2.5 GEOLOGY AND SEISMOLOGY

2.5.1 General

The regional geologic features in the Browns Ferry site area and the local geologic formations in the immediate plant area have been investigated. The results of extensive drilling, excavation, and testing are presented in this subsection. These results show that the underlying bedrock will provide more than adequate foundation for Browns Ferry plant structures.

The seismicity of the site area has also been studied. Data from many earthquakes have been used to compile the seismic history of the area and to evaluate the earthquake hazard at the site.

2.5.2 Geology

2.5.2.1 Introduction

The Browns Ferry area was explored first in 1962 as one of several possible sites in the vicinity for a fossil fuel plant. The area is underlain by flat-lying, undeformed limestone of Mississippian age. Exploration indicated and subsequent excavation proved that no significant geologic problems would be encountered in developing satisfactory foundations for the major structures.

2.5.2.2 Geological Investigative Program

2.5.2.2.1 Site

Initial geologic investigations were made at the Browns Ferry site from January to May, 1962, and again during February and March, 1963. During these periods 80 holes totaling 5658.6 linear feet were drilled on 200-foot spacing along lettered ranges roughly parallel to the shoreline of Wheeler Lake and numbered sections at right angles to the ranges as shown in Figure 2.5-1. At this time the site was under consideration for a fossil fuel plant. For this reason drilling depth was limited to 10 feet of sound, unweathered rock and, therefore, the majority of the holes penetrated less than 20 feet of rock. The geologic conditions revealed by this drilling are shown on Figures 2.5-2 and 2.5-3.

2.5.2.2.2 Plant Foundations

With the decision in 1966 to utilize the site for a nuclear plant, additional exploration was done. As shown on Figure 2.5-4, 29 additional core drill holes and 95 percussion holes were drilled in the main plant area to provide additional geologic information. Graphic logs of the additional core drill holes are shown on

Figure 2.5-5. A summary of geological investigative programs from 1972 to 1980 is presented on Figure 2.5-5a.

2.5.2.2.3 Access Highway Bridge

In April of 1972 foundation investigations were conducted for the construction of an access highway bridge over a relocated channel. Ten Nx-wireline core holes were drilled and visually logged. The drill layout is presented on Figure 2.5-5b and the logs are on Figures 2.5-5c through 2.5-5l.

2.5.2.2.4 Deleted

2.5.2.2.5 Low-Level Radwaste Storage Facility

During the period from January to April 1980, TVA conducted foundation investigations at the site of the proposed Low Level Radwaste Facility (LLRW) (Figures 2.5-S1 through 2.5-S4). Core borings and various geophysical methods were used to determine the depth to rock, the configuration of the top of rock, and to locate and zone deficiencies in order to assess the ability of the bedrock to support the overlying soils and thus the LLRW facility. Three vertical Nx-wireline core holes were drilled through the near-horizontally bedded limestones of the Mississippian aged Fort Payne Formation into the undifferentiated shales, sandstones, and limestones below (Figure 2.5-S5). The cores from these holes were logged visually, and the holes were logged geophysically. The bedrock was found to lie an average of 50 feet below the ground surface and to have an undulating to slightly undulating top-of-rock surface containing narrow near-vertical solution features. Subsurface geophysical investigations that were carried out in 102 percussion holes drilled 50 feet into the rock, revealed no buried sinkholes or large near-surface cavities which could potentially collapse and cause settlement.

2.5.2.3 Regional Geology

2.5.2.3.1 Geological History

The Browns Ferry area lies on the southeastern flank of the Nashville structural dome where it merges into the foreland slope of the Appalachian geosyncline. Throughout most of the Paleozoic Era the region was at or slightly below sea level. During this time more than 5,000 feet of limestone, dolomite, and shale were deposited. Since the end of the Paleozoic Era, some 250,000,000 years ago, the area has been above sea level and has been subjected to numerous cycles of erosion resulting in a general peneplanation. During its history this immediate region has been one of little structural deformation. Major folds and faults are entirely absent. The rock strata are only slightly warped with regional dips of less than 1 degree to the southeast away from the Nashville dome and toward the foreslope of the Appalachian geosyncline.

