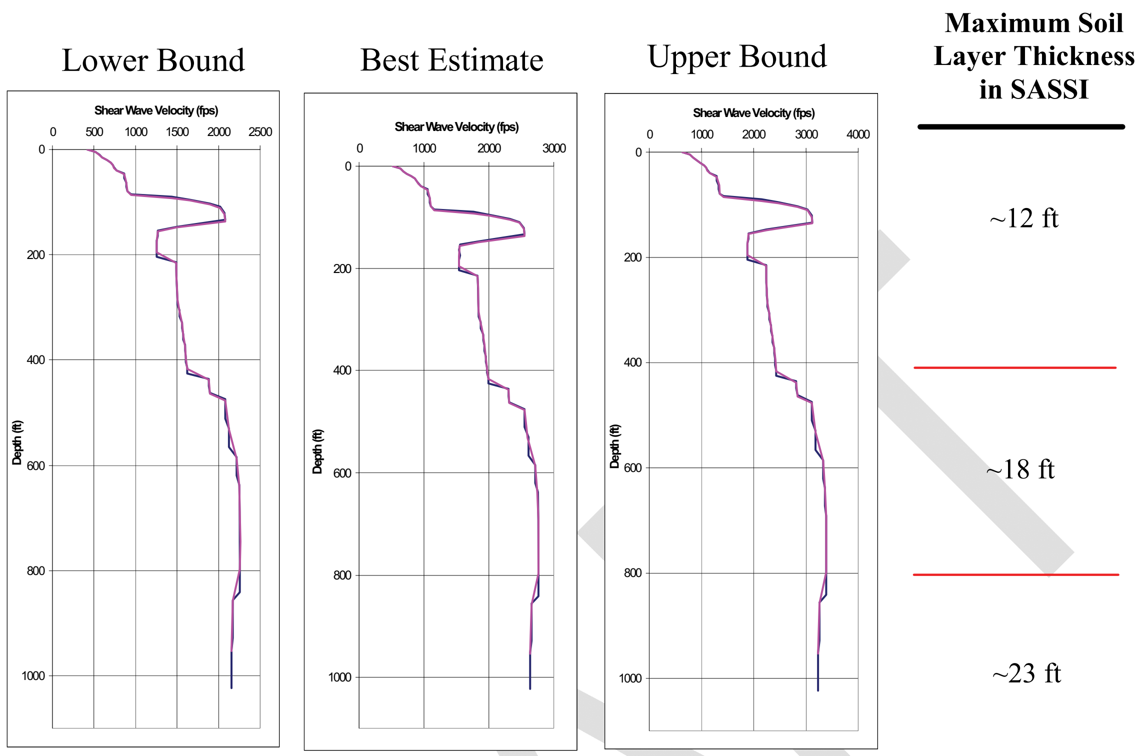


Figure 4.1-1 – 2D Soil Structure Interaction Model



Note: The blue line indicates the data provided by Bechtel and the red line indicates the data used by Westinghouse. The difference is due to layer thicknesses which is minor.

Figure 4.1-2 – ESP Shear Wave Velocity Profiles for 2D Site Soil Layers

4.2 Adjacent Buildings (Annex, Turbine, and Radwaste Buildings)

Since the Vogtle site is a deep soil site with a shallow inversion as shown in Figure 4.1-2, adjacent buildings (Annex, Radwaste, and Turbine buildings) dynamic models are included in the SASSI analyses. By including these buildings into the SASSI model, the adjacent building seismic demands can be obtained. The SASSI models with these building models are shown in Figures 4.2-1 and 4.2-2. It is noted that the Radwaste and Turbine buildings are represented as a lump mass at grade, whereas the Annex buildings are represented as sticks.

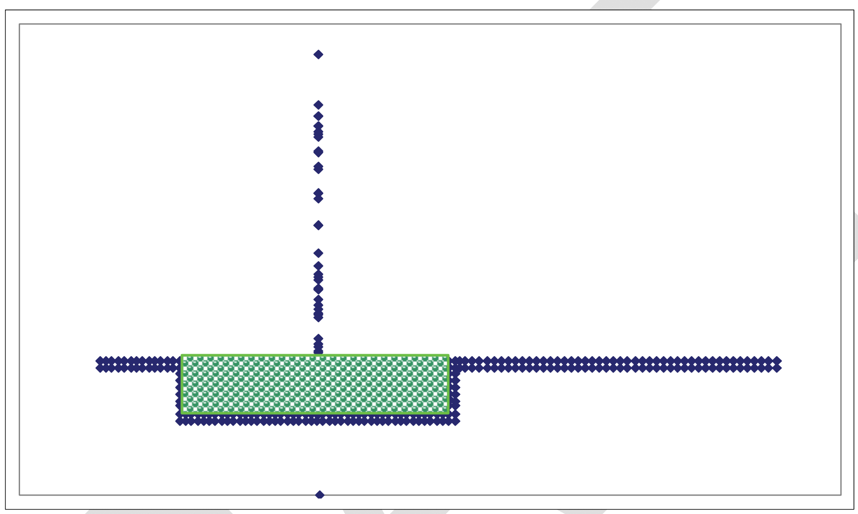
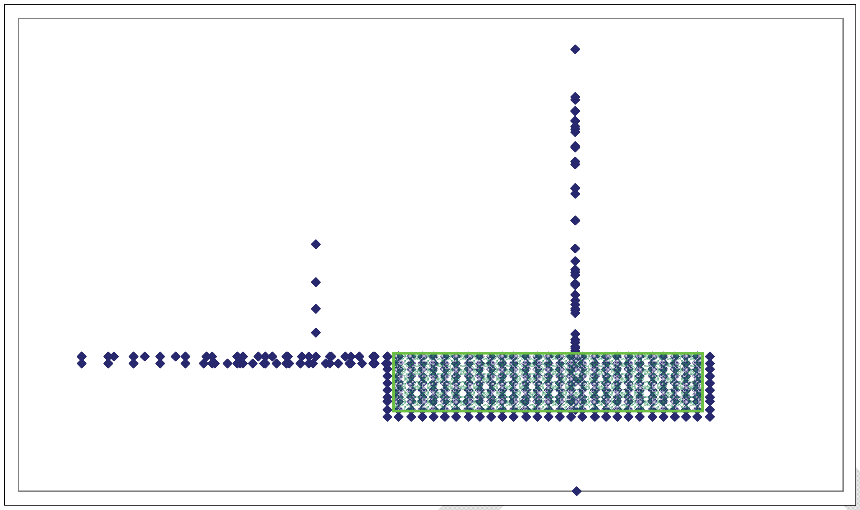


Figure 4.2-1 – Turbine and Radwaste Adjacent Buildings



East-West Model (y-dir)

Figure 4.2-2 – Annex Adjacent Building

5.0 Soil Cases and SSI Analyses

The SSI analyses are performed for the Early Site Permit (ESP) and Sensitivity (SEN) soil cases. The ESP and SEN each have three soil cases: lower bound, best estimate, and upper bound. Figure 5-1 is the shear wave velocity profiles to a depth of 160’ for each of the soil cases evaluated. Figure 5-2 shows the FIRS and GMRS locations used in the Vogtle site specific SASSI AP1000 NI Analyses.

Note that the SEN soil cases are based on a complete recalculation of site response assuming a higher shear wave velocity profile of the backfill than used for the ESP soil profiles. This resulted in a new set of FIRS, time histories, and strain compatible lower bound, best estimate, and upper bound soil profiles. The purpose of this study is to determine the sensitivity of the Vogtle AP1000 NI seismic response to a much wider range of backfill shear wave velocity profiles.

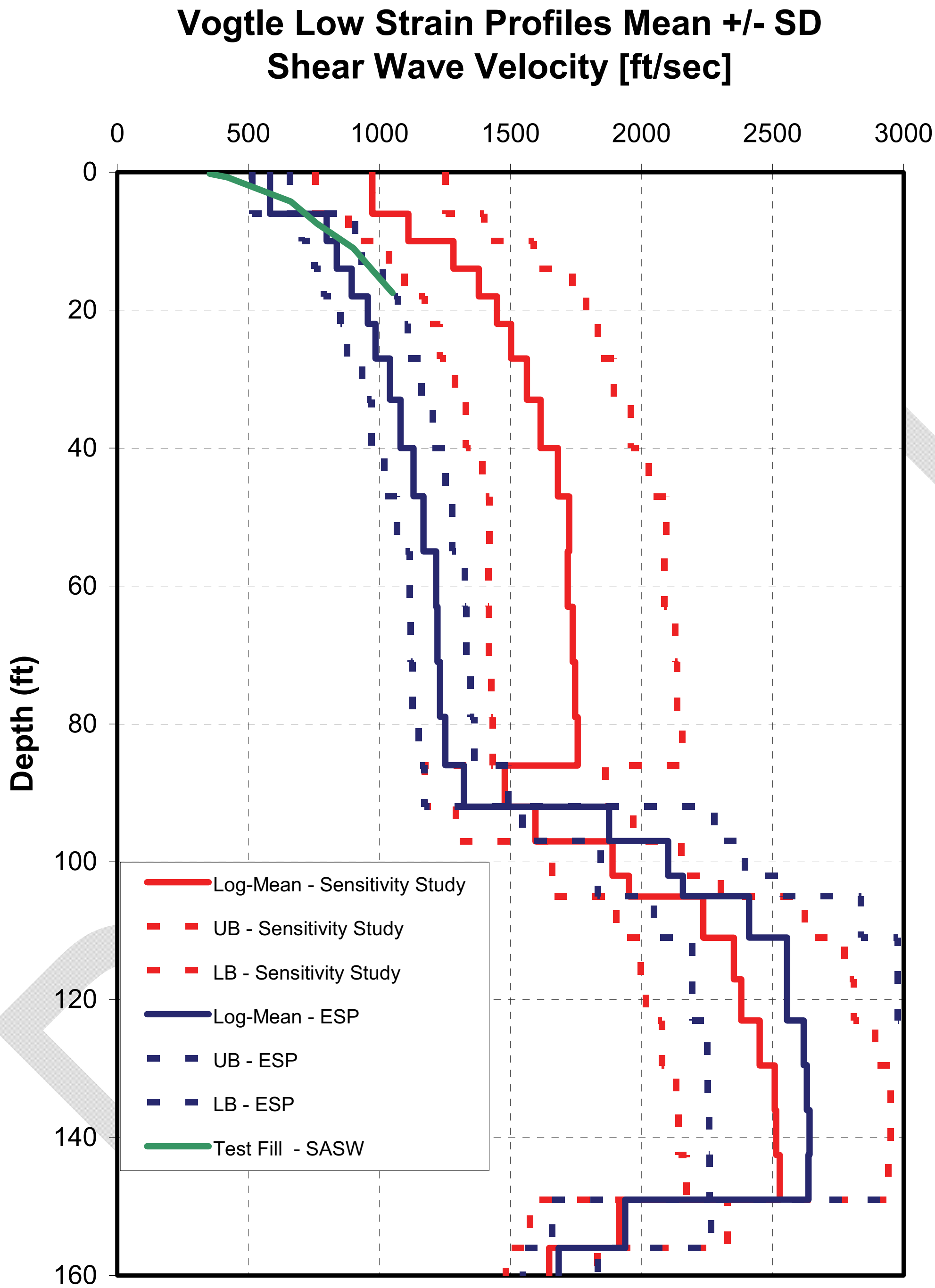


Figure 5.0-1 – Shear Wave Velocities for the ESP and SEN Soil Cases

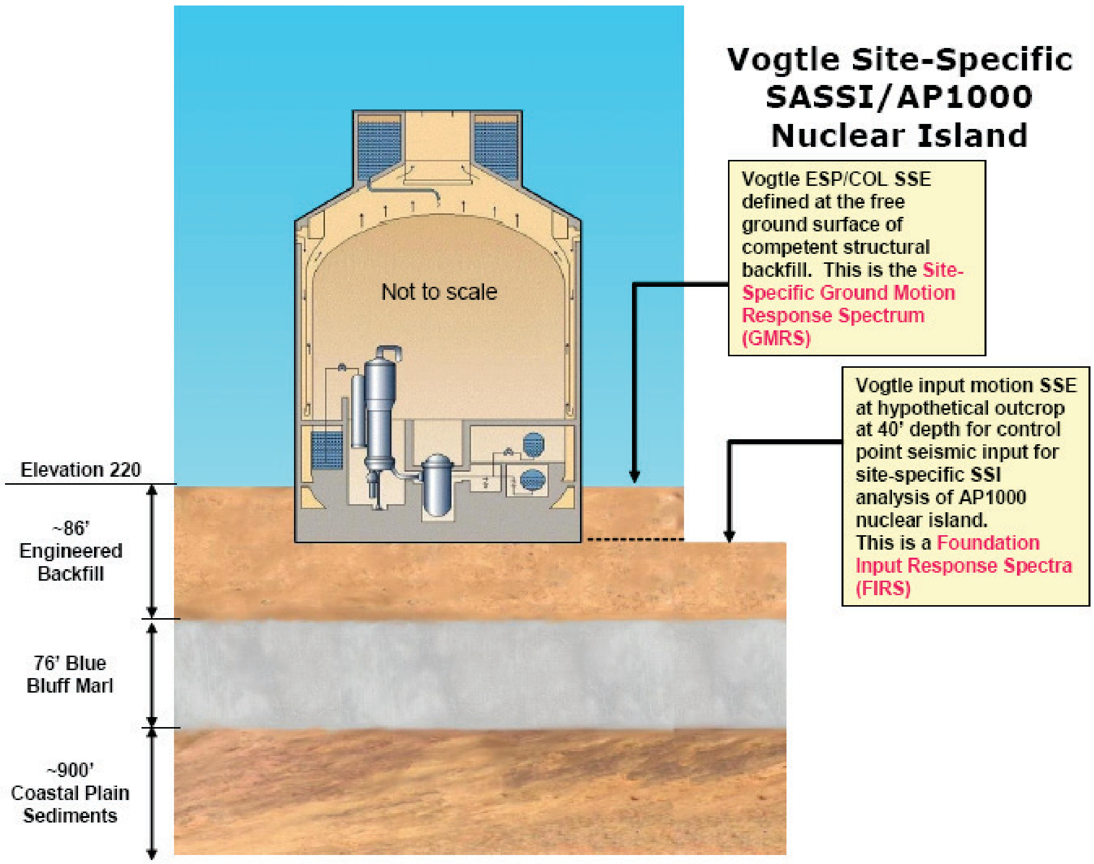


Figure 5.0-2 – FIRS and GMRS Locations in Vogtle Site Specific SASSI AP1000 NI Analyses

5.1 2D SASSI Analyses and Parameter Studies

This section describes the SASSI and parametric analyses performed using the 2D models that include the adjacent structures described in Section 4.2. The Vogtle site-specific soil cases are analyzed.

Figures 5.1-1 to 5.1-18 are the response spectra with 5% damping compared to the AP1000 SSI Envelope for the ESP soil cases. Figures 5.1-19 to 5.1-36 are the comparisons for the SEN soil cases. These spectra are also at 5% damping. The floor response spectra are given at the six key locations as defined in Table 5.1-1.

As seen from these spectra, there is a slight exceedance in the range from 0.5 hertz to 0.6 hertz in the NS direction and 0.45 hertz to 0.65 hertz in the EW direction. The only dynamic response in this region is due to tank sloshing. As seen from Table 5.1-2, the sloshing frequencies are away from this region of exceedance. Sloshing within the tanks will not affect the AP1000 plant design.

Table 5.1-1 – Critical Nodes Selected

Nodes	AP 1000 Generic Plant Elevation (ft)	Description
4041	99.00	NI at Reactor Vessel Support Elevation
4061	116.5	Auxiliary Shield Building at Control Room Floor
4120	179.56	ASB Auxiliary Building Roof Area
4310	327.41	ASB Shield Building Roof Area
4412	224	Steel Containment Vessel near Polar Crane
4535	134.25	Containment Internal Structure at Operating Deck

Table 5.1-2 – Sloshing Frequencies

Tank and Seismic Response Direction	Frequency Hertz
Fuel Area	
Fuel Pool, EW	0.39
Fuel Pool, NS	0.26
Fuel Transfer Canal, EW	0.68
Fuel Transfer Canal, NS	0.26
Cask Loading Pit, EW	0.39
Cask Loading Pit, NS	0.37
Cask Washdown Pit, EW	0.39
Cask Washdown Pit, NS	0.36
IRWST Tank	
Steel Wall, EW	0.41
Steel Wall, NS	0.25
NE Wall, EW	0.36
North Wall Pressurizer, NS	0.29
West Wall, EW	0.29
South Wall, NS	0.29
Shielding Building	
PCCS Tank	0.136

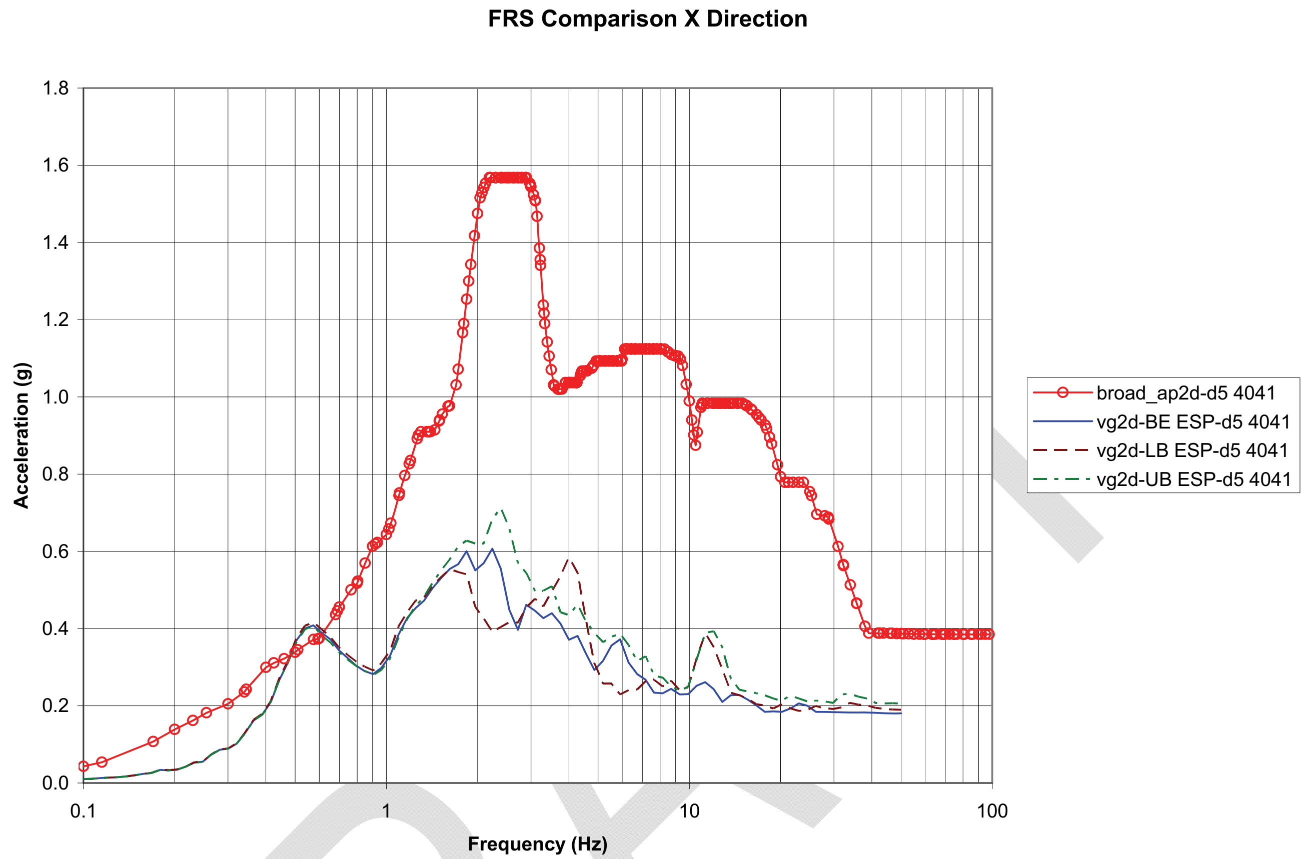


Figure 5.1-1 - Comparison of Node 4041 ESP Response to AP1000 SSI Envelope, NS Dir.

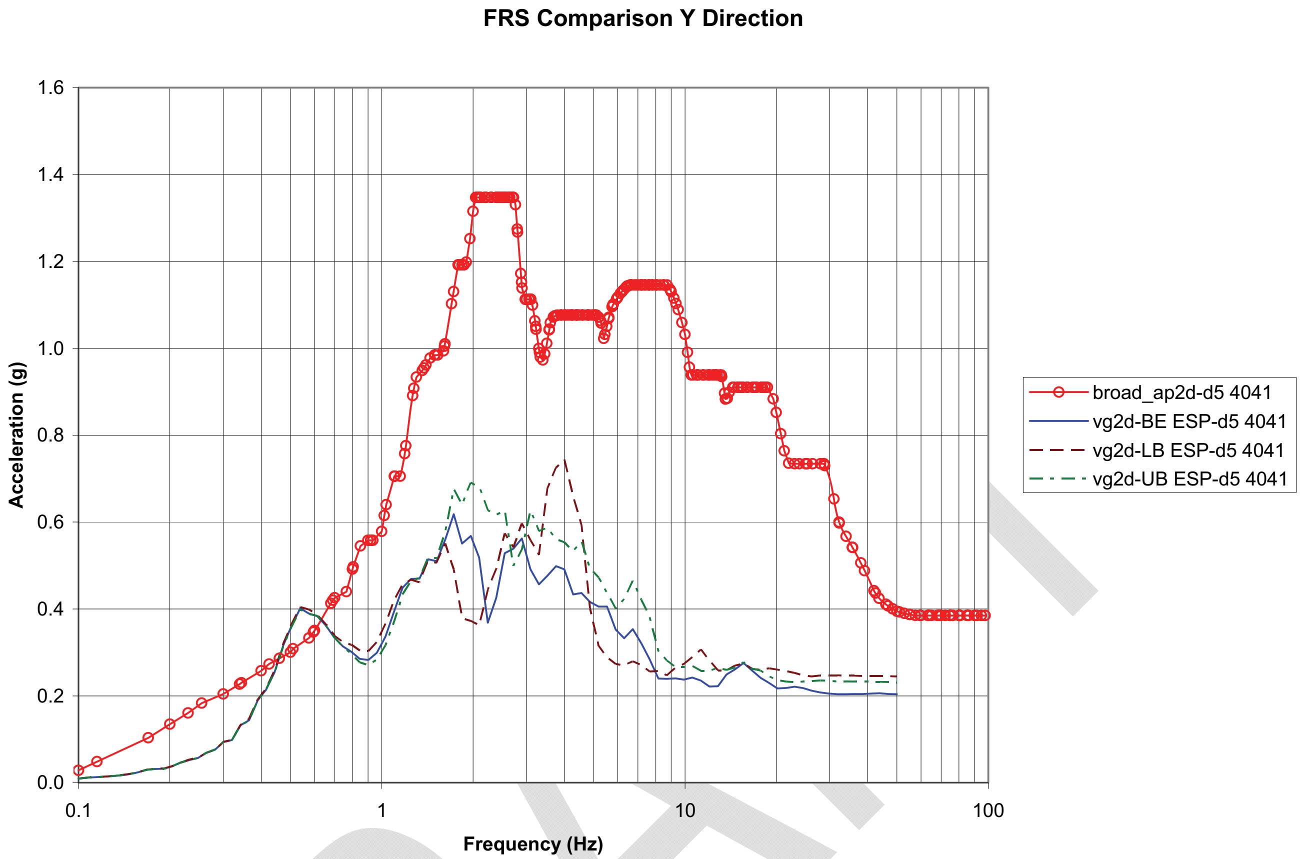


Figure 5.1-2 - Comparison of Node 4041 ESP Response to AP1000 SSI Envelope, EW Dir.

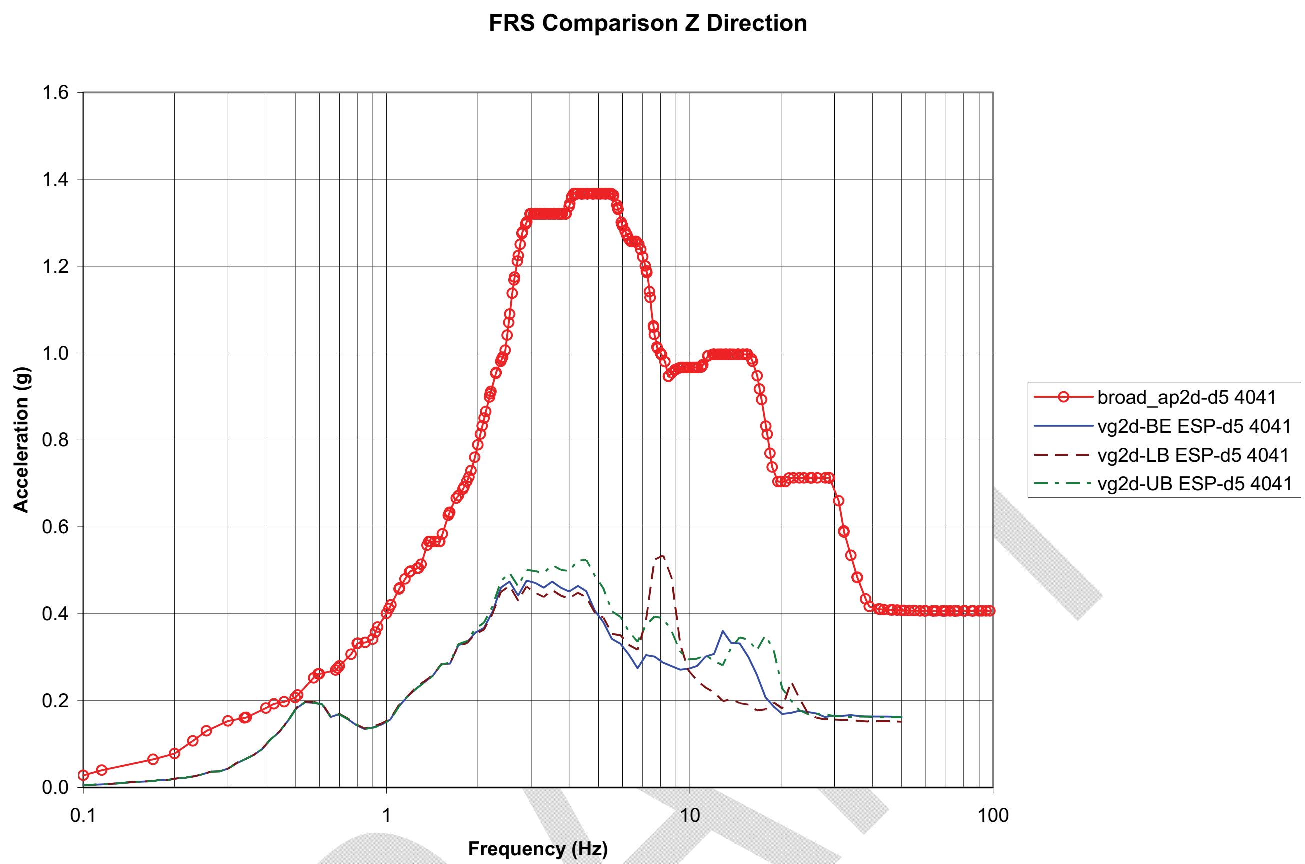


Figure 5.1-3 - Comparison of Node 4041 ESP to AP1000 SSI Envelope, Vertical Dir.

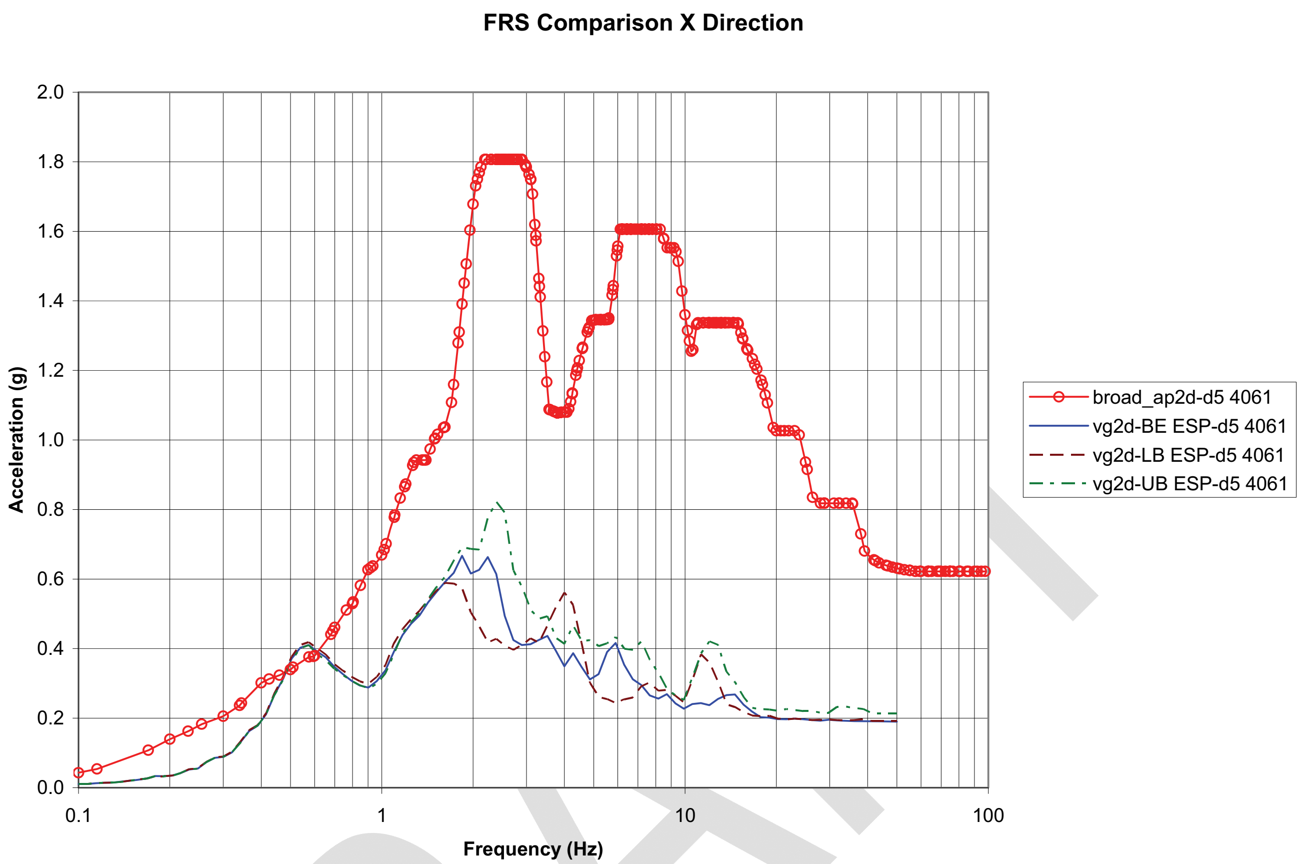


Figure 5.1-4 - Comparison of Node 4061 ESP to AP1000 SSI Envelope, NS Dir.

