SEISMIC ANALYSIS AND STRUCTURE CAPACITY EVALUATION OF THE BELENE NUCLEAR POWER PLANT

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ABSTRACT

The seismic analysis and structure capacity evaluation of the Belene Nuclear Power Plant, a two-unit VVER 1000, was performed. The principal objective of the study was to review the major aspects of the seismic design including ground motion specification, foundation concept and materials, and the Unit 1 main reactor building structure response and capacity.

The main reactor building structure/foundation/soil were modeled and analyzed by a substructure approach to soil-structure interaction (SSI) analysis. The elements of the substructure approach, implemented in the family of computer programs CLASSI, are:

- Specification of the free-field ground motion
- Modeling the soil profile
- SSI parameters
- Modeling the structure
- SSI-response analyses

Each of these aspects is discussed.

The Belene Unit 1 main reactor building structure was evaluated to verify the seismic design with respect to current western criteria. The structural capacity evaluation included criteria development, element load distribution analysis, structural element selection, and structural element capacity evaluation.

Equipment and commodity design criteria were similarly reviewed and evaluated.

Methodology results and recommendations are presented.

INTRODUCTION

The seismic analysis and structure capacity evaluation of the Belene Nuclear Power Plant (NPP), a two-unit VVER 1000, was performed. The principal objective of the study was to review the major aspects of the seismic design including ground motion specification, foundation concept and materials, and the Unit 1 main reactor building structure response and capacity (Reference 1). This paper reports on the seismic response and capacity aspects of the study.

In the following, the main reactor building structure, and components housed therein, are described. Modeling of the structure/foundation/soil is discussed–the overall approach and each element as it pertains to the Belene configuration. The substructure approach to soil-structure interaction (SSI) analysis was applied. Elements of this approach are: specification of the free-field ground motion, modeling the soil profile, calculating SSI parameters, modeling the structure, and combining these elements in the SSI response analysis. The results of the analyses are structure loads for capacity evaluation and in-structure response spectra for evaluation of equipment and commodities.

The Belene Unit 1 main reactor building structure was evaluated to verify the seismic design with respect to current western criteria. The structural capacity evaluation included criteria development, element load distribution analysis, structural element selection, and structural element capacity evaluation.

Equipment and commodity design criteria were similarly reviewed and evaluated.

BELENE NPP MAIN REACTOR BUILDING

The Belene main reactor building consists of four substructures which differ from both structural and functional points of view (Figures 1 and 2). Starting from foundation base mat [El. (-) 4.2m below grade], the substructure rises up to a second concrete base mat (El. 13.2m) which supports the reactor containment building, the reactor internal structure and a peripheral auxiliary building called the outer building.

The substructure is a three story building which houses the main control room and auxiliary systems equipment including several large tanks. The building has a square shape of 66x66 meters with an even distribution of orthogonal walls. Skewed walls at El. 0m correspond to the main entrance corridor. The corridor is used to transport the fuel and major equipment under the main hatch which goes up through the containment base mat.

The containment is made of a post-stressed concrete shell with a steel liner. The post tensioning cables are anchored in a stiff ring girder at the junction of the cylinder with the elliptical dome. The wall thickness is 1.2 m for the cylinder and 1.1 m for the dome.

The internal structure is a massive concrete structure supporting the primary components, including the reactor vessel, four horizontal steam generators, reactor coolant pumps, and pressurizer, as well as accumulator tanks and some other auxiliary equipment (Figure 2). The core consists of one thick cylindrical shield wall around the vessel and two groups of pools and cavities containing the spent fuel. There are other shear walls of complex geometry which are interconnected by intermediate floor slabs and partially connected to the core. The upper floor (El. 36.9m) serves as working decks.

The Outer Building houses miscellaneous equipment, including HVAC, electrical and the main steam piping going to the turbine building at El. 28.8m.

Most of the interior walls and slabs in the substructure are made of precast concrete panels serving as formwork for cast-in-place concrete, as explained in later sections. The exterior walls of the substructure and all walls and slabs above El. 13.2m are made of monolithic concrete.

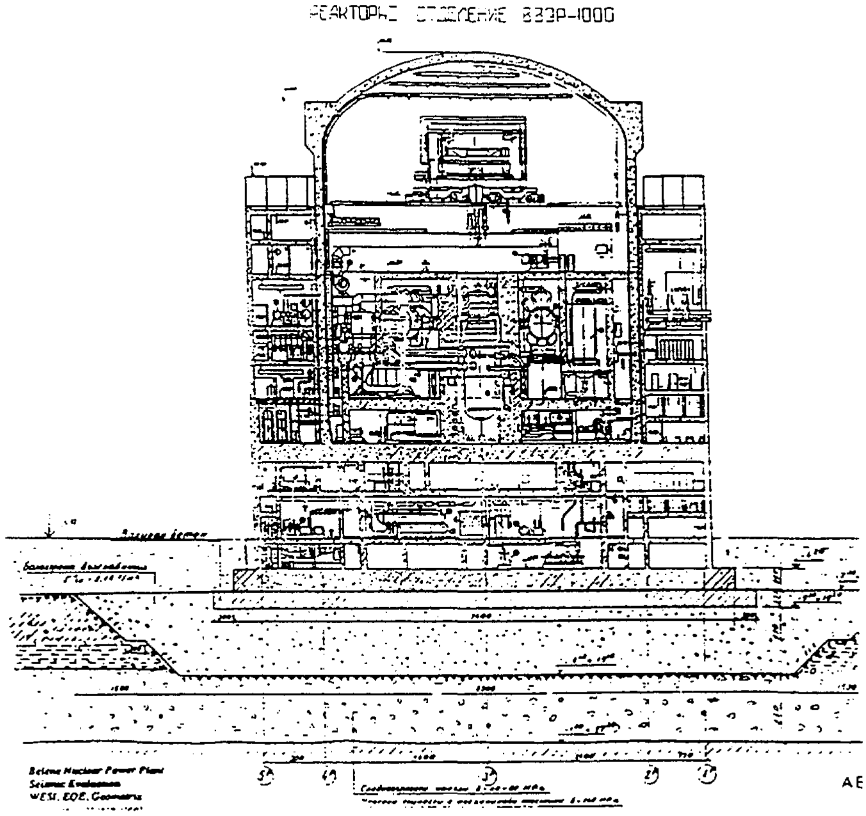
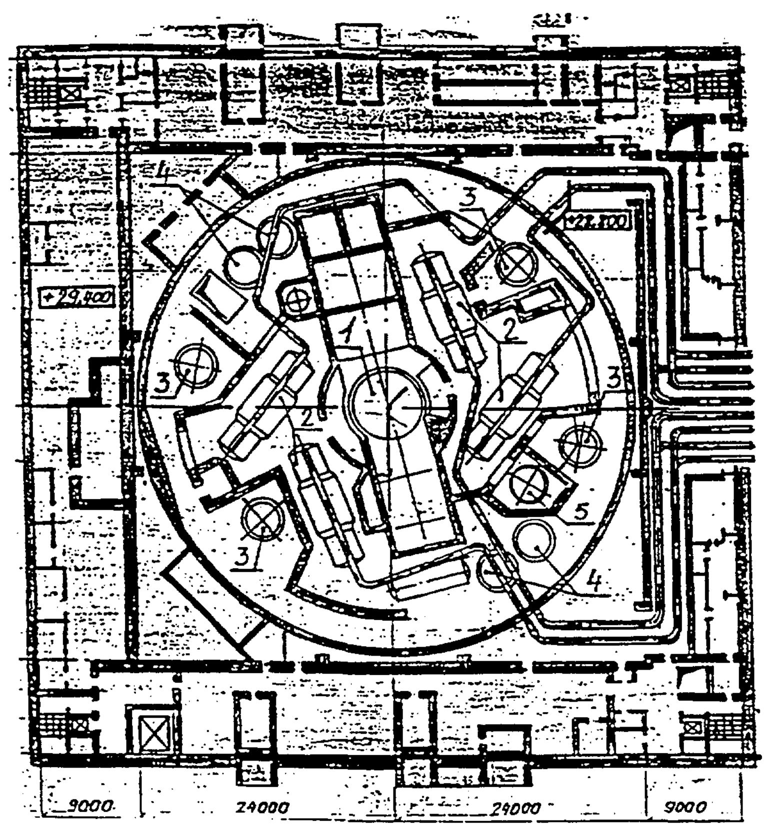


Figure 1: Reactor building cross section



Legend
1. Reactor vessel
2. Steam generators
3. Reactor coolant pumps
4. Accumulator tanks
5. Pressurizer

Figure 2: Plan view of reactor building and primary components

SEISMIC RESPONSE ANALYSIS

Method of Analysis

A substructure method of performing soil-structure interaction (SSI) analysis of structures (Reference 2) was applied to the Belene Nuclear Power Plant main reactor building. Several assumptions apply to the analysis procedure. Most notably, the foundations are assumed to behave rigidly. Full bonding is assumed between the embedded portion of the structure and the foundation with the soil. Strictly speaking, the analysis procedure is linear. Soil is modeled as a series of linear viscoelastic horizontal soil layers. Nonlinear soil material behavior is modeled in an equivalent linear fashion, i.e., equivalent linear soil shear moduli and material damping values for each layer. The remaining constants for the material model are mass density, Poisson's ratio, and water table location. The substructure approach, as it applies here, separates the SSI problem into a series of simpler problems, solves each independently, and superposes the results. The elements of the substructure approach as applied to structures with assumed rigid foundations subjected to earthquake excitations are: specifying the free-field ground motion; defining the soil profile; calculating the foundation input motion; calculating the foundation impedances; modeling the structure; and performing the SSI analysis, i.e., combining the previous steps to calculate the response of the coupled soil-structure system.

Free-field Ground Motion

Specification of the free-field ground motion entails specifying the control point, the frequency characteristics of the control motion (typically, time histories or response spectra), and the spatial variation of the motion. In general, the control point is specified on the free surface of soil or rock. The control motion is acceleration time histories. In most cases, vertically incident plane waves are assumed, which define the spatial variation of motion once the soil properties are identified.