2.5.2.3.2 <u>Regional Stratigraphy (References 1 and 2)</u>

The low plateau on which the Browns Ferry site lies is underlain by near-horizontal limestone strata of Mississippian age having an aggregate thickness of slightly over 1,000 feet. In ascending order the formations and their maximum thicknesses, according to the Alabama Geological Survey, are: Fort Payne, 207 feet; Tuscumbia, 200 feet; Ste. Genevieve, 43 feet; Bethel, 40 feet; Gasper, 160 feet; Cypress, 7 feet; Golconda, 70 feet; Hartselle, 200 feet; and Bangor, 90 feet. Bedrock is mantled by varying thicknesses of cherty clay, silt, sand, and gravel of residual and alluvial origin.

The only formations involved directly in the site area are the unconsolidated materials overlying bedrock and the Tuscumbia limestone and the Fort Payne Formation. A brief description of each of these follows.

Unconsolidated Deposits - Within the site area bedrock is mantled by an average thickness of 54 feet of red and yellow clay containing some residual chert boulders and lenses of sand and gravel. This material varies in thickness from a known minimum of 41 feet to a known maximum of 69 feet.

Tuscumbia Limestone - Only the lower 50 feet of the Tuscumbia formation was encountered at the Browns Ferry site. The Tuscumbia is characterized by medium-to-thick beds of light-gray, medium-to-coarse-crystalline, fossiliferous limestone. Inasmuch as the Tuscumbia Limestone is a relatively pure limestone, it is more affected by solution (than the Fort Payne Formation). Practically all the cavities encountered at the site were developed in this formation.

Fort Payne Formation - The maximum known thickness of the Fort Payne formation in northern Alabama is slightly over 200 feet. At the Browns Ferry site the total thickness, penetrated in one drill hole, is 145 feet. The formation consists of medium-bedded, medium to dark gray, silty dolomite and siliceous limestone with a few thin horizons of shale. Near the top of the formation, some of the beds are cherty and some of the cores showed zones which were slightly asphaltic. The most distinguishing lithologic feature is the presence of quartz-and calcite-filled vugs up to 1 inch in diameter. The silty, siliceous nature of the Fort Payne formation inhibits the development of solution cavities and very few were found in cores drilled from this formation. In general, excavation grades for the major structures of the Browns Ferry plant were set in the Fort Payne formation.

2.5.2.3.3 <u>Regional Structure</u>

The regional structure in the Browns Ferry area is controlled by the Nashville dome. The area lies on the southeast flank of this dome and the regional dip is a degree or less to the southeast. This regional trend is commonly obscured by the slight local variations in dip caused by minor warping and folding. In general, axes of these secondary flexures trend northwest-southeast which is compatible with a regional "cross grain" that has developed on the flank of the Nashville dome.

In the immediate site area the beds of the Tuscumbia and Fort Payne formations are essentially horizontal. Calculations based on the elevations at which the contact between the Tuscumbia and Fort Payne formations was encountered indicate that the direction of dip varies considerably but has an overall westward major component.

As is to be expected in near-horizontal strata, bedrock is cut by a pattern of near-vertical joints. Close to the surface of bedrock, solution channels have developed along these joints especially in the Tuscumbia Limestone. At depth, however, in the less soluble Fort Payne, the joints are tight and most are cemented with calcite.

Faulting is not a significant factor in considering the geologic structure in the Browns Ferry area. No active faults showing recent surface displacement are known within a 200-mile radius of the site. The nearest known ancient fault is in Lawrence County, Alabama, 16.5 miles to the west-southwest from the Browns Ferry site and is one of three apparently related near-vertical faults. The vertical displacement varies from 0 to 60 feet and cuts Mississippian bedrock.

At the site, the only indications of any rock movement are small shears along bedding planes which represent minor readjustments between beds when the area was uplifted at the end of the Paleozoic Era some 250,000,000 years ago. No accurate measurement of these displacements can be made, but movement was probably on the order of a few hundredths of a foot.

2.5.2.4 Site Geology

2.5.2.4.1 Physiography

The area surrounding the Browns Ferry site lies near the southern margin of the Highland Rim section of the Interior Low Plateaus as defined by Fenneman.[1] This physiographic subdivision is characterized by a young-to-mature plateau of moderate relief. The general level of the ground rises gradually from 600 feet above sea level at the north shore of Wheeler Lake to around 800 feet above sea level 10 miles north in the vicinity of Athens, Alabama. This surface is modified by the drainage patterns of Poplar, Round Island, and Mud Creeks, which flow across it from northeast to southwest.