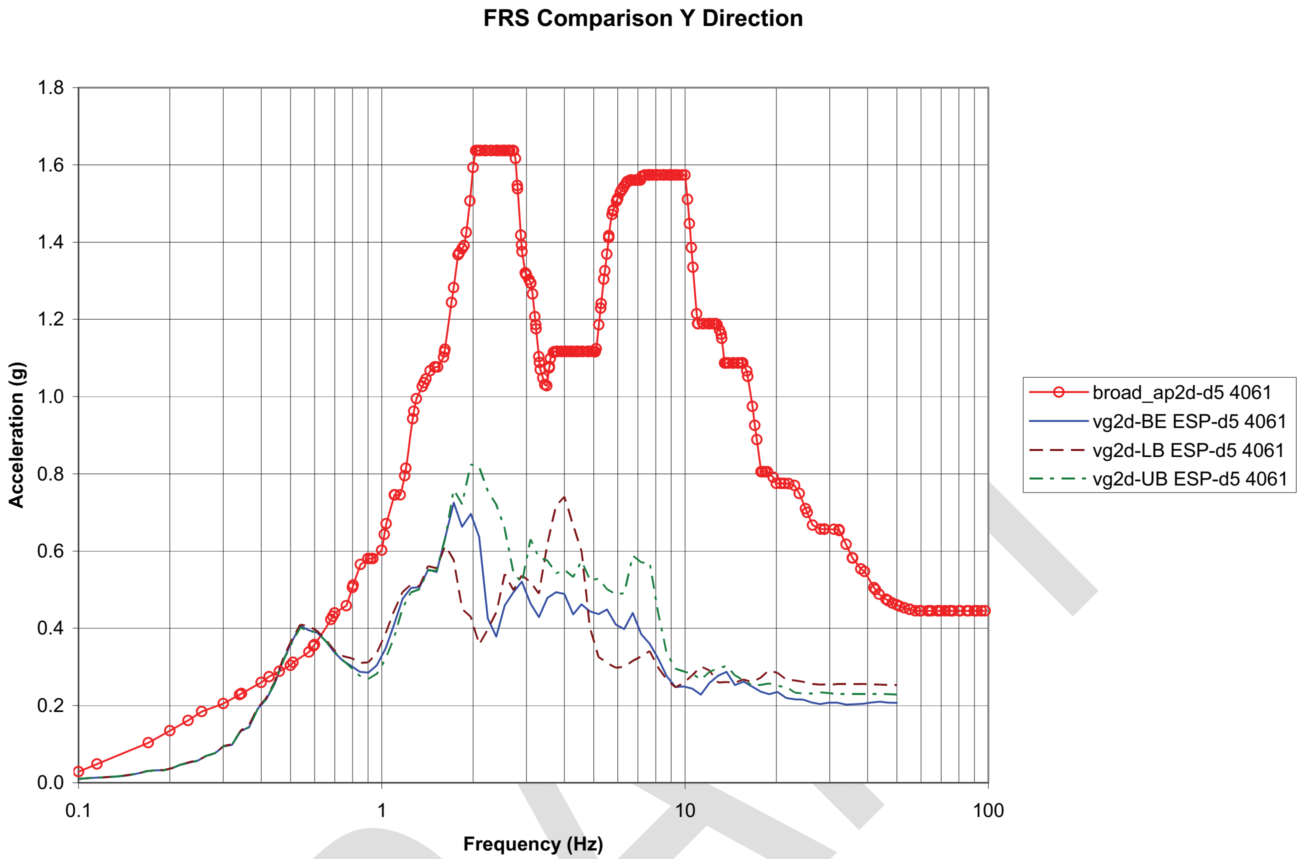


Figure 5.1-5 - Comparison of Node 4061 ESP to AP1000 SSI Envelope, EW Dir

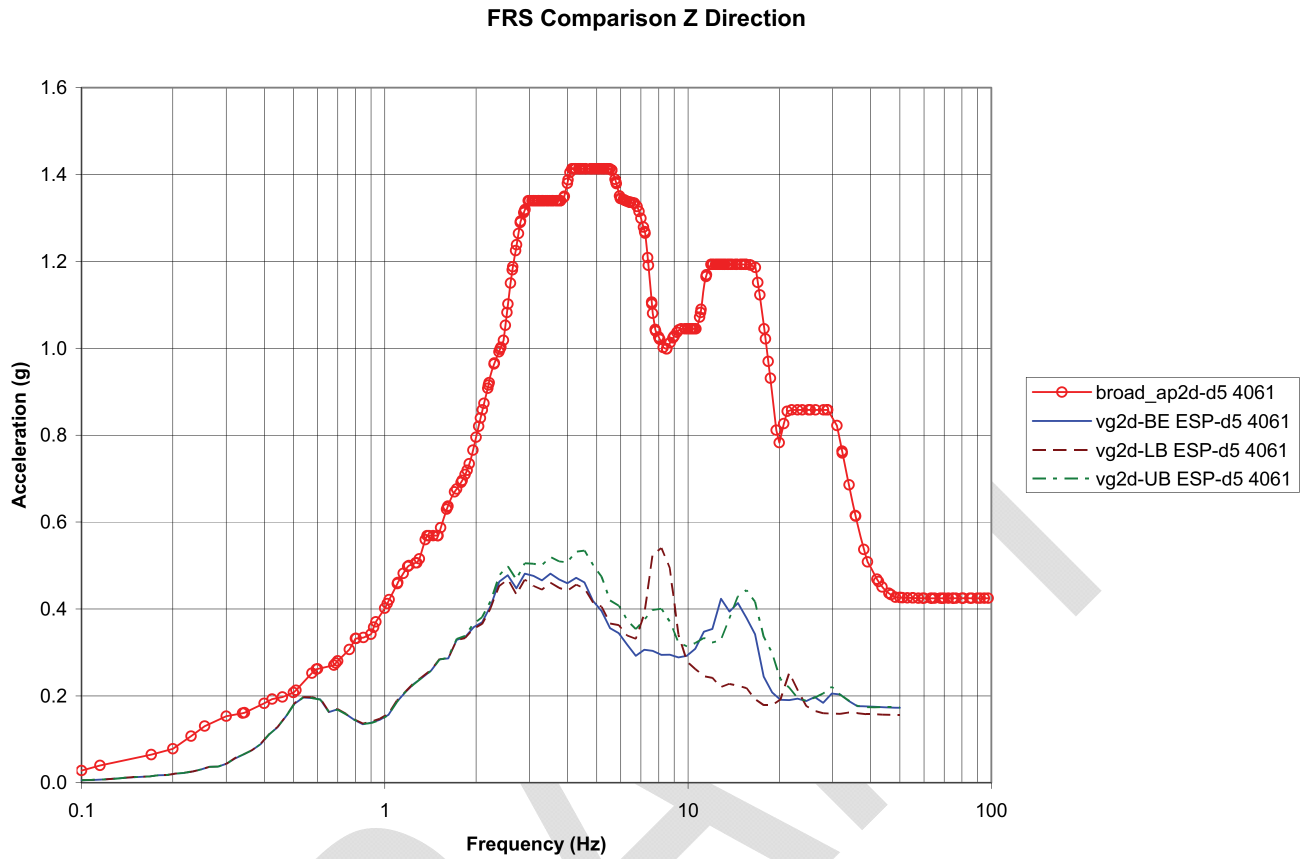


Figure 5.1-6 - Comparison of Node 4061 ESP to AP1000 SSI Envelope, Vertical Dir

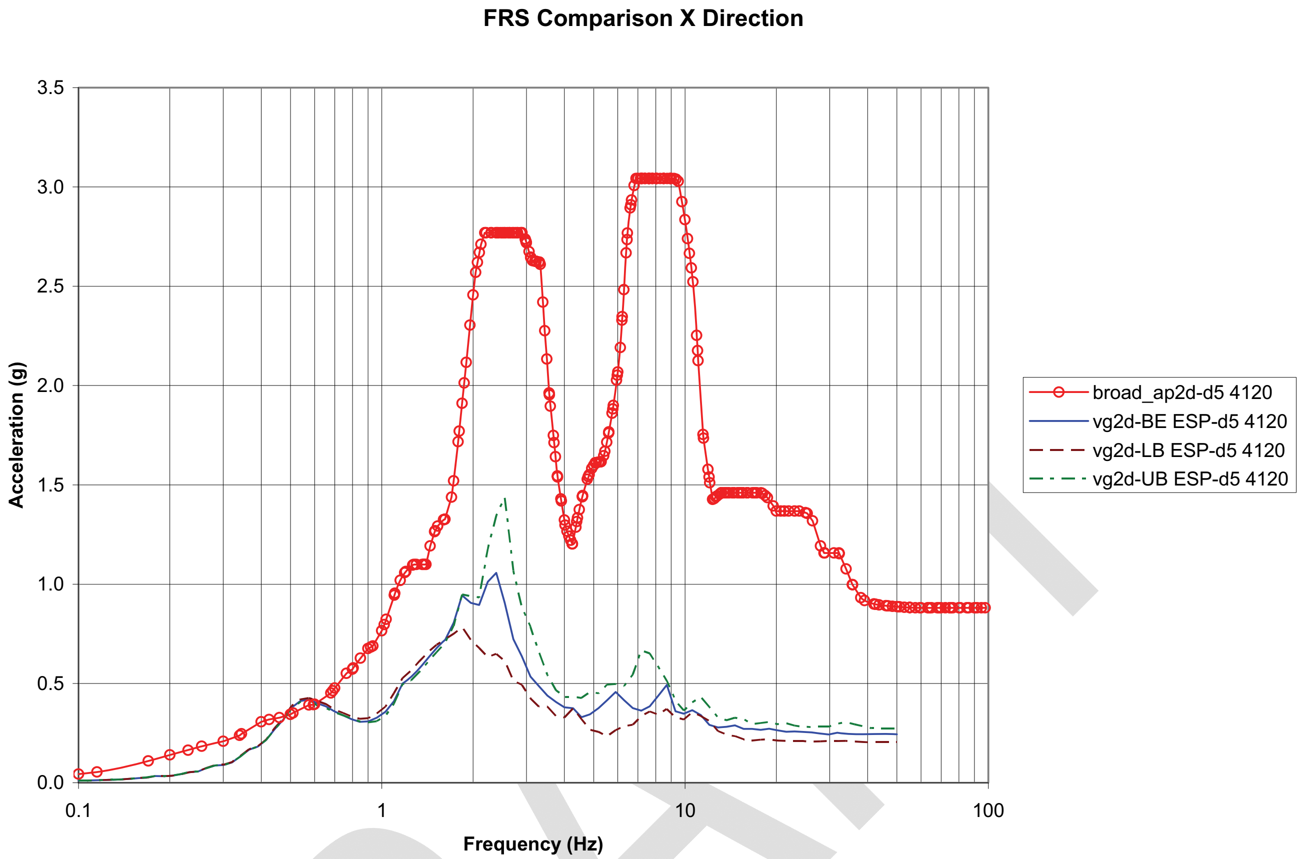


Figure 5.1-7 - Comparison of Node 4120 ESP to AP1000 SSI Envelope, NS Dir

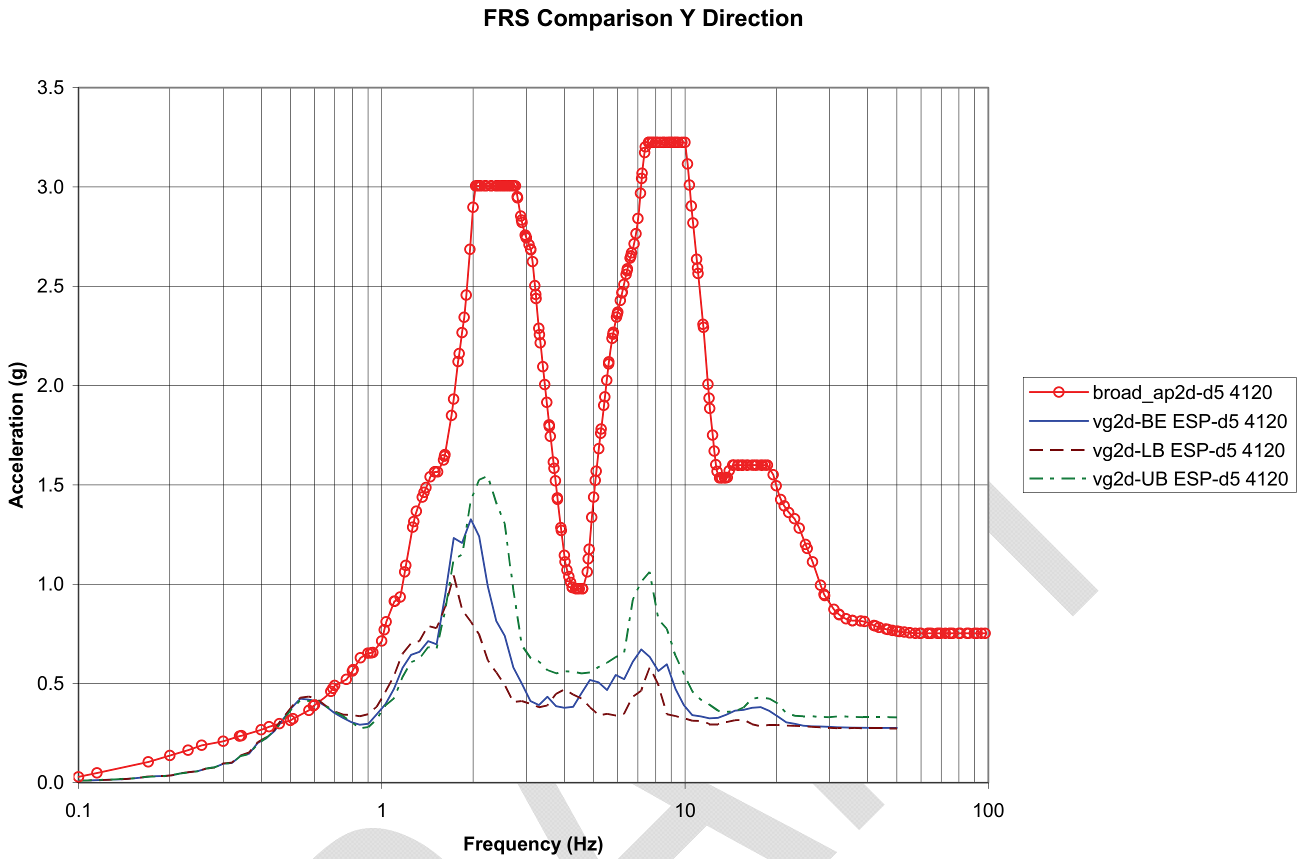


Figure 5.1-8 - Comparison of Node 4120 ESP to AP1000 SSI Envelope, EW Dir

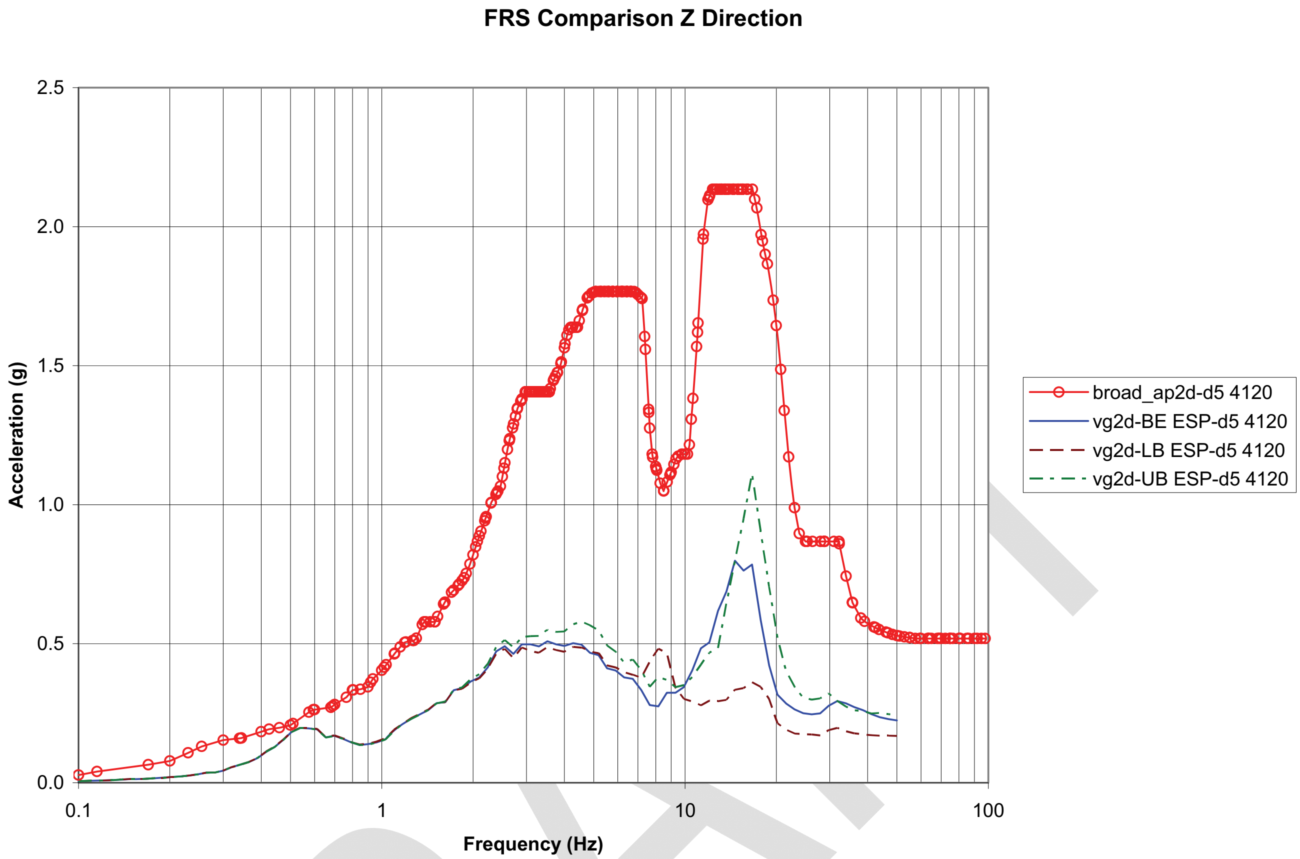


Figure 5.1-9 - Comparison of Node 4120 ESP to AP1000 SSI Envelope, Vertical Dir

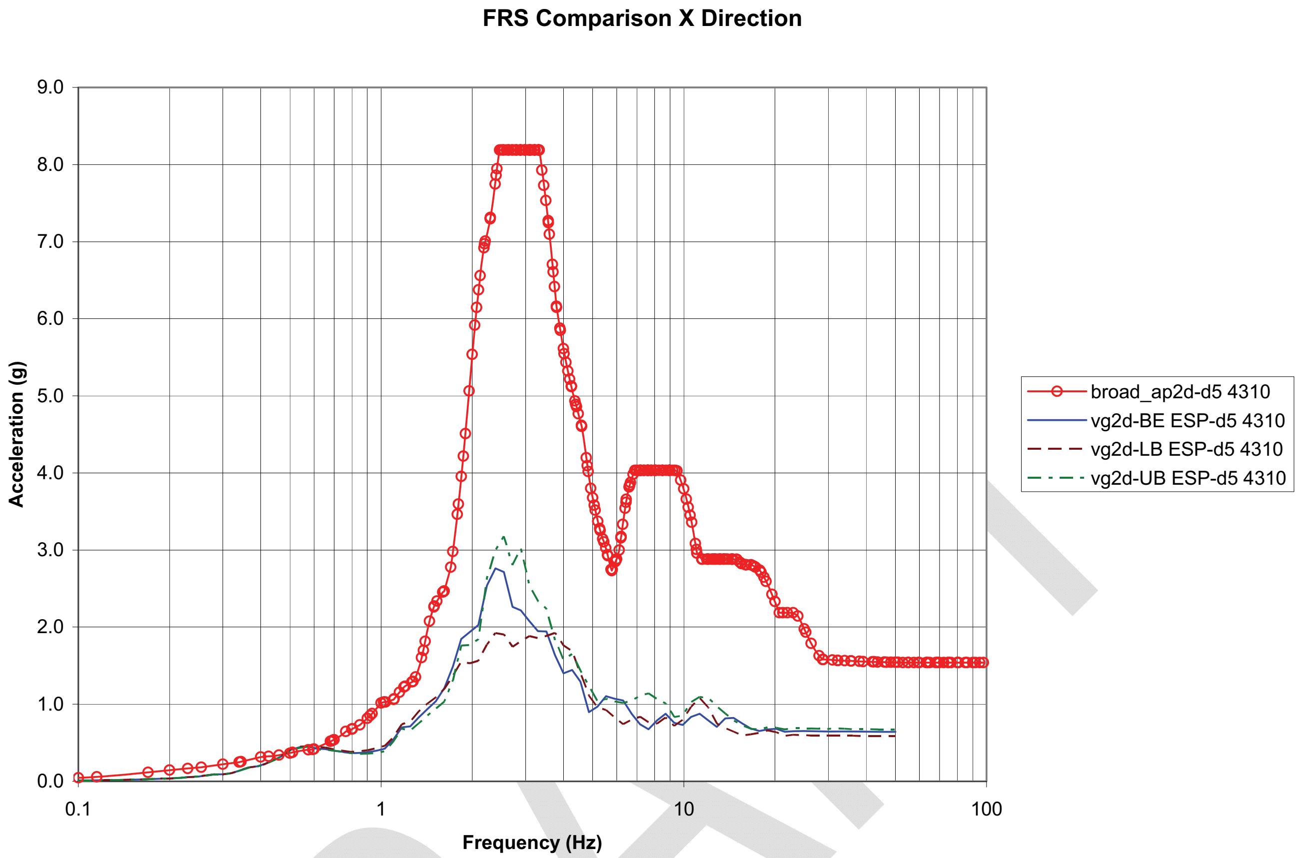


Figure 5.1-10 - Comparison of Node 4310 ESP to AP1000 SSI Envelope, NS Dir

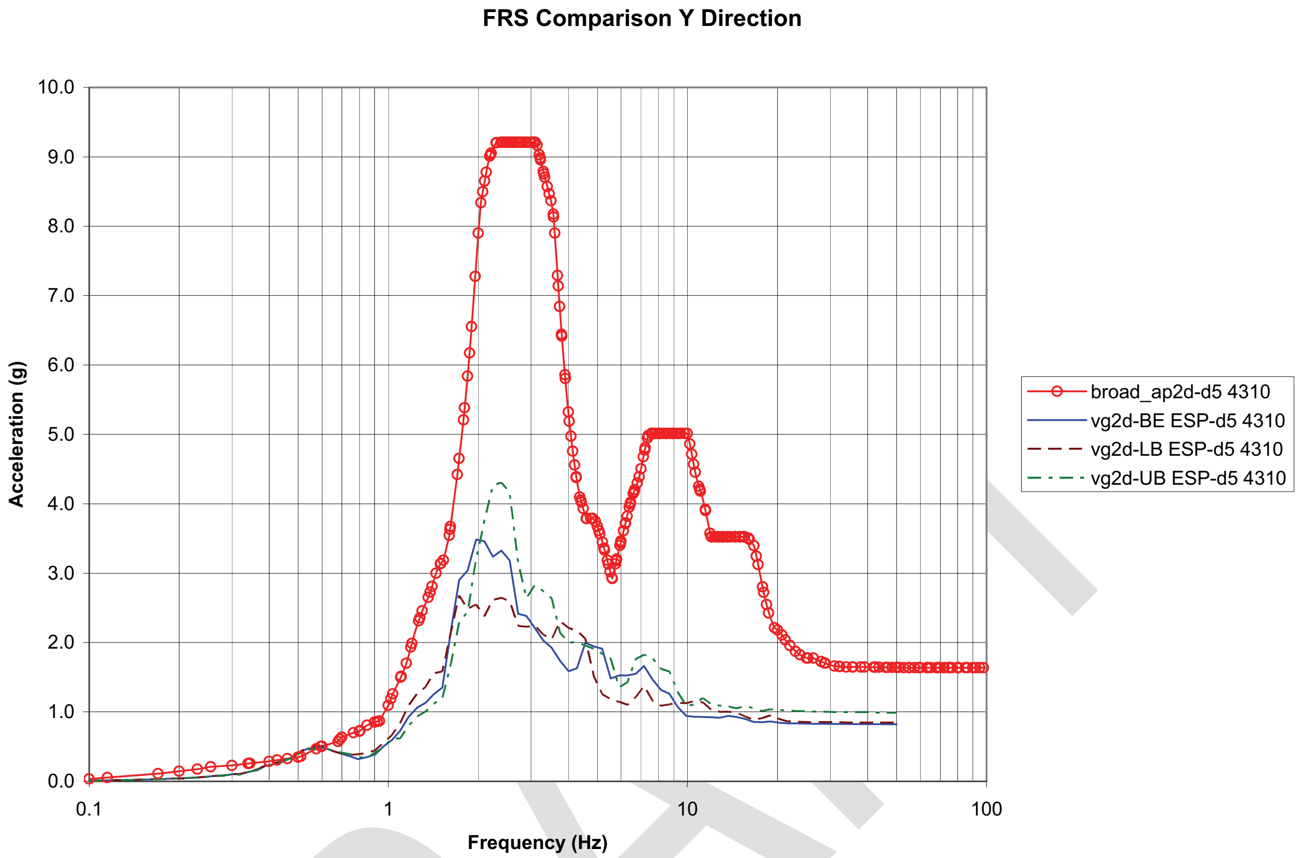


Figure 5.1-11 - Comparison of Node 4310 ESP to AP1000 SSI Envelope, EW Dir

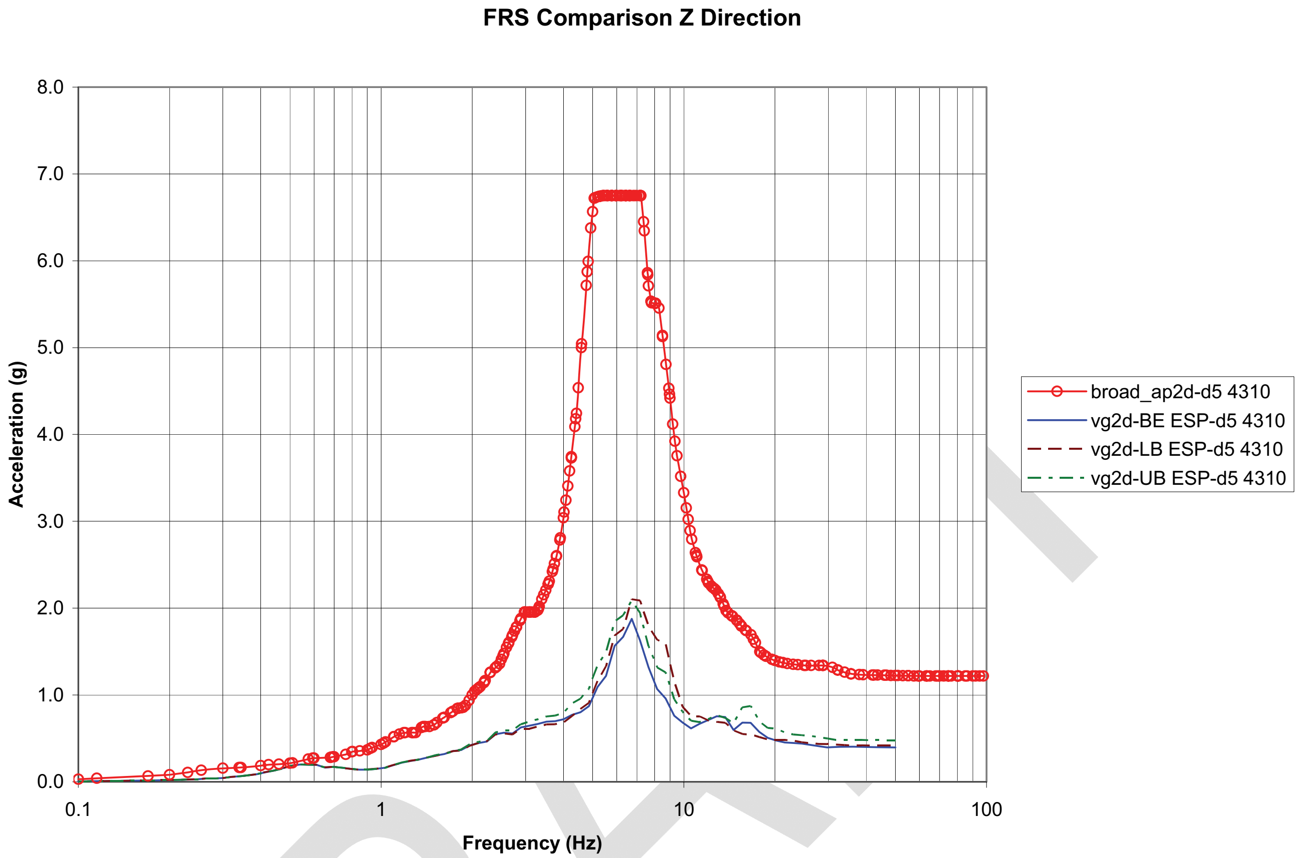


Figure 5.1-12 - Comparison of Node 4310 ESP to AP1000 SSI Envelope, Vertical Dir

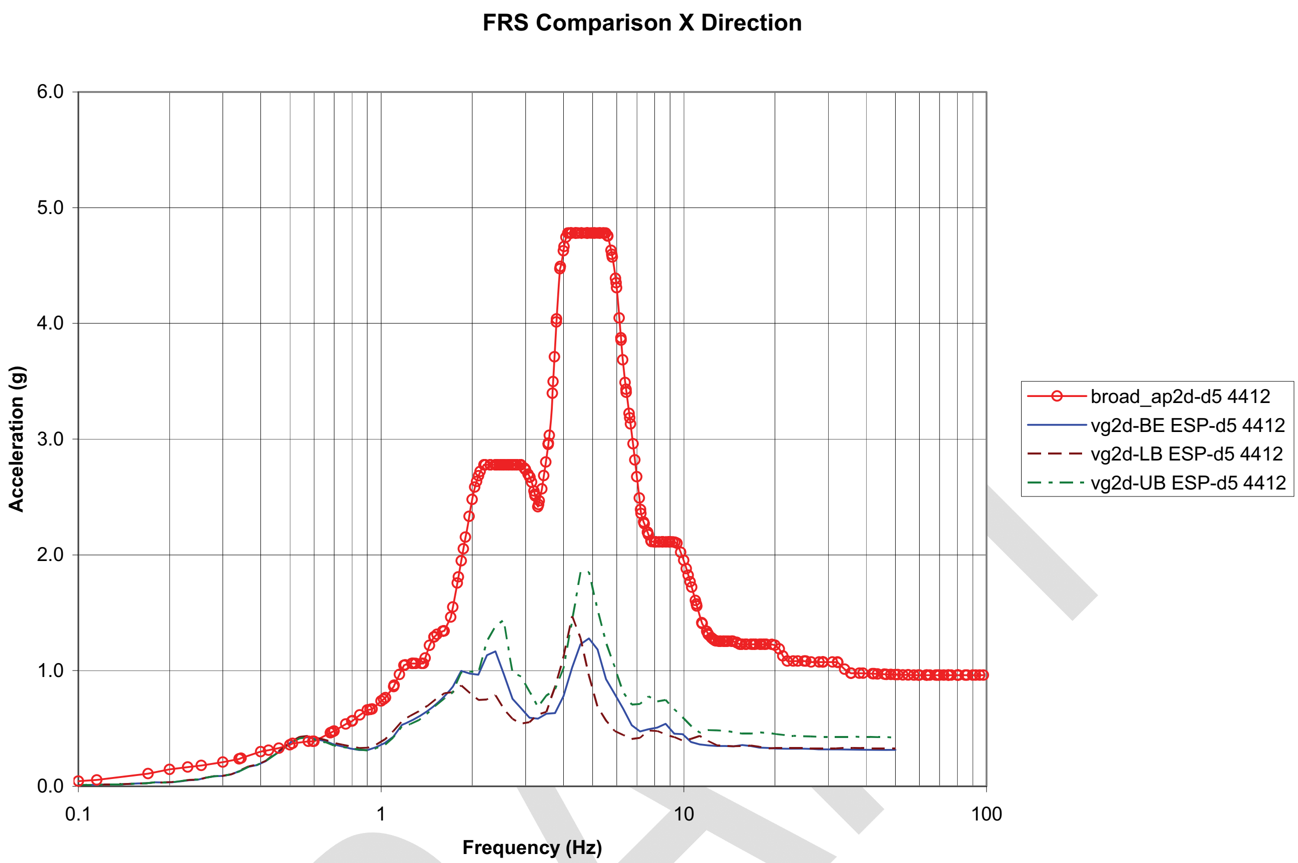


Figure 5.1-13 - Comparison of Node 4412 ESP to AP1000 SSI Envelope, NS Dir

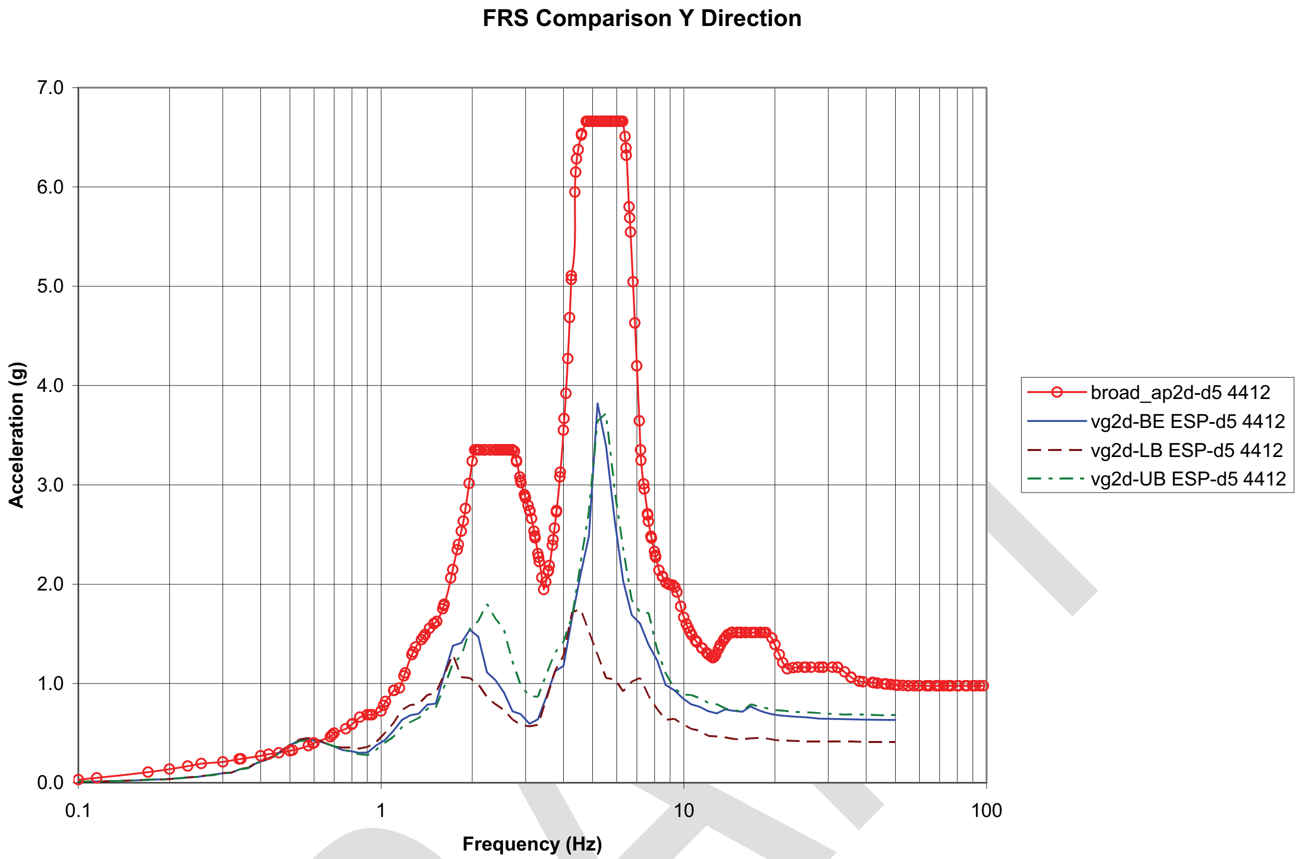


Figure 5.1-14 - Comparison of Node 4412 ESP to AP1000 SSI Envelope, EW Dir

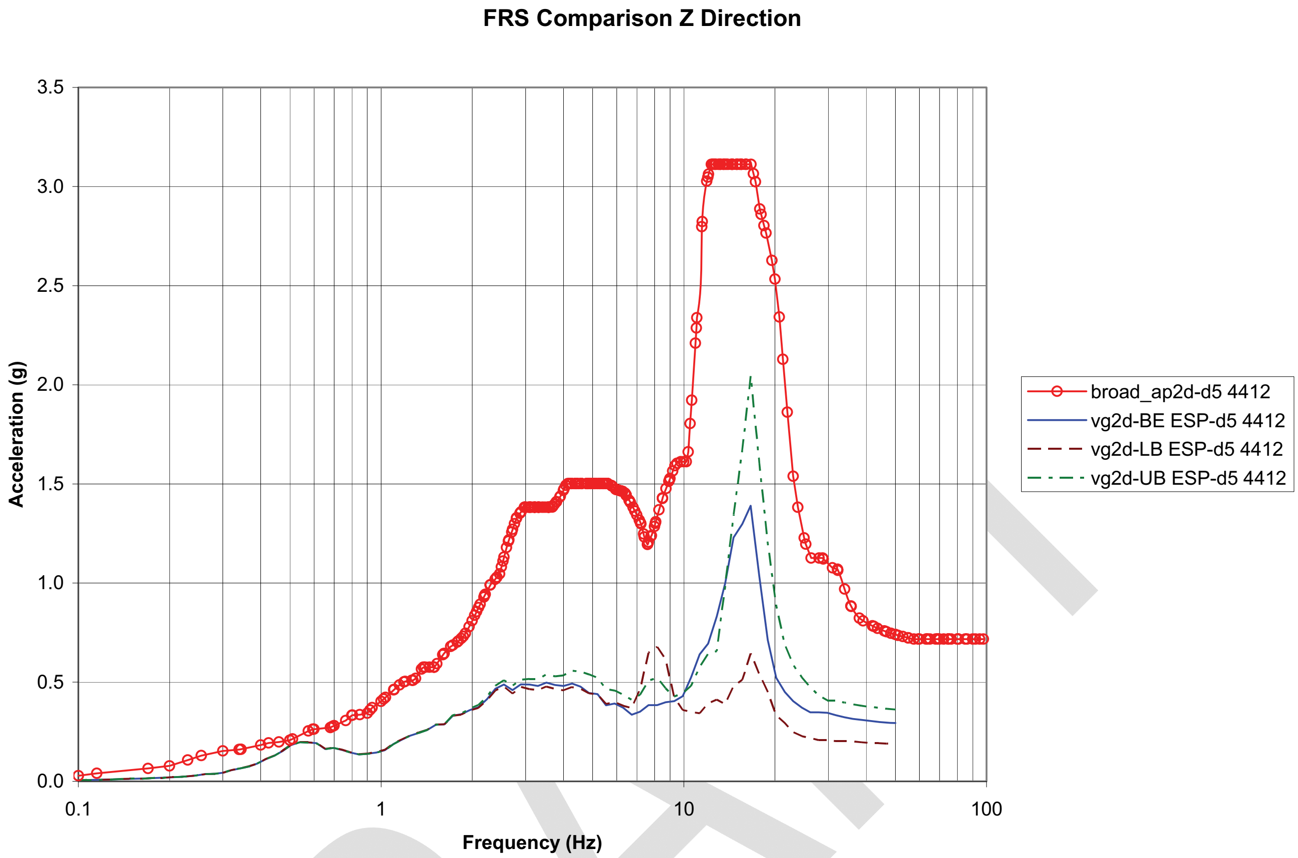


Figure 5.1-15 - Comparison of Node 4412 ESP to AP1000 SSI Envelope, Vertical Dir

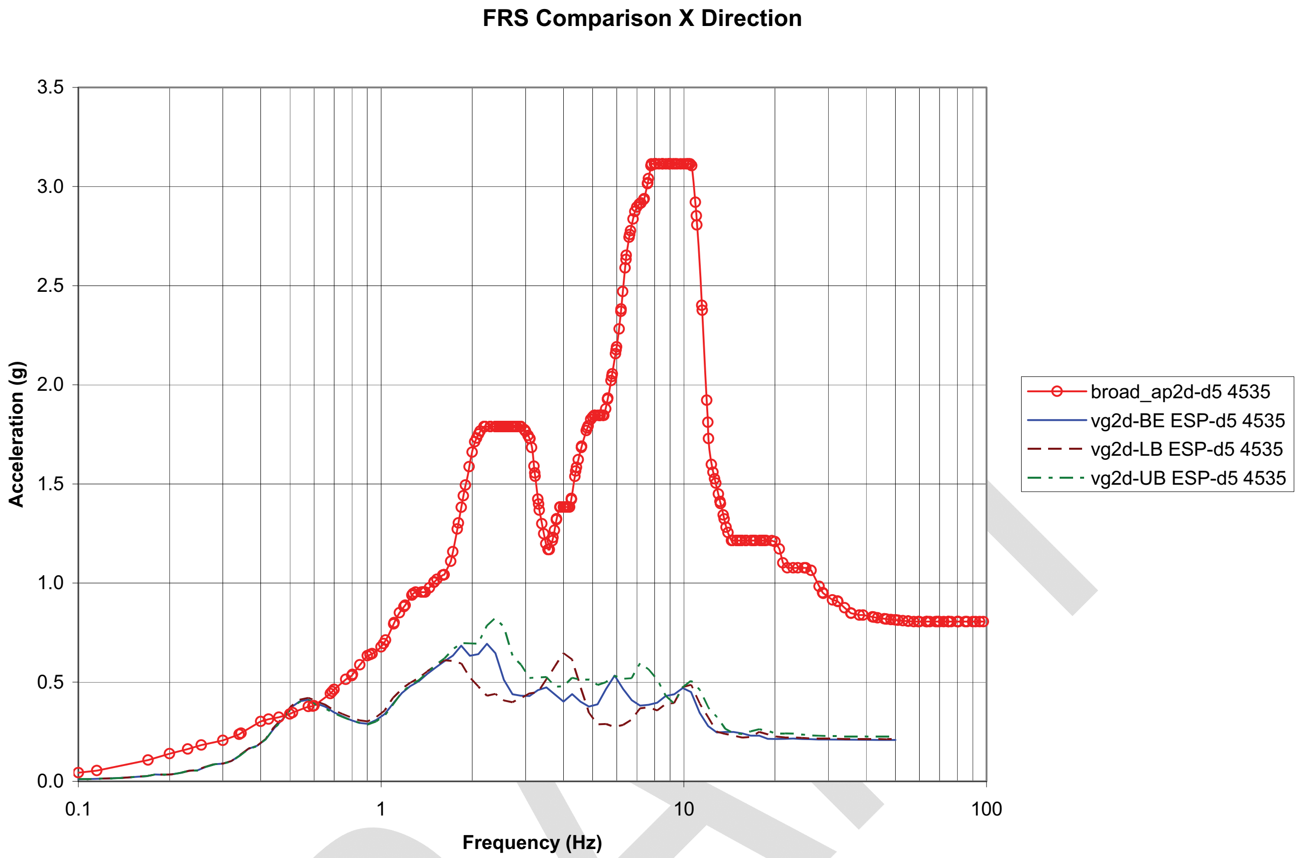


Figure 5.1-16 - Comparison of Node 4535 ESP to AP1000 SSI Envelope, NS Dir

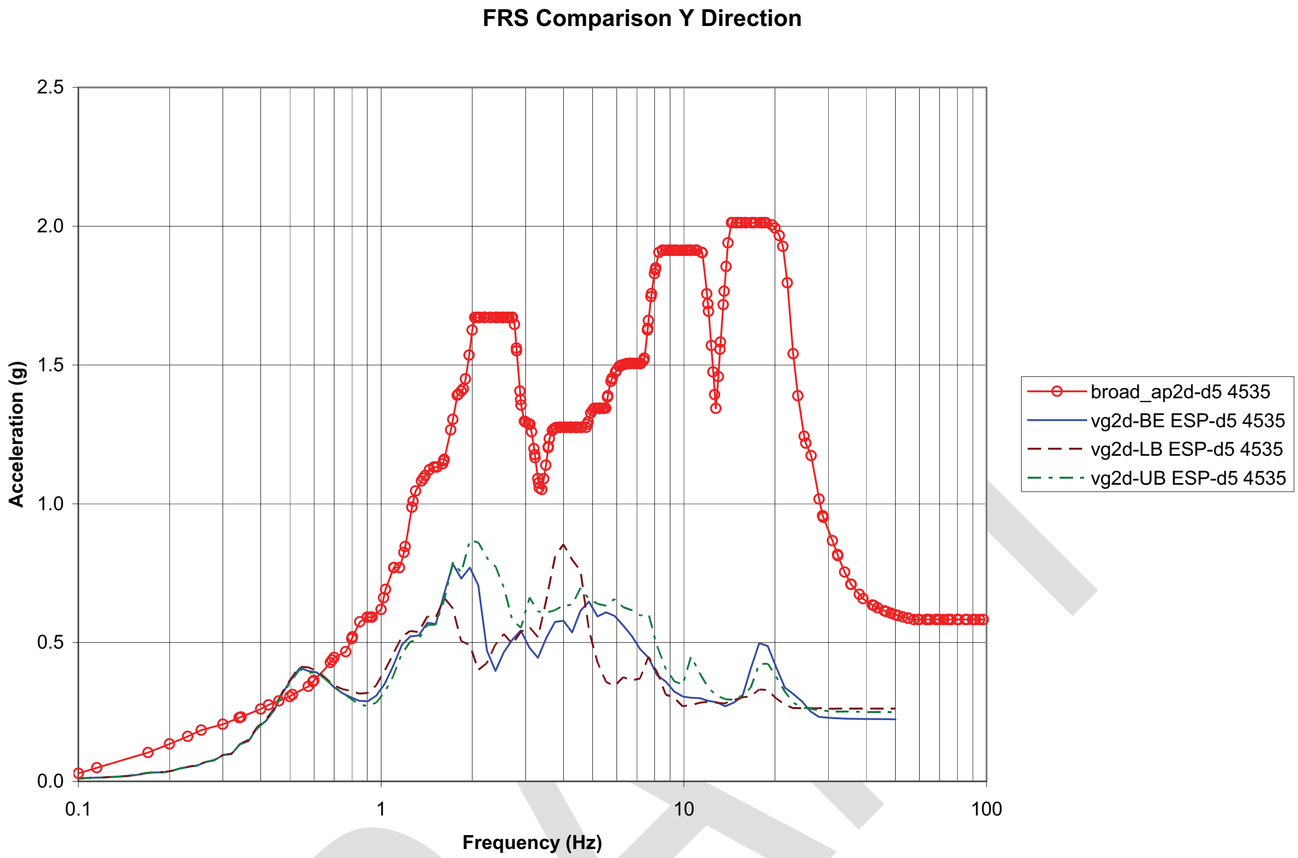


Figure 5.1-17 - Comparison of Node 4535 ESP to AP1000 SSI Envelope, EW Dir

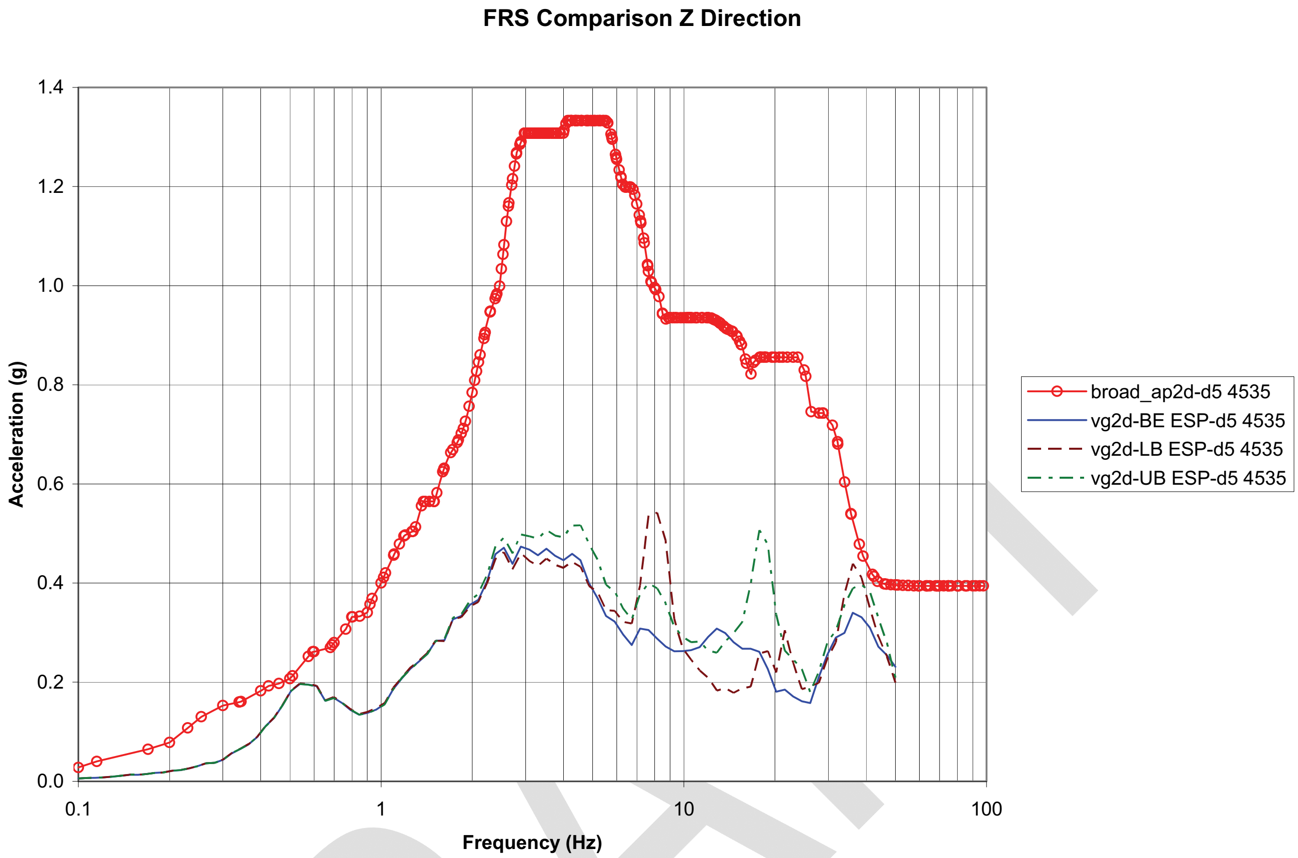


Figure 5.1-18 - Comparison of Node 4535 ESP to AP1000 SSI Envelope, Vertical Dir

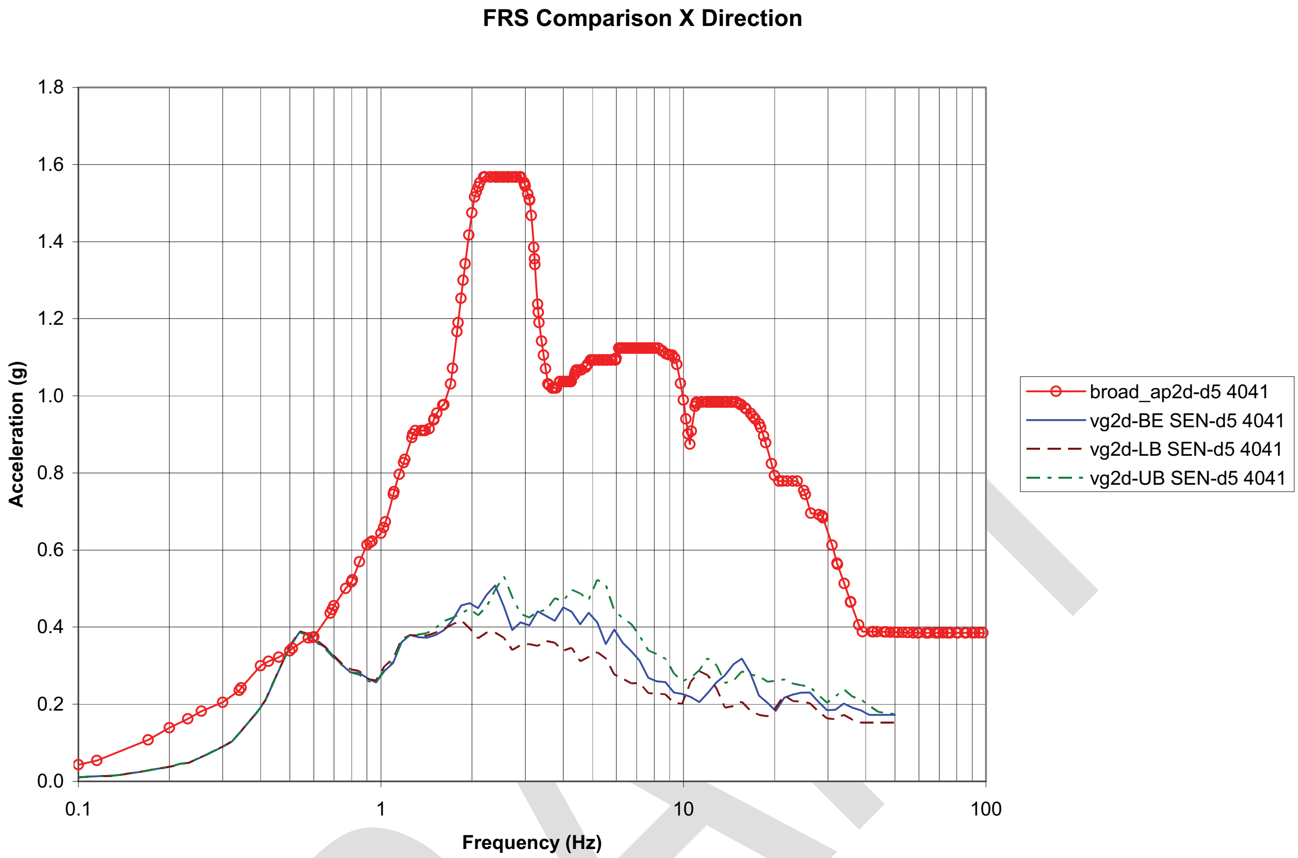


Figure 5.1-19 - Comparison of Node 4041 SEN to AP1000 SSI Envelope, NS Dir

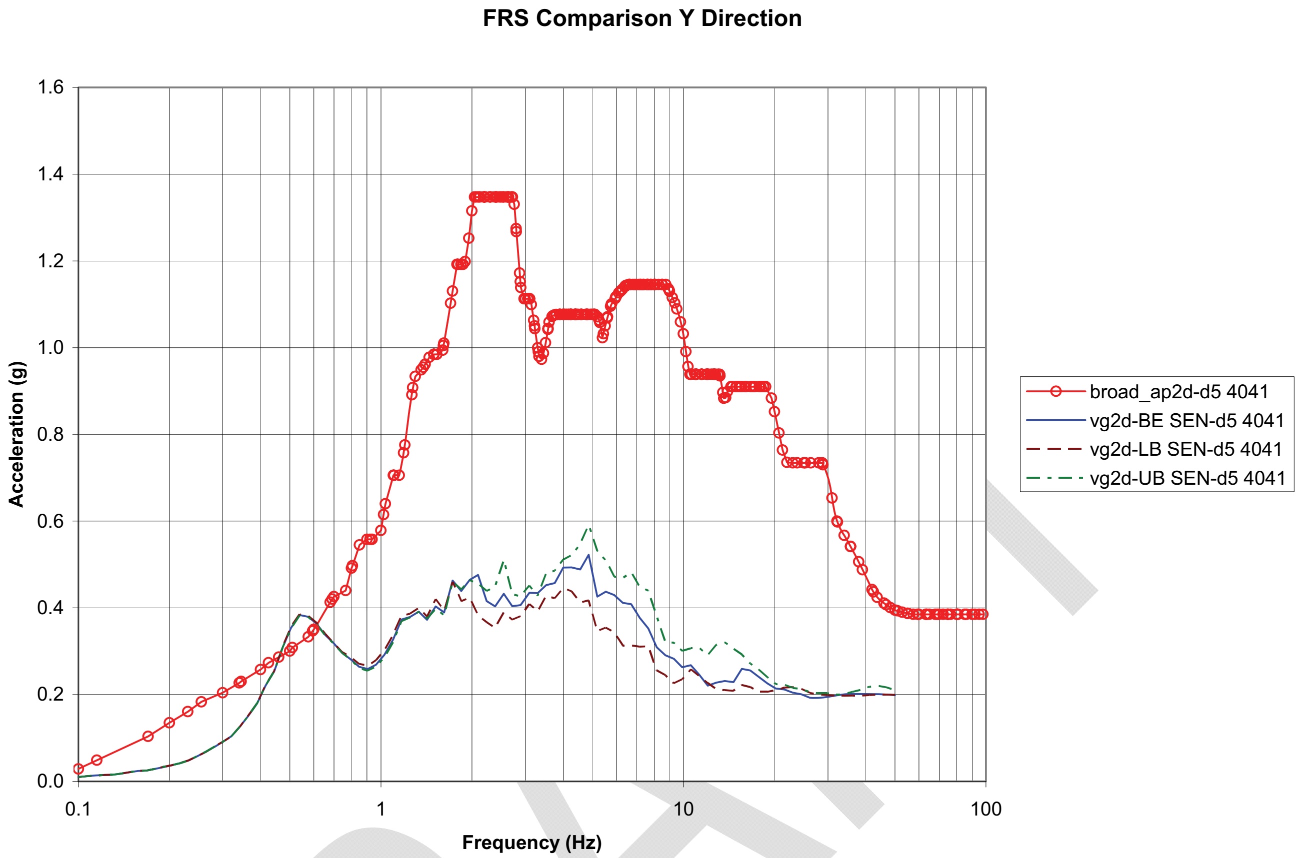


Figure 5.1-20 - Comparison of Node 4041 SEN to AP1000 SSI Envelope, EW Dir

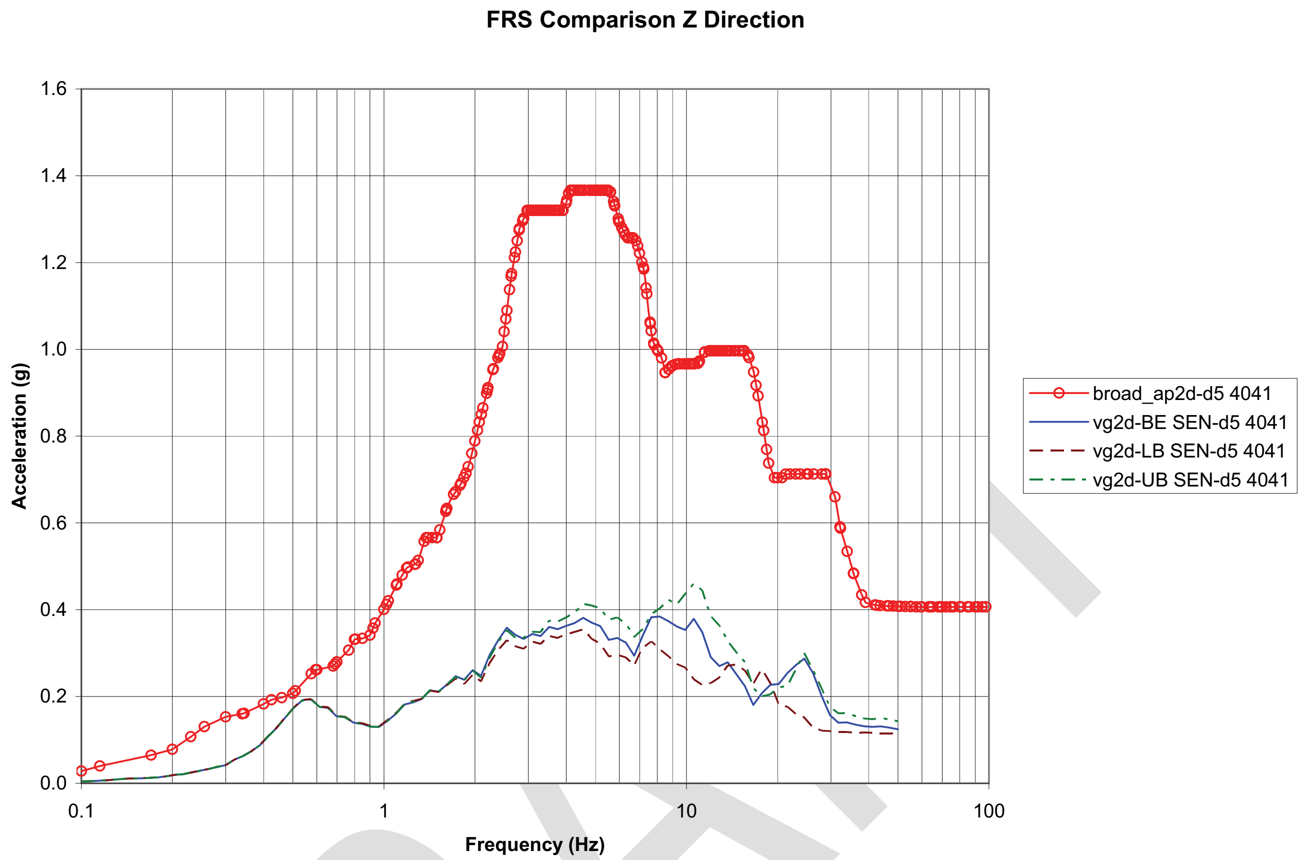