Soil Profile

Defining the soil profile for SSI parameter development first involves defining the low strain soil properties as a function of depth. The important parameters are soil shear modulus, soil material damping, Poisson's ratio, mass density, and water table location – all as a function of depth in the soil. An additional aspect of defining the soil properties is the variation in soil shear modulus and soil material damping with shear strain level, i.e., the reduction in shear modulus and the increase in damping as shear strain increases.

Foundation Input Motion

The foundation input motion differs from the free-field ground motion in all cases, except for surface foundations subjected to vertically incident waves. The motions differ primarily for two reasons. First, the free-field motion varies with soil depth. Second, the soil-foundation interface scatters waves because points on the foundation are constrained to move according to its geometry and stiffness. For vertically propagating seismic waves impinging on rigid surface foundations, the foundation input motion is the same as the free-field motion.

Foundation Impedances

Foundation impedances describe the force-displacement characteristics of the soil. They depend on the soil configuration and material behavior, the frequency of the excitation, and the geometry of the foundation. In general, for a linear elastic or viscoelastic material and a uniform or horizontally stratified soil deposit, each element of the impedance matrix is complex-valued and frequency dependent. For each rigid foundation, the impedance matrix is a 6 by 6 which relates a resultant set of forces and moments to the six rigid-body degrees-of-freedom.

Structure Model

The dynamic characteristics of the structures to be analyzed are described by their fixed-base eigensystem and modal damping factors. Modal damping factors are the viscous damping factors for the fixed-base structure expressed as a fraction of critical damping. The structures' dynamic characteristics are then projected to a point on the foundation at which the total motion of the foundation, including SSI effects, is determined.

SSI Analysis

The final step in the substructure approach is the actual SSI analysis. The results of the previous steps – foundation input motion, foundation impedances, and structure model – are combined to solve the equations of motion for the coupled soil-structure system. For a single rigid foundation, the SSI response computation requires solution of, at most, six simultaneous equations – the response of the foundation. The SSI analyses of the Belene Nuclear Power Plant main reactor building were carried out using the CLASSI system of computer programs (Reference 3).

Elements of the Belene NPP SSI Analysis

Specification of the Free-Field

The seismic design ground motion for the Belene nuclear plant was based on a maximum site intensity of MSK VIII and an equivalent peak ground acceleration of 0.2g. The design evaluation response spectrum was based on analyses of the recordings at Bucharest and Nish from the 1977 Vrancea Earthquake.

As part of the overall effort (Reference 1), the seismic evaluation team reviewed the adequacy of the seismic design motions utilizing readily available information and interpretations from past investigations and applying common practices used to evaluate critical facilities. Both deterministic and probabilistic ground motion evaluations were conducted. From the deterministic evaluation, best estimates for site peak ground acceleration were 0.14g (median) and 0.21g (84th percentile). These estimates are consistent with a maximum MSK intensity of VIII for the site. Intensity VIII appears to be reasonable and appropriate and represents one intensity unit above the largest intensity reported for the site. In current U.S. practice, the 84th percentile ground motion is usually considered appropriate for critical facilities. At the 84th percentile level, the estimated peak ground acceleration for the Belene site exceeds only slightly the value of 0.20g that has been used in design.

The deterministic best estimate for the 84th percentile response spectrum is lower than the design evaluation response spectrum for the plant in the period range of 0.1 to 0.5 seconds (frequency range of 2 to 10 Hz) which is of greatest importance to the seismic response of the plant. At shorter and longer periods, the best-estimate 84th percentile spectrum only slightly exceeds the design evaluation spectrum. Figure 3 compares the design evaluation response spectrum with the deterministic median and 84th percentile spectra.

From the probabilistic analysis, the estimated annual probability of exceeding the design peak ground acceleration of 0.20g is 4.1×10^{-4}. This is within the range of values that have been obtained in probabilistic analyses of many U.S. nuclear plants. The annual probability of exceeding the design evaluation response spectrum in the critical period range of 0.1 to 0.5 seconds is 0.7 to 4.0×10^{-4}.

On the basis of the above evaluations, the seismic evaluation team concluded that the design peak acceleration of 0.20g and the design evaluation response spectrum comprise an adequately conservative ground motion design basis for the plant.

Three artificial acceleration time histories – two horizontal components (E-W and N-S) and a vertical component were developed for the analyses. These time histories were generated such that their response spectra approximate the target. An additional criterion was applied to the three components, i.e., they were verified to be statistically independent with cross-correlation coefficients less than 0.2. Figure 4 plots response spectra comparisons of the specified ground response spectrum and the calculated values from the artificial time histories. The vertical component was specified as one-half of the horizontal.

The control point for the specification of the ground motion is on the free surface at finished grade elevation. The wave propagation mechanism is assumed to be vertically propagating waves. Both of these assumptions are consistent with the current state-of-the practice within the U.S. and consistent with recorded data.

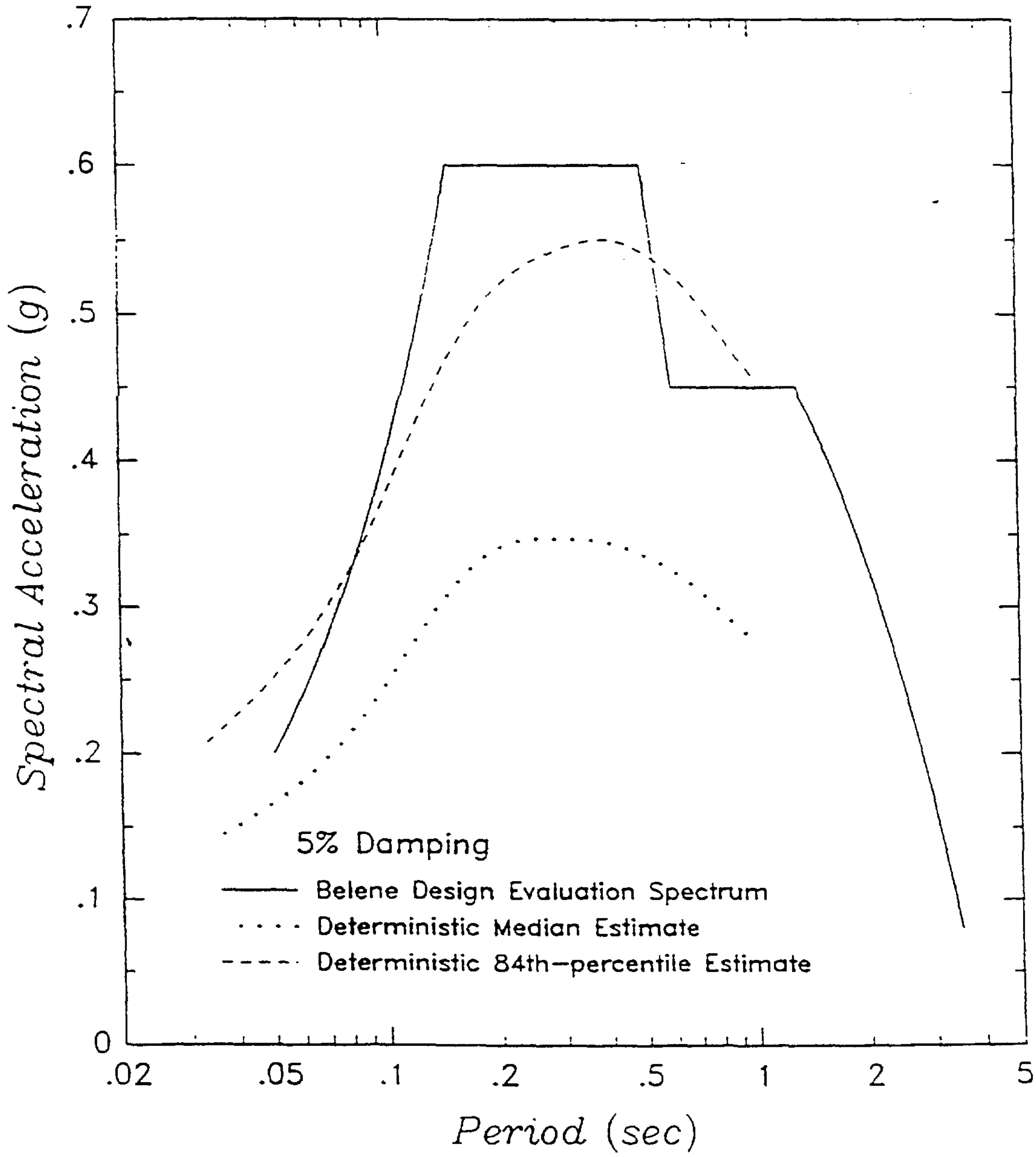


Figure 3: Comparison of the median and 84th percentile deterministic response spectra for the Vrancea source (magnitude 7.5 earthquake at an epicentral distance of 230 km) with Belene NPP design evaluation spectrum.