1Fenneman, N.M., "Physiography of the Eastern United States," pp.415-427.

The plant site is located on an old river terrace surface with an average elevation of 575 feet above sea level. This surface represents an old flood plain of the Tennessee River developed when the river was flowing at a higher level. The most recent flood plain is now inundated by the waters of Wheeler Lake. Plant grade is at El. 565.

2.5.2.4.2 Bedrock

The general description of the Tuscumbia and Fort Payne formations has been covered in Section 2.5.2.3.2, "Regional Stratigraphy." This section will deal with the foundation conditions as encountered and treated during the construction period.

The first bedrock uncovered was the lower portion of the Tuscumbia Limestone. As had been expected from the results of the exploratory drilling, the Tuscumbia was cut by frequent near-vertical solution channels developed along steeply dipping joints. Before El. 515 had been reached, the near-vertical solution channels had pinched out and the major indications of solution were near-horizontal zones of weathering developed along the bedding.

At this stage additional exploratory drilling was done to determine the detailed foundation conditions under the reactor building, turbine building, intake structure, and chimney areas. As shown on Figure 2.5-4 a combination of core drill and percussion holes was drilled. The core holes furnished detailed stratigraphic control, while the primary purpose of the closer spaced percussion holes was to ensure that no large cavities existed below the foundation.

Eleven specimens of core selected from holes drilled in the reactor area were tested for unconfined compressive strength in the TVA materials laboratory (Reference 3). The maximum value obtained was 17,200 psi; the minimum was 11,419 psi; and the average was 14,175 psi (Reference 4). Additional core samples from the same area were sent to John A. Blume and Associates for testing of other physical properties. The following data are taken from their report (Reference 5):

Elastic Modulus	8,200,000 psi
Shear Modulus	2,300,000 psi
Constrained Modulus..	10,000,000 psi
Poisson's Ratio	0.252

Ten selected percussion holes in the Unit 1 and Unit 2 areas were inspected with a borehole television camera to confirm the results of the drilling, and the continuity of horizontal seams was checked in four core drill holes by resistivity and gamma ray logs. In addition, test pits were dug at the centers of Units 1 and 2.

The exploration and initial excavation disclosed a persistent weathered zone at the base of the Tuscumbia Limestone, between El. 509 and 508 at the west side of the Unit 1 portion of the reactor building, which contained one or more partially open seams 0.1 foot to 0.15 foot thick. In some instances this seam was open, and in others it was clay-filled. This weathered zone sloped upward to the east along the contact between the Tuscumbia and Fort Payne formations. Midway between Unit 1 and Unit 2 this contact intersected the rock surface at approximately El. 511. As a result, only the Fort Payne formation is the foundation rock in the Unit 2 and Unit 3 areas, and no cavities or seams were present.

Special foundation treatment was required to ensure an adequate foundation for the structures. In the Unit 1 portion of the reactor building this was accomplished by a system of underpinning. Trenches were excavated through the Tuscumbia Limestone to the underlying Fort Payne Formation. These trenches are under the perimeter walls of the reactor building and a doughnut-shaped trench is under the periphery of the central mass under the drywell. These trenches were backfilled with concrete to El. 513, the bottom of the reactor building slab. (Figure 2.5-19)

Under the Unit 2 and Unit 3 portions of the reactor building where the Fort Payne formation was below the design elevation of 513, fill concrete was placed to that elevation over the entire area.

Foundation for the bearing pile cluster locations in the turbine building area was provided by grouting the seam at the Tuscumbia-Fort Payne contact by conventional consolidation grouting methods.

Surficial Deposits

Plant Area - A soils investigation program was initiated in the spring of 1966 for the purpose of establishing the allowable bearing value for soil-supported structures and identifying adequate borrow material (Reference 6).

The original ground surface occurred at approximately 15 feet above final plant grade in the area of plant structures, and approximately 2 feet above the final transformer and switchyard area grade.

The top 15-20 feet is classified as a red-to-reddish-brown, sandy clay and lean-to-medium silty clay with a maximum thickness of 30 feet. This is designated as the preferred borrow material.