Figure 5.1-21 - Comparison of Node 4041 SEN to AP1000 SSI Envelope, Vertical Dir

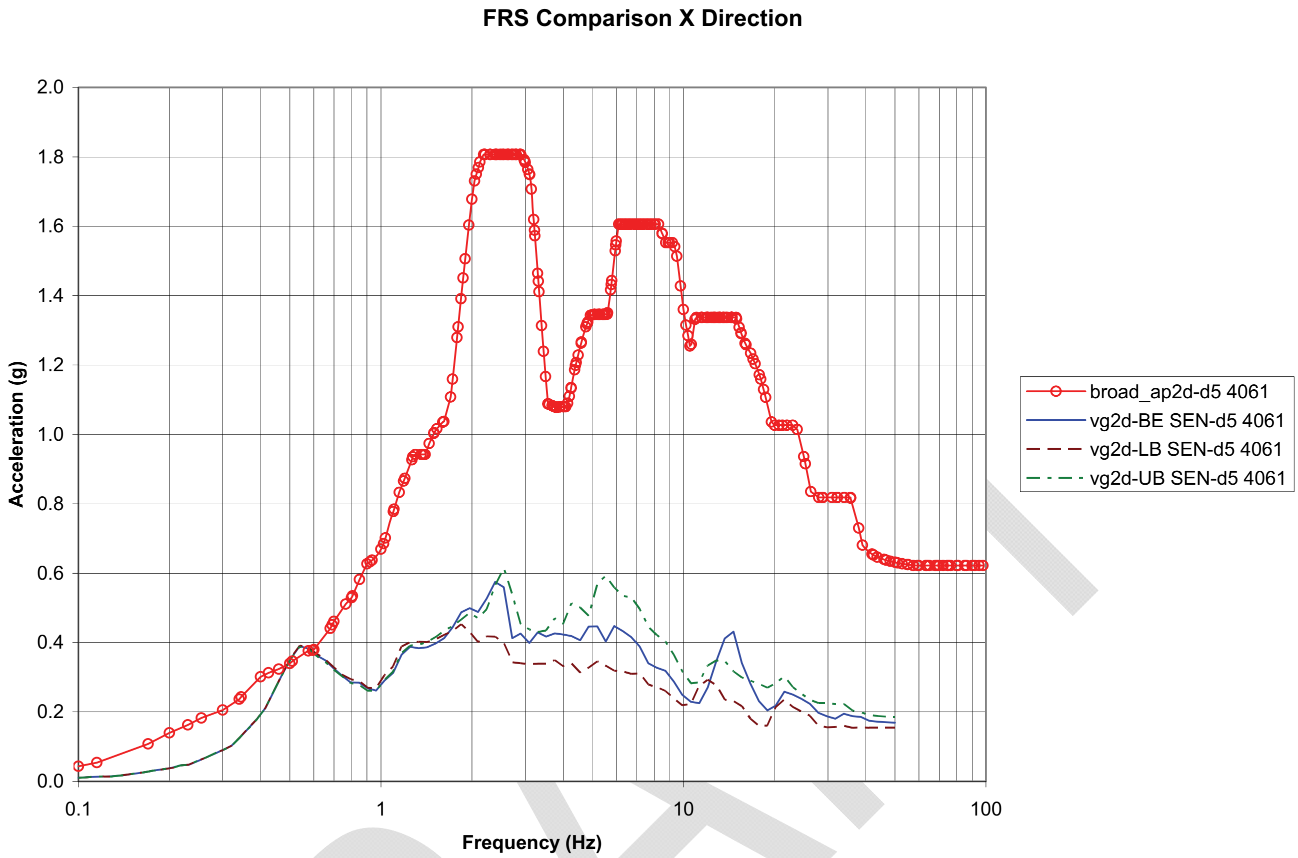


Figure 5.1-22 - Comparison of Node 4061 SEN to AP1000 SSI Envelope, NS Dir

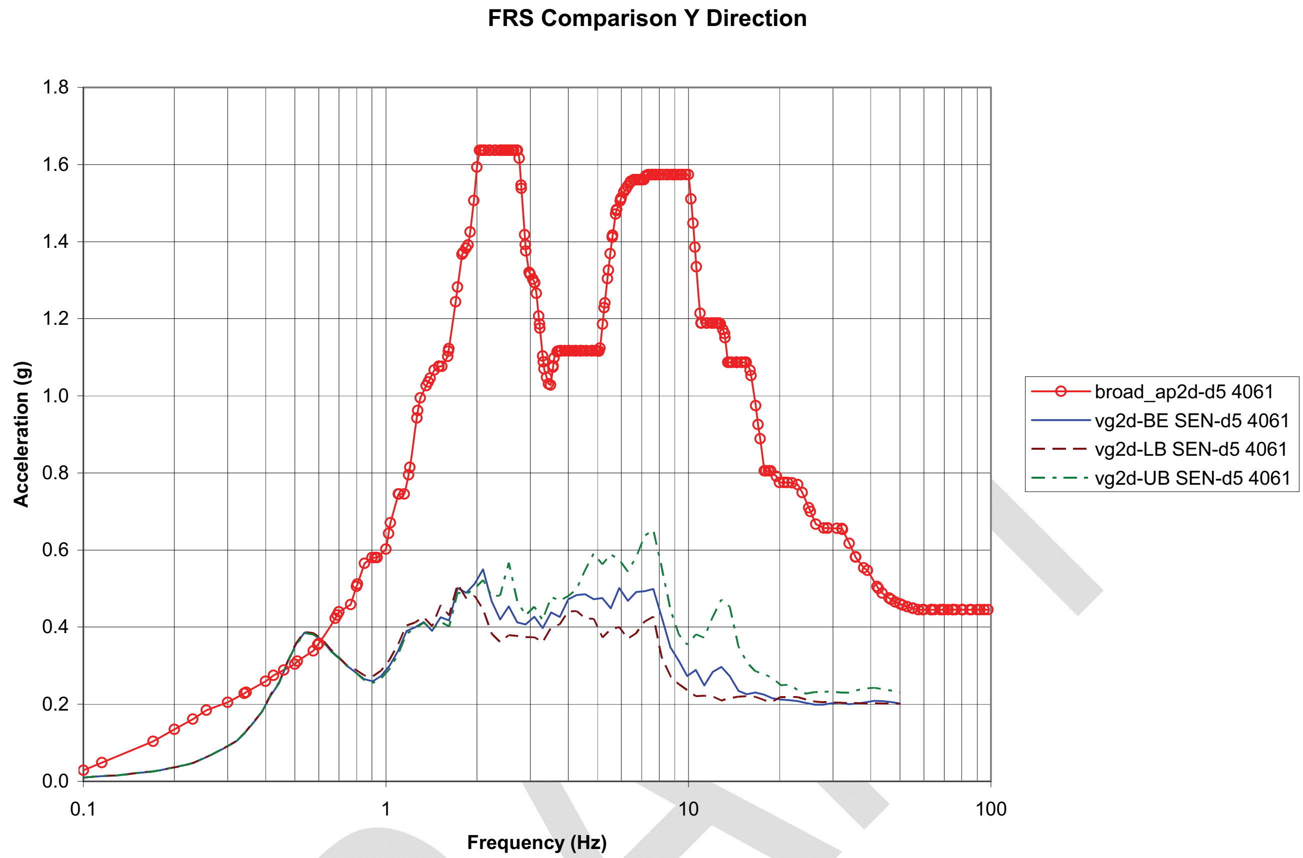


Figure 5.1-23 - Comparison of Node 4061 SEN to AP1000 SSI Envelope, EW Dir

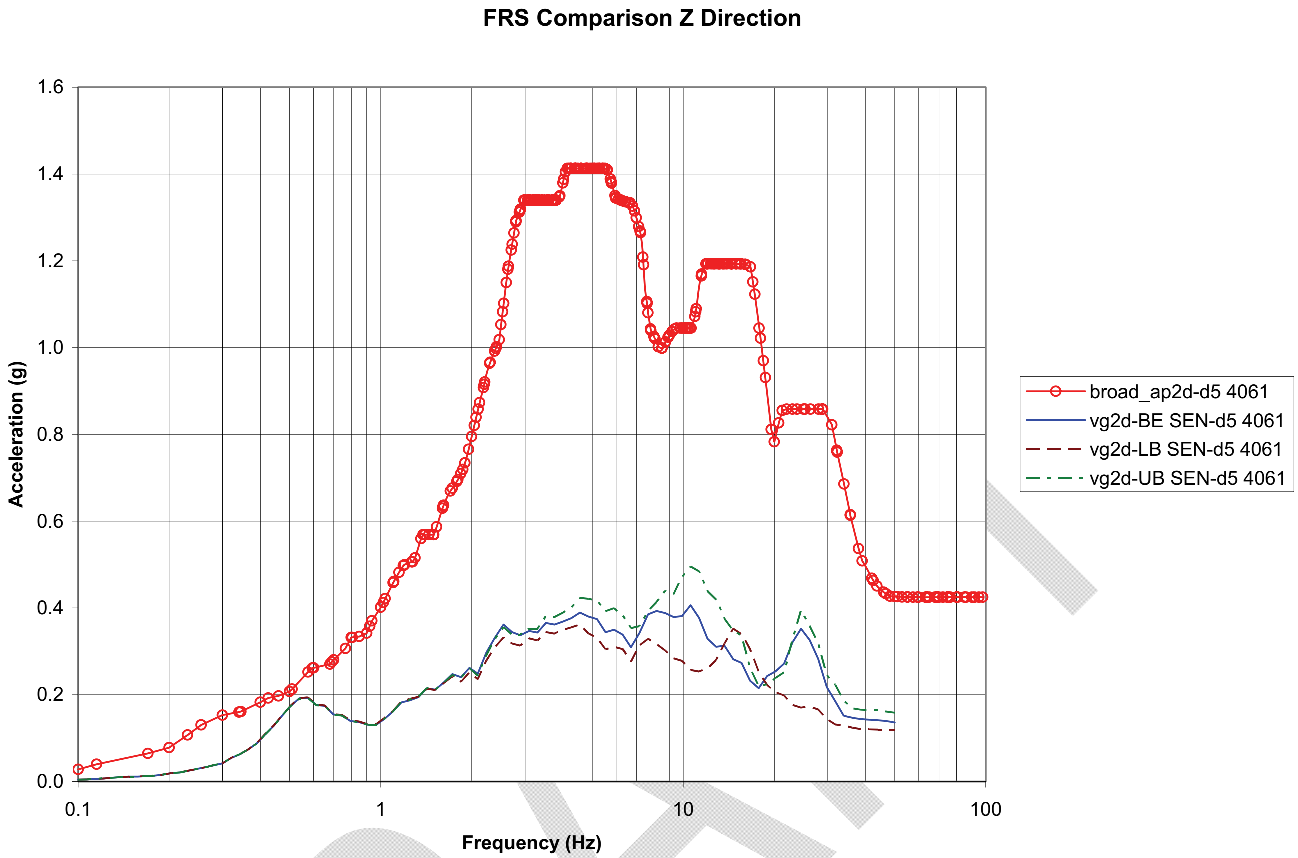


Figure 5.1-24 - Comparison of Node 4061 SEN to AP1000 SSI Envelope, Vertical Dir

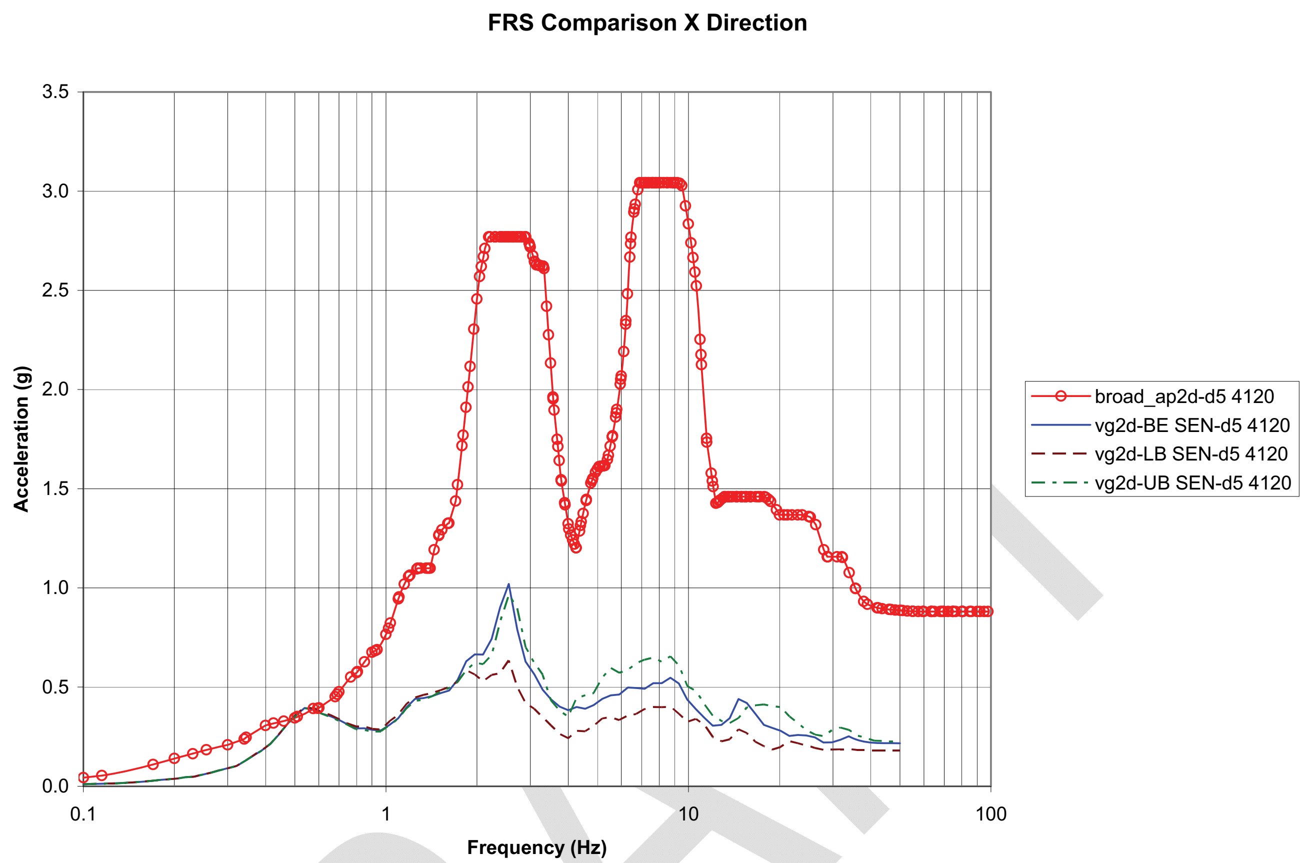


Figure 5.1-25 - Comparison of Node 4120 SEN to AP1000 SSI Envelope, NS Dir

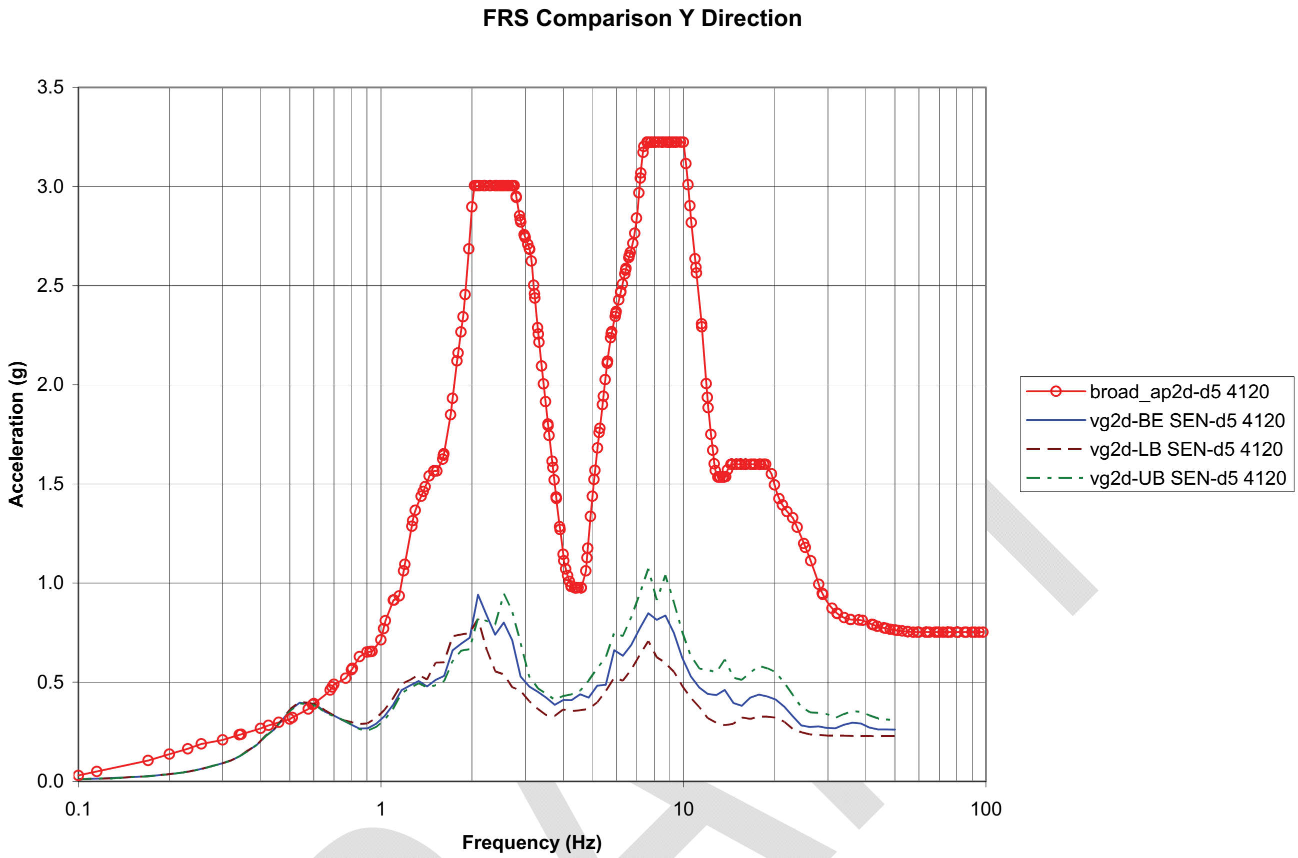


Figure 5.1-26 - Comparison of Node 4120 SEN to AP1000 SSI Envelope, EW Dir

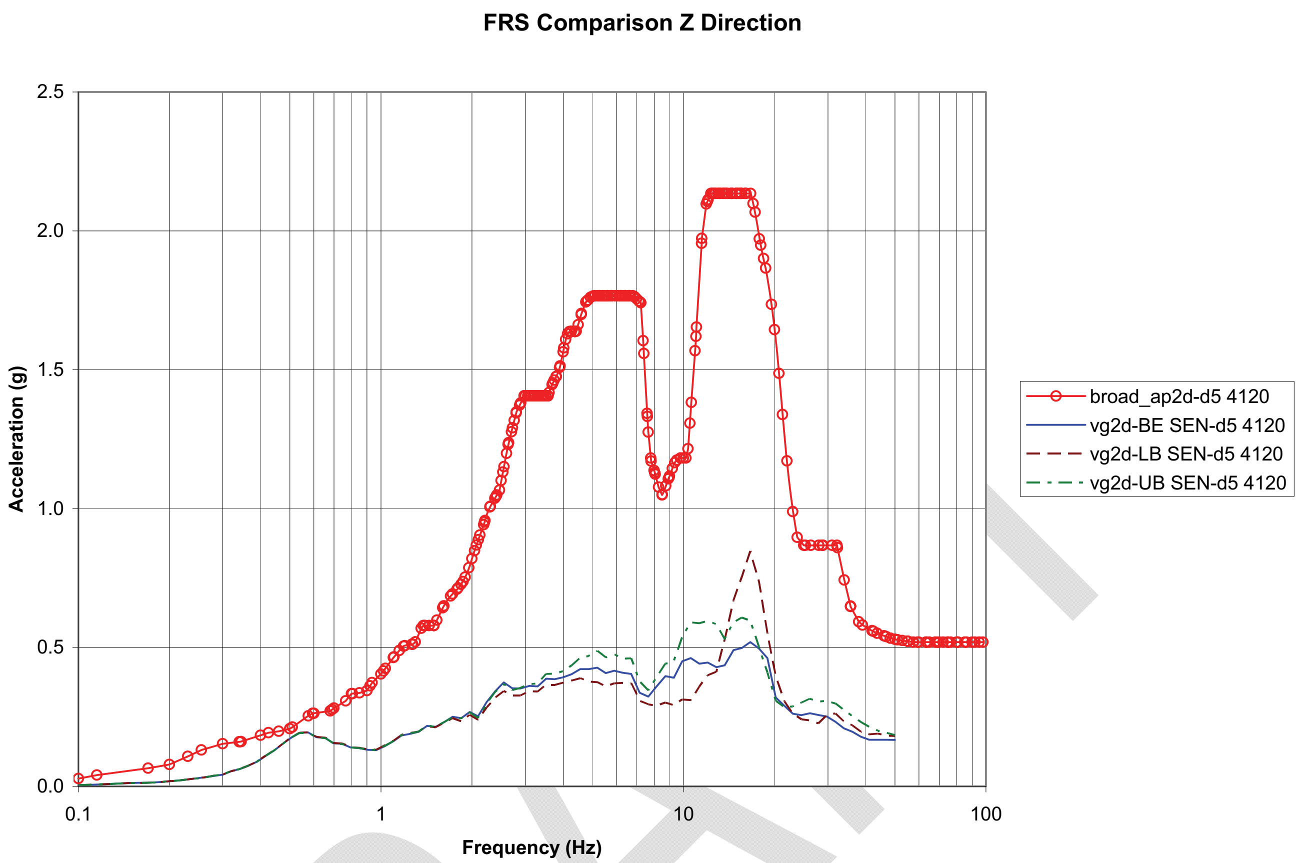


Figure 5.1-27 - Comparison of Node 4120 SEN to AP1000 SSI Envelope, Vertical Dir

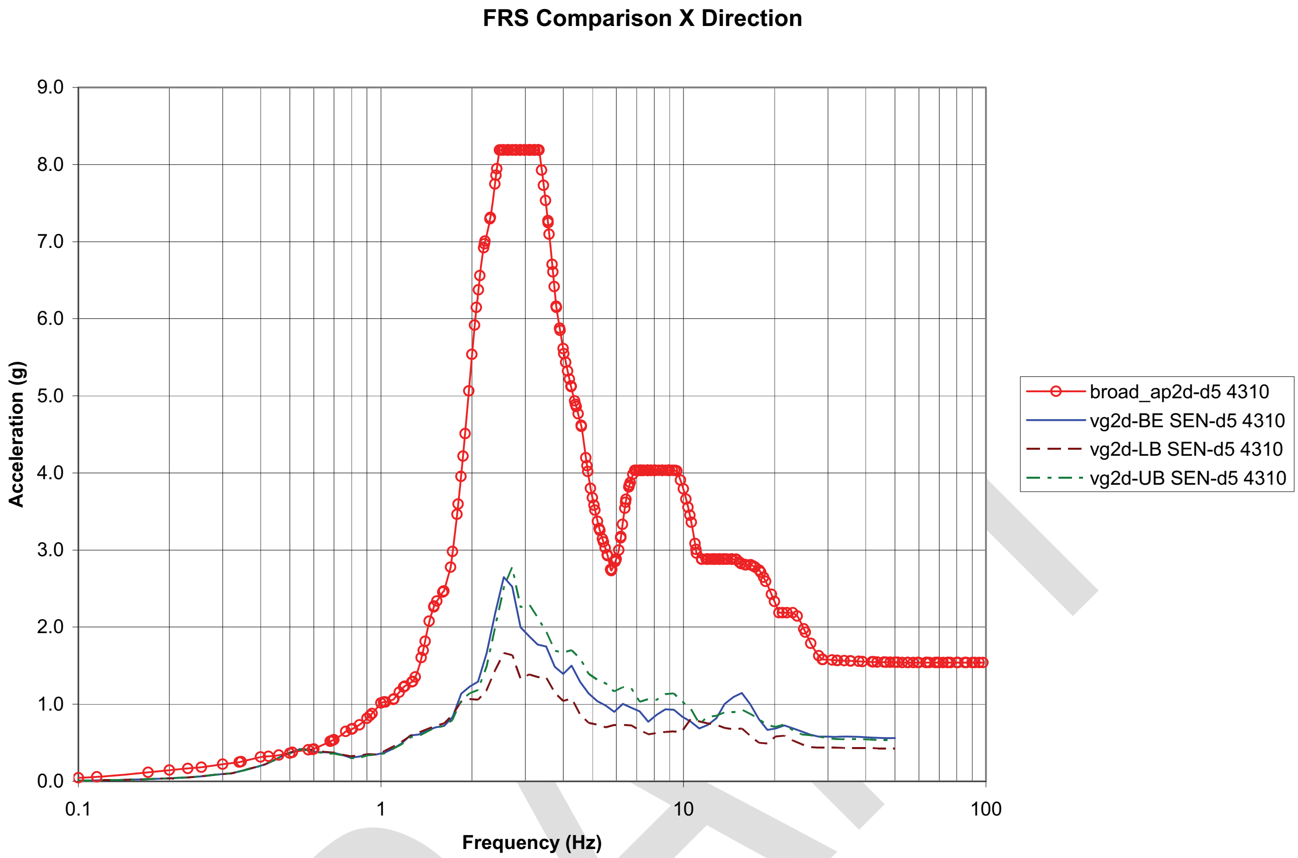


Figure 5.1-28 - Comparison of Node 4310 SEN to AP1000 SSI Envelope, NS Dir

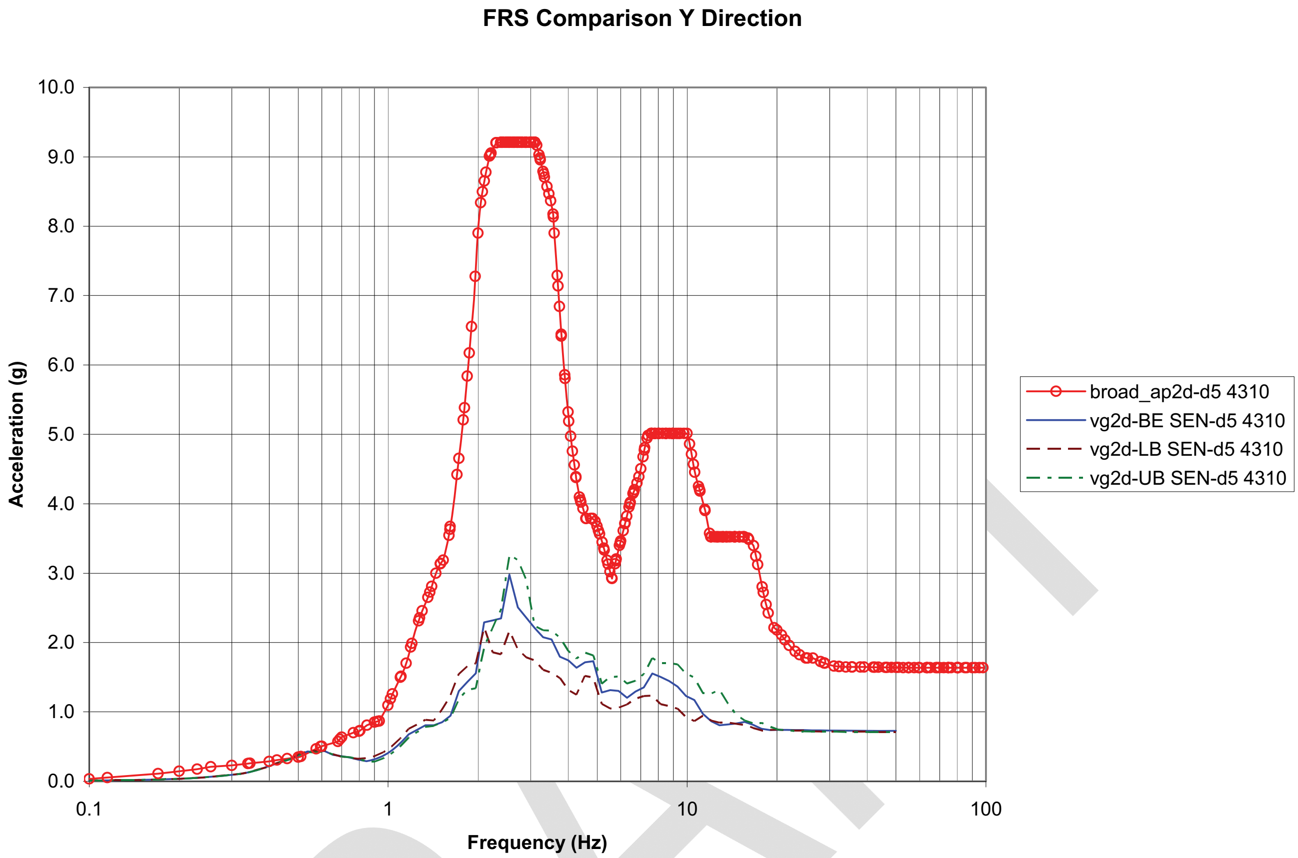


Figure 5.1-29 - Comparison of Node 4310 SEN to AP1000 SSI Envelope, EW Dir

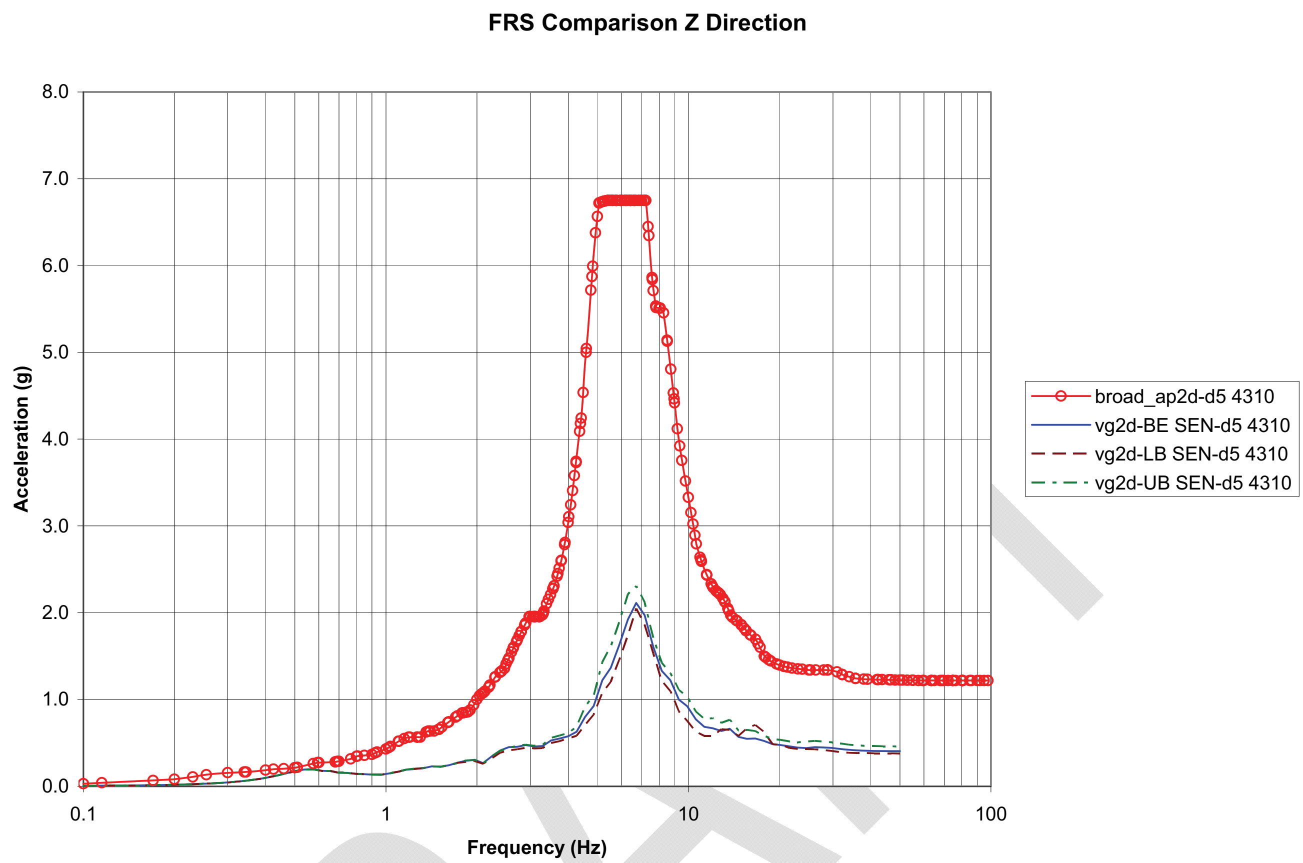


Figure 5.1-30 - Comparison of Node 4310 SEN to AP1000 SSI Envelope, Vertical Dir

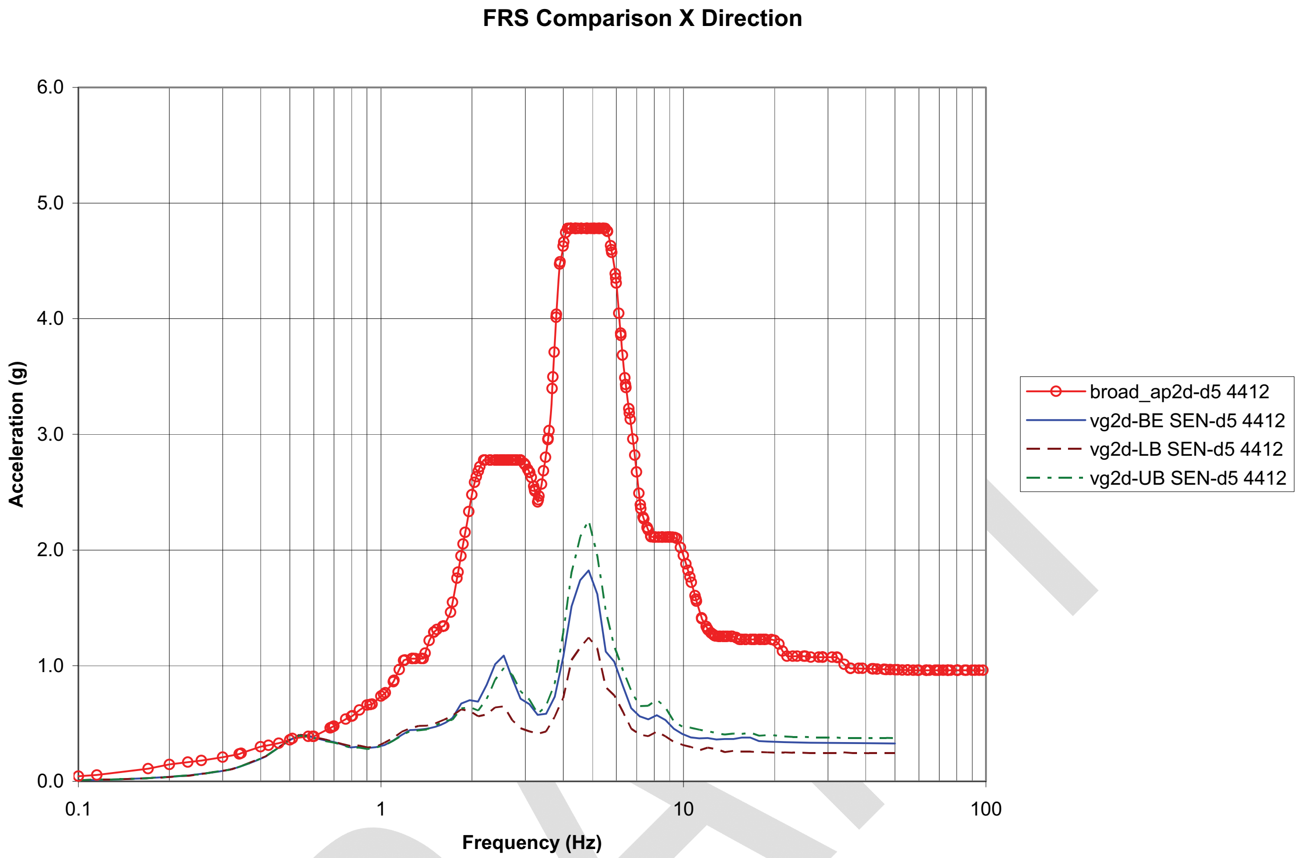


Figure 5.1-31 - Comparison of Node 4412 SEN to AP1000 SSI Envelope, NS Dir

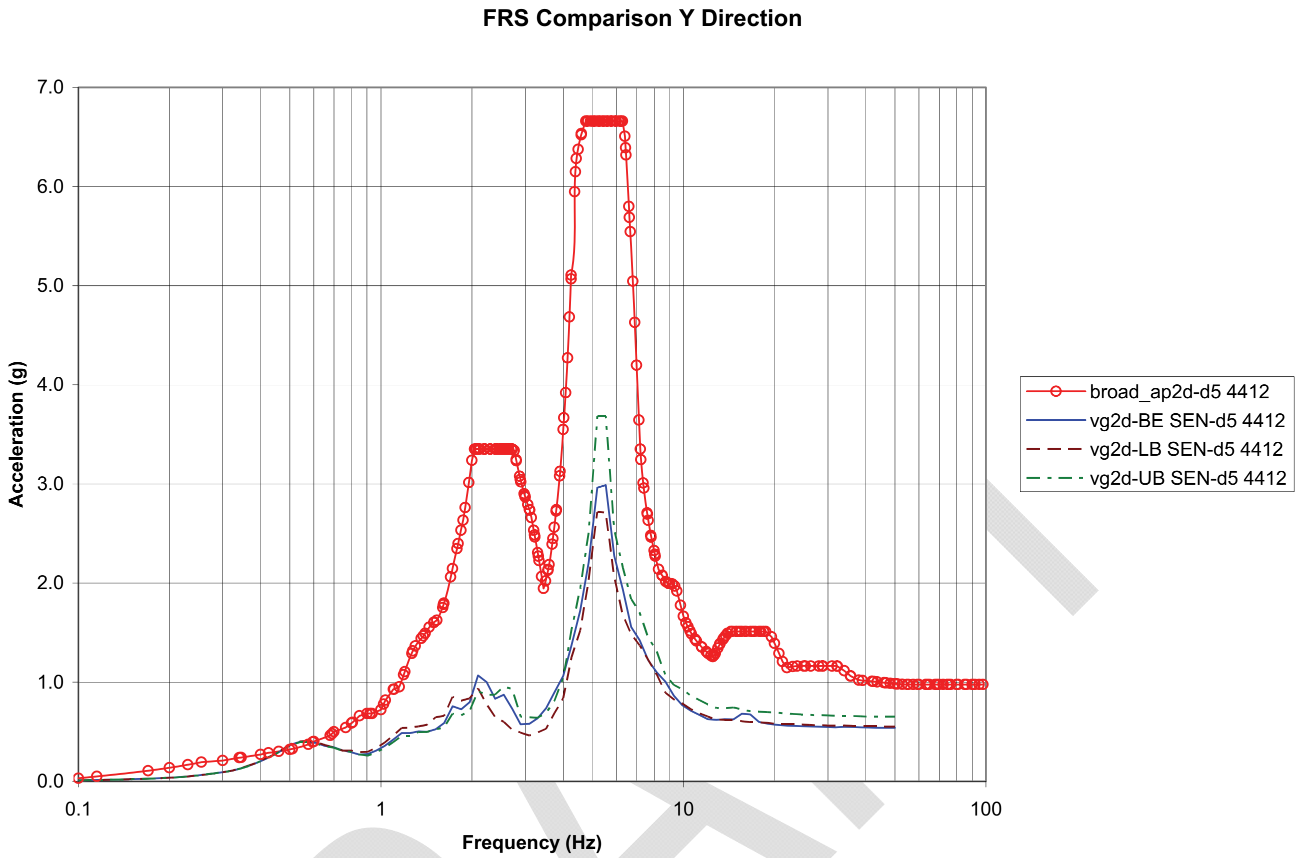


Figure 5.1-32 - Comparison of Node 4412 SEN to AP1000 SSI Envelope, EW Dir

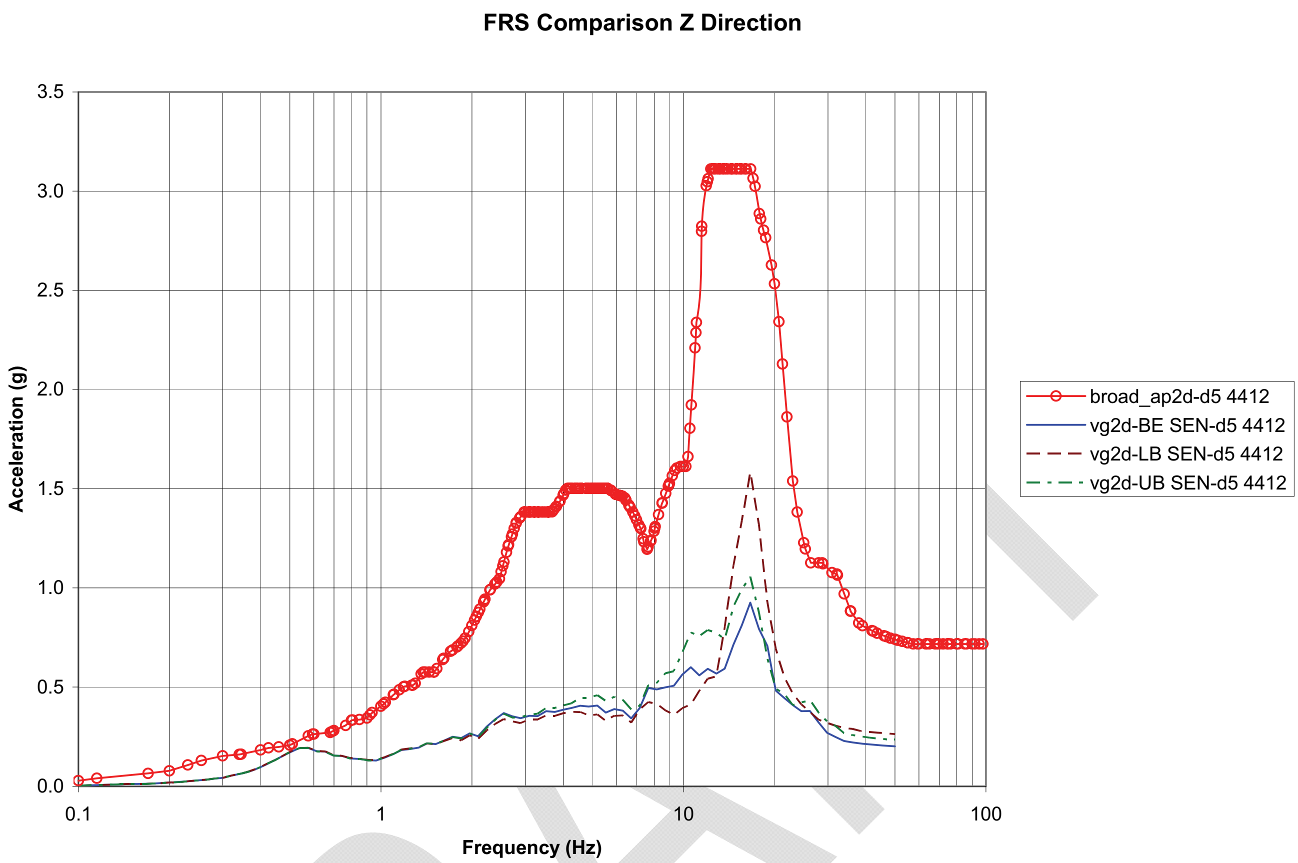


Figure 5.1-33 - Comparison of Node 4412 SEN to AP1000 SSI Envelope, Vertical Dir

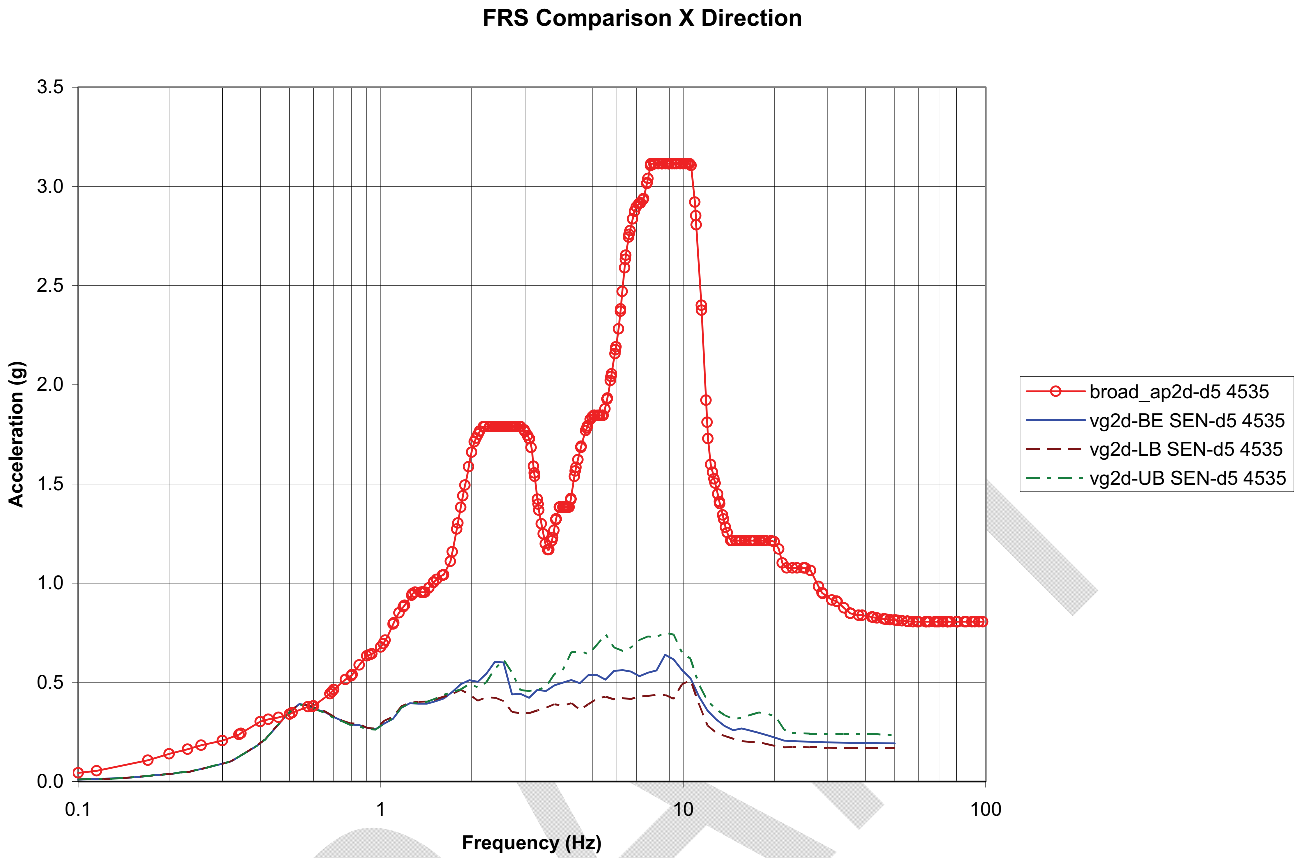


Figure 5.1-34 - Comparison of Node 4535 SEN to AP1000 SSI Envelope, NS Dir

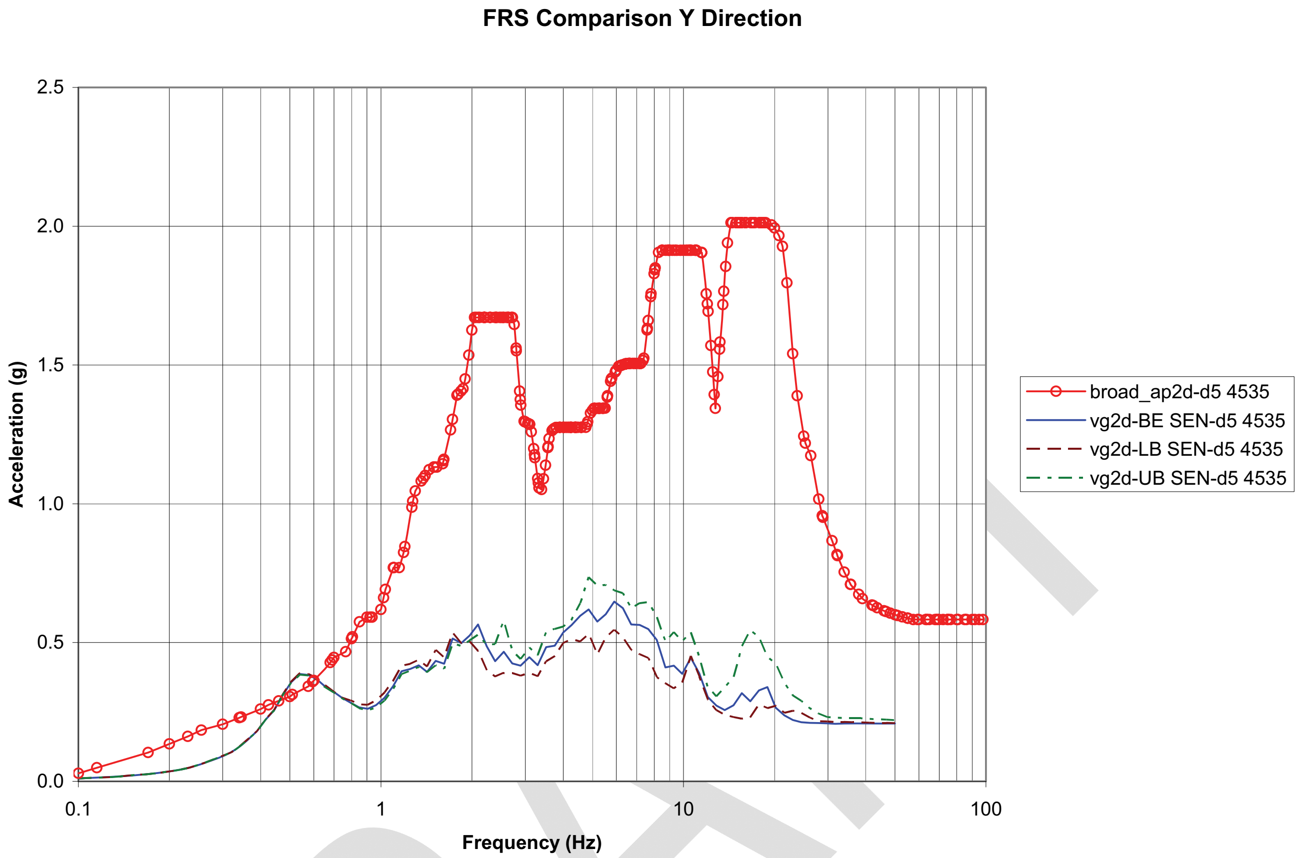


Figure 5.1-35 - Comparison of Node 4535 SEN to AP1000 SSI Envelope, EW Dir

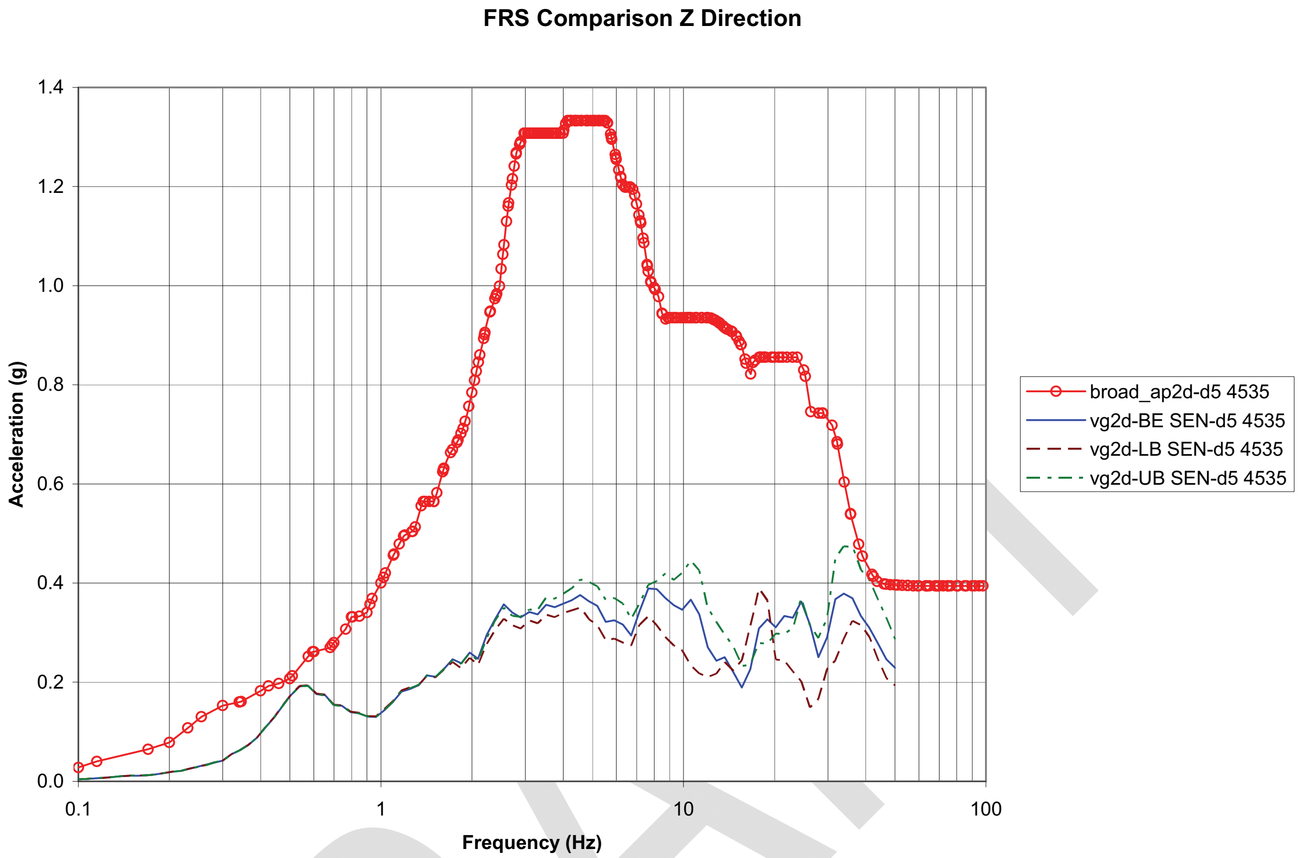


Figure 5.1-36 - Comparison of Node 4535 SEN to AP1000 SSI Envelope, Vertical Dir

5.2 Adjacent Building Seismic Demand