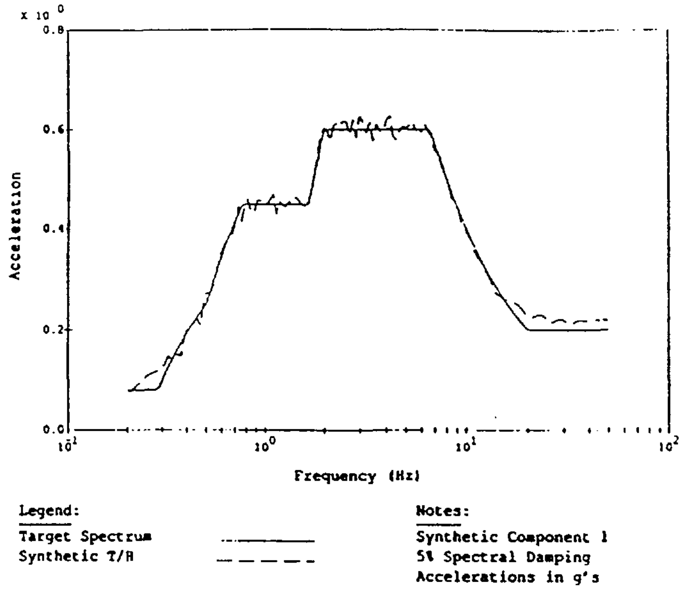


Figure 4a: Comparison of synthetic T/H's response to Belene target spectrum (Component 1)

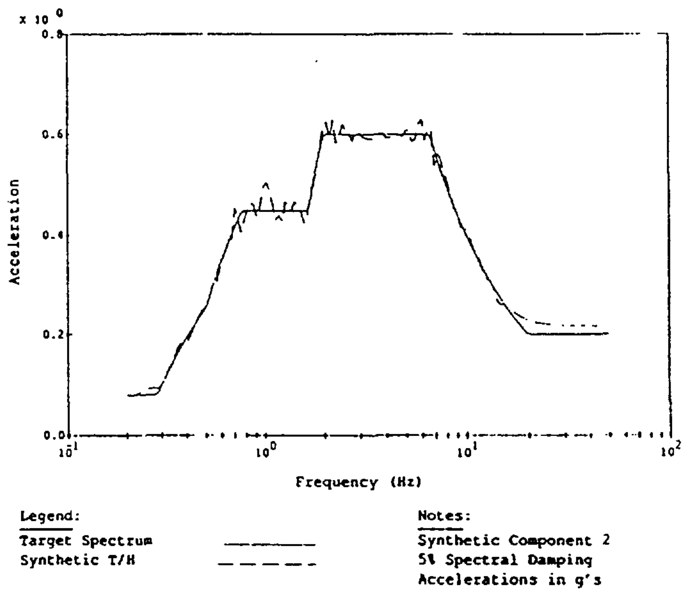


Figure 4b: Comparison of synthetic T/H's response to Belene target spectrum (Component 2)

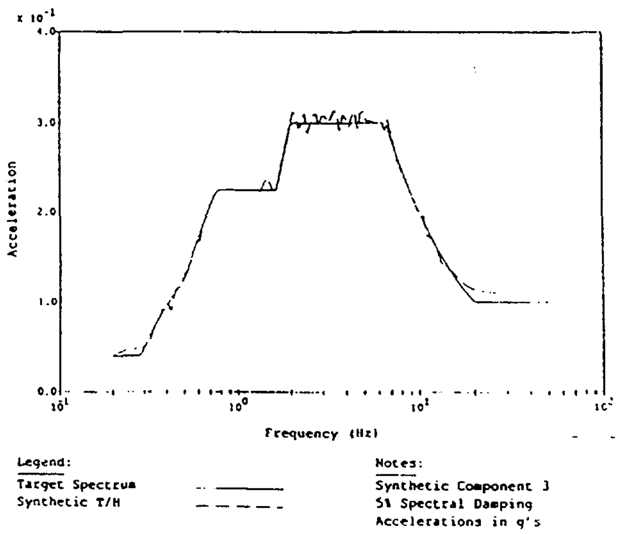


Figure 4c: Comparison of synthetic T/H's response to Belene target spectrum (Component 3)

Soil Profile

The original site conditions at Belene consist of alluvial deposits underlain by marl. The alluvial deposits consist of clayey and sandy soils to depths of approximately 9 to 12 m below the existing grade assumed at El. 0m for this analysis. In the plant area, the clayey and sandy soils were removed and replaced by compacted sandy gravel fill, and the foundation mat is founded directly on this compacted fill which overlies the natural gravel deposit and marl.

Figure 5 depicts a representative soil profile in the vicinity of the reactor buildings. The finished grade is at El. 0m, the bottom of the foundation mat at El. -7m, the bottom of the gravel fill at El. -9m, and the bottom of the natural gravel deposit at El. -27.5m. The water table is reported to be approximately at El. -11m under typical conditions.

The soil material properties to be used in the SSI analysis of the main reactor building at the Belene site need to be compatible with the soil shear strains expected during the design basis earthquake. One starts with the low strain properties in the free-field which were developed based on results of field measurements provided to the project. The stiffness properties of the gravel fill and natural gravel beneath the foundation mat are adjusted upward to account for the increased effective stresses due to the weight of the structure. These properties are then modified as a result of site response analyses effectively reducing their stiffness to account for the state of strain due to the earthquake motion itself. The resulting estimate of the strain compatible soil profile is shown in Figure 6 where shear wave velocity and soil material damping are plotted as a function of depth in the soil.

SSI analyses of the Belene main reactor building were performed for four soil profiles. Three profiles assumed the site to be represented by a uniform half-space with median, and a lower and upper range on soil stiffness. This approach is typical for a smoothly varying profile such as shown in Figure 6. The fourth case modeled the distinct layering as shown in Figure 6. Results from the four cases were enveloped for the structure capacity evaluation. In-structure response spectra at 2% and 5% damping were calculated for the layered system. The 5% damped in-structure response spectra are presented and discussed later. These spectra provide insight to overall soil-foundation-structure behavior. The 2% damped in-structure response spectra were smoothed, peak-broadened, and enveloped for comparison with the specified design floor response spectra. The comparison is also presented later.

The best estimate equivalent half-space properties were taken to be the soil properties at a depth of one-half of an equivalent foundation radius below the bottom of the foundation. For the Belene main reactor building analysis, this approach led to best estimate equivalent half-space properties of V_S = 365 m/sec. Two additional analyses were performed for a stiffer profile (1.5 times stiffer) and for a softer profile (0.67 times softer). These three cases are shown in Figure 6. The fourth case models the best estimate layered profile, again as shown in Figure 6.

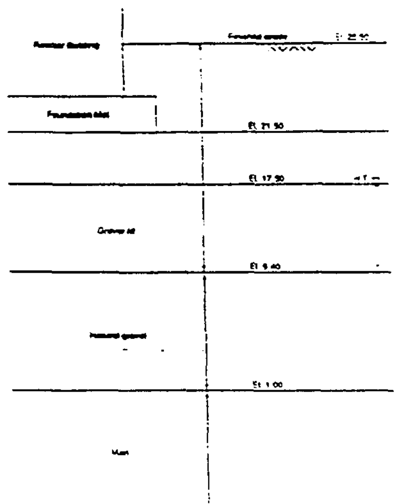


Figure 5: Representative soil profile beneath Reactor Building

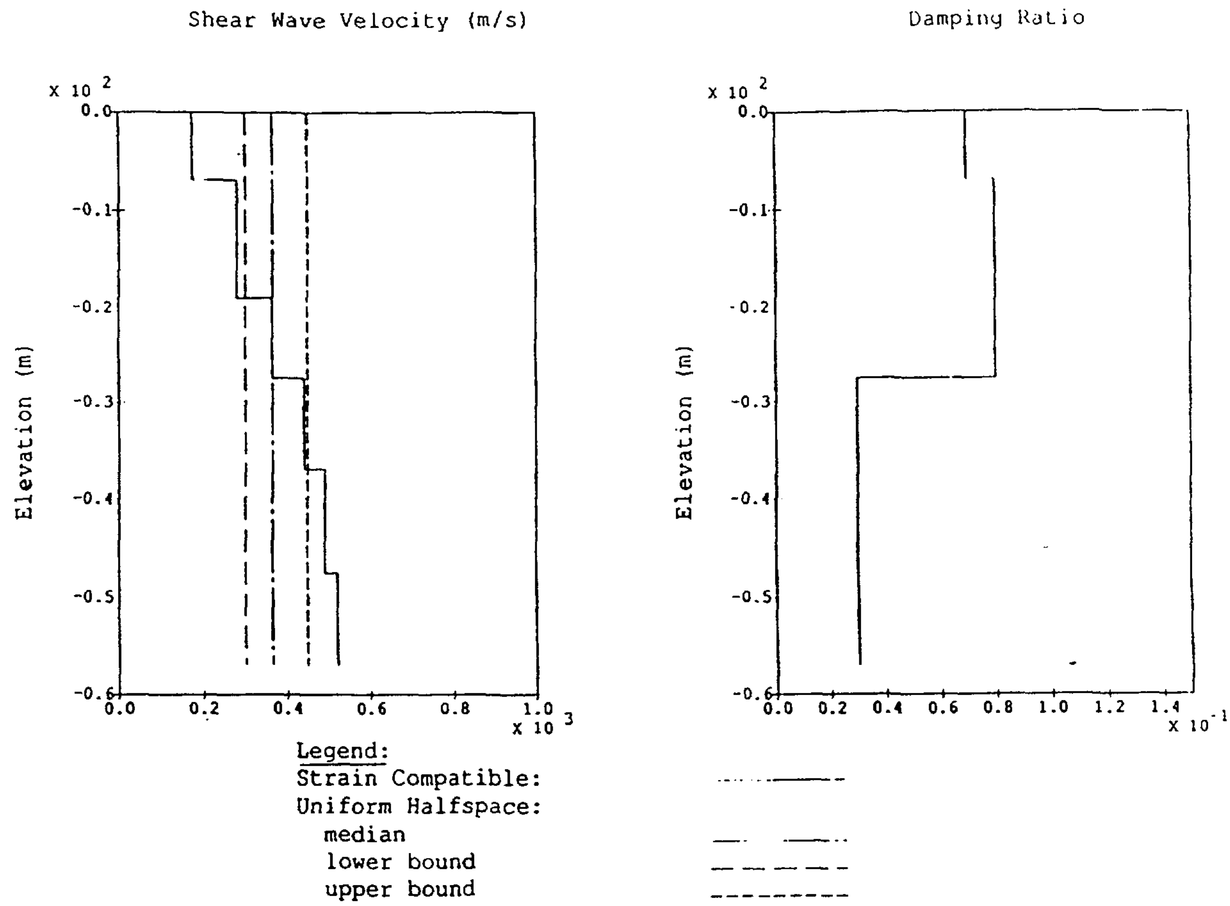


Figure 6: Variation of soil shear wave velocity and damping ratio with depth

SSI Parameters

The SSI parameters here refer to the foundation input motion and foundation impedances that characterize the kinematic and inertial behavior of the foundation-soil system.

Foundation input motion

Foundation input motion denotes the effective motion of a hypothetical massless, rigid foundation. This motion differs from the free-field ground motion for two reasons: the free-field motion varies with depth in the soil and the soil-foundation interface scatters waves as points on the foundation are constrained to move according to the foundation's geometry and stiffness. When the effective stiffness of the foundation is large relative to the soil, rigid behavior can be assumed; the motion of the foundation is uniquely defined by six rigid body degrees-of-freedom--three translations and three rotations. Foundation input motion is related to each component of free-field ground motion through a transformation defined by a complex-valued, frequency dependent scattering vector. This relationship is defined at each frequency by a 6 x 3 scattering matrix relating the six rigid body degrees-of-freedom to the three free-field ground motion components.