Underlying these alluvial terrace deposits are approximately 40 feet of residual medium-to-fat clays and plastic silts interbedded with lenses of medium layers of gravelly chert. The ground water table was established at El. 555.1 foot, which corresponds to the level of Wheeler Reservoir.

A total of 13 borings was made. Of these, undisturbed samples were obtained from seven, using 3-inch and 5-inch Shelby Tubes. The remaining six were drilled with a 4-inch power auger, with disturbed samples taken. Undisturbed samples were taken at 3-foot intervals, while disturbed samples were taken at 5-foot intervals. Standard penetration tests were made using a 2-inch split-spoon.

Laboratory testing consisted of index tests, soil classification, consolidation tests, and vane shear tests. The laboratory testing and the standard penetration resistance result gave an allowable soil bearing capacity of 1.5 tons per square foot for a mat foundation and 2.0 tons per square foot for spread footings.

Intake Channel - To determine the seismic stability of materials in the intake channel, borings and samplings were made in depth at five locations in the side slopes of the channel connecting the intake structure with Wheeler Reservoir. Laboratory tests and analysis then established shear and other design values of this material (see Chapter 12). A vibroseismic survey was also made at the site to identify the shear wave and compression wave velocities of the soils. Using the values obtained, the seismic stability was evaluated and is described in Section 12.2.7.

Low Level Radwaste Storage Facility - A soils investigation program was initiated in the Winter of 1980 for the purpose of determining the in-situ and borrow soil properties. Figure 2.5-S1 shows the location of the LLRW facility in relation to main plant and Figure 2.5-S2 shows the location of in-situ soil borings for the LLRW facility.

The overburden thickness varies from 37 to 50 feet with existing grade varying from El. 588 to El. 570. Final grade is at approximate El. 580. Figures 2.5-S3 and -S4 are generalized soil profiles in the LLRW area.

A predominantly red lean clay layer extends from the ground surface to depths ranging from 2 to 18 feet and averaging 16 feet. This layer is continuous except where previously excavated for borrow. Typically, blow counts (N) from the standard penetration test (ASTM D-1586) fall within the stiff to very stiff consistency range. However, surficial weakness revealed by blow counts less than 10 is not uncommon. These clayey soils represent ancient terrace alluvium.

Underlying the lean clay is an intermediate layer consisting of predominantly tan to red, medium to highly plastic residual clay. This layer is not continuous. In places, it has a thickness up to 26 feet and averages about 16 feet. Inclusions of gravelly clay, lean clay, and silt are present. Penetration resistance indicates very stiff consistency with only scattered isolated weakness.

The intermediate layer extends to bedrock and is underlain by a basal cherty clay or clayey chert. The fine fraction of these soils is usually highly plastic, showing a wide variation in color ranging from red to yellow and tan to black. The gravelly clay containing over 10 percent gravel represents a transitional zone between the fat clay and the clayey gravel as shown on the general cross sections, Figures 2.5-S3 and 2.5-S4.

The residual clayey gravels sampled with the 2-inch split-spoon usually showed some particle breakage due to the penetration of the samplers. Thus, the in-place material is coarser than the samples grain size tests indicate. This gravel layer is continuous across the site and averages about 18 feet in thickness. Soil consistencies indicate wide areas of relative weakness ($N \leq 10$) which is most pronounced and persistent immediately above bedrock.

Water level readings were taken in borings 1 hour and 24 hours after completion of drilling. Water levels varied from El. 575 in the western area of structures A-1 and A-2 to about El. 569 on the eastern end of A and B structures and the southern end of the C structures. In some cases, water was not initially apparent at the established elevations. On penetrating the clayey gravel (GC) soils, however, water rose and adjusted rapidly, indicating the water is confined.

Laboratory testing consisted of index testing of split-spoon samples for classification. Moisture contents were obtained on all samples. Undisturbed samples were tested for moisture content, grain size, Atterberg limits, specific gravity, and unit weight. In addition, representative undisturbed samples were selected for determining engineering properties. Specifically, triaxial compression unconsolidated-undrained (Q-test), triaxial compression consolidated-undrained (R-test), and direct shear consolidated-drained (S-test) strengths were determined. Consolidation and permeability tests were conducted.