The 2D SASSI east-west model, nuclear island and Annex building (Seismic Category II building), was used to obtain the relative displacement between nuclear island and at top of the annex building at NI elevation 179'-7" and annex building elevation 182'-8". The maximum relative displacement between nuclear island and at top of the Annex building for the ESP Best Estimate soil case is 2", which is less than the 4 inch gap between nuclear island and annex building. The response spectra at the location of the Seismic Category II Annex building are given in Figures 5.2-1 and 5.2-2 for the horizontal and vertical directions. The response spectra is compared to the AP1000 SSI Envelope (identified as ap2d) for the ESP best estimate soil case (identified as vg2d) at 5% damping.

The seismic accelerations at the base of the seismic Category III Turbine and Radwaste buildings are given in Table 5.2-1 for the soil cases associated with ESP and SEN soil cases. For the AP1000 generic analysis, the seismic maximum seismic acceleration in each building is greater than 0.5g.

The Vogtle specific maximum bearing pressures for the Radwaste, Annex, and Turbine buildings are given in Section 7.0.

Table 5.2-1 – Turbine and Radwaste Base Seismic Accelerations

(Units: g)

	BE ESP	UB ESP	LB ESP	Max. ESP	BE SEN	UB SEN	LB SEN	Max SEN
Turbine Building								
South Side	0.18	0.20	0.19	0.20	0.17	0.15	0.15	0.17
Center	0.26	0.27	0.31	0.31	0.22	0.22	0.22	0.22
North Side	0.32	0.32	0.41	0.41	0.25	0.25	0.25	0.25
Radwaste Building								
South Side	0.19	0.20	0.20	0.20	0.18	0.16	0.16	0.18
Center	0.29	0.31	0.37	0.37	0.21	0.22	0.22	0.22
North Side	0.21	0.22	0.23	0.23	0.18	0.18	0.18	0.18

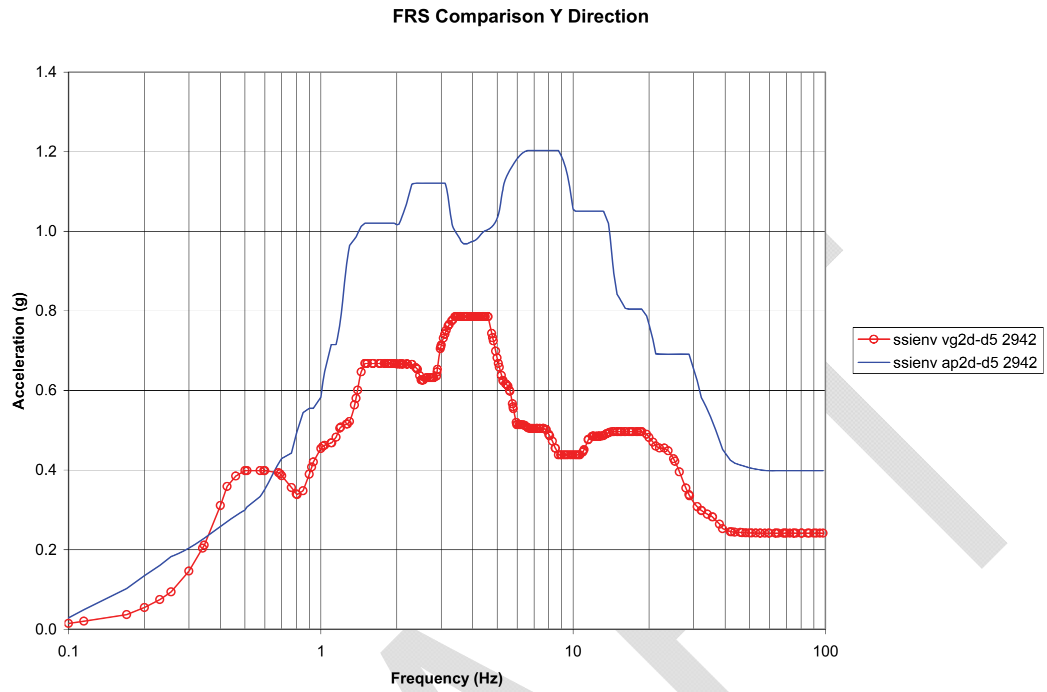


Figure 5.2-1 - Horizontal Seismic Response Spectra at Base of Annex Building

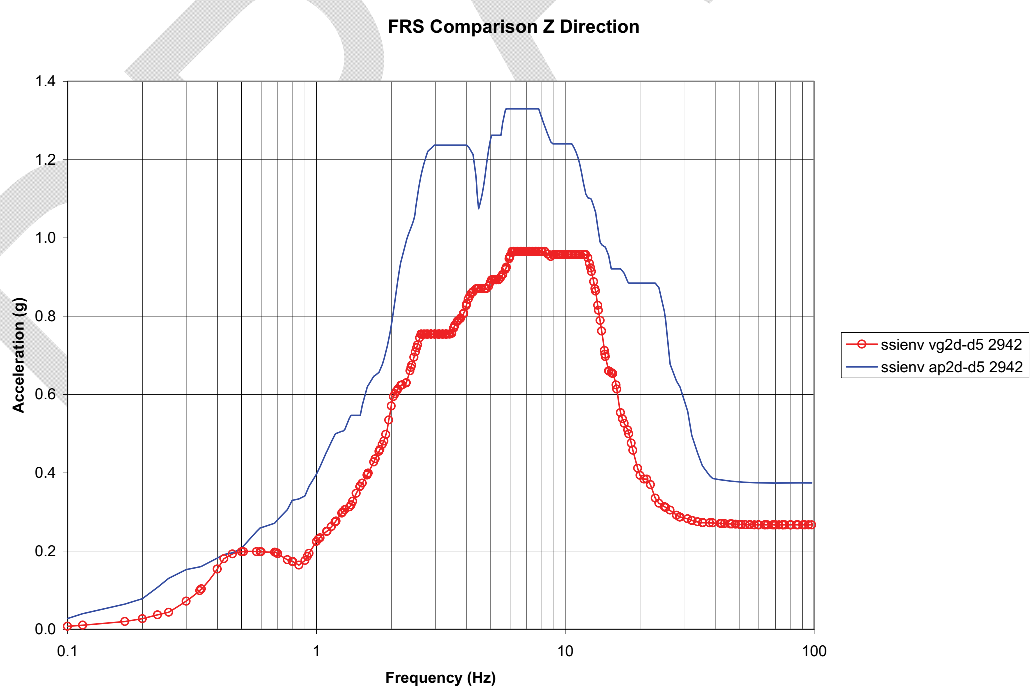


Figure 5.2-2 – Vertical Seismic Response Spectra at Base of Annex Building

5.3 Sensitivity Study of Backfill Behind MSE Wall