For the Belene main reactor building, this phenomenon occurs due to the 7 m of embedment side soil. The free-field ground motion is defined on the free surface of finished grade. Scattering functions were calculated based on this control point location, vertically propagating shear and dilatational waves, soil material properties, and foundation geometry. Separate scattering functions were derived for each soil profile assumption--uniform half-space and layered half-space.

Foundation impedances

Foundation impedance functions define the force-displacement characteristics of the soil-foundation system. For an assumed rigid foundation, the foundation impedance matrix is 6 x 6, complex-valued and frequency-dependent, i.e., at each frequency of interest, a 6 x 6 complex-valued matrix defines the impedances. The impedance functions for the main reactor building foundation, which is 74 m square and embedded 7 m, are derived for the uniform half-space and the layered soil profile.

Impedance functions for surface founded rigid rectangular foundations on a uniform half-space are available from published tables (Reference 4). Impedance functions for a square foundation and soil properties compatible with the Belene site were used. The impedance function tables are presented as nondimensional quantities in both impedance value and frequency. The uniform half-space impedance functions for the different values of shear modulus (median, lower and upper stiffness) are then scaled directly. These surface-founded impedances were corrected for embedment side soil by the procedures of References 5, 6, and 7.

For the fourth case, layered half-space, impedance functions were calculated directly using CLASSI (Reference 3) assuming a surface-foundation and then corrected for embedment. The embedment corrections were applied frequency-by-frequency (References 5, 6, and 7). These foundation impedance correction terms physically represent the increase in stiffness and radiation damping due to the side soil.

Modeling the Structure

An extremely powerful aspect of the substructure approach to SSI analysis (References 2 and 3) is the representation of the structure's dynamic characteristics by its fixed-base modes and modal damping characteristics. The effect of kinematic and inertial soil-structure interaction is, then, included in the solution procedure as the equations of motion are solved at the foundation reference point. Detailed structure response is calculated from this foundation motion.

A brief description of the different portions of the main reactor building--substructure, outer building, containment, and internal structure was presented previously. A detailed discussion of the load-carrying system for each substructure is presented later. The modeling approach and sequence of analyses is described here.

Initially, a separate model was defined for each of the four major portions of the structure, substructure, outer building, containment and internal structure. The final coupled model was formed by joining the structures at El. 13.2m with rigid links between the substructure mass and those of the other portions of the building.

Each substructure was modeled by a lumped mass finite element model. Equivalent beam elements modeled stiffness properties between mass points--these beam elements are denoted "equivalent" because each element represents the group of walls between floor levels for all sub-structures except containment. Containment was discretized into a large number of elements, as shown in Figure 7. The containment stiffness properties were developed assuming beam behavior, also, a common and appropriate assumption. The basic modeling procedure for each sub-structure proceeded as follows. Lumped masses were defined at each floor level. These lumped masses were comprised of tributary contributions of one-half the wall height above and below the floor, the floor slab itself, and equipment at that floor elevation. These lumped masses act at their centers-of-gravity. Modeling stiffness between floor slabs is next. Modeling stiffness between floor slabs with a single beam element assumes floor slabs are rigid and plane sections remain plane. This assumption was critically reviewed during model construction and in the load distribution aspect of the evaluation and was judged to be appropriate throughout the three sub-structures except for the upper two mass points of the internal structure. For these two elements, stiffnesses were derived which compensated for non-rigid behavior, i.e., bending stiffnesses were modified which affected axial load distribution. In fact, this modification had minimal effect on the overall response of the internal structure since it remained governed by shear deformation.

Each of the separate models were completed, reviewed, and checked. Upon satisfactory completion, the four models were combined to a single model by joining them at El. 13.2m. The fixed-base eigensystem for this combined model was extracted and used in the SSI analyses.

Figure 7 shows the combined model schematically. Table 1 lists fixed-base frequencies, modal masses as a percentage of the total, and a notation of global behavior for the first 20 modes. The lowest two modes are at 3.9 Hz in the two horizontal directions and are principally containment modes, each representing about 43.5% of the total main reactor building mass. Modes 3 and 4 are dominated by X and Y motion of the outer building, 33-35% of the total main reactor building mass is participating. Mode 5 is a substructure mode, i.e., with maximum response at the floor slab, El. 13.2m. Modes 7 and 8 are the first two dominant internal structure modes. Note, the first major vertical mode is mode 9, 10.82 Hz. Note, also, the most complicated dynamic behavior is that of the internal structure, as expected, with many modes identifiable in the first twenty.

The twenty-nine modes listed in Table 1 were used in the SSI analyses. Total mass participating in these modes is 96.1%, 95.9%, and 89.2% for the X, Y, and Z directions, respectively. Consequently, these modes well represent the dynamic behavior of the system. In the SSI analyses, modal damping of 7% of critical was assumed for all structure modes.

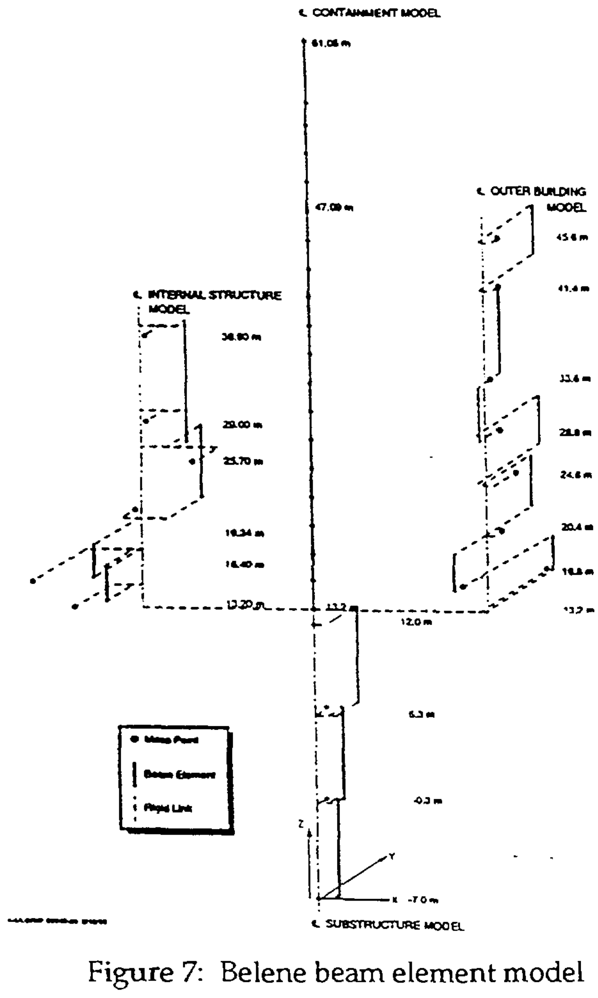


Figure 7: Belene beam element model

Table 1

MAIN REACTOR BUILDING MODAL FREQUENCIES
AND PERCENT MASS PARTICIPATION

Mode No	Freq (Hz)	x	y	z	Description*
1	3.91	43.661	1.422	0.001	CB-x
2	3.91	1.424	43.405	0.003	CB-y
3	5.73	33.081	0.004	0.001	OB-x
4	5.80	0.007	35.260	0.004	OB-y
5	6.30	0.714	0.030	0.000	SS-x
6	8.86	0.090	0.008	0.000	IS-x
7	9.59	6.427	0.012	0.002	IS-x
8	9.62	0.020	4.339	0.068	IS-y
9	10.82	0.013	0.056	53.895	CB-z
10	12.02	0.007	6.190	0.121	IS-y
11	12.19	5.021	0.004	0.052	IS-x
12	12.62	0.255	0.003	0.002	IS-x,y
13	14.66	0.000	0.323	17.632	CB-y
14	14.85	1.374	0.001	0.000	CB-x
15	14.94	0.000	1.145	11.695	CB-y
16	16.98	0.132	0.001	0.001	IS-x
17	17.32	0.000	0.568	0.404	IS-y
18	17.54	0.967	0.001	0.001	IS-x
19	20.50	0.000	0.065	0.007	IS-y
20	21.52	0.827	0.000	0.002	OB-x
21	22.78	0.000	0.739	0.021	
22	23.22	0.135	0.000	0.009	
23	23.51	0.000	1.060	0.024	
24	24.54	0.000	0.005	5.208	
25	26.45	0.795	0.000	0.002	
26	27.28	0.002	0.000	0.000	
27	30.38	0.000	0.001	0.000	
28	32.88	0.006	1.173	0.017	
29	33.68	1.158	0.073	0.000	
Total Pct Mass		96.119	95.887	89.172	

* CB = Containment
IS = Internal Structure
OB = Outer Building
SS = Substructure

SSI Response Analyses

Four SSI analyses were performed.

Case	1	Best estimate uniform half-space Vs = 365 m/sec
	2	Lower range uniform half-space
	3	Upper range uniform half-space
	4	Best estimate layered half-space

Cases 1-3 were analyzed first to provide initial load distributions for the seismic capacity evaluation and to represent a reasonable estimate of the range of dynamic behavior expected. Case 4 was analyzed and compared to the envelope of Cases 1-3. In general, Case 4 most closely matched Case 3 in terms of structure loads. For structure evaluation purposes, the highest of the envelope of Cases 1-3 and Case 4 was considered in the evaluation. Generally, either Case 3 or Case 4 governed.

Figures 8, 9, 10, and 11 present in-structure response spectra throughout the height of each substructure. All figures include the free-field ground response spectra and the foundation response to place the individual spectra in context. In general, the amplification through the structure is relatively little. This is due, principally, to the effects of SSI on the response including embedment effects. The two horizontal directions vary somewhat in response which is likely due to a combination of effects including selected non-prismatic behavior in the outer building structure model and the ground motions acceleration time histories in the two directions. Finally, these figures graphically demonstrate the importance of the ground response spectra in the frequency neighborhood of 2 Hz.