Three borrow areas were investigated to locate a suitable quantity of an acceptable borrow material. Each area was sampled with auger borings to collect bag samples for laboratory testing. Standard compaction curves (ASTM D-698) were developed for each borrow area, and shear strength, consolidation and permeability tests were conducted on remolded samples of each identified borrow soil class. Each borrow area was evaluated and determined to provide an adequate source for borrow material.

An evaluation of the in-situ and borrow soils was made. The results were used to determine the allowable bearing capacity and predicted settlement, which were found to be less than the design criteria limits.

In-situ dynamic soil properties were determined using cross hole, uphole, and downhole geophysical tests. The tests were performed at three locations. The tests were evaluated, and the results indicated a range of shear wave velocities between approximately 500 and 2300 feet per second.

2.5.3 Seismology (References 7 and 8)

2.5.3.1 Introduction

The Browns Ferry Nuclear Plant is located in an area far removed from any centers of significant seismic activity. A few major earthquakes centered at distant points, several light-to-moderate shocks at distant points, and several light-to-moderate shocks with nearer centers have affected the area at low-to-moderate intensity (Reference 9).

2.5.3.2 Seismic Investigation Program

In order to evaluate the earthquake hazard at the plant site, a study was made of the known seismic history of a large surrounding area. This study was greatly facilitated by research carried on over a period of more than three decades on the seismicity of the southeastern United States in general and the Tennessee Valley region in particular. Voluminous files of earthquake data, collected from a number of sources, were used in the compilation of seismic histories of the several states. By plotting the epicenters of hundreds of these earthquakes, the areas of continuing seismic activity became apparent. The more active areas are as follows:

a. Mississippi Valley, especially the New Madrid region of Missouri, Arkansas, Tennessee and Kentucky. This area has been seismically active since the appearance of the white man and very probably long before. The area has been the center of a few great earthquakes and very numerous lighter shocks which are still occurring at intervals. The New Madrid region is about 170 miles northwest of the plant site.

b. The Lower Wabash Valley of Indiana and Illinois. This area has been the center of several moderately strong earthquakes, some of which were felt as far south as Tennessee. The Lower Wabash Valley is about 225 miles north-northwest from the plant site.

c. Charleston area, South Carolina. One of the country's greatest earthquakes was centered in the Charleston area. Many other light-to-moderate earthquakes have occurred in this area and the activity has continued to the present time. Charleston is about 420 miles east of the plant site.

d. The Southern Appalachian area of western North Carolina and eastern Tennessee. Light-to-moderate earthquakes occur in this area at an average frequency of one or two per year. This area is centered about 200 miles east of Decatur.

In addition to these areas, shocks of light-to-moderate intensity have occurred at many other localities in the southeastern states at various distances from the Browns Ferry plant site. At many of these localities, only a few light-to-moderate shocks from widely scattered centers are known. Seismic history in the vicinity of the plant is discussed in Section 2.5.3.4.

Several scales have been devised to evaluate the force of earthquake shocks. Scales which rate earthquakes on the degree of shaking at any given locality are known as intensity scales. In general, the intensity of an earthquake is highest in the epicentral area and diminishes in all directions from the point of maximum intensity. Another factor affecting the intensity of an earthquake is the character of the ground in a given locality. The shaking is much less severe, other things being equal, at a place on bedrock than one on thick alluvium. The most widely used intensity scale is the modified Mercalli scale which has 12 degrees of intensity (see Figure 2.5-7). The degrees of intensity are expressed by Roman numerals from I to XII.

The Richter magnitude scale applies to an earthquake as a whole rather than the observations made at some point, or points, within the area affected. Earthquake magnitude is calculated from measurements on seismograms and is expressed by an Arabic numeral and a fraction, such as 6-3/4 or 5.5.

2.5.3.3 Geologic and Tectonic Background

As discussed in Section 2.5.2, Browns Ferry Nuclear Plant is founded on a thick succession of essentially horizontal sedimentary rocks. The site is 16.5 miles away from the nearest known inactive fault and approximately 200 miles from the New Madrid region of the Mississippi Valley. Since the site area is very low on the southeastern flank of the Nashville structural dome, it has undergone no tectonic movement except simple uplift. This movement probably ceased at the close of the Paleozoic Era.