A sensitivity study of the Vogtle site-specific NI SSI seismic responses was performed to evaluate the effect of reduced backfill shear wave velocity (V_s) directly behind the MSE wall due to the use of different backfill compaction methods adjacent to the MSE wall. This sensitivity analysis was performed using two-dimensional (2D) seismic soil structure interaction SASSI models. The first model assumes the V_s of the backfill is the same throughout, and is the ESP best estimate (BE) backfill V_s (515-909 fps). The second model utilizes the same BE backfill V_s, except for an area extending from the face of the wall five feet into the backfill for the full height of the wall. Figure 5.3-1 shows the MSE wall configuration. For this area of fill, the ESP LB V_s (421-755 fps) is used. For the sensitivity analysis the ESP BE input time histories are used.

Figure 5.3-2 through 5.3-19 show the FRS comparisons between the Vogtle 2D model with the reduced shear wave velocity directly behind the MSE wall (VG2dMSE-BE ESP-d5) and the Vogtle ESP BE 2D SASSI (vg2d-BE ESP-d5) model at Nodes as shown in Table 5.1-1. These figures also show the AP1000 SASSI 2D SSI FRS envelope.

The FRS for the model that included the LB backfill V_s directly behind the MSE wall were almost identical to the FRS of the same model without any reduction in V_s directly behind the MSE wall. Therefore, the potentially reduced shear wave velocity of the backfill directly behind the MSE wall does not affect the Nuclear Island building responses.

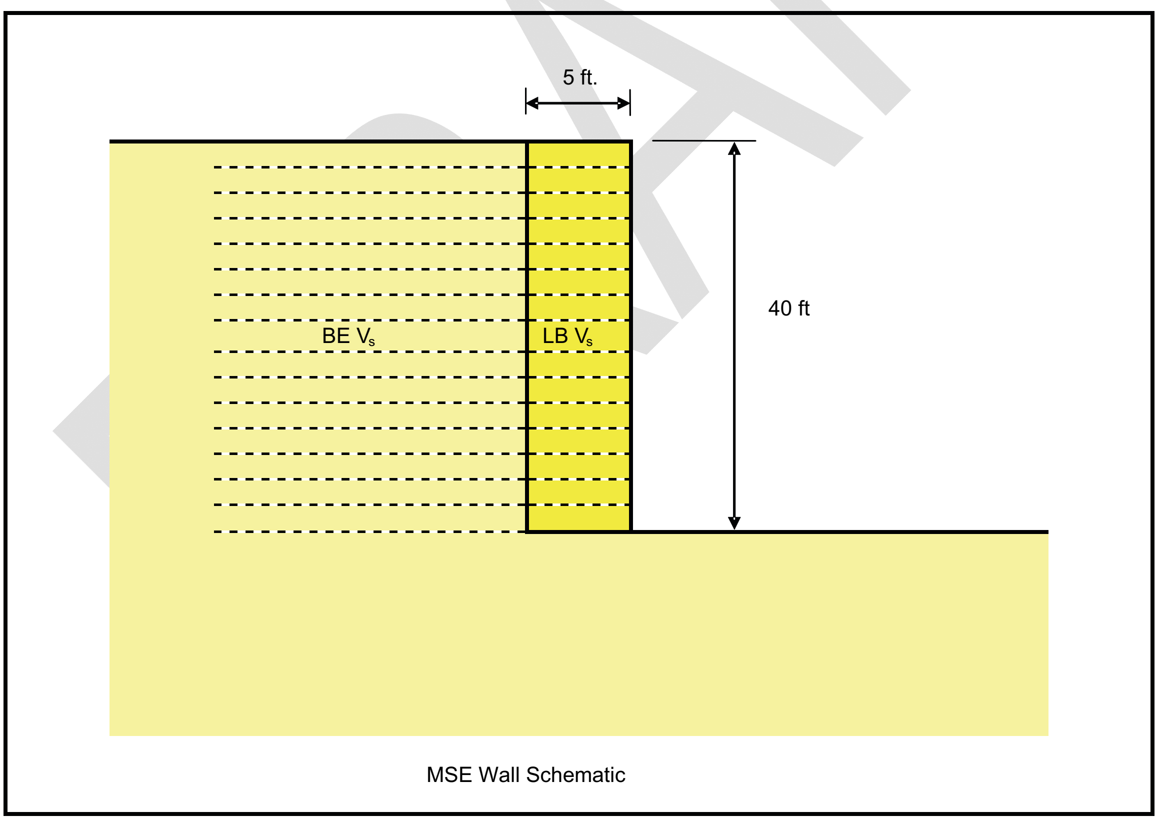


Figure 5.3-1 –2D SASSI Model with MSE Wall

Figure Not To Scale

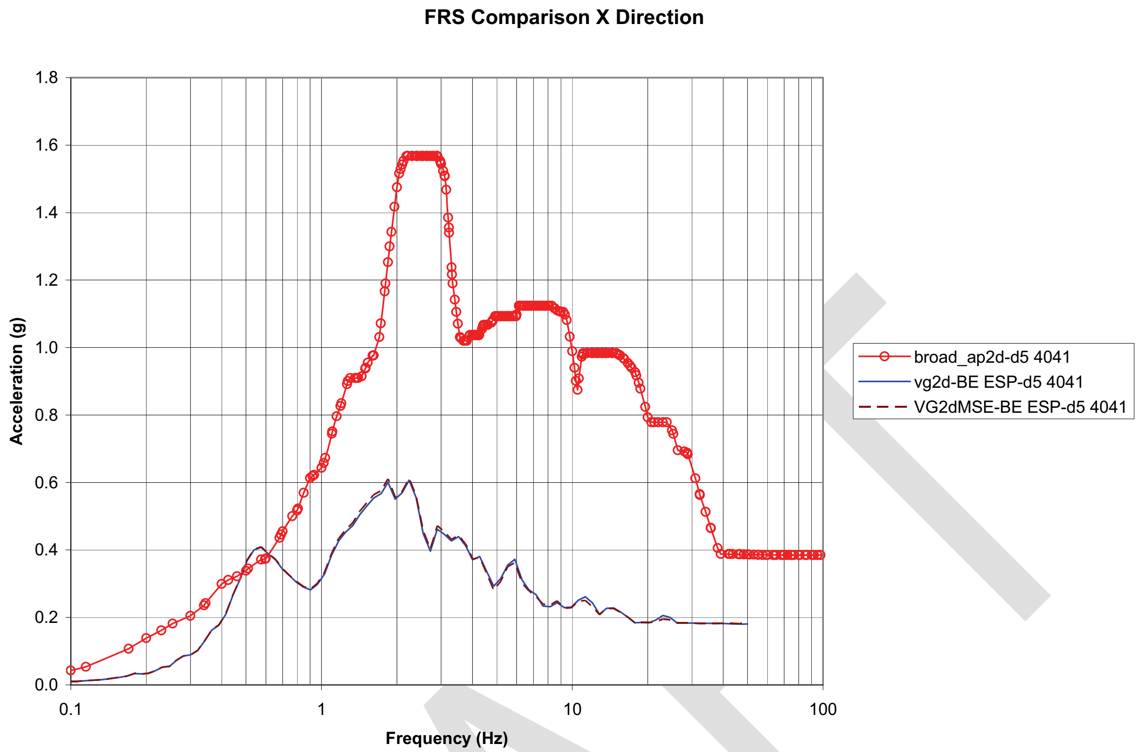


Figure 5.3-2 – FRS Comparison at Node 4041 (X Direction)

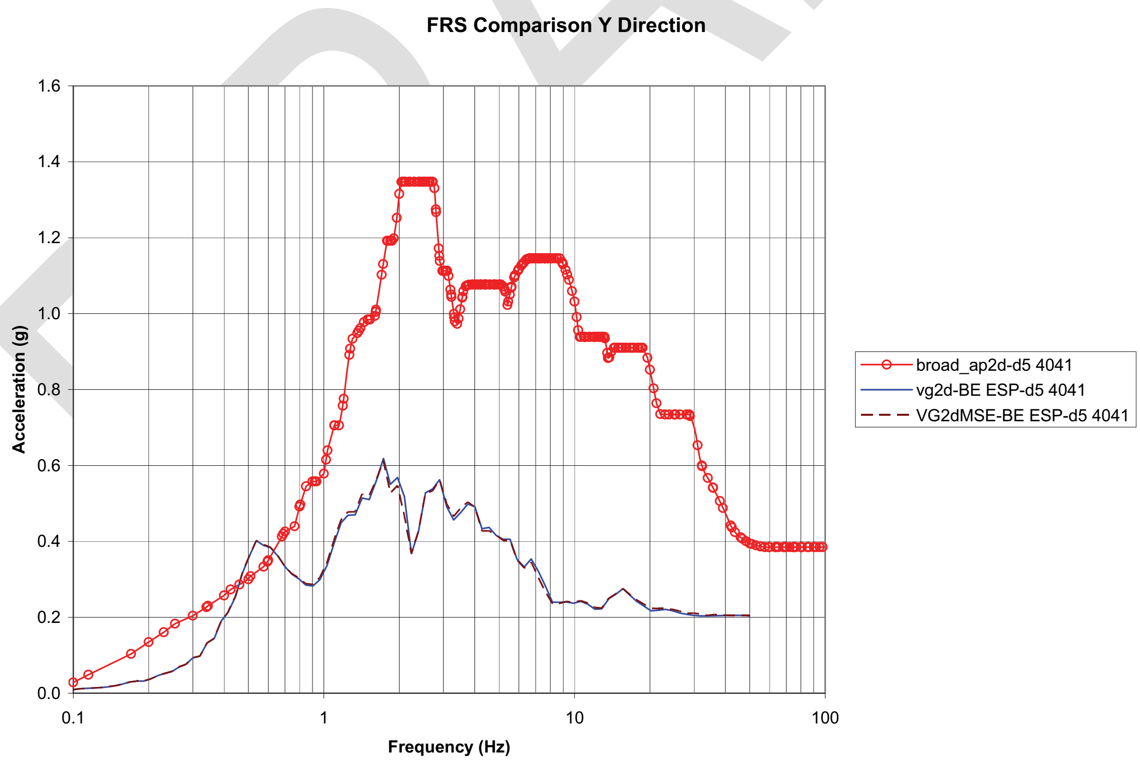


Figure 5.3-3 – FRS Comparison at Node 4041 (Y Direction)

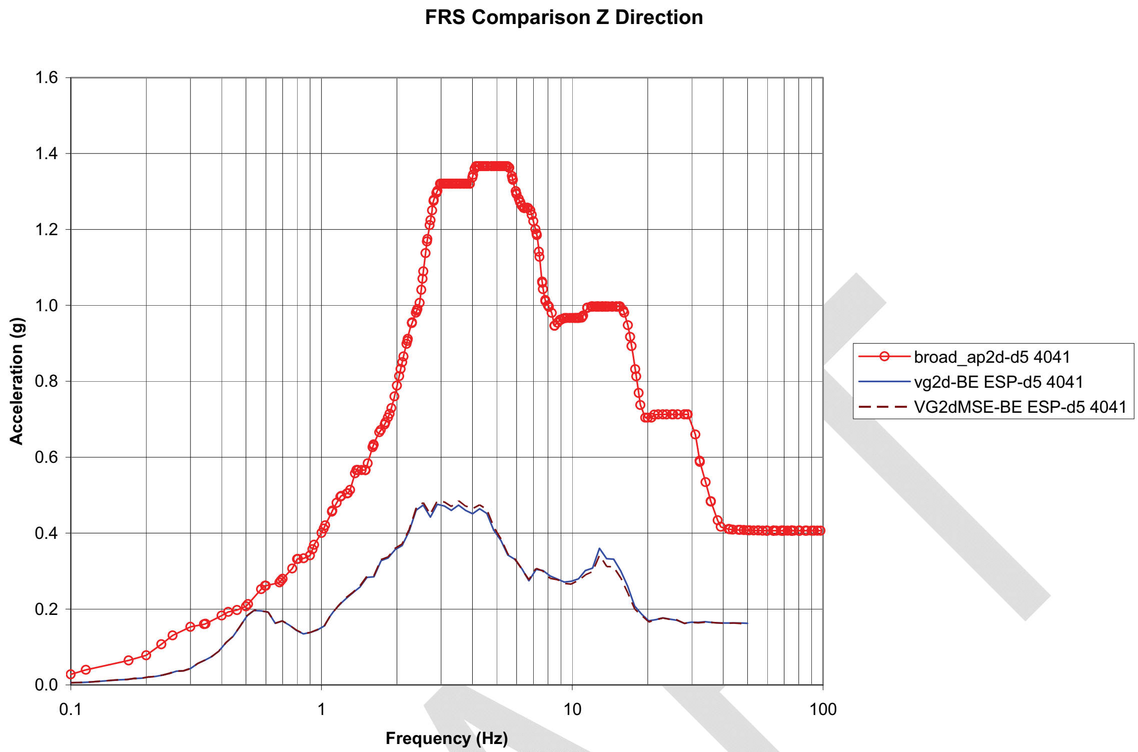


Figure 5.3-4 – FRS Comparison at Node 4041 (Z Direction)