STRUCTURAL CAPACITY EVALUATION

An evaluation of the main reactor building structure was performed to determine if it has sufficient capacity to withstand loads resulting from the design evaluation earthquake. The key features of the evaluation are as follows:

- Evaluation criteria development
- Element load distribution analysis
- Structural element selection
- Structural element capacity evaluation

Development of the evaluation criteria included selection of load combinations, material strengths, allowable stresses and loads, and methods for combination of responses due to the three ground motion components. These criteria were based upon current U.S. Nuclear Regulatory Commission (USNRC) criteria for the design of nuclear power plants that are specified in the Standard Review Plan (SRP) (Reference 8). These criteria are appropriate for new design and are thus very conservative.

The seismic response analysis described previously obtains overall seismic loads acting on each story within the structure. These overall loads were distributed to the individual structural elements (i.e., shear wall or floor slab) by the element load distribution analysis. Different analytical methods appropriate for the different seismic load-resisting systems were used. Similarly, estimates of element loads due to normal operating conditions (i.e., gravity, prestress, etc.) were calculated.

Specific structural elements were selected for detailed evaluation. This sampling included elements from each of the different parts of the structure. The selected elements are judged to be those with the highest applied loads in comparison to their allowable capacities. In general,

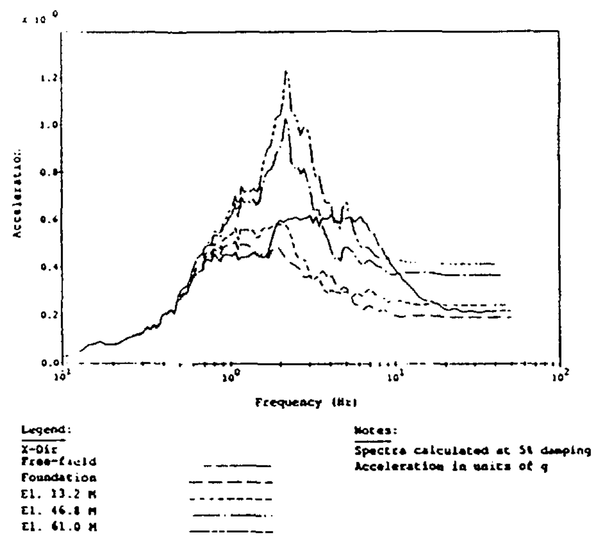


Figure 8a: Belene NPP containment building structural response layered case

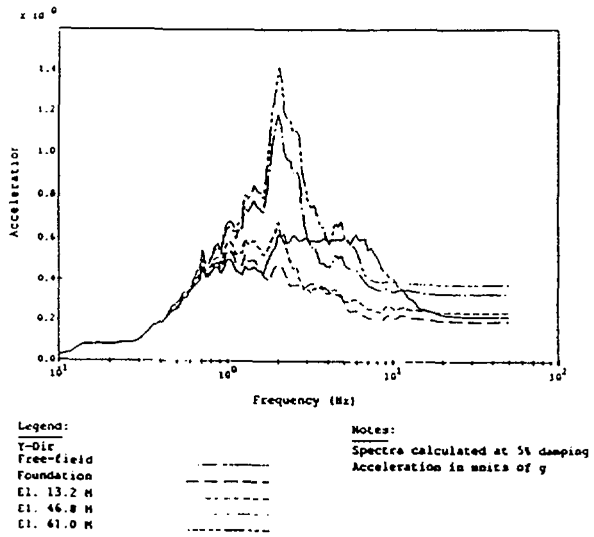


Figure 8b: Belene NPP containment building structural response layered case

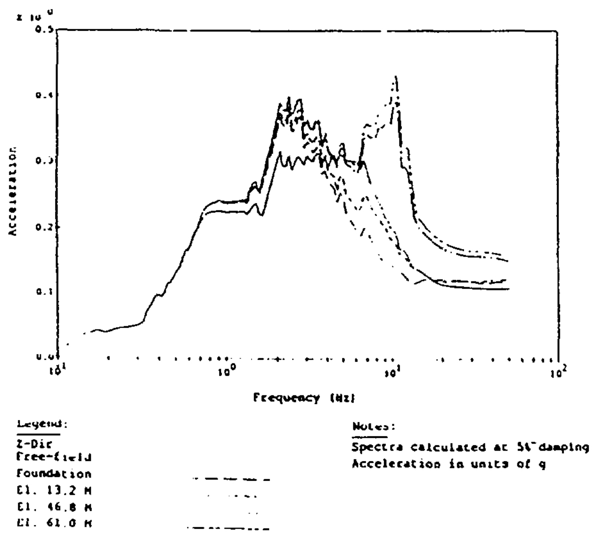


Figure 8c: Belene NPP containment building structural response layered case

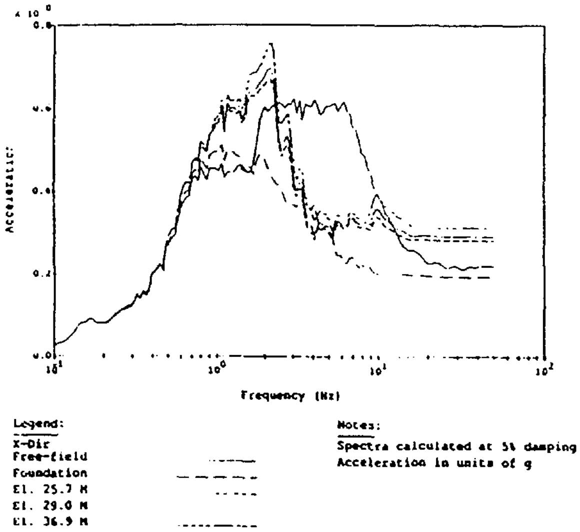


Figure 9a: Belene NPP internal building structural response layered case

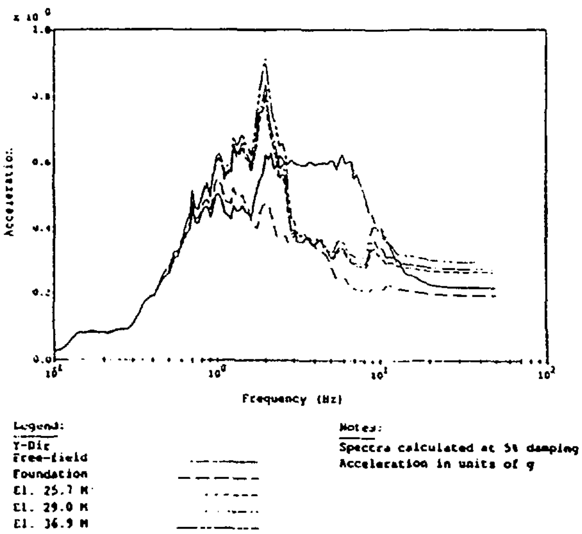


Figure 9b: Belene NPP internal building structural response layered case

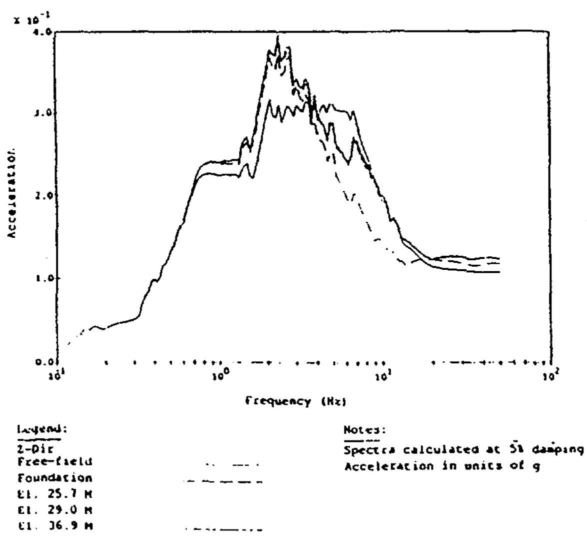


Figure 9c: Belene NPP internal building structural response layered case

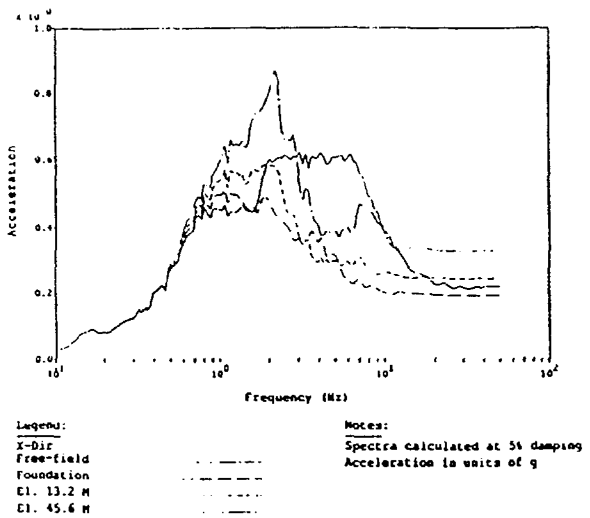


Figure 10a: Belene NPP outer building structural response layered case

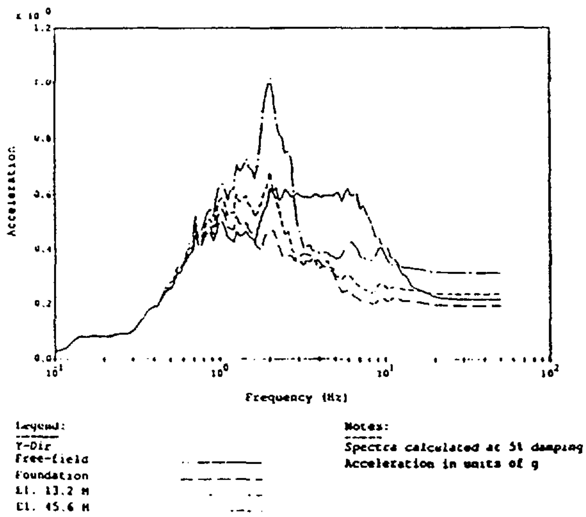


Figure 10b: Belene NPP outer building structural response layered case

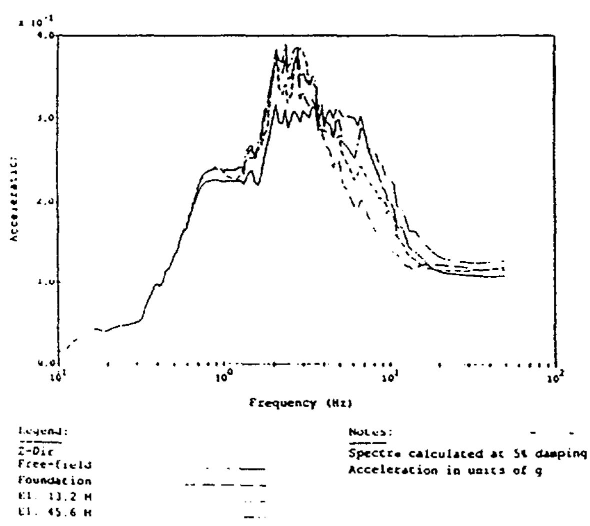


Figure 10c: Belene NPP outer building structural response layered case

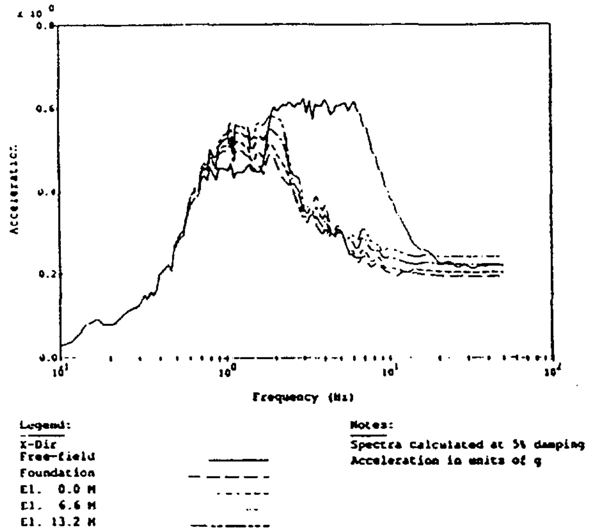


Figure 11a: Belene NPP substructure response layered case

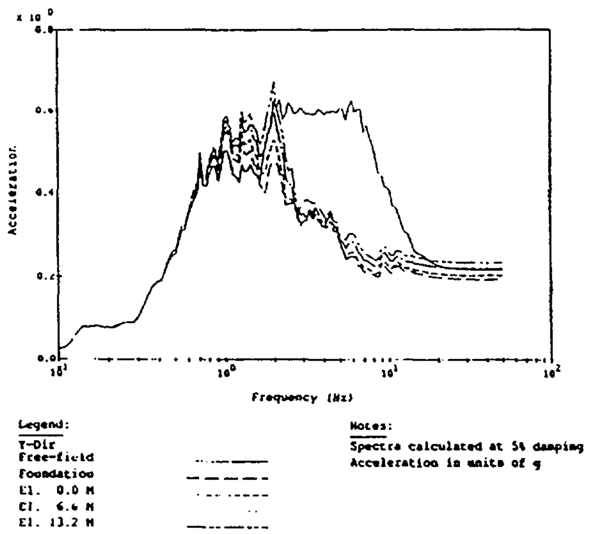


Figure 11b: Belene NPP substructure response layered case

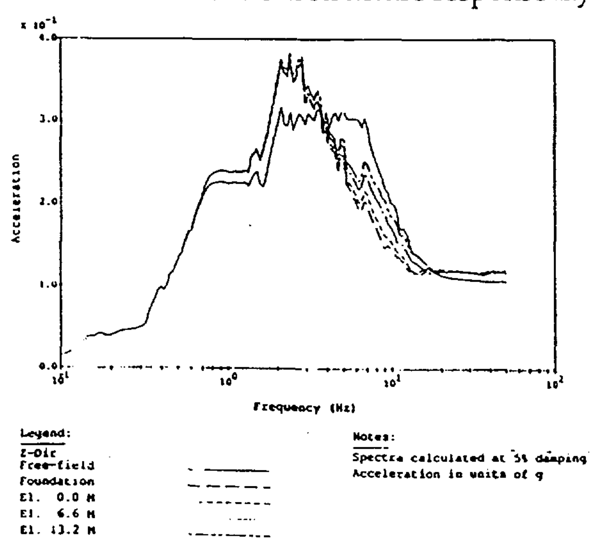


Figure 11c: Belene NPP substructure response layered case

these elements tend to be the heavier shear walls at the lower portions of the structures where lateral loads are the greatest, floor slabs at higher elevations, and sections of the containment are of particular interest.

The element selection was based only on a brief review of limited available drawings. Drawings showing complete structural details and detailed element load distributions were unavailable. The selection of critical elements relied significantly on engineering judgement and past experience in similar evaluations. While it may be possible that not all of the most critical elements were selected, the element sampling is sufficient to establish the general adequacy of the structure as a whole.

In the structural element capacity evaluation, allowable stresses/loads for the selected elements were compared to the applied stresses/loads obtained by the load distribution analyses. The final results are expressed in terms of factors of safety, FS. The factor of safety is defined as the ratio of the allowable stress/load to the applied stress/load. A factor of safety of unity or greater signifies that the structural element meets the acceptance criteria.

Load Combinations

SRP criteria specify that safety-related nuclear plant structures be designed to withstand various combinations of individual load cases. Structure loads due to some of these load cases were not available. Examples include loads on the containment structure due to internal pressure and temperature associated with a loss-of-coolant-accident (LOCA). Analysis to determine the structure loads due to these load cases was not included in the scope of work.

The following load cases were considered in the evaluation:

D - Dead load
L - Live load
F - Prestress load
E' - Loads due to the safe shutdown earthquake (SSE)

Loads due to the Belene design evaluation earthquake were used for load case E'.

The following SRP load combinations include the load cases listed above and were used in the structural evaluation:

Containment structure

$D + L + F + E'$

Substructure, outer building, and internal structure

$D + L + E'$
$0.9D + E'$

Acceptance Criteria

Acceptance criteria for the structure capacity evaluation were based on U.S. NRC SRP criteria. These acceptance criteria are in the form of allowable stresses or loads.