2.5.3.4 Seismic History

A list of seismic events within a 200-mile radius of the plant site that occurred from 1699 through 1980 is presented in Table 2.5-1. Epicenters of earthquakes within a 200-mile radius of the plant site based on Table 2.5-1 are shown on Figure 2.5-6.

2.5.3.5 Seismicity of the Area

Light shocks have been centered near Huntsville, Hazel Green, Anniston, Cullman, Easonville, and Birmingham, but most of them were felt as low intensity shocks or not at all in the Decatur area. The shocks felt most strongly in the area have been major earthquakes centered at distant points, especially in the Mississippi Valley.

There is continuing seismic activity in the Mississippi Valley, and the possibility of another great earthquake in the New Madrid region of Missouri, Arkansas, Tennessee, and Kentucky cannot be discounted. An earthquake of intensity XI or XII at New Madrid might be felt in the Decatur area with an intensity of VII.

2.5.4 Conclusions (References 8 and 10)

The site is underlain by massive formations of bedrock, thus providing adequate foundations for all plant structures.

The major seismic activity experienced at the site has been caused by distant major earthquakes, especially those at New Madrid and Charleston. For design purposes, a conservative assumption was made that a seismic event at an unstated location could cause an intensity of VII at the plant site. Thus, the design of structures and equipment important to the plant safety features was based on a horizontal ground motion due to a peak acceleration of 0.10g. In addition, the design is such that the plant can be safely shut down during a peak horizontal ground acceleration of 0.20g. Vertical accelerations are two thirds of the horizontal accelerations. Details of the earthquake design of these structures are given in Chapter 12. Design based upon these ground accelerations provides a margin of protection against either minor shocks originating near the site or major shocks originating at New Madrid or Charleston.

Since the site is located in an area of extremely low seismicity, it has been principally affected, if at all, by distant, strong earthquakes. The response spectra chosen for the site are shown in Figures 2.5-8 and 2.5-9 for the OBE and DBE, respectively. The time-history method was used in analyzing all structures; and the El Centro, May 1940, N-S component was chosen for this purpose. This record was determined to adequately represent any potential threat to the site. A comparison of the response spectrum produced by this record and the spectrum used for design is shown in Figure 2.5-10. As an alternate basis for design of subsystems, an artificial acceleration time history input ground motion was used. Figure 2.5-11 depicts the acceleration, velocity, and displacement time history of this record. Figures 2.5-12 through 2.5-16 compare the response spectra of this time history to the site design spectra for the various damping levels. The artificial time history meets the enveloping requirements of Section 3.7.1, "Standard Review Plan."

2.5.5 Seismic Instrumentation Program

Seismic instrumentation is provided in order to assess the effects on the plant of earthquakes which may occur that exceed the ground acceleration for the Operating Basis Earthquake (OBE = 0.10g ground acceleration). The seismic instrumentation is not safety-related and does not have any effect on safety-related systems or equipment. The seismic instruments were selected to emphasize accuracy and reliability, while at the same time minimizing the maintenance and surveillance resources required to support the system. The instrumentation that is provided is described in the following sections.

2.5.5.1 Location and Description of Instrumentation

The seismic instrumentation locations are shown in Figure 2.5-17. It is solid state digital instrumentation which will enable the processing of data at the plant site within 4 hours of a seismic event. One of the sensors is located at the top-of-rock where the OBE design response spectrum is defined. Therefore, this instrumentation is sufficient to adopt the OBE exceedance criteria described in Reference 2.5.6-11 through 15.

The instrumentation consists of the following:

1. A strong-motion triaxial time-history accelerograph at each of the following locations:

   a. El. 519.5, Unit 1 Reactor Building, south corner of the Northwest Quad Room,

   b. El. 621.75, Unit 1 Reactor Building, upper level in Electrical Board Room 1A, and

   c. El. 566.0, Unit 1 and 2 Diesel Generator Building, on the base slab in Room B.

   These accelerographs have a full scale range of ±2g. The internal recorder is capable of digitally recording a minimum of 25 minutes of data with a minimum of 3 seconds of pre-event memory and 5 seconds of post-event memory. An internal seismic trigger with a bandwidth of 0.1Hz - 12.5 Hz actuates the recording system when a threshold acceleration level is sensed. The unit is equipped with an internal rechargeable battery and an external plug-in type battery charger.

