LOUISIANA ENERGY

Post Office Box 1004
Charlotte, NC 28201-1004

July 29, 1992

Mr. John W. N. Hickey, Chief
Fuel Cycle Safety Branch
Division of Industrial and
Medical Nuclear Safety
Office of Nuclear Material Safety
and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Docket No.: 70-3070
Louisiana Energy Services
Claiborne Enrichment Center
Requests For Additional Information
File: MTS-6046-00-2001.01

Dear Mr. Hickey:

As requested by your letter to LES dated May 20, 1992, provided in Attachment A is information related to selected environmental, safety and quality assurance issues. Also enclosed are "Information Only" copies of the sections of the License Application (LA), Environmental Report (ER), and Safety Analysis Report (SAR) that will be revised as a result of providing this information. A formal update to the license application documents will be made in the near future. The responses to the information requests regarding geology and seismology starting on the enclosed page S-1 reflect the meeting with members of your staff on June 9, 1992.

If there are any questions concerning this, please do not hesitate to call me at (704) 373-8466.

Sincerely,

Peter G. LeRoy

Peter G. LeRoy
Licensing Manager

PGL/N56.792

Enclosures

NF04

xc: (w/ enclosures)

Ms. Diane Curran, Esquire
Harmon, Curran, Gallagher, & Spielberg
2001 S Street, NW, Suite 430
Washington, DC 20009-1125

Ms. Nathalie Walker
Sierra Club Legal Defense Fund
400 Magazine Street
Suite 401
New Orleans, LA 70130

Mr. R. Wascom
Office of Air Quality and Radiation Protection
Louisiana Department of Environmental Quality
PO Box 82135
Baton Rouge, Louisiana 70884-2135

QUESTIONS AND REQUESTS FOR ADDITIONAL INFORMATION: ENVIRONMENTAL

2.1.3 DECONTAMINATION AND DECOMMISSIONING

1. Adjust decommissioning cost analysis to 1996 dollars.

Section 4.4.4 and Table 4.4-2 of the Environmental Report (ER) have been revised to provide decommissioning costs adjusted to 1996 dollars. Section 8.1 has also been updated, as well as Safety Analysis Report Section 11.8. There is no ER Section 2.1.3.

2.2.2 LAND USE

1. Provide information for any significant natural resources <u>at</u> the proposed site and discuss plans for utilization of these resources. If resources are to be extracted, identify potential environmental impacts of this utilization.

There are several limited outcroppings of iron ore on the LES property. LES has no plans for utilization of this material now nor in the future. Petroloum resources potentially exist beneath the LES property but LES will not recover any such resource now nor in the future. LES' only intended activity at the CEC is the enrichment of uranium for use as commercial power reactor fuel.

2.6 METEOROLOGY

1. The NRC staff interpretation is that the data presented for the low wind speed (1-3 knot) category in ER Table 2.6-5 includes periods of calms. The assumptions for the treatment of calms are not provided. For this low wind speed category, provide a listing of frequencies of calms and of low wind speed observations for each stability class and direction.

The meteorological data in Table 2.6.-5 includes periods of calms. The discussion of the treatment of calm wind speeds was, however, limited to those sections of the Environmental Report (ER) and Safety Analysis Report (SAR) that dealt with the XOQ/DOQ air modeling and results. Appendix A-1 for ER Section 4.2.1.2.3 states that the guidance provided in Regulatory Guide 1.111 was used to include calm observations into the dataset. Specifically, the calms were assigned to wind directions in proportion to the directional distribution within an atmospheric class of the lowest noncalm windspeed class. This same information is provided in Section 3.3.3.1.2 of the SAR. Neither the joint frequency distribution of stability, wind speed and direction without adjustment for calms, nor the number of calm wind observations in each stability, were included in either report. A complete listing of this data is attached (Tables A-G).