Material strengths were converted from Bulgarian or European conventions to U.S. conventions where necessary--concrete compressive strengths and yield and ultimate strength of reinforcement and structural steel.

For the concrete containment, the SRP requires applied stresses to be less than allowable values specified in the ASME Section III, Division 2 Code (Reference 9), with a few exceptions.

Allowable loads for containment internal structures and other seismic Category I structures are specified in SRP Sections 3.8.3 and 3.8.4. For concrete structures, applied loads must be less than allowable values determined in accordance with the strength design methods of the ACI 349-85 Code (Reference 10). Thus, allowable loads for concrete structural elements of the internal structure, substructure, and outer building were determined using the ACI 349-85 provisions. The most significant forces on these structural elements are in-plane shears and moments.

Seismic Load Distributions

Overall seismic loads acting on the structure were determined from the seismic response analysis. For lumped mass structures whose stiffnesses are represented by vertical beam elements, these loads consist of axial forces, horizontal shears, torsional moments, and overturning moments. To conduct the structural evaluation, it is necessary to distribute these overall structure seismic loads to the individual structural elements (i.e., shear walls, floor slabs) that compose the load-resisting system. The seismic load distribution methods employed were selected as being appropriate for the different load-resisting systems encountered.

Overall structure seismic loads were obtained from the seismic response analyses for the best estimate, upper bound, and lower bound effective soil stiffness cases. The upper bound effective soil stiffness case essentially provided enveloping structure seismic loads. The structural element seismic load distributions were thus calculated for this case.

The containment structure wall is essentially a uniform cylindrical shell. Past experience in seismic analysis of other containments has shown that overall seismic response is typically dominated by the fundamental beam bending mode. The seismic stress distributions for this mode of response are adequately calculated by elementary elastic beam theory assuming that plane sections remain plane. For the containment wall, the horizontal seismic responses result primarily in tangential shear and meridional membrane stresses. These stresses were distributed around the containment wall according to the first harmonic. Axial (vertical) loads result in uniformly distributed wall meridional membrane stress. Containment stresses due to higher modes are typically much smaller and unlikely to result in structural failure.

The seismic load-resisting walls of the internal structure are typically interconnected in the lower part of the structure where global seismic loads are the greatest. Under these conditions the walls are considered to be constrained to have the same global deformations. Accordingly, vertical stresses due to vertical load and overturning moment were also determined by elementary beam theory using the cross-sectional properties assigned to the various elements of the dynamic model. Overall horizontal shears from the response analysis were distributed to the individual walls in proportion to their contributions to the overall dynamic model shear stiffness.

The lateral load-resisting system for the substructure and outer building consist of concrete shear walls interconnected by concrete floor slabs. The floor slabs act as diaphragms distributing horizontal seismic inertial loads to the shear walls, which in turn transmit these loads down through the structure to the foundation. If the diaphragm is relatively stiff, the shear walls of a story are constrained to displace laterally as a unit, and the horizontal loads can be distributed to the walls in proportion to their relative rigidities.

The seismic response analysis calculates overall shears and torsional moments at each story of the structure. These overall structure shear loads and torsional moments cause shear loads on the individual wall elements. The overall structure loads were distributed to the individual walls in proportion to their relative rigidities. Overall structure overturning moments are calculated at the nodes of the structure dynamic model. The incremental change in overturning moment in a story (i.e., the difference in overturning moment between the top and bottom floors of the story) was distributed to the individual walls in the same proportion as the distribution of horizontal shears to the walls. In effect, the overturning moment increments were also distributed to the walls in proportion to their relative rigidities. The total wall overturning moment at a particular elevation was taken as the sum of the incremental moment changes for the stories from the top of the wall to that elevation.

Wall element story stiffness estimates are required to determine wall relative rigidities. In-plane wall stiffnesses considered both shear and local flexural deformations, assuming a condition of rotational fixity at the floors above and below the wall. Out-of-plane wall stiffnesses were neglected, resulting in conservative in-plane load distributions.