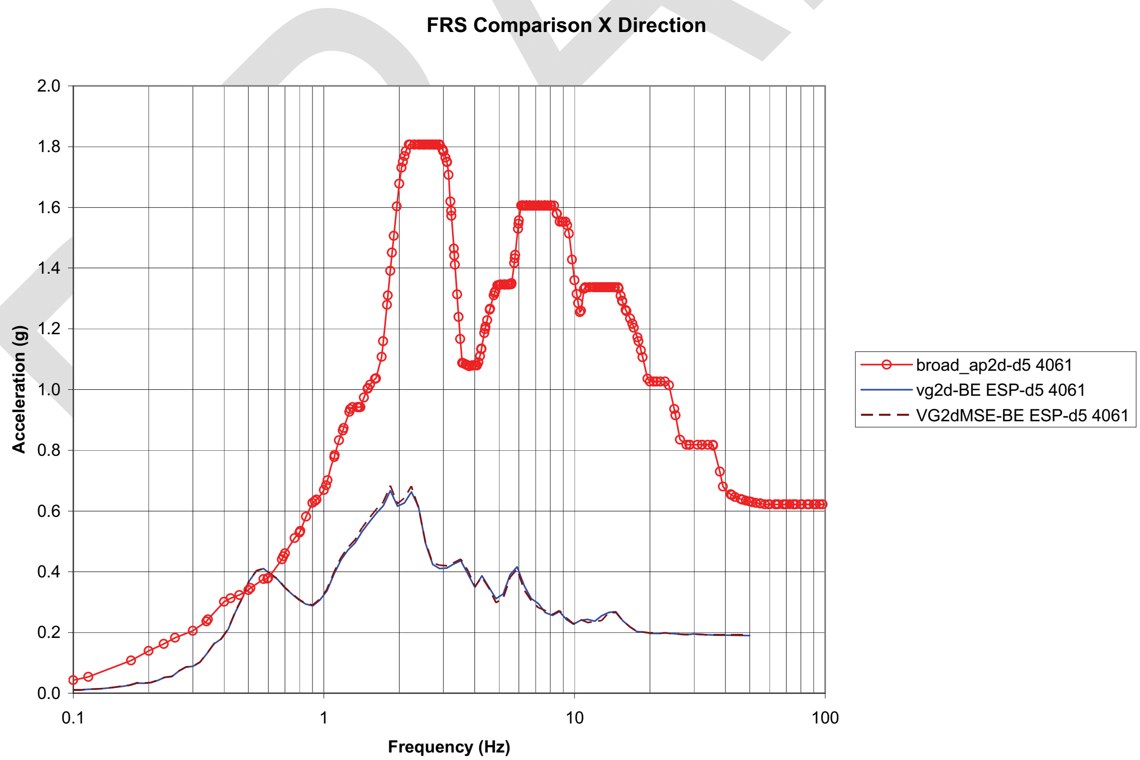


Figure 5.3-5 – FRS Comparison at Node 4061 (X Direction)

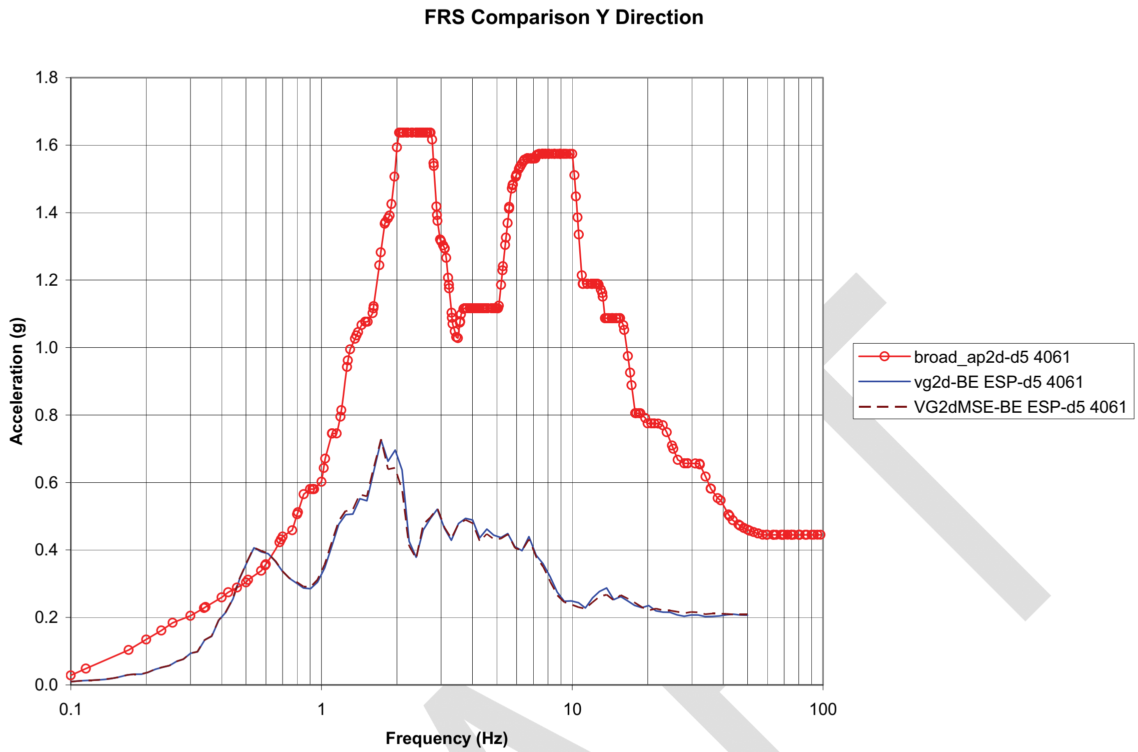


Figure 5.3-6 – FRS Comparison at Node 4061 (Y Direction)

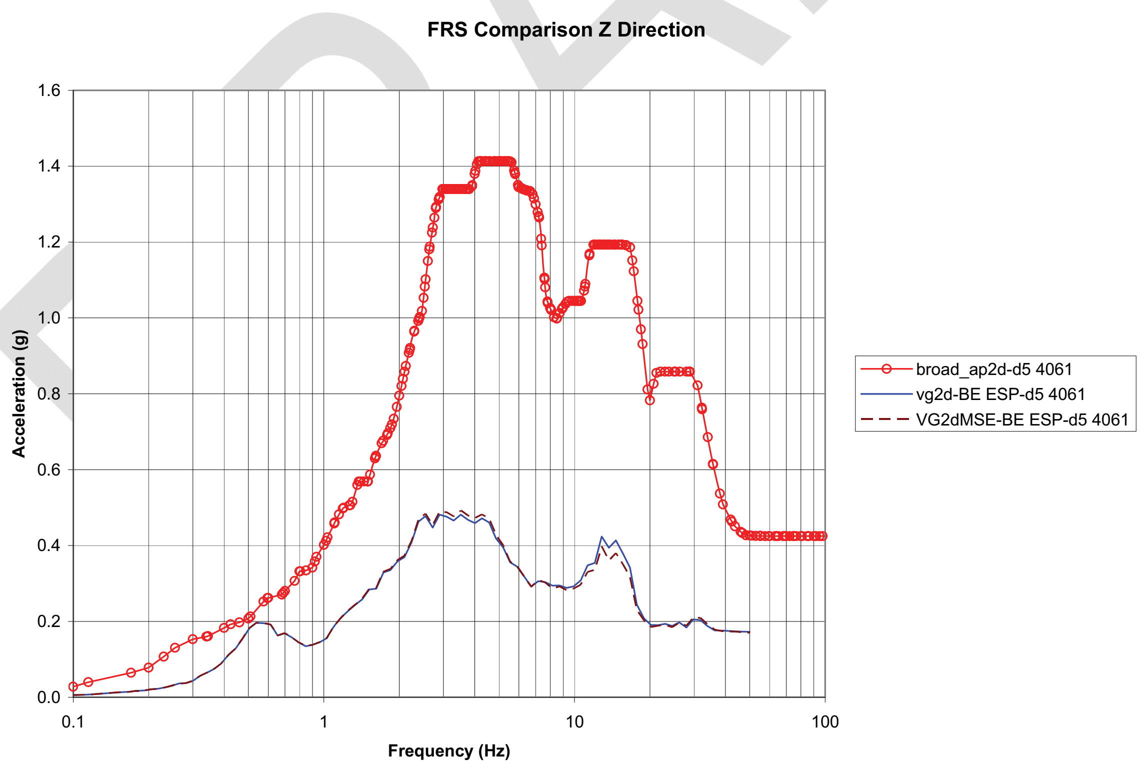


Figure 5.3-7 – FRS Comparison at Node 4061 (Z Direction)

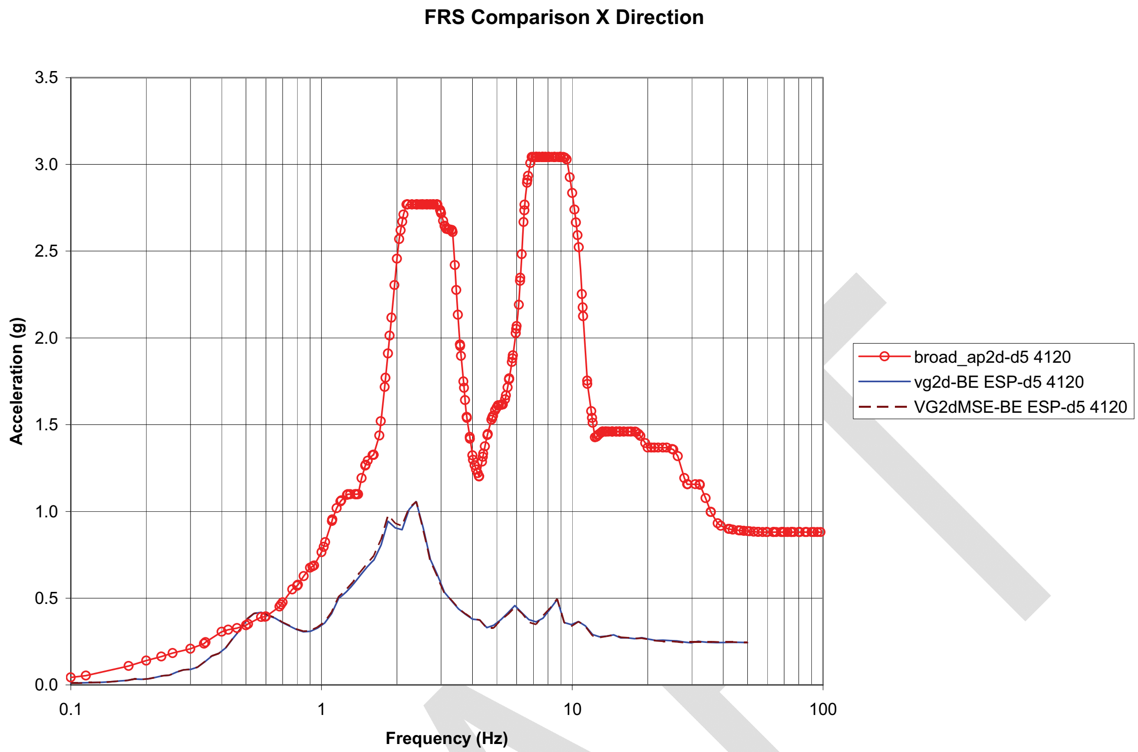


Figure 5.3-8 – FRS Comparison at Node 4120 (X Direction)

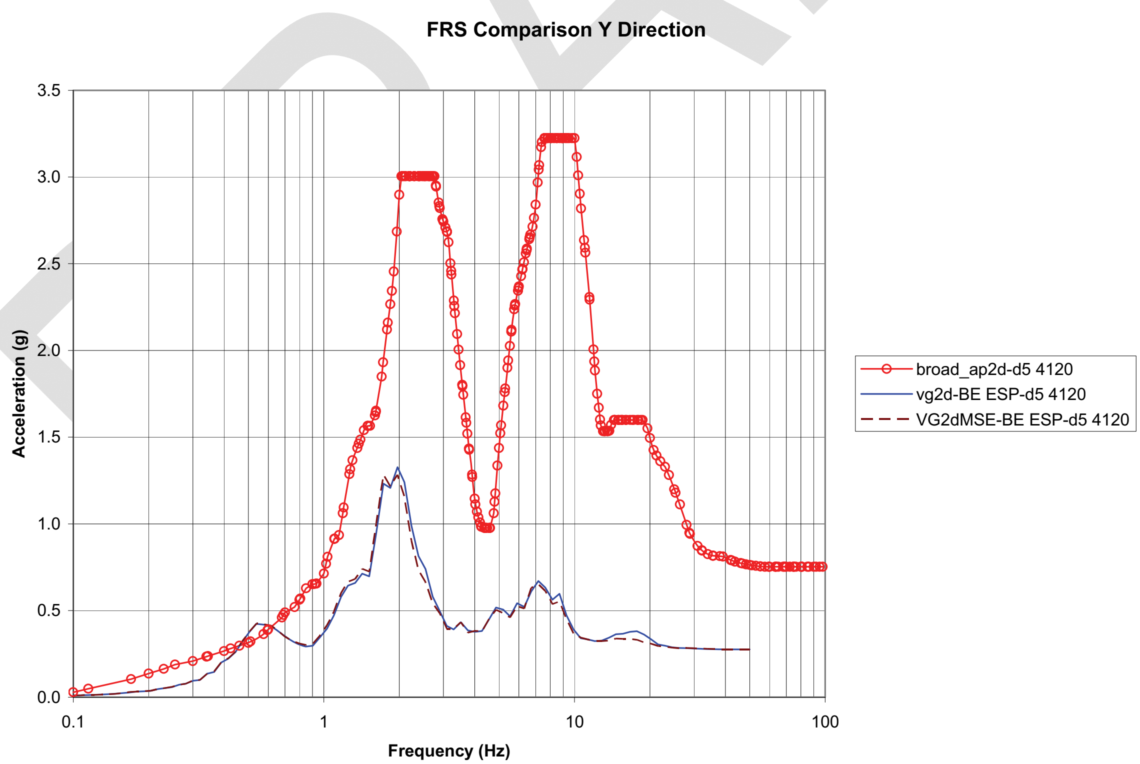


Figure 5.3-9 – FRS Comparison at Node 4120 (Y Direction)

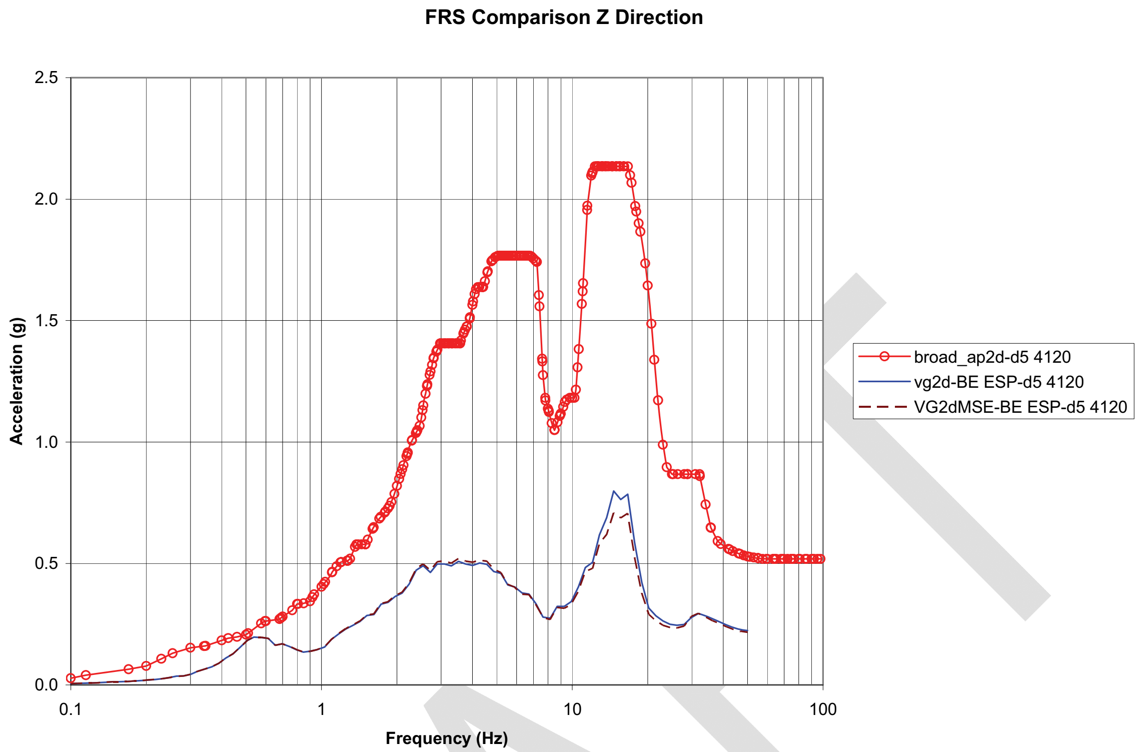


Figure 5.3-10 – FRS Comparison at Node 4120 (Z Direction)

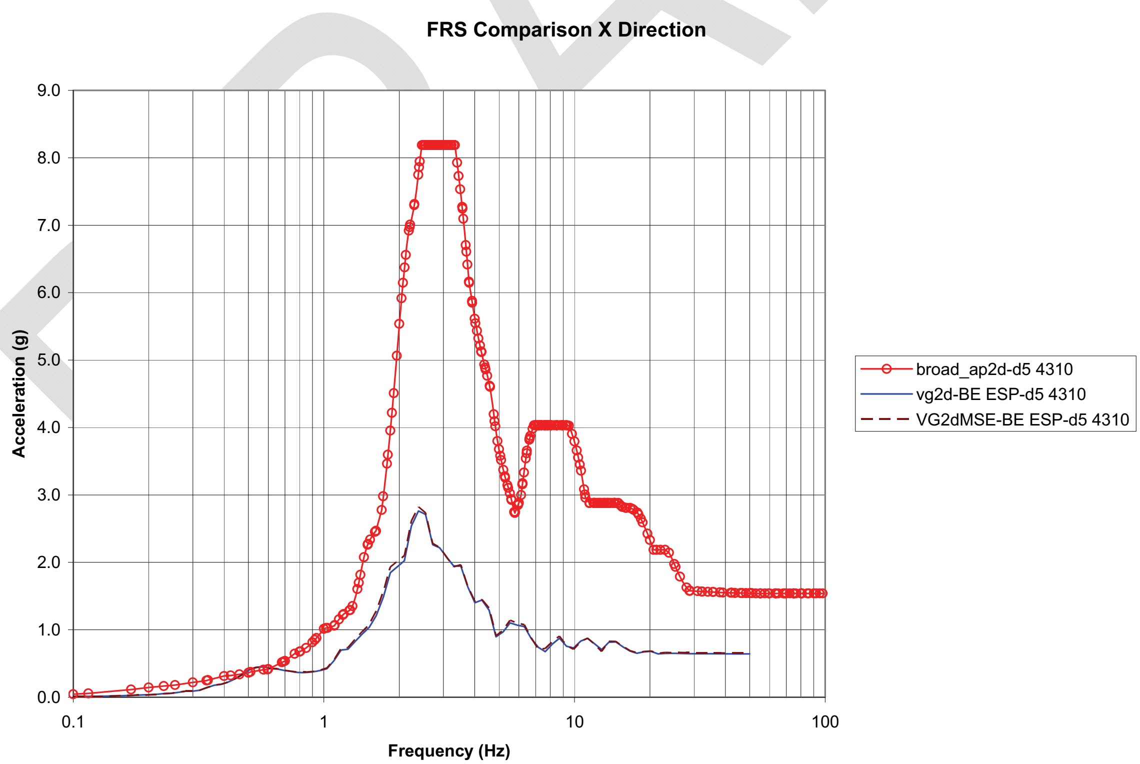


Figure 5.3-11 – FRS Comparison at Node 4310 (X Direction)

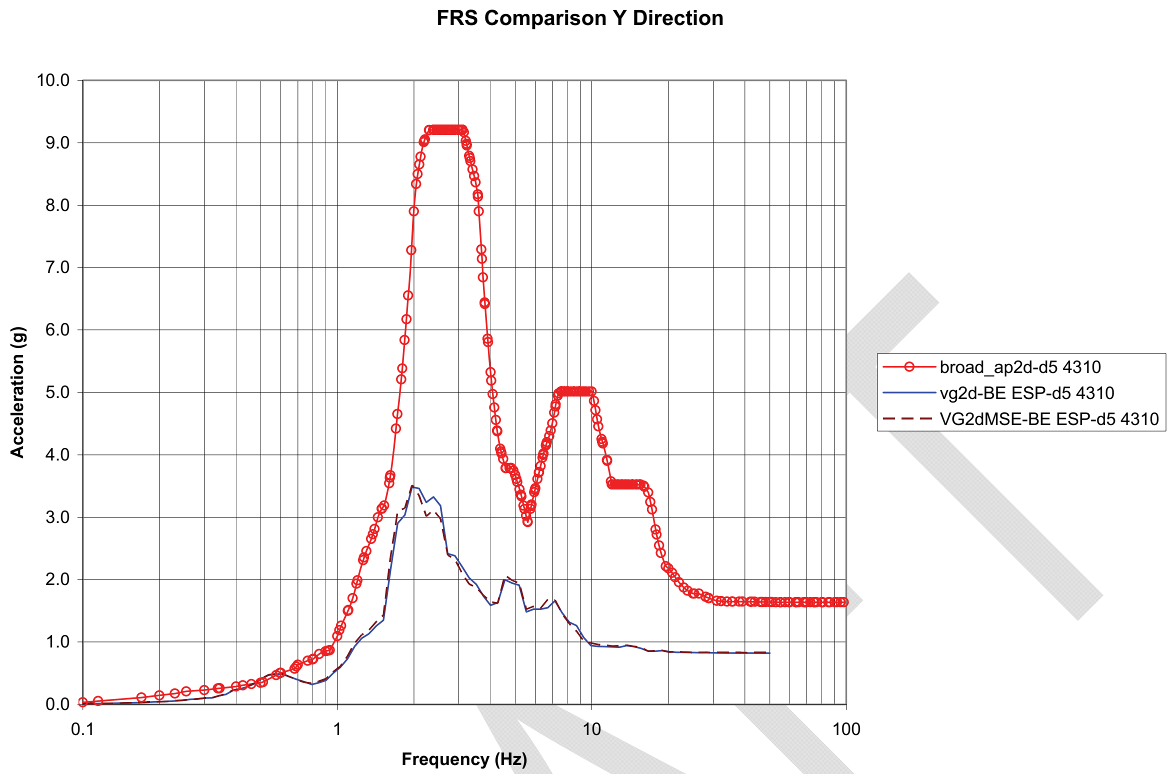


Figure 5.3-12 – FRS Comparison at Node 4310 (Y Direction)

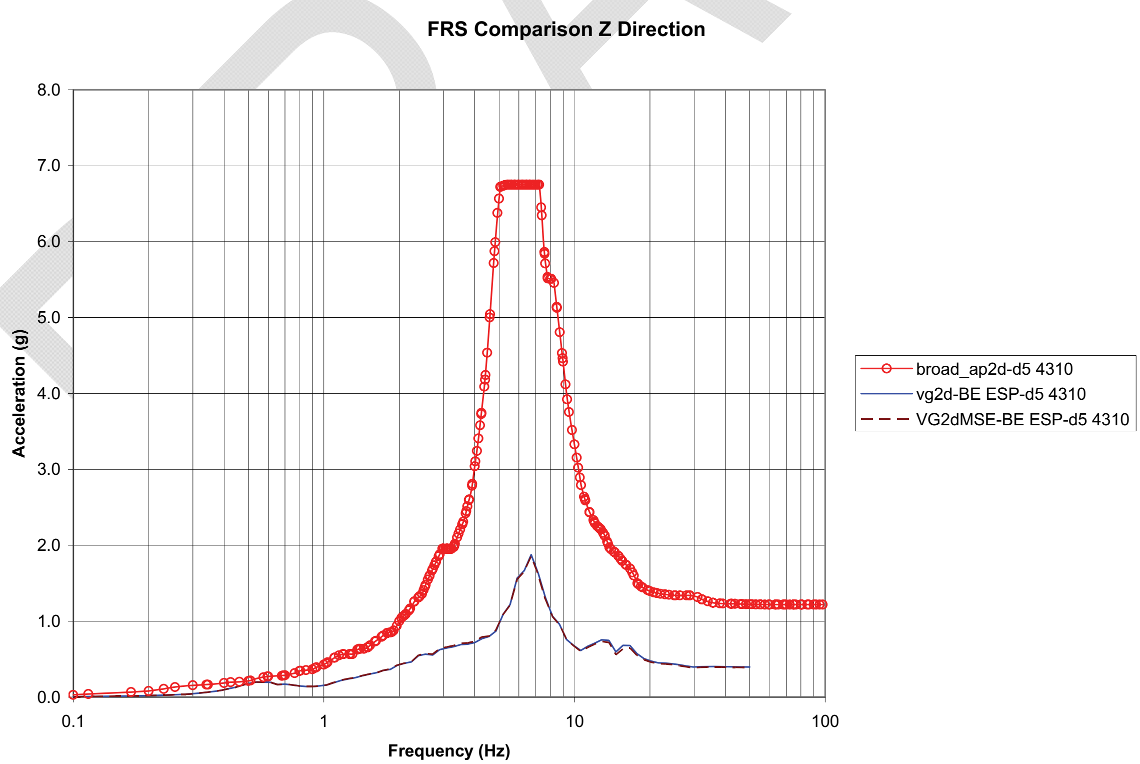


Figure 5.3-13 – FRS Comparison at Node 4310 (Z Direction)

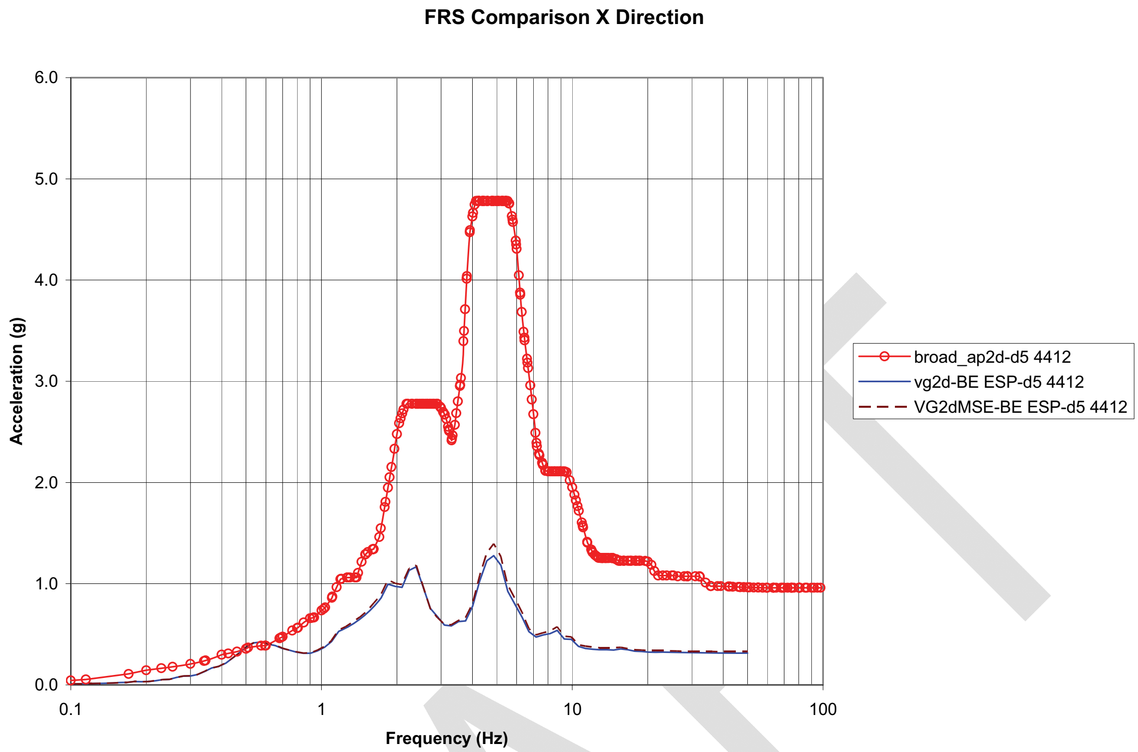


Figure 5.3-14 – FRS Comparison at Node 4412 (X Direction)

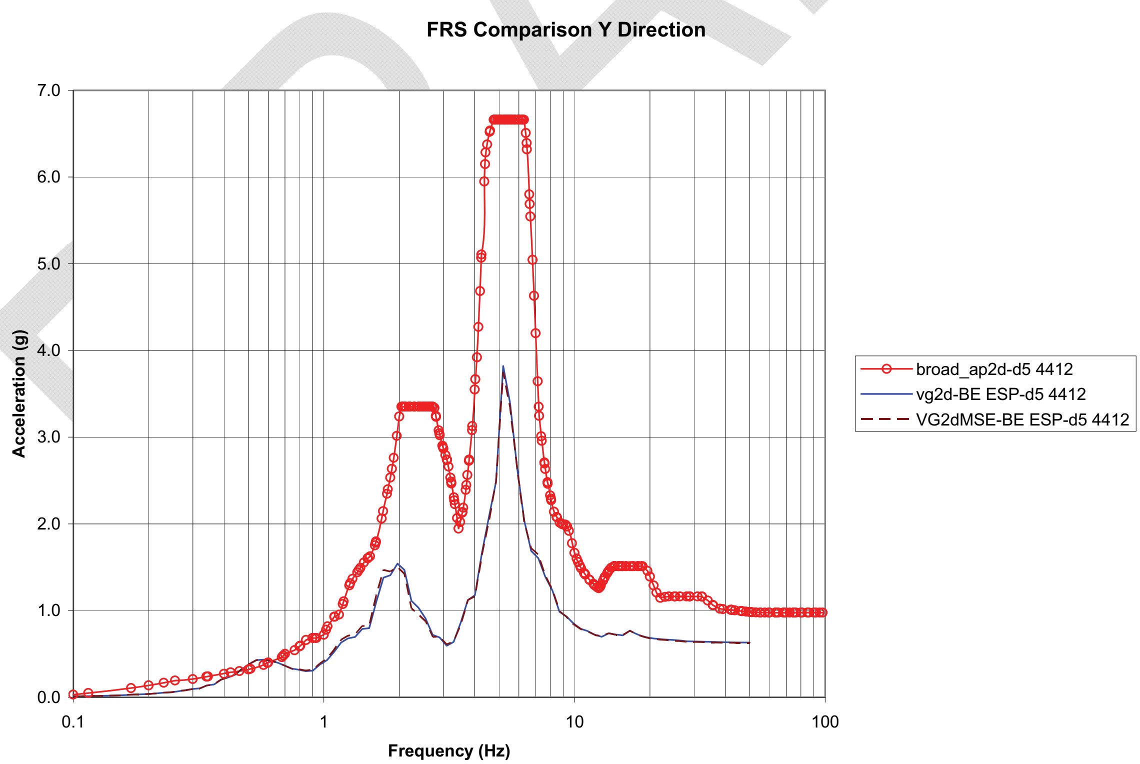


Figure 5.3-15 – FRS Comparison at Node 4412 (Y Direction)

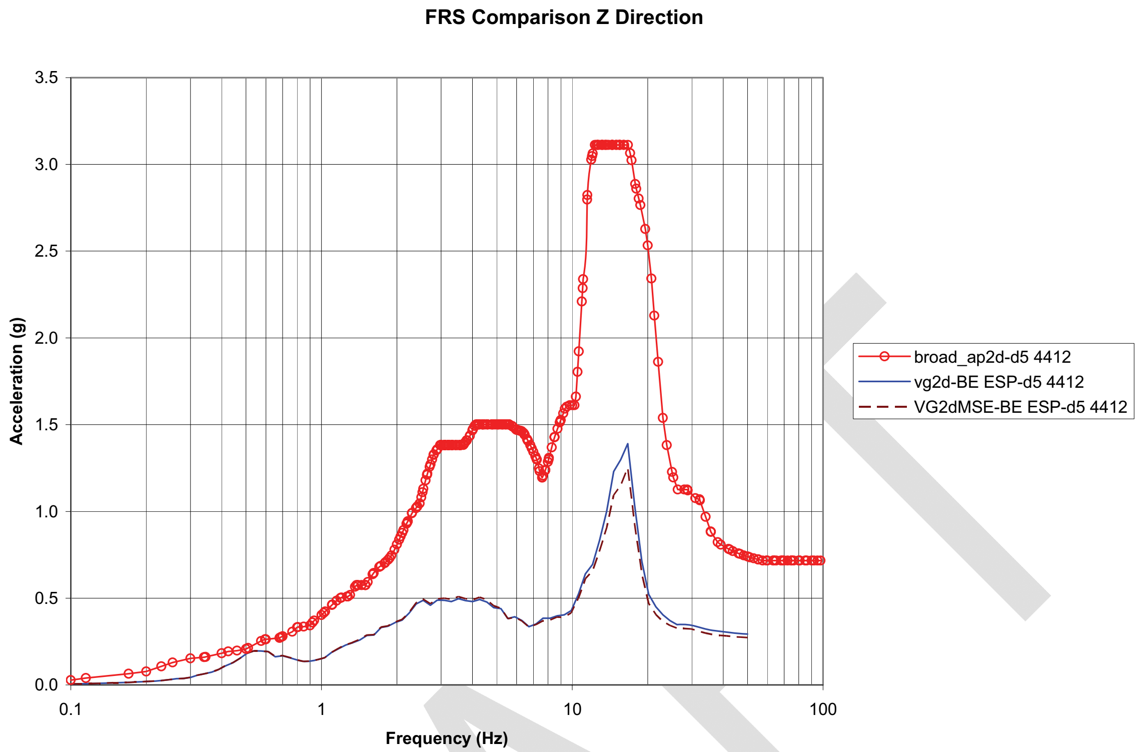


Figure 5.3-16 – FRS Comparison at Node 4412 (Z Direction)