TABLE A

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY A

Direction	SPEED (KNOTS) 1-3	4-6	7-10	11-16	17-21	>21
N	7	46	0	0	0	0
NNE	4	14	0	0	0	0
NE	4	24	0	0	0	0
ENE	4	12	0	0	0	0
E	6	23	0	0	0	0
ESE	5	18	0	0	0	0
SE	6	30	0	0	0	0
SSE	7	23	0	0	0	0
S	11	42	0	0	0	0
SSW	8	16	0	0	0	0
SW	7	34	0	0	0	0
WSW	7	30	0	0	0	0
W	8	29	0	0	0	0
WNW	5	17	0	0	0	0
NW	9	20	0	0	0	0
NNW	3	11	0	0	0	0
TOTAL	101	389	0	0	0	0

# of occurrences of A Stability = 657
# of calms with an A Stability = 167

TABLE B

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY B

Direction	1-3	4-6	7-10	11-16	17-21	>21
	SPEED (KNOTS)					
N	38	138	97	0	0	0
NNE	18	72	56	0	0	0
NE	17	81	54	0	0	0
ENE	17	69	41	0	0	0
E	34	108	77	0	0	0
ESE	30	92	58	0	0	0
SE	30	128	73	0	0	0
SSE	32	95	68	0	0	0
S	52	186	156	0	0	0
SSW	25	88	77	0	0	0
SW	28	102	100	0	0	0
WSW	34	116	78	0	0	0
W	35	107	78	0	0	0
WNW	22	60	29	0	0	0
NW	17	81	46	0	0	0
NNW	21	67	41	0	0	0
T O T A L	450	1590	1129	0	0	0

# of occurrences of B Stability = 3461
# of calms with a B Stability = 292

TABLE C

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY C

	SPEED (KNOTS)					
Direction	1-3	4-6	7-10	11-16	17-21	>21
N	8	140	336	34	0	0
NNE	5	61	138	18	0	0
NE	5	76	140	18	0	0
ENE	3	55	116	12	0	0
E	11	101	229	13	0	0
ESE	8	107	187	20	2	0
SE	16	130	274	28	0	0
SSE	14	106	237	41	0	0
S	15	221	505	96	3	1
SSW	9	94	197	29	0	0
SW	6	119	214	28	1	0
WSW	15	117	218	27	1	0
W	6	78	156	28	0	0
WNW	4	50	107	15	2	0
NW	3	61	154	27	1	0
NNW	3	60	164	22	0	0
T O T A L	131	1576	3372	456	10	1

# of occurrences of C Stability = 5712
# of calms with a C Stability = 166

TABLE D

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY D

Direction	1-3	4-6	SPEED (KNOTS) 7-10	11-16	17-21	>21
N	35	346	813	554	53	1
NNE	17	228	509	195	15	1
NE	15	174	395	147	1	0
ENE	16	195	413	130	2	0
E	25	294	462	119	4	0
ESE	23	345	359	81	4	2
SE	54	452	750	273	12	1
SSE	36	322	878	407	31	6
S	35	462	1330	1078	76	5
SSW	11	154	383	211	14	0
SW	17	160	276	180	6	0
WSW	16	121	160	125	14	4
W	6	115	169	180	25	3
WNW	11	93	248	236	25	16
NW	12	133	360	487	69	8
NNW	11	110	311	351	34	4
T O T A L	340	3704	7816	4754	385	51

# of occurrences of D Stability = 17432
# of calms with a D Stability = 382

TABLE E

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY E

	SPEED (KNOTS)					
Direction	1-3	4-6	7-10	11-16	17-21	>21
N	0	159	281	0	0	0
NNE	0	105	139	0	0	0
NE	0	85	107	0	0	0
ENE	0	89	152	0	0	0
E	0	213	132	0	0	0
ESE	0	250	72	0	0	0
SE	0	402	129	0	0	0
SSE	0	328	212	0	0	0
S	0	463	531	0	0	0
SSW	0	141	174	0	0	0
SW	0	129	92	0	0	0
WSW	0	121	62	0	0	0
W	0	122	98	0	0	0
WNW	0	72	193	0	0	0
NW	0	75	243	0	0	0
NNW	0	51	150	0	0	0
T O T A L	0	2805	2767	0	0	0