Containment Structure Capacity Evaluation

The containment structure consists of a cylindrical wall with an elliptical cap dome. It is founded on the 2.4 m thick base mat at El. 13.2m. The cylindrical wall is constructed of prestressed concrete with a total thickness of 1.2m and outside radius of 23.7 m. It is prestressed by a total of 96 post-tensioned tendons arranged in a helical pattern at an angle of about 35 degrees from horizontal. All of the wall tendons are anchored at a ring girder at the top of the wall. The dome is also constructed of prestressed concrete with a total thickness of 1.1 m. Additional steel reinforcement is provided in both the wall and the dome. The containment is lined at its inside face by 8 mm thick steel plate.

The following elevations of the containment wall were selected for capacity evaluation:

- El. 13.2m - Base of the wall
- El. 15.57m - Transition to 1.2m wall thickness
- El. 21.92m - Reduction in the reinforcement provided
- El. 46.8m - Polar crane support
- El. 51.6m - Transition to the ring girder

The containment wall was typically evaluated for meridional membrane stress and membrane direct tension stress.

A net meridional tension due to combined gravity, prestress, and seismic loads typically does not occur. Thus, the factors of safety are typically controlled by concrete stress. Reinforcement stress factors of safety are much higher. The lowest factor of safety of 1.59 was obtained at El. 46.8m. This results from the very conservative estimates of loads imposed by the polar crane. The actual factor of safety is probably higher.

The direct tension stress was evaluated in terms of the applied and allowable tangential shear force due to earthquake, which is the primary source of seismic-induced direct tension. The lowest factor of safety of 3.01 was obtained at El. 15.57m.

As shown by capacity evaluation of the containment structure, allowable stresses/loads are well in excess of applied stresses/loads. This is consistent with past experience in the evaluation of containment structures. The design of such structures is typically controlled by the pressure and temperature associated with a loss-of-coolant-accident. Seismic loads are relatively small in comparison to LOCA loads. Thus, acceptable performance of the containment under earthquake loads in combination with loads occurring at normal operating conditions would be expected.

Internal Structure

The internal structure is a concrete structure consisting of a primary shield, outer shield walls, and spent fuel pool. There are numerous walls in the internal structure which are interconnected by concrete floor slabs at El. 36.9m, 29.0m, 25.7m, and 19.34m. The entire internal structure is supported on the 2.4m thick concrete slab at El. 13.2m. The internal structure walls are encased in steel modules constructed of 6 mm to 8 mm thick steel plates which are internally braced to serve as forms. The concrete walls are typically unreinforced. The steel plates function as wall reinforcement. Composite action between the concrete and steel plates are provided by 12mm diameter shear studs which are welded to vertical steel angles which in turn are welded to the inside faces of the steel plates. Additional composite action is developed by other internal steel members.

The wall selected for evaluation is the outer wall at Line I between El. 13.2m and El. 19.34m. Applied loads considered in the evaluation included the following:

- Axial load due to the global seismic moment and vertical seismic response
- In-plane shear
- Local in-plane moment due to the in-plane shear

The following structural components along the wall load path were evaluated:

- Shear transfer between the concrete slab at El. 19.34m and the top of the selected wall
- Shear capacity of the wall
- Shear transfer at the base of the wall
- Combined axial load and local moment

The lowest factor of safety of the code capacity over the applied load is 1.53 corresponding to shear strength of the wall. This is a conservative value as the strength of the steel plates was not considered. The next lowest factor of safety is 1.61 for shear transfer at the wall base. It is concluded that the internal structure wall has adequate seismic capacity against the design evaluation earthquake.

Substructure Capacities

The substructure constitutes the part of the building between the base mat at El. (-)4.2m and the containment base mat at El. 13.2m. It provides support for the containment, internal structure, and outer building. The substructure is constructed of reinforced concrete. Resistance against seismic-induced lateral loads is provided primarily by shear walls. The floors typically consist of precast concrete panels covered with cast-in-place concrete to a total depth of 0.6 m.

The interior walls of the substructure are typically 0.6 m thick. These walls are constructed of a combination of precast and cast-in-place concrete. Precast panels bolted to intermediate steel trusses are used as forms. Column reinforcement are placed in the wall at the precast panel joints which are typically spaced at 3 m. Horizontal beam reinforcement is also placed at the bottom of the wall just above the joint with the floor slab. The space between the vertical and horizontal reinforcement is typically left unreinforced. After placement of the panels and reinforcement, cast-in-place concrete is poured between the panels. The precast panels, which form the wall faces, are left in place.

Removeable forms were in the construction of the exterior walls. In addition, uniformly distributed reinforcement was placed at both faces of the wall.

The shear walls selected for capacity evaluation are all located at the lowest story of the substructure between El. (-)4.2m and El. 0m where lateral seismic loads are the greatest. The selected walls are expected to be the stiffest and thus be subjected to larger seismic loads than other walls at this story.

The precast panels are lightly anchored to the cast-in-place concrete. The ability of these anchorages to develop composite action between the precast panels and cast-in-place concrete in resisting seismic loads is unknown. The substructure interior wall capacity was evaluated using the following two assumptions:

- Precast panels assumed to be ineffective in resisting loads on the wall
- Precast panels assumed to be 40% effective in resisting shear loads on the wall

The former represents a lower bound on interior wall load capacity. The latter represents an approximate upper bound on the realistic load capacity.

Different allowable shear capacities corresponding to the two assumptions on precast panel effectiveness were calculated for the interior wall. For the 0% effectiveness case, the potential strength contribution from the panels was not included. For the 40% panel effectiveness case, 40% of the allowable shear strength for the panels was included.

For the 0% panel effectiveness case, the lowest factor of safety of 1.01 was calculated. For the 40% panel effectiveness case, the lowest factor of safety of 1.27 was calculated. These comparisons illustrate the sensitivity of the final results to panel effectiveness.

The precast panel reinforcement is considered to be ineffective in resisting in-plane wall moment. This is because the bars are discontinuous at the panel base, which is the critical section for in-plane moment.

Shear wall factors of safety based on moment capacity were calculated. For the 0% panel effectiveness case, the lowest factor of safety of 1.49 was obtained. For the 40% panel effectiveness case, the lowest factor of safety of 1.57 was obtained.

To summarize, the lowest factor of safety for the substructure shear walls is 1.01 for the 0% panel effectiveness case, and 1.27 for the 40% panel effectiveness case. Therefore, the actual effectiveness should be studied in more detail, starting with the search of any test data or real world experience data on actual resistance of similar concrete constructions during past earthquakes.

Outer Building

The outer building constitutes the part of the structure above El. 13.2m that encloses the containment. It is isolated from the containment by separation gaps reportedly 110 mm wide. Similar to the substructure, the outer building is constructed of reinforced concrete with resistance against seismic-induced lateral loads provided by shear walls.

The original construction drawings for the outer building were based upon the use of precast wall and slab panels similar to those employed for most of the substructure. However, this construction concept has been revised for the outer building. The walls are constructed monolithically with removable forms. Instead of concentrated column reinforcement, they are reinforced by uniformly spaced horizontal and vertical bars. The floor slabs are constructed of steel decking covered with concrete to a total thickness of 0.6 m. The slab concrete contains uniformly spaced reinforcement bars.

The shear walls selected for capacity evaluation are located at the lowest story of the outer building between El. 13.2m and El. 16.8m where lateral seismic loads are the greatest. The selected walls are expected to be the stiffest and thus be subjected to larger seismic loads than the interior walls.

The shear wall shear capacity evaluation lowest factor of safety of 1.97 was calculated. The lowest factor of safety of 2.47 was obtained in the shear wall moment capacity evaluation.

Portions of the floor slabs at El. 41.4m and El. 20.4m were evaluated to investigate their ability to transmit their own inertia back to the supporting shear walls. The floor slab evaluation consisted of the following two parts:

- Local response analysis
- Capacity evaluation

In the local response analysis, the floor slabs were modeled as horizontal beams vibrating under their own inertia. The slab horizontal frequencies were determined by closed form solutions. Applied seismic loads were determined by factoring the slab and wall mass by the seven percent damped spectral acceleration obtained from the broadened floor spectra. In-plane shears and moments were then calculated by closed-form solutions.

In-plane floor slab shear and moment capacities were determined following the same provisions for shear walls.

The results of the floor slab capacity evaluations are as follows. For shear, the lowest factor of safety of 6.3 was obtained for the slab at El. 41.4m. For moment, the lowest factor of safety of 2.18 was obtained for the slab at El. 41.4m. Based on these values, it is concluded that the floor slabs of the outer building have adequate seismic capacity against the design evaluation earthquake.

EVALUATION OF EQUIPMENT AND COMMODITIES

The VVER 1000 reactors being constructed at Belene are a standard design for which the equipment and commodities (piping, cable trays conduit, ducting) are designed for a seismic input defined as envelope response spectra covering 4 elevation zones of the main reactor building (Table 2). Site specific floor response spectra have been developed for the Belene site design evaluation ground motion spectrum. Representative results were shown in Figures 8 to 11 for 5% damping. Additional in-structure response spectra for 2% damping were generated for comparison with the envelope design spectra shown in Figure 12. These floor spectra, when compared to the design spectra show some exceedances in certain frequency ranges. Selected comparisons are shown in Figures 13 to 16. The effect of these exceedances on equipment and commodities must be assessed. The method of assessment depends upon the dynamic characteristics of the component or commodity being considered, the frequency range of exceedance, the magnitude of exceedance and the status of the design or construction.