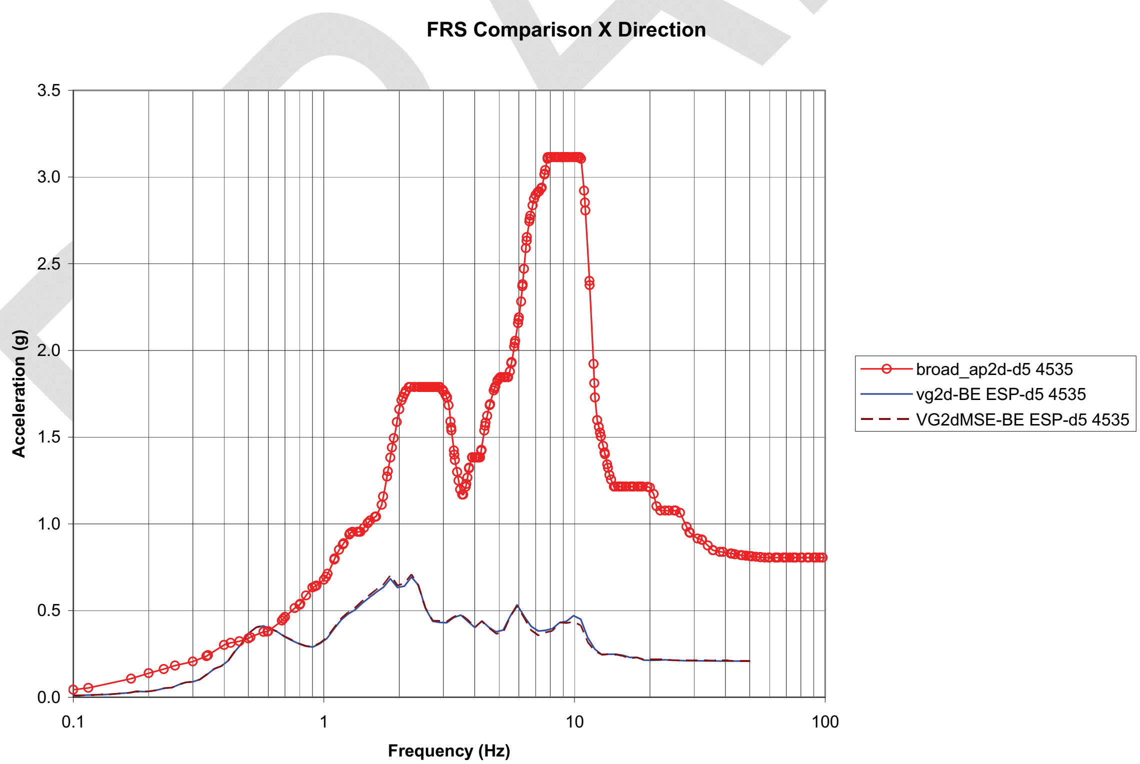


Figure 5.3-17 – FRS Comparison at Node 4535 (X Direction)

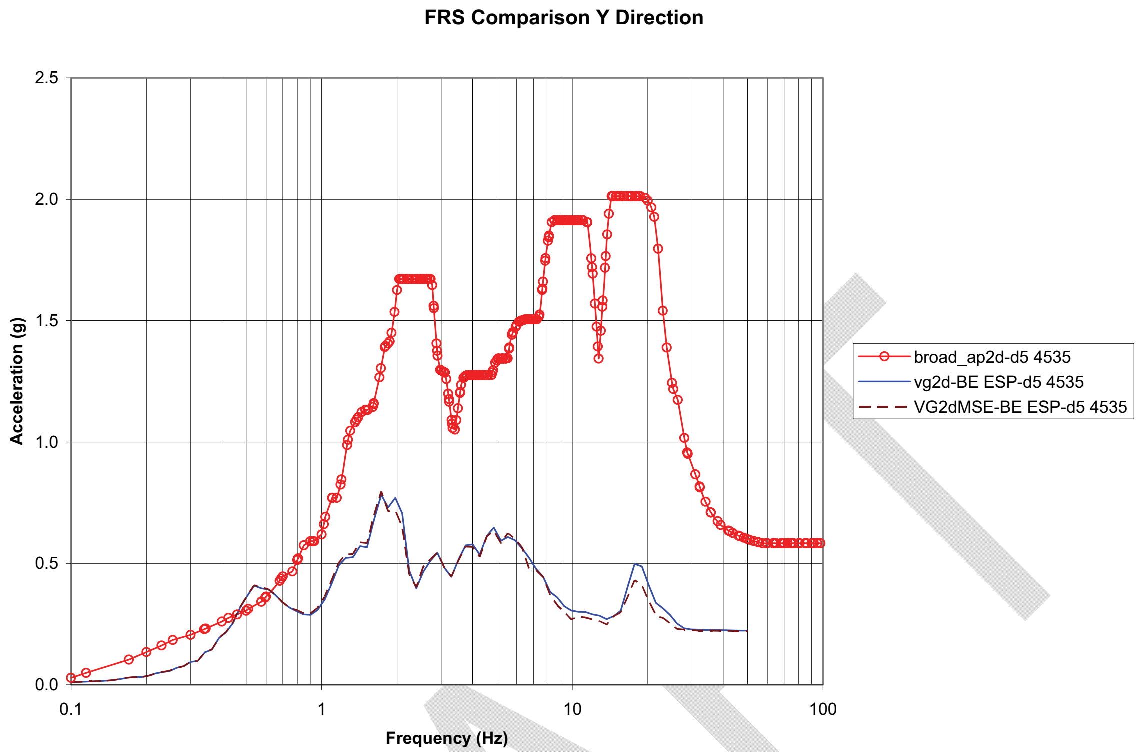


Figure 5.3-18 – FRS Comparison at Node 4535 (Y Direction)

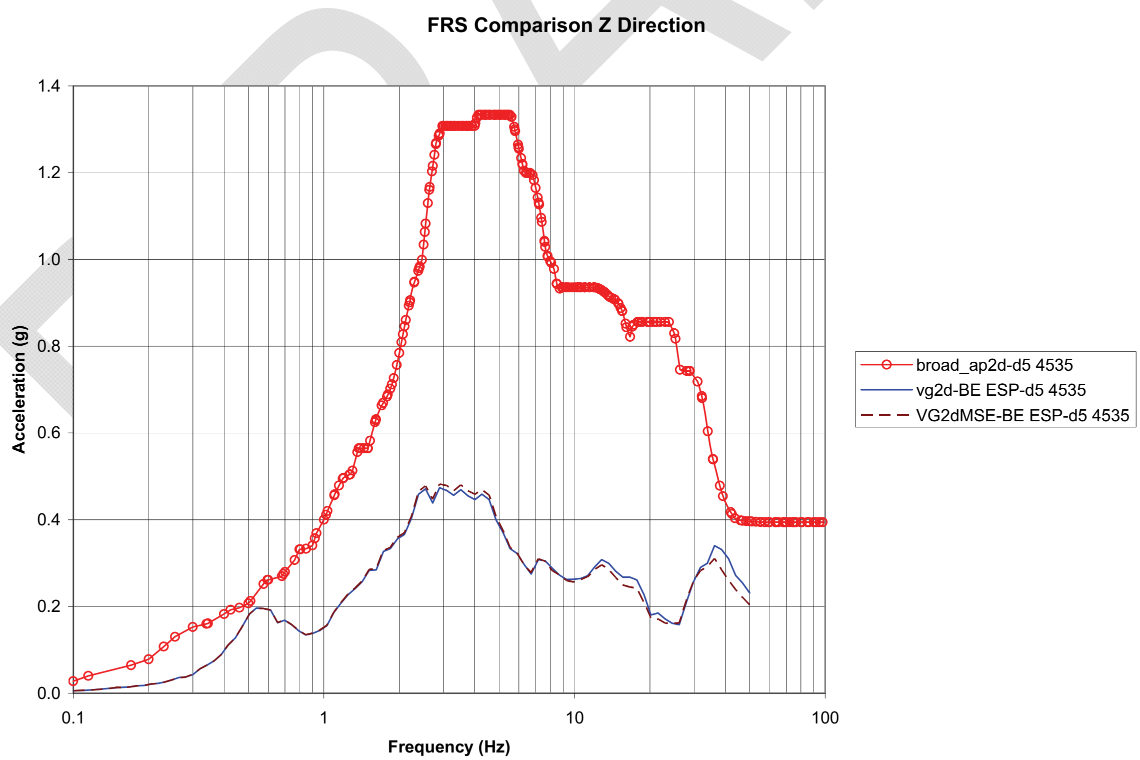


Figure 5.3-19 – FRS Comparison at Node 4535 (Z Direction)

6.0 Stability Analyses

A stability analysis of the Nuclear Island (NI) has been performed with factors of safety determined for:

- Flotation for ground water and maximum flood effect
- Overturning and sliding during tornado/wind/hurricane conditions
- Overturning and sliding during the SSE

These “generic” analyses have been performed for six site profiles: hard rock, firm rock, soft rock, upper bound soft-to-medium soil, soft to medium soil, and soft soil. The stability factors of safety for non-seismic loading for the AP1000 NI are given in Table 6-1. The sliding analyses have been performed using a coefficient of friction of 0.7. For the Vogtle site, a sliding coefficient of friction of 0.45 is used. Since all of the sliding factors of safety are larger than 10, it is not necessary to calculate new sliding factors of safety for these cases using a coefficient of friction of 0.45 since there is a lot of margin between the factors of safety and the limit. The minimum sliding factor of safety will be above 7 for the Vogtle site if a sliding coefficient of friction of 0.45 is considered.

The water table at the Vogtle site is below the NI basemat. The water table is at Vogtle site elevation 165’, and the lowest point of the basemat is at Vogtle site elevation 180’. The Vogtle seismic stability analyses reflect this and, therefore, the NI dead weight is not reduced by the buoyancy force.

A seismic stability analysis has been refined for the Vogtle site for the following soil cases using results from the 2D SASSI analysis:

Early Site Permit Soil Cases (ESP)

- Lower Bound
- Best Estimate
- Upper Bound

Sensitivity Soil Cases (SEN)

- Lower Bound
- Best Estimate
- Upper Bound

The acceptability of using seismic response from the 2D SASSI model has been documented by Westinghouse generically. It has been shown that the shear and overturning moments compare closely between the 3D (NI20) shell model and 2D analyses. In addition it has been shown that there are very little changes in the seismic factors of safety associated with overturning and sliding when using the 2D SASSI model or NI20 3D shell model.

Table 6-1 – Stability Factors of Safety for Non-Seismic Loading for AP1000 NI

Load Combination	Sliding		Overturning		Flotation	
	Factor of Safety	Limit	Factor of Safety	Limit	Factor of Safety	Limit
D + H + B + W	Design Wind					
North-South	23.2	1.5	51.5	1.5	–	–
East –West	17.4	1.5	27.9	1.5	–	–
D + H + B + Wt	Tornado Condition					
North-South	12.8	1.1	17.7	1.1	–	–
East –West	10.6	1.1	9.6	1.1	–	–
D + H + B + Wh	Hurricane Condition					
North-South	18.1	1.1	31	1.1	–	–
East –West	14.2	1.1	16.7	1.1	–	–
	Flotation					
D + F	–	–	–	–	3.51	1.1
D + B	–	–	–	–	3.7	1.5

6.1 Seismic Stability Formulas

The calculation of the seismic stability factors of safety are based on the following formulations for overturning and sliding given below.

Overturning Seismic Stability Formula

FS = (MR + MP) / (MO + MAO) (6.1)

FS = Factor of safety against overturning from a safe shutdown earthquake
MR = Nuclear Island's resisting moment against overturning (due to the force of deadweight - buoyancy)
MO = Maximum SSE induced overturning moment acting on the nuclear island
MP = Resistance moment associated with passive pressure
MAO = Moment due to lateral forces caused by active and overburden pressures

Maximum SSE overturning moments are calculated about column line I, west side of shield building, and column lines 1 and 11. Column line I is located along the east side of the auxiliary building. Shield Building West (SB West) is located on the west side of the shield building. Column line 11 is the north side of the auxiliary building and column line 1 is the south side.

Sliding Seismic Stability Formula

FS = [Ff + Fp] / [FSSE + FAO] (6.2)

FS = Factor of safety against Sliding
Ff = Sliding resistance based coefficient of friction factor of 0.45
Fp = Passive soil pressure resistance
FAO = Active soil pressure + Overburden
FSSE = Seismic Shear

6.2 Vogtle Site-Specific Stability Evaluation

The site-specific evaluation is performed using the seismic response of the NI from the lower bound, best estimate, and upper bound soil cases as described in Section 3.0. The site specific stability factors of safety for the Vogtle site are summarized in Tables 6.2-1 and 6.2-2 for the soil profiles associated with the Early Site Permit and Sensitivity cases. As seen from this table, all of the factors of safety are well above the limit, and reflect much more margin than the AP1000 generic all soil analysis.

Increasing the Vogtle site-specific seismic response spectra at the 40' outcrop location to be equal to 0.3g (ZPA), new stability factors of safety are calculated. The increase in seismic level is 1.2 in the horizontal directions, and 1.36 in the vertical direction. The summary of stability factors of safety are given in Tables 6.2-3 and 6.2-4, and they are well above the limits.

The sliding factors of safety are based on full passive pressure. The sensitivity of the sliding factor of safety versus passive pressure is shown in Figure 6.2-1 using the upper bound ESP estimate that has the lowest sliding factors of safety. It is noted that at the limit of the sliding factor of safety (1.1) the passive pressure is close to the At Rest Pressure (15% Passive Pressure vs. At Rest Pressure which is 11% of the Passive Pressure).

It can be concluded from the stability evaluation for the Vogtle site that:

- Seismic stability factors of safety for the Vogtle site have significant margin.
- The Vogtle site seismic stability factors of safety have significant margin above the AP1000 generic design considering a 0.3g (ZPA) level. The reduction of seismic load at Vogtle is due to SSI effects that do not exist in the hard rock case.
- The seismic stability factors of safety for sliding have significant margin using a sliding coefficient of friction of 0.45. Even though the Vogtle coefficient of friction is less than that standard AP1000 certified design value, the standard design capacity would not be reduced for the Vogtle site.
- When the seismic stability factor of safety for sliding is equal to the factor of safety limit of 1.1, the passive pressure is close to the At Rest Pressure. Therefore, the NI at the Vogtle site will not slide and consideration of a dynamic coefficient of friction in the stability calculation for sliding is not required.

Table 6.2-1 –Comparison of Seismic Stability Factors of Safety - ESP

Stability Factors of Safety	AP1000 Generic Analyses	Vogtle Site Specific Lower Bound Estimate	Vogtle Site Specific Best Estimate	Vogtle Site Specific Upper Bound Estimate	Limit
Sliding NS earthquake	1.28	2.07	1.97	1.83	1.1
Sliding EW earthquake	1.33	2.10	2.08	1.96	1.1
Overturning NS earthquake	1.35	3.74	3.52	3.37	1.1
Overturning EW earthquake	1.12	2.77	2.67	2.45	1.1

Table 6.2-2 –Comparison of Seismic Stability Factors of Safety - SEN

Stability Factors of Safety	AP1000 Generic Analyses	Vogtle Site Specific Lower Bound Estimate	Vogtle Site Specific Best Estimate	Vogtle Site Specific Upper Bound Estimate	Limit
Sliding NS earthquake	1.28	2.17	2.04	2.02	1.1
Sliding EW earthquake	1.33	2.24	2.19	2.10	1.1
Overturning NS earthquake	1.35	3.97	3.80	3.68	1.1
Overturning EW earthquake	1.12	3.01	2.90	2.81	1.1

Table 6.2-3 –Comparison of Seismic Stability Factors of Safety – ESP
Increased to 0.3g at 40' Outcrop

Stability Factors of Safety	AP1000 Generic Analyses	Vogtle Site Specific Lower Bound Estimate	Vogtle Site Specific Best Estimate	Vogtle Site Specific Upper Bound Estimate	Limit
Sliding NS earthquake	1.28	1.89	1.78	1.64	1.1
Sliding EW earthquake	1.33	1.92	1.89	1.77	1.1
Overturning NS earthquake	1.35	2.85	2.69	2.58	1.1
Overturning EW earthquake	1.12	2.23	2.15	1.98	1.1

Table 6.2-4 –Comparison of Seismic Stability Factors of Safety – SEN
Increased to 0.3g at 40' Outcrop

Stability Factors of Safety	AP1000 Generic Analyses	Vogtle Site Specific Lower Bound Estimate	Vogtle Site Specific Best Estimate	Vogtle Site Specific Upper Bound Estimate	Limit
Sliding NS earthquake	1.28	1.99	1.86	1.84	1.1
Sliding EW earthquake	1.33	2.06	2.00	1.91	1.1
Overturning NS earthquake	1.35	2.99	2.89	2.80	1.1
Overturning EW earthquake	1.12	2.38	2.32	2.25	1.1

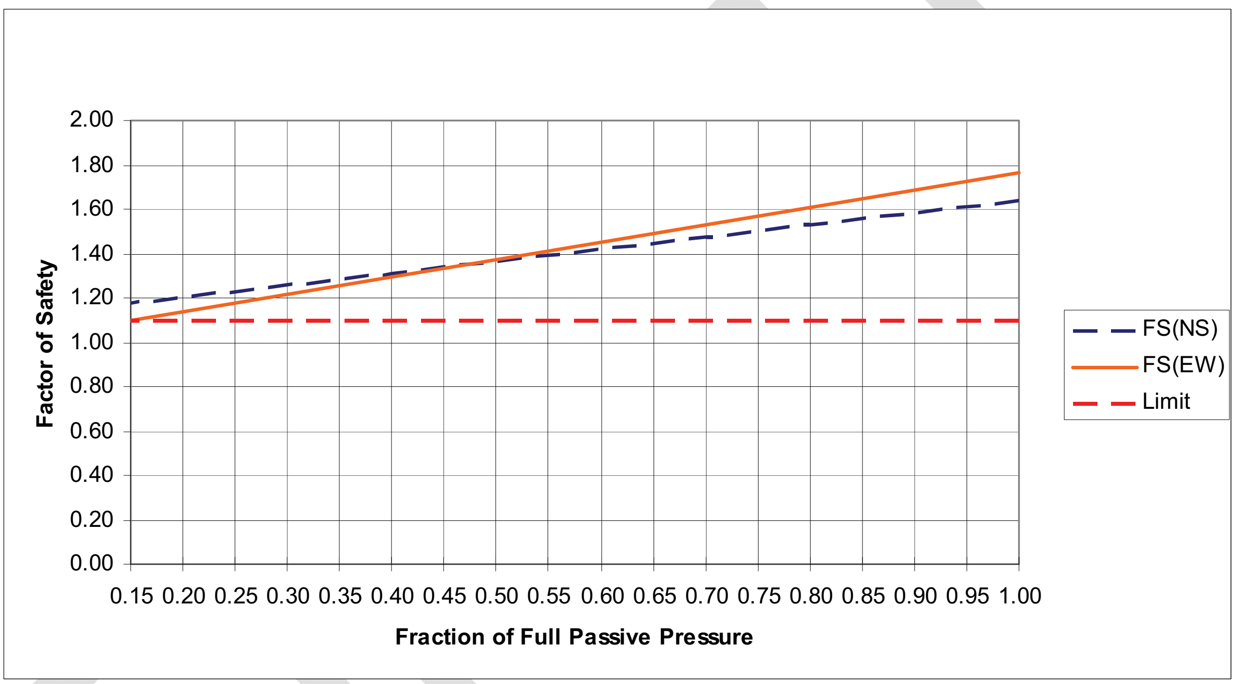


Figure 6.2-1 – Sliding Factor of Safety versus Passive Pressure
ESP- Increased to 0.3g at 40 ft Outcrop – UB

7.0 Foundation Bearing Pressures

The foundation bearing pressures from the Vogtle site specific evaluation are the maximum pressures associated with the dead load plus seismic load cases. To account for 3D seismic effects, the maximum bearing pressures were calculated using the square root sum of squares of the seismic responses in the NS, EW, and vertical directions. The maximum bearing pressure given in Table 7-1 includes the bearing pressure due to seismic excitation plus the building dead load pressure.

Table 7-1 – Vogtle Specific Maximum Pressures Under Basemat (ksf)

	Nuclear Island	Radwaste	Annex	Turbine
Best Estimate ESP	16.53	1.43	3.73	2.48
Best Estimate SEN	15.44	1.54	4.92	2.42
Lower Bound ESP	15.71	1.50	3.77	2.54
Lower Bound SEN	14.77	1.26	7.20	2.42
Upper Bound ESP	17.95	1.38	6.92	2.49
Upper Bound SEN	15.59	1.68	5.35	2.41

8.0 Settlement of Foundations

The details of the foundation settlement analysis for the Vogtle site are reported in Reference 3. The foundation mat displacements for the AP1000 buildings due to potential elastic settlements of the subgrade soils at the Vogtle site are documented in this reference. The evaluation examines the construction conditions including the structural loading sequences and calculates the resulting settlements and the foundation mat displacement time histories. Based on the evaluation, it estimates maximum foundation mat displacements rotations and the maximum differential displacements between buildings.

The settlement analysis and the subsequent structural evaluation estimates foundation mat vertical displacements, tilting, and differential displacements between buildings. The analysis is based on the proposed construction schedule and sequence.

The predicted maximum settlements are less than 3 inches total and 1/2 inch in 50 feet tilt across the basemat (Reference 3). The settlements calculated for the Vogtle site are given in Table 8-1 below.

Table 8-1 – The maximum settlements for the Vogtle site

Building	Settlement Summary				Differential Settlement (center-to-center)			
	Max. Settlement	Min. Settlement	Tilt (inch per 50') NS	Tilt (inch per 50') EW	Nuclear Island	Turbine Bldg.	Radwaste Bldg.	Annex Bldg.
Nuclear Island	2.66	1.39	0.06	0.39	---	0.44	0.12	0.32
Turbine Bldg.	1.85	1.51	0.05	0.01	0.44	---	0.32	0.75
Radwaste Bldg.	2.00	1.98	0.00	0.00	0.12	0.32	---	0.44
Annex Bldg.	2.47	2.37	0.01	0.05	0.32	0.75	0.44	---

Note: All values in inches unless noted otherwise.
Settlement calculation assumes non-NI buildings are constructed at an elevation relative to the settled NI.

The maximum foundation tilt is 0.39" in 50' and occurs in the east-west direction of the Nuclear Island(less than ½" in 50' tilt across basemat in Reference 3). The most significant differential settlement between the Nuclear Island and adjacent buildings is 0.44 inch, between the Nuclear Island and the Turbine Building.

9.0 Settlement Monitoring Program

The proposed settlement monitoring plan addresses the expected heave or rebound during the excavation and dewatering phase, as well as the settlements due to the building construction loads. Compared to the predicted settlements for the generic profiles considered in the standard design, the calculated settlements for the Vogtle conditions are relatively small and all within the limiting settlement parameters utilized in the standard design.

The proposed settlement monitoring program will include:

- Piezometers to measure pore water pressures in the Blue Bluff Marl and the Lower Sand layer. Vibrating wire piezometers are preferred for this purpose, as they are adequately sensitive and responsive and easily record positive and negative changes on a real-time basis.

- Settlement monuments placed directly on concrete, preferably on the mud mat and on the corners of the structures at grade that are accessible with conventional surveying equipment.

Settlements will be monitored continuously during all construction stages to verify structural displacements due to construction loads. Figure 9-1 presents a distribution of the suggested monitoring points located at the building's foundation mats.

Monitoring will be performed continuously for all the structures during and after construction, particularly when large loads are applied early in the NI construction (CV Head, M20, M21).

Particular emphasis will be placed on the rotation about the north-south axis of the Nuclear Island, as this study estimates that a noticeable tilt may occur in this direction.

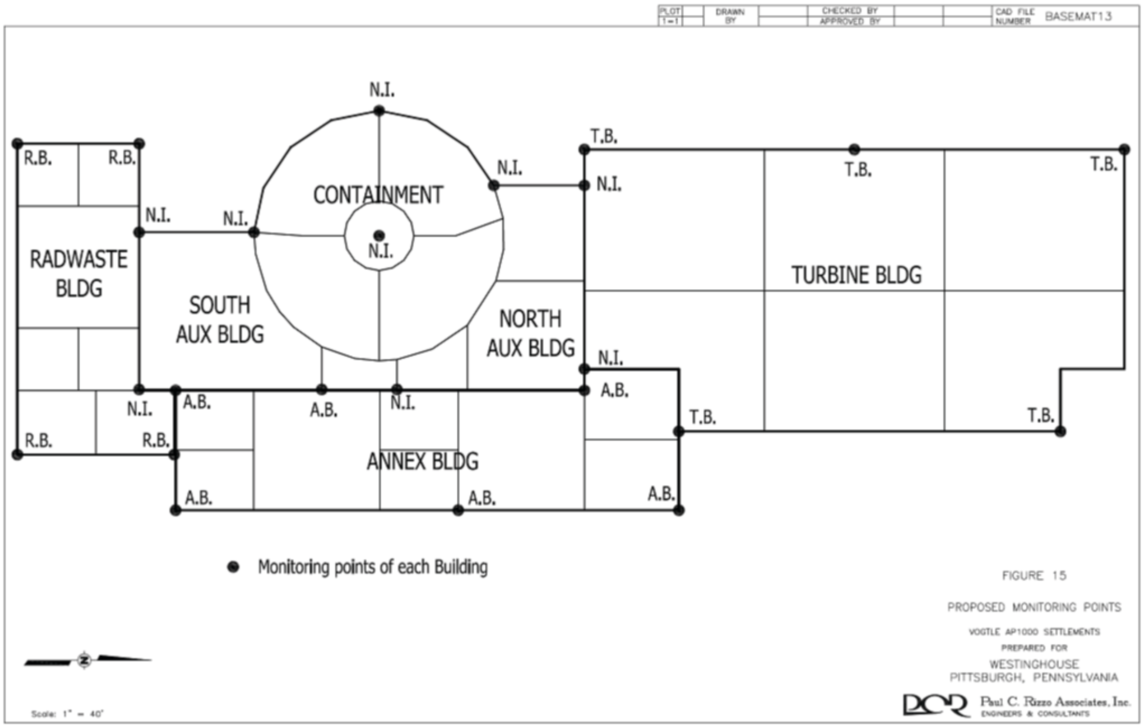


Figure 9-1 – Proposed Monitoring Points for Vogtle

10.0 References

1. Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants, Rev. 1.
2. APP-1000-S2C-025, Rev. 2 "AP1000 2D SSI Analysis with Adjacent Buildings."
3. Foundation Mats Settlements, Vogtle AP1000 Foundations, Report No. 05-3423-R1, Rev. 0, Paul C. Rizzo Associates.
4. Calculation of Transfer Functions Using WEC Response Time History, SNC Calculation Number SV0-SSAR-XSC-2018 Bechtel National, Inc.,

Appendix A: Vogtle 2D Bathtub Model and AP 2D Model Comparison

Due to the large volume of excavation and the lateral extent of the backfill at the Vogtle site, the backfill layers were modeled as free-field soil layers in the characterization of the soil profile for both the site amplification for development of ground motion (GMRS and FIRS) and the site-specific seismic SSI analysis of the AP1000. A sensitivity analysis was performed to assess the sensitivity of this modeling assumption to the explicit modeling of the geometry of the excavation and backfill on site response and seismic SSI response. The results provided here pertain to the SSI portion of the sensitivity analysis for the Vogtle site-specific seismic SSI analysis of the AP1000.

A 2D SASSI bathtub model (Bathtub Model-d5) was developed to represent the E-W cross section of the Vogtle excavation and backfill soil condition as shown in Figure A-1. The bathtub model is constructed with the nuclear island and adjacent annex building with the backfill soil modeled as part of the structural model. For this model the strain-compatible soil properties for the in-situ upper sand layer were used as part of the free-field SASSI model. The seismic response of the bathtub model is compared to the seismic response obtained using the standard Vogtle site-specific 2D SASSI model (2D-AP-d5). The input time histories and the compatible soil profiles were provided for both models from the site response portion of this sensitivity analysis. For each of the two 2D SASSI models, the SSI response was limited to using the mean soil profile and one time history from the site response portion of the sensitivity analysis. The input motions for the two SSI analyses were from the respective 1D SHAKE analysis from the site response portion of the sensitivity analysis and consistent with the free-field soil characterization used in each SASSI model. For the bathtub model the single time history is called the “in-situ time history” since it is developed from a 1D soil column that included the 86 foot in-situ upper sand layer. For the standard Vogtle site-specific 2D SASSI model the single time history is called “backfill time history” since it is developed from a 1D soil column where the top 86 feet is backfill.

Figures A-2 and A-7 are the horizontal floor response spectra 5% damping comparisons of the Vogtle 2D SASSI bathtub model (Bathtub Model-d5) using in-situ time history and the Vogtle site-specific 2D SASSI AP model (2D-AP-d5) using the backfill time history. The AP1000 2D standard design enveloped floor response spectra are shown to provide an overall assessment of the available margin. The floor response spectra are compared at six critical locations. It should be noted that the single time histories that were used represent time histories from 1D soil columns analyses with a high frequency rock input time history which resulted in input motion that had relatively low frequency content. This does not invalidate the one to one comparison needed for a sensitivity analysis. The backfill model, Bathtub Model-d5, used an in-situ time history. The AP model, 2D-AP-d5, used a time history that was developed assuming the backfill was of infinite extent.

Figures A-8 to A-13 compare the Vogtle 2D AP model and the Vogtle bathtub model transfer functions at the same six key locations. Bath-tub represents the Bathtub Model and Backfill represents the AP model. Transfer functions represent the harmonic amplification of the input motion at base rock to the response motion at the selected locations in the SSI model. This result is documented in Bechtel National, Inc. “Calculation of Transfer Functions Using WEC Response Time History”, SNC Calculation Number SV0-SSAR-XSC-2018 (Reference 4).

Although the bathtub model shows slightly higher transfer function peaks, the transfer function comparisons are very close and indicated no significant differences. The response spectra obtained from the 2D bathtub and 2D AP model are similar. There are some variations between the 2D bathtub model and the 2D AP model due to the modeling of the backfill but these differences are small. There is significant margin between the AP1000 2D and the Vogtle 2D results compared to the AP1000 generic spectra. From the analyses performed using the 2D AP model and the backfill model, it can be concluded that the AP1000 plant design is acceptable for the Vogtle plant site. The difference between the AP model and the bathtub model are negligible.