2. The centralized seismic instrumentation panel components are located in the Unit 1 Main Control Room (Elevation 621.0'). This panel contains: a) central controller, b) alarm panel, and c) display panel. A description of each item is given below.

   a. A central controller consisting of an industrial computer and custom software, which provides a user interface in a multi-tasking operating system that supports simultaneous seismic data acquisition and interrogation. The controller is powered by 120V AC power. The central controller retrieves data files from the internal digital recorders in each remote accelerograph after an event and, for the top-of-rock sensor, performs automatic analysis of the data. The event-analysis capabilities include computation of the Cumulative Absolute Velocity (CAV), spectral content of the recorded data, and automatic comparison to the site OBE (OBE = ½ SSE) response spectrum. Hard copies of the operational data and event analysis will be printed to a Unit 1 Main Control Room printer. The central controller's software capabilities also include automatic event alarm and annunciation (See item 2b). The event analysis functions of the central controller may be performed off-line, if necessary.

b. An alarm panel containing visual alarms to indicate that a seismic event has been recorded, the OBE response spectrum has been exceeded in a damaging frequency range, and system trouble including either loss of AC or DC power. The seismic event alarm is triggered by the accelerographs, while the OBE exceedance and system trouble alarms are triggered by the central controller. Activation of either the event alarm, exceedance alarm, or system trouble alarm also causes corresponding windows on an annunciator panel in the Unit 1 Main Control Room.

c. A display panel to provide a visual display for operation of the centralized system.

3. Annunciator lights in the Unit 1 Main Control Room:

a. Window Legends: START OF STRONG MOTION ACCELEROGRAPH - Any one of the three strong-motion accelerographs (Item 1) will activate this window if an acceleration greater than or equal to 0.01g is sensed.

b. Window Legends: ½ SSE RESPONSE SPECTRUM EXCEEDED - The central controller in conjunction with the alarm panel (Items 2a and 2b) will activate this window if the central controller has determined the OBE response spectrum to be exceeded. This determination is based only on input received from the Reactor Building base slab accelerograph (Item 1a).

c. Window Legends: SEISMIC MONITORING SYSTEM TROUBLE - The alarm panel will activate this window if the central controller detects a system trouble or if there is loss of AC or DC power.

The basis for selecting the Reactor Building for installation of seismic instruments is that it is the rock-supported building most important to safety. The basis for selecting the Diesel Generator Building is that it is the soil-supported building most important to safety.

2.5.5.2 <u>Control Room Operator Notification</u>

The seismic monitoring system provides three independent alarm windows in the Unit 1 Main Control Room. The first annunciator window indicates system trouble which serves to provide warning of equipment operability problems under normal power conditions as well as following a seismic event. The next annunciator window is provided by the accelerographs (Item 1) via the controller, Section 2.5.5.1, which alerts the operator that a seismic event is being recorded. This annunciation indicates that at least one of the accelerographs triggers has sensed seismic motion in excess of 0.01g.

The final annunciator window is actuated later and is provided by the central controller (Item 2a), Section 2.5.5.1, and is only received if the event-analysis software indicates that the site OBE design basis response spectrum has been exceeded in a potentially damaging frequency range, as described in Section 2.5.5.3.

The basis for establishing the OBE design basis response spectrum for the levels at which control room operator notification is required is that the design of Structures, Systems, and Components (SSCs) for loading combinations, which include OBE, are to design basis allowable stress levels which are well within the elastic limit of the materials.

2.5.5.3 Controlled Shutdown Logic

The operator will utilize input from multiple sources to determine the need for a controlled shutdown following the seismic event. The decision for a controlled shutdown will be based primarily on an assessment of the actual damage potential of the event, which will be available within 4 hours, and on the results of short-term inspections, which will be available within 8 hours. The operator may also confirm that ground motion was sensed by plant personnel and/or confirm the occurrence of the seismic event with the National Earthquake Information Center. The purpose of these actions are 1) to perform a preliminary assessment of the effect of the earthquake on the physical condition of SSCs, and 2) to determine if shutdown of the plant is warranted based on observed damage to SSCs, or because the OBE has been exceeded.