Table 2

APPLICABLE DESIGN SPECTRA			
		Spectra	
Structure	Elev. Range	Horizontal	Vertical
Containment	Above 35.8M	1	6
Containment	24.5M-35.8M	2	6
Containment	13.2M-24.5M	3	6
Int. Structure	25.7M-36.6M	2	6
Int. Structure	13.2M-24.5M	3	6
Out. Structure	Above 41.4M	1	6
Out. Structure	28.8M-41.4M	2	6
Out. Structure	13.2M-28.8M	3	6
Substructure	-4.2M-13.2M	4	5

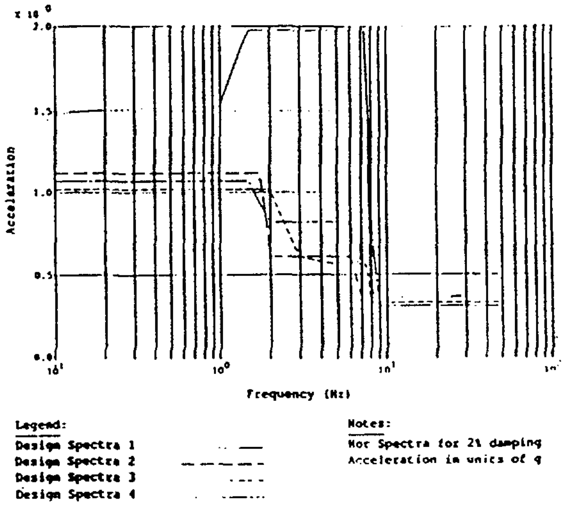


Figure 12a: Horizontal design spectra

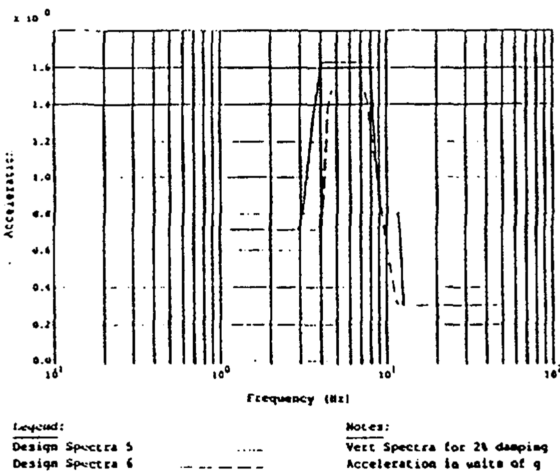


Figure 12b: Vertical design spectra

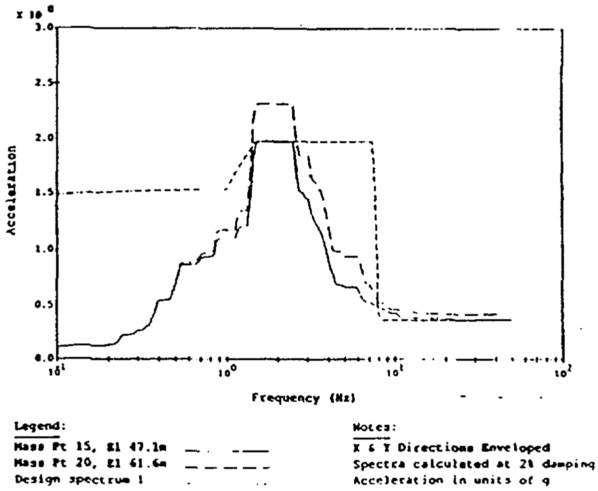


Figure 13: Containment building comparison of horizontal response to design spectrum 1

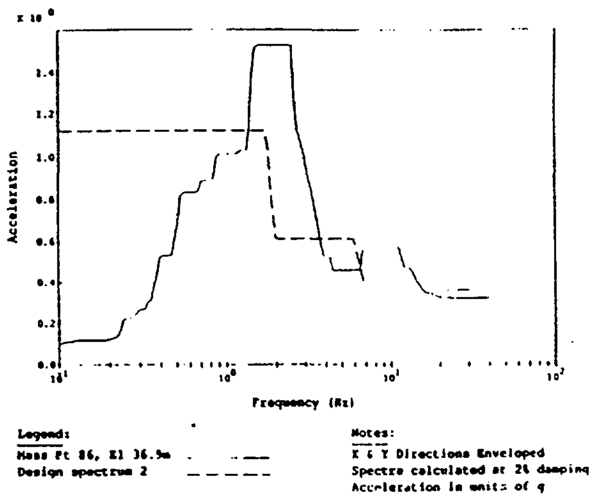


Figure 14: Internal structure comparison of horizonal response to design spectrum 2

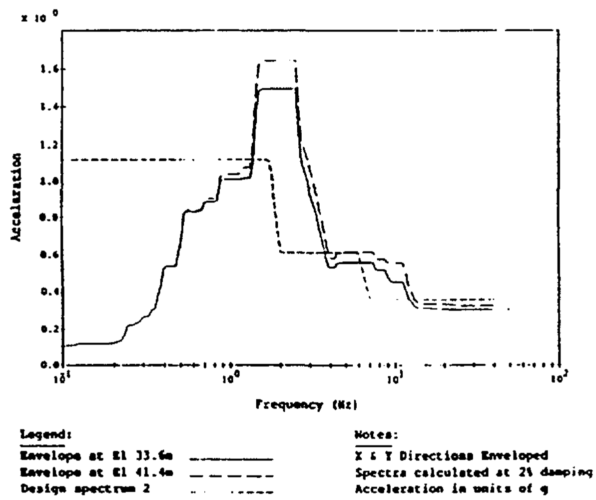


Figure 15: Outer building comparison of horizontal response to design spectrum 2

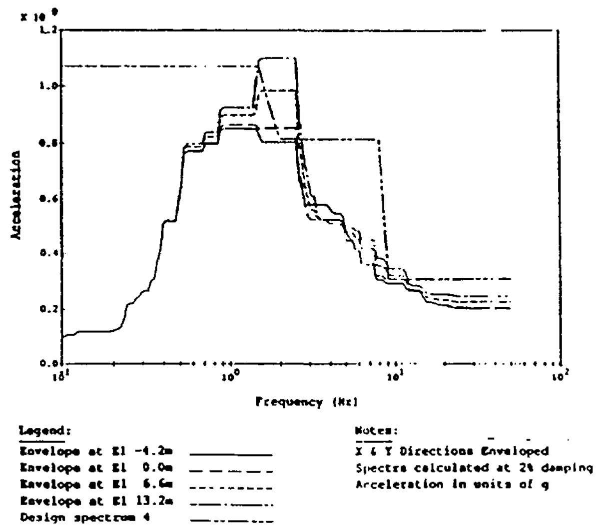


Figure 16: Substructure building comparison of horizontal response to design spectrum 4

While these exceedances are shown in some cases to be greater than 100%, the greatest exceedances are in a relatively low frequency range (2 to 3 Hz) whereas most equipment will be much stiffer and will not be affected. Also, the peak spectral acceleration values, defined at 2% damping, are not excessive compared to design spectra utilized in the United States. The most significant impact will be on equipment for which this design, fabrication, and qualification has been completed. In almost all cases, the requalification by analysis or test will not result in hardware changes, only the generation of more documentation.

The most labor intensive issues will be for flexible commodities. Although these commodities have been demonstrated to be inherently rugged, the analytical reevaluation process will be voluminous and may result in changes in support configurations.

It would be most cost effective to conduct the reevaluation of these commodities using more empirical seismic experience based criteria. In this manner, the inherent seismic ruggedness of the commodities and many classes of equipment can be utilized to minimize the impact of newly defined seismic qualification loadings. These seismic experience based methods as described in References 11 to 14 and are being utilized in the United States for demonstrating the functionality of equipment and commodities which were not qualified to current standards or to demonstrate the seismic margin of equipment and commodities for seismic events greater than the original design basis. It is recognized that it must ultimately be demonstrated to the satisfaction of the regulators that the seismic experience based methods are applicable to non-U.S. equipment. For most equipment though, it is believed that this is achievable without further extensive seismic experience data gathering.

CONCLUSIONS

The critical aspects of predicting seismic response of the main reactor building soil-structure system are two-fold. First, the ground response spectra in the neighborhood of 2 Hz are extremely important to structure response prediction. Since, this is the frequency range of transition from an amplification factor of 2.25 to 3.0 on peak ground acceleration, increases in the lower frequency amplification will impact calculated structure loads and in-structure response spectra. Second, soil stiffness plays a major role in response prediction. As evidenced by the comparison of structure loads for the four soil cases, increases in soil stiffness lead to increases in loads. These increases are due to overall soil-structure system frequencies being in a more highly amplified portion of the ground response spectra (i.e., greater than 2 Hz) and due to changes in overall dynamic behavior of the system as the soil stiffens. Any changes in these two aspects of the evaluation will require a re-assessment of dynamic response of the soil-structure system.

For this VVER 1000 configuration on stiff soil or rock, higher seismic response will occur for the same ground motion. Hence, the conclusions for this study will change unless design changes have been implemented.

If the design evaluation earthquake level increases or site conditions change, i.e., are stiffer, the seismic capacity of the structure may not satisfy the conservative design acceptance criteria adopted for this evaluation. Different options are available to verify if the structure has sufficient capacity to maintain its integrity under a larger earthquake event, including the following:

- Testing of the walls with precast panels
- Nonlinear finite element analysis of the walls with precast panels
- Reevaluation using alternative acceptance criteria

Laboratory testing to conclusively quantify the effectiveness of the precast panels in resisting seismic loads acting on the interior substructure walls can be performed, if such data are not already available. The test program should be developed so that the actual panel and wall construction, and the actual loads due to a seismic event, are accurately simulated.

As an alternative to testing, precast panel effectiveness may also be demonstrated by detailed finite element analysis. Such an analysis would attempt to simulate the actual seismic loading, panel and wall construction, panel connectivity, and potential nonlinearities. The seismic evaluation team has the necessary computer software and previous experience to perform this type of analysis.

The acceptance criteria used in the structural capacity evaluation are appropriate for new design of nuclear power plants, and are therefore conservative. The seismic evaluation team has data demonstrating that these criteria are very conservative for the monolithic shear walls. The team has used alternative acceptance criteria for reevaluation of nuclear plant structures constructed in the monolithic manner. These alternative criteria are still conservative, but are also more realistic than the SRP acceptance criteria. They would be expected to result in higher factors of safety for the monolithic shear walls.

Finally, if the test and analytical methods described above still do not conclusively demonstrate that the structure is seismically adequate, retrofit of the existing structure and design changes to the parts of the structure still to be constructed may be considered. For example, retrofits to the existing substructure interior walls might consist of providing additional anchorage between the precast panels and the cast-in-place concrete, and providing continuity for the panel reinforcement at the horizontal and vertical panel edges. The seismic evaluation team can provide guidance on conceptual retrofits and design changes, based on our past experience in these areas.

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