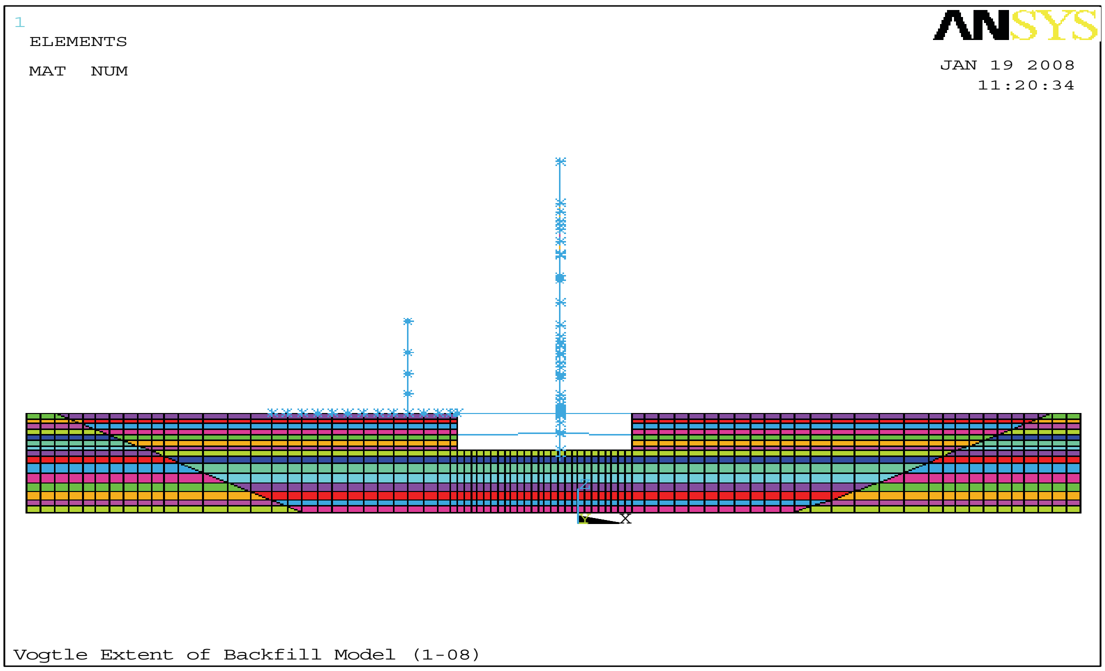


Figure A-1: 2D SASSI Backfill Model

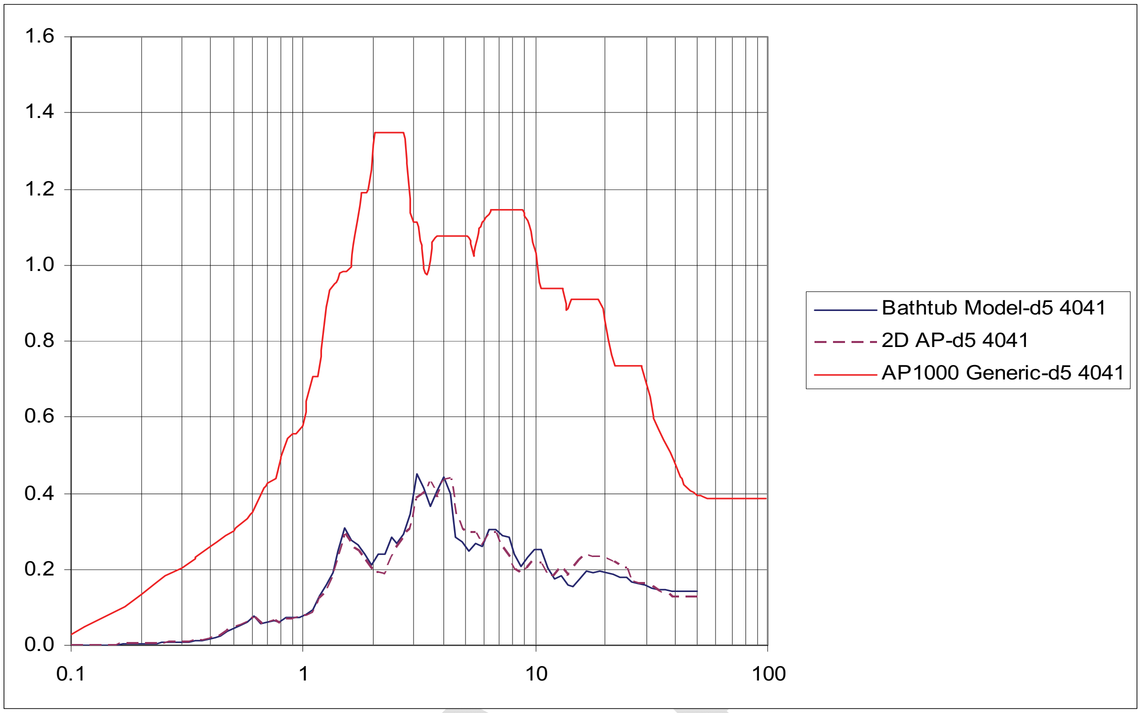


Figure A-2: FRS Comparison at Node 4041

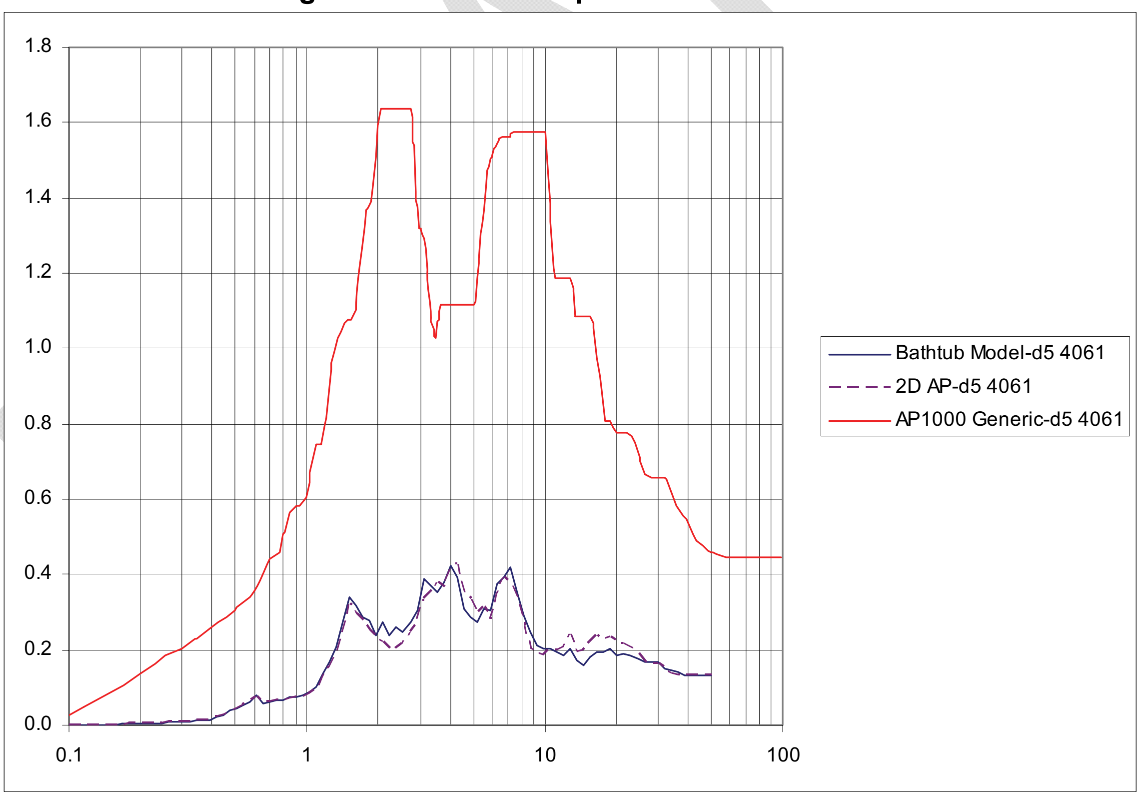


Figure A-3: FRS Comparison at Node 4061

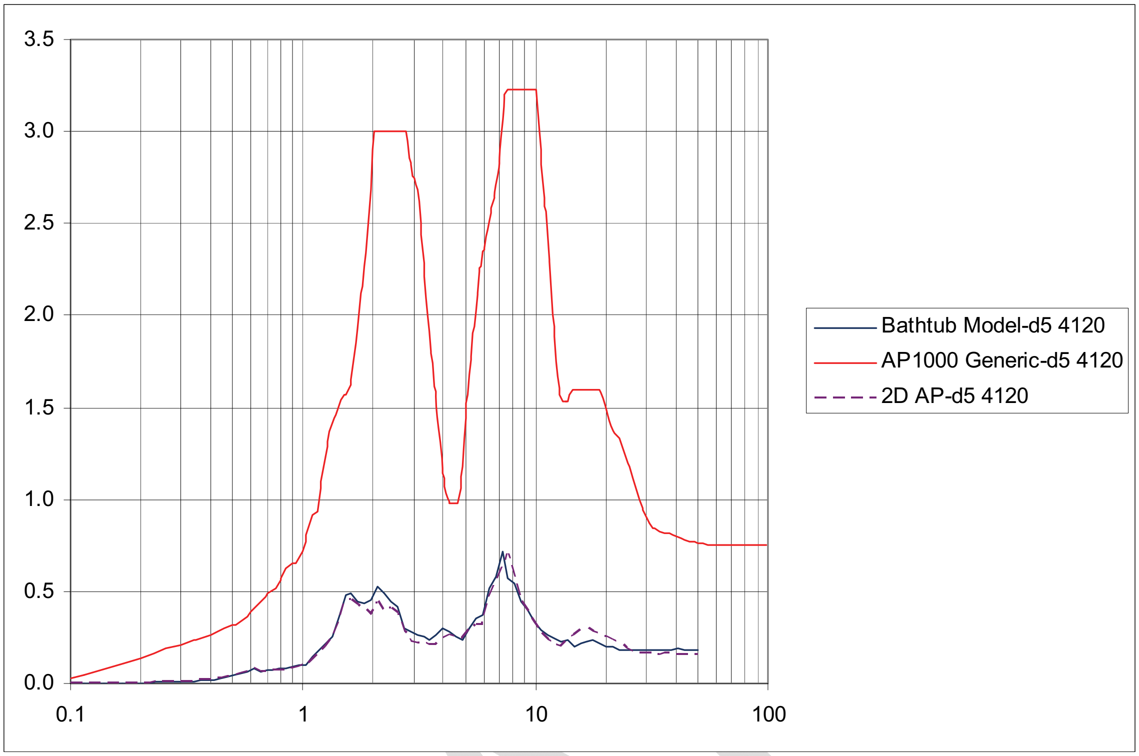


Figure A-4: FRS Comparison at Node 4120

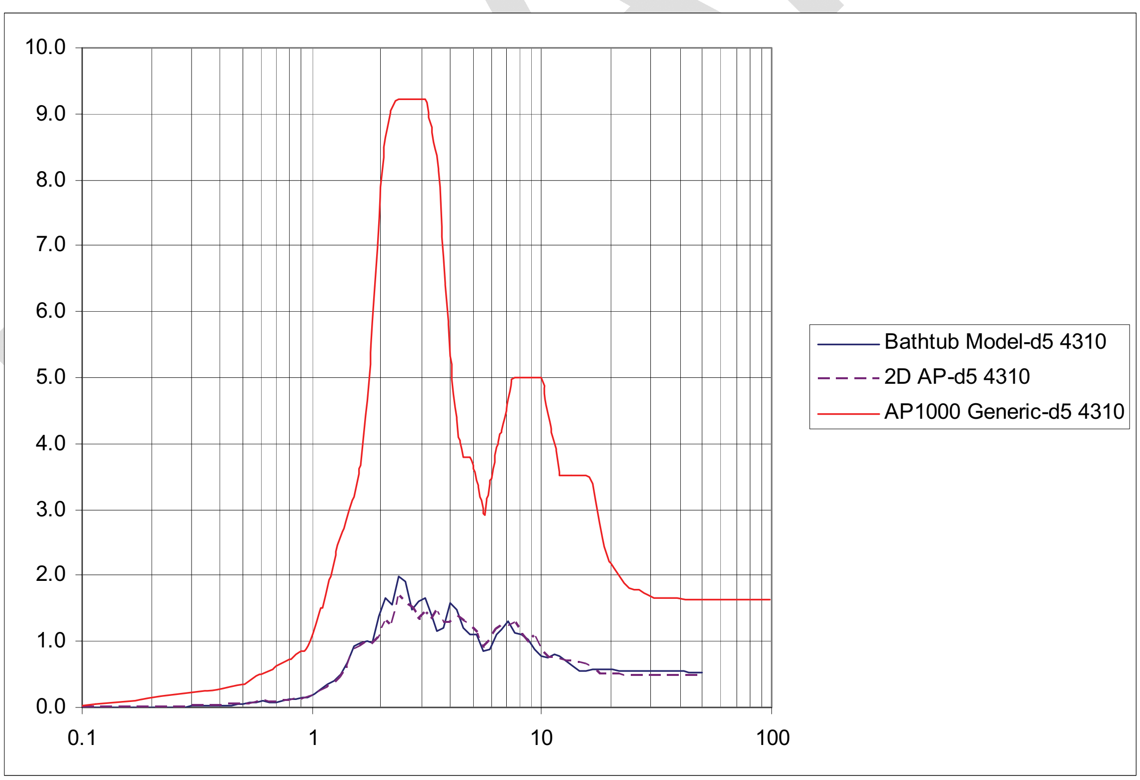


Figure A-5: FRS Comparison at Node 4310

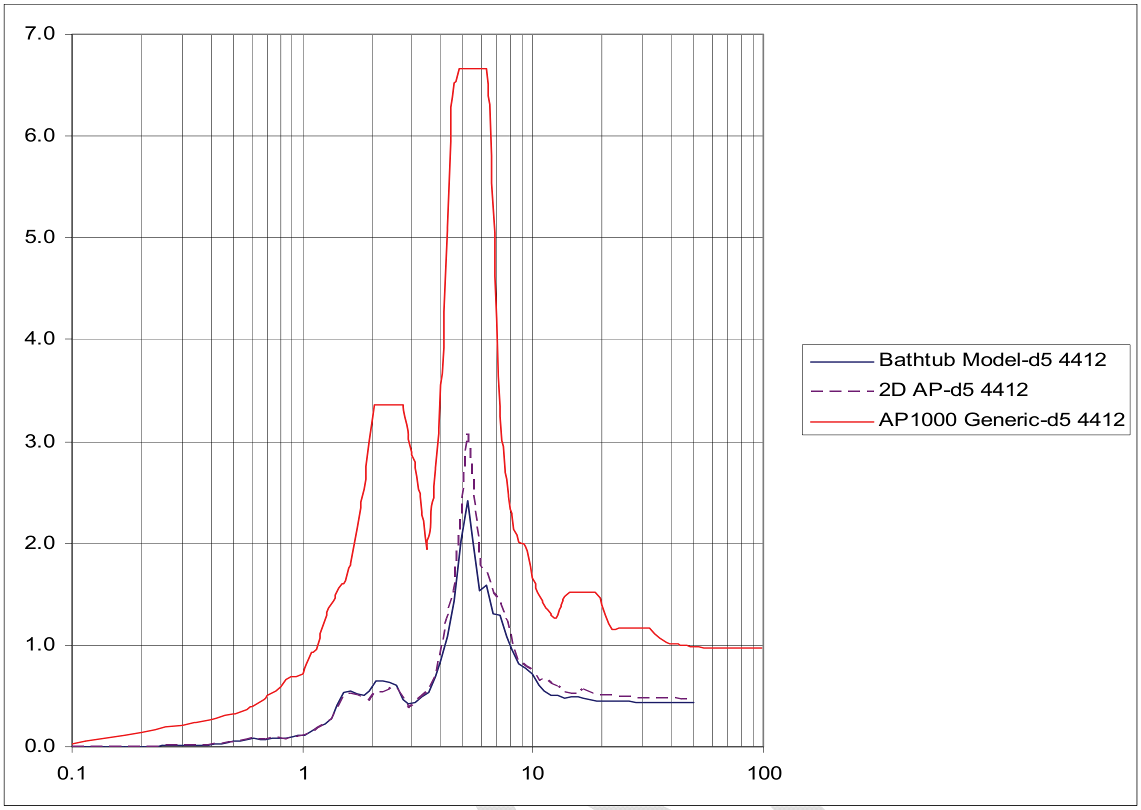


Figure A-6: FRS Comparison at Node 4412

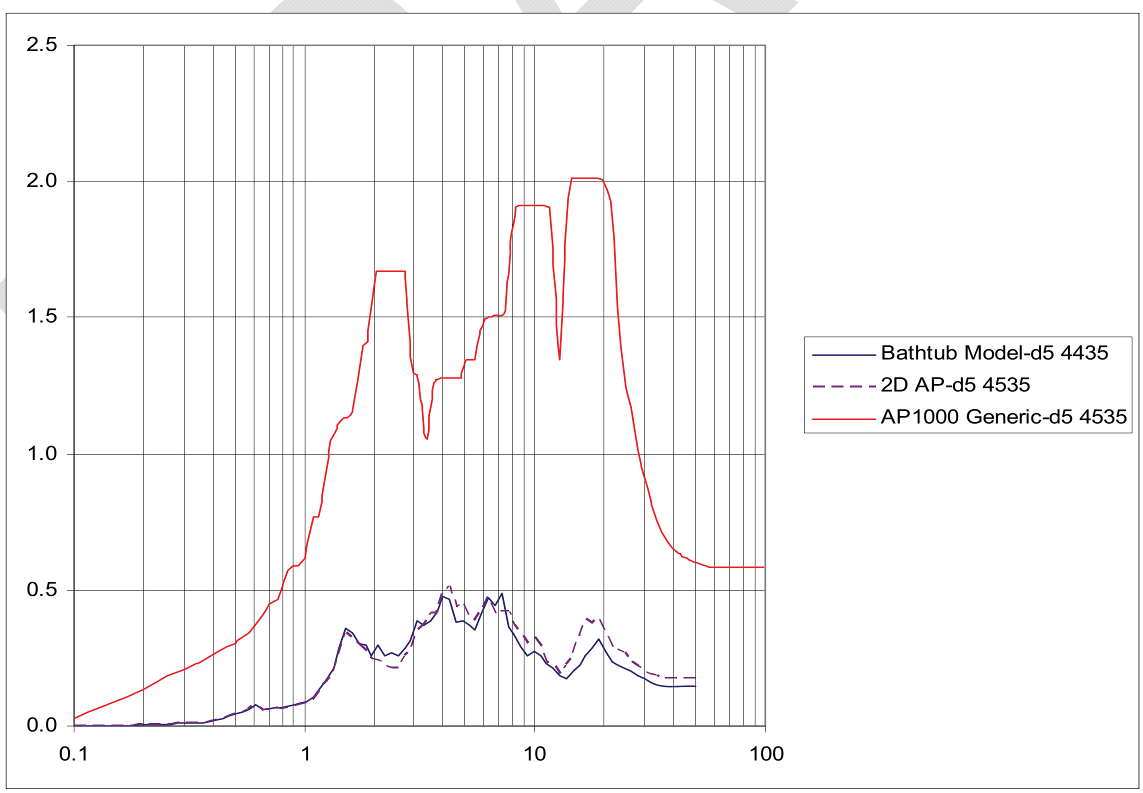


Figure A-7: FRS Comparison at Node 4535

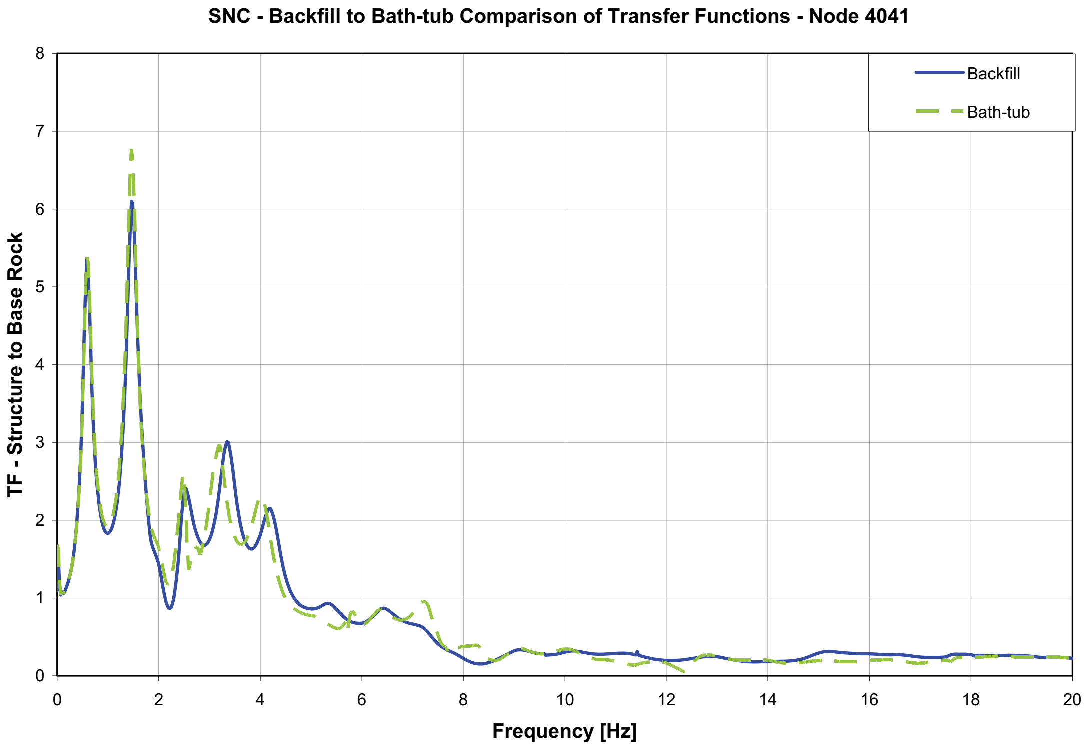


Figure A-8: Vogtle AP vs. BT Transfer Function from Bed Rock to Node 4041

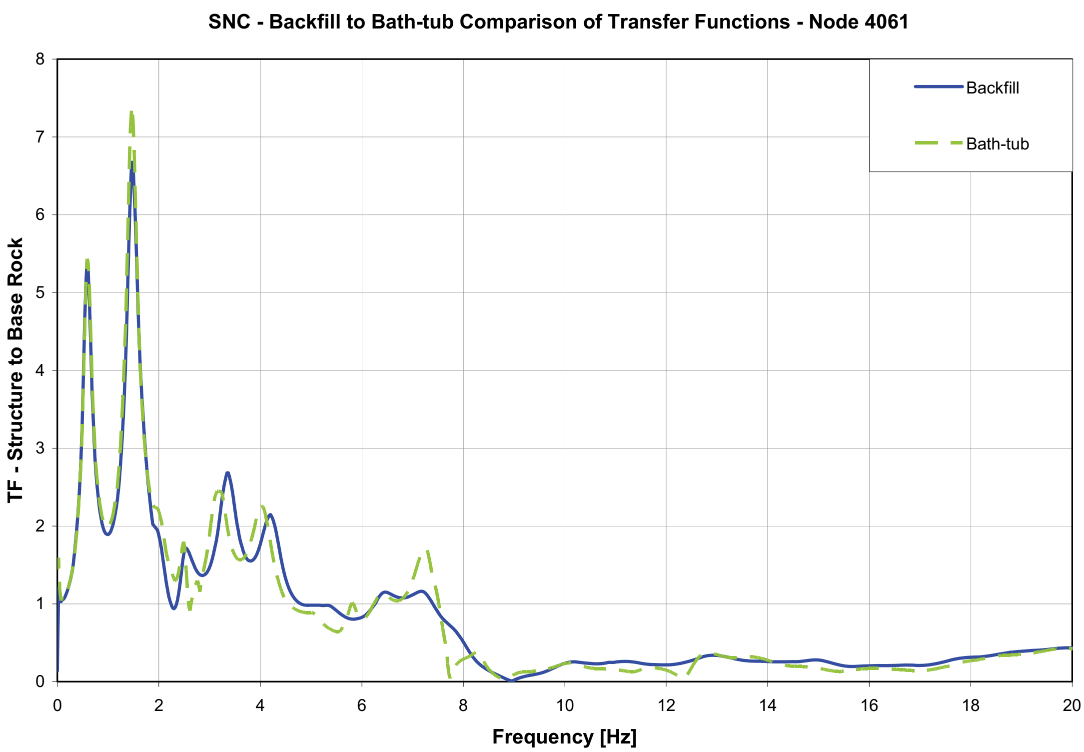


Figure A-9: Vogtle AP vs. BT Transfer Function from Bed Rock to Node 4061

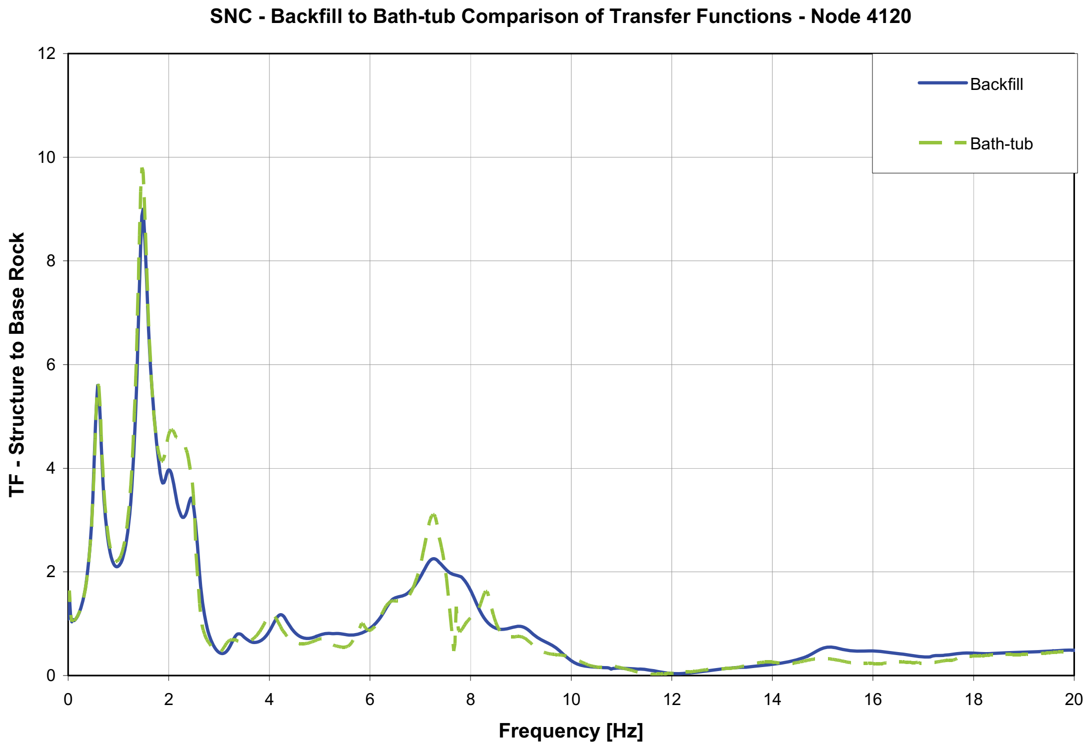


Figure A-10: Vogtle AP vs. BT Transfer Function from Bed Rock to Node 4120.

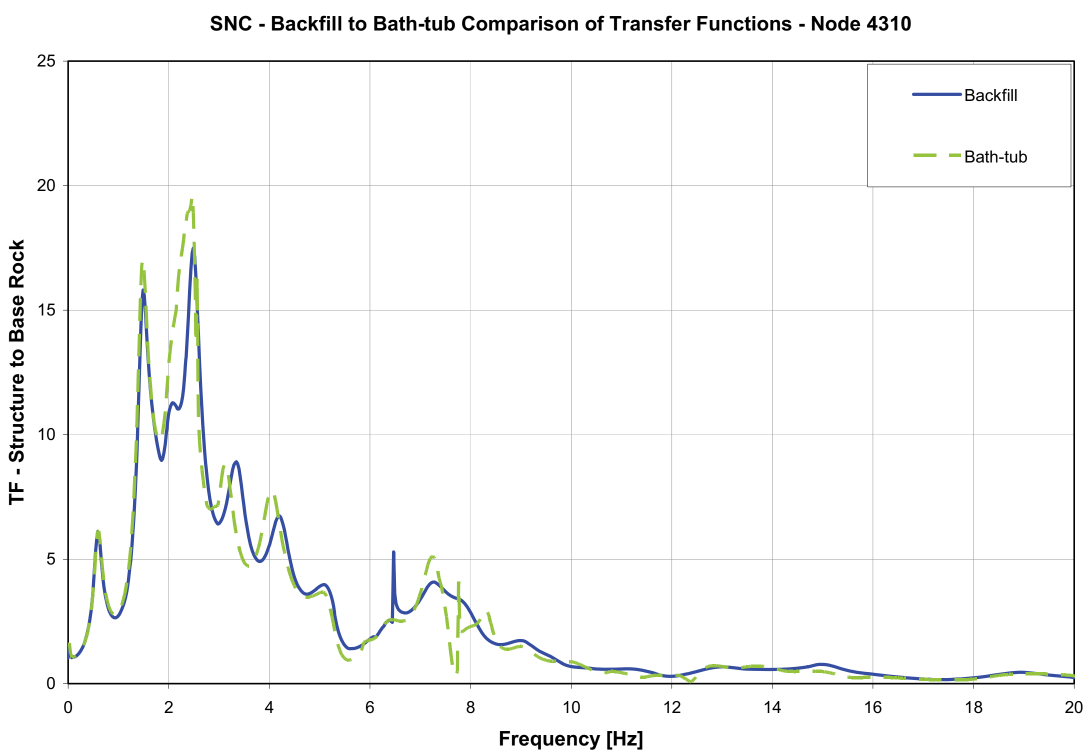


Figure A-11: Vogtle AP vs. BT Transfer Function from Bed Rock **to Node 4310**

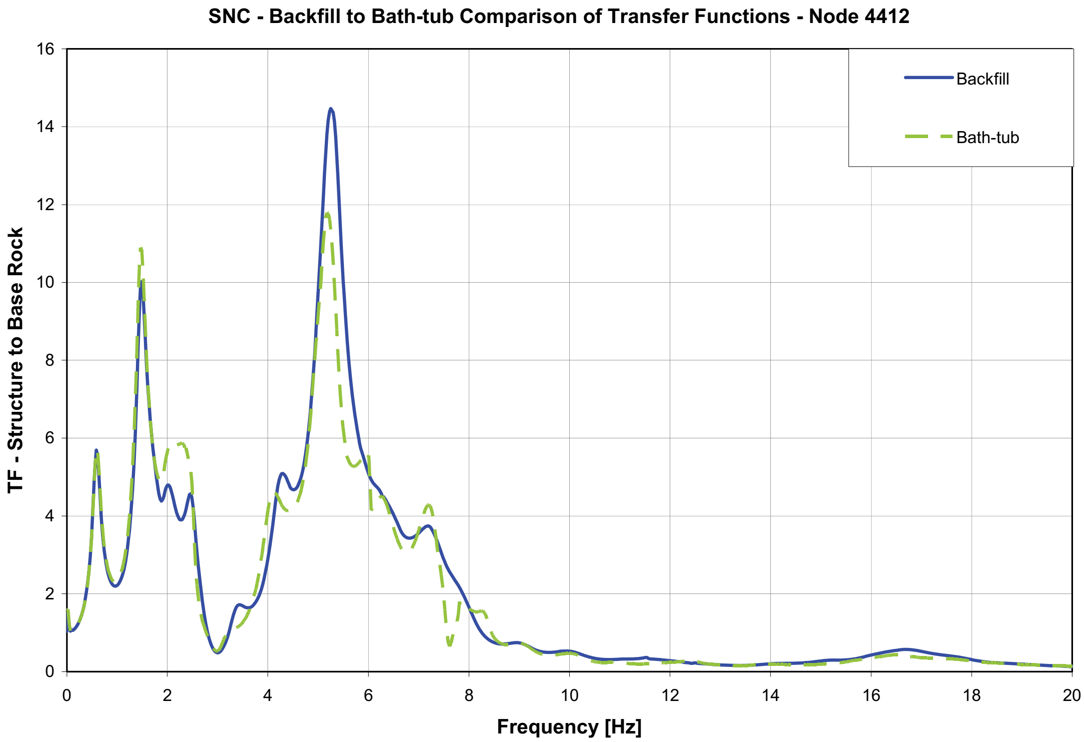


Figure A-12: Vogtle AP vs. BT Transfer Function from Bed Rock to Node 4412

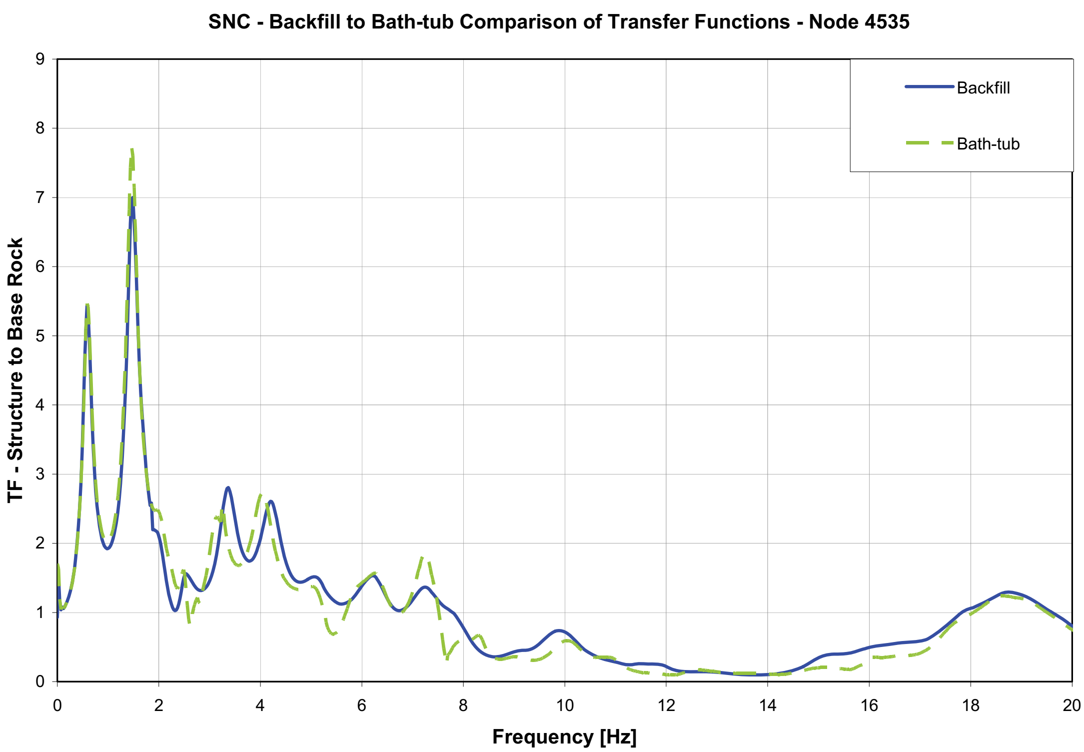


Figure A-13: Vogtle AP vs. BT Transfer Function from Bed Rock to Node 4535