# of occurrences of E Stability = 5572
# of calms with an E Stability = 0

TABLE F

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY F

Direction	1-3	4-6	SPEED (KNOTS) 7-10	11-16	17-21	≥21
N	19	295	0	0	0	0
NNE	13	190	0	0	0	0
NE	16	93	0	0	0	0
ENE	11	152	0	0	0	0
E	40	252	0	0	0	0
ESE	57	305	0	0	0	0
SE	109	394	0	0	0	0
SSE	59	392	0	0	0	0
S	68	782	0	0	0	0
SSW	25	311	0	0	0	0
SW	21	219	0	0	0	0
WSW	61	297	0	0	0	0
W	54	395	0	0	0	0
WNW	16	267	0	0	0	0
NW	7	218	0	0	0	0
NNW	2	85	0	0	0	0
T O T A L	578	4647	0	0	0	0

# of occurrences of F Stability = 6450
# of calms with a F Stability = 1225

TABLE G

ANNUAL FREQUENCY DISTRIBUTION
SHREVEPORT, LA
NWS 1984-1988

STABILITY G

Direction	1-3	4-6	7-10	11-16	17-21	>21
	SPEED (KNOTS)					
N	31	0	0	0	0	0
NNE	19	0	0	0	0	0
NE	17	0	0	0	0	0
ENE	14	0	0	0	0	0
E	29	0	0	0	0	0
ESE	93	0	0	0	0	0
SE	137	0	0	0	0	0
SSE	138	0	0	0	0	0
S	112	0	0	0	0	0
SSW	51	0	0	0	0	0
SW	44	0	0	0	0	0
WSW	136	0	0	0	0	0
W	181	0	0	0	0	0
WNW	59	0	0	0	0	0
NW	13	0	0	0	0	0
NNW	13	0	0	0	0	0
T O T A L	1087	0	0	0	0	0

# of occurrences of G Stability = 4516
# of calms with a G Stability = 3429

3.2 PLANT OPERATION

Plant Features

1. "It is estimated that 20,000 gallons of diesel fuel will be stored on site. It is necessary to assure that if leaks from the storage tanks occur, that they will be detected and stopped immediately." Therefore, provide more information for the leak detection and monitoring program that will be adopted at the site.

The diesel fuel tank design has been changed from underground storage tanks to tanks located above ground. The tanks are provided with a secondary containment shell to contain leaks and spills of diesel fuel. This design change was reflected in the March 30, 1992 revisions to SAR Section 6.4.11. The above ground tank design allows for visual inspection of the containment reservoir area within the secondary containment shell for leaks and spills and for visual inspection of the soil surrounding the tank shell for evidence of a release beyond the secondary containment barrier. Leak detection will be accomplished by daily visual inspection of the area between the primary containment tank and the secondary containment shell. Daily inspection of the soil surrounding the containment shell will also be conducted. Because the secondary containment shell will be designed to contain any diesel fuel that is spilled or leaks from the primary tank, no further leak detection is required beyond visually verifying that no leaks or spills have reached the containment reservoir within the secondary containment shell. Since the secondary containment shell is designed to contain a leak until cleanup can be accomplished, "immediate" detection of a leak is not necessary. Additional information about leak detection has been added to SAR Section 6.4.11.

4.0 EFFECTS OF SITE PREPARATION AND PLANT CONSTRUCTION

Socioeconomics and Land Use

1. It is stated that no activities unrelated to the plant operations are to occur within the 65-acre site for primary plant operations. However, the site property boundary consists of 442 acres. Are any activities unrelated to the operations of the plant to occur on this property?

No activities unrelated to CEC operations are to occur on the 442 acres of property owned by LES.

4.4 DECOMMISSIONING AND DISMANTLING

4.4.27 Disposal

This section provides an estimate of the total volume (i.e., 100 cubic meters) of radioactive warte produced on D&D activities. Identify and provide a volume estimate of all hazardous waste, including any potential mixed waste