The walkdowns of plant SSCs in accessible areas of the plant will be performed witin 8 hours following the seismic event. The walkdowns will be performed using the general guidance in Chapter 4 of the Electrical Power Research Institute (EPRI) Report NP-6695 (ref. 2.5.6-12). These walkdowns will include a check of the neutron flux monitoring sensors for changes and an inspection of the containment isolation system to ensure continued containment integrity. The walkdown data will be compared to data previously obtained from baseline and Maintenance Rule inspections in order to obtain a clear understanding of any seismically induced damage.

The assessment of the damage potential of the event will be made within 4 hours following the event using the OBE Exceedance Criteria developed by EPRI and documented in references 2.5.6-11 through -15. As noted above, the indication of damage potential will be provided by event-analysis software installed on the centralized seismic monitoring system described in Section 2.5.5.1. The analysis will be performed for the uncorrected accelerograms recorded from the strong motion triaxial accelerograph located in the Unit 1 Reactor Building on the base slab (item 1a of Section 2.5.5.1). Use of the uncorrected accelerograms is known to be conservative. The basis for use of the seismic motion on the Reactor Building base slab is that the site OBE design response spectrum is defined at top-of-rock, which corresponds to the Reactor Building base slab location.

The EPRI OBE Exceedance Criteria uses two indicators of damage potential. The first indicator of damage potential is specified as the Cumulative Absolute Velocity (CAV) of the accelerogram. A meaningful usage of the CAV requires that the recorded data be obtained by the accelerometer mounted in the free-field. As noted above, the OBE design spectrum for Browns Ferry is defined as occurring at top-of-rock (i.e., foundation level of the rock-supported structures); whereas, free-field is defined as top-of-soil at sufficient distance from nearby structures to preclude interference/interaction effects. The Seismic Monitoring System for Browns Ferry does not have a free-field accelerometer. Therefore, the shutdown logic adopted for Browns Ferry will concede CAV exceedance and base the assessment of damage potential solely on the second indicator, as discussed below.

In the absence of data from a free-field accelerometer, the second indicator is an evaluation of the frequency range in which the OBE spectrum is exceeded. This criteria is based on research which indicates that exceedances above a frequency of 10 Hz are not damaging to nuclear plant SSCs. The following two measures of damaging potential are used.

- The OBE site design basis response spectrum is exceeded if the 5 percent damping response spectra generated for any one of the three components of the uncorrected accelerograms from the Reactor Building foundation accelerometer is larger than:

    1. The corresponding OBE design basis response spectral acceleration in a frequency range between 2-10 Hz, or

    2. The corresponding OBE design basis response spectral velocity for frequencies between 1-2 Hz.

Therefore, Browns Ferry will base the assessment of damage potential of the event on either a spectral acceleration exceedance between 2-10 Hz or a spectral velocity exceedance between 1-2 Hz.

Once the results of the walkdown and the assessment of damage potential of the event are available, the operators will determine 1) if a controlled shutdown is required and 2) the condition of the equipment needed to safely achieve shutdown. If the assessment of damage potential indicates that the OBE Exceedance Criteria were not met, and the walkdown results are favorable, the plant will continue to operate. Basing shutdown logic on the actual damage potential of the event and on the results of short-term inspection avoids unnecessary shutdowns while ensuring that the operator has the plant status information needed to make an informed reactor shutdown decision.

Post-shutdown actions, including retrieval of data, recalibration of seismic instruments, and comparison of measured and predicted responses will be based on the guidance in Chapters 5 and 6 of EPRI Report NP-6695 (Ref. 2.5.6-12).

2.5.6 References

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4. Memorandum - R. O. Lane to B. C. Moneymaker, "Tims Ford and Browns Ferry Sites - Rock Cores - Compressive Strength Test Results," June 10, 1966. (R40990309902)

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11. EPRI Report NP-5930, “A Criterion for Determining Exceedance of the Operating Basis Earthquake,” July 1988.

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13. EPRI Report TR-100082, “Standardization of the Cumulative Absolute Velocity,” December 1991.

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15. Nuclear Regulatory Commission Regulatory Guide 1.12, “Nuclear Power Plant Instrumentation for Earthquakes,” March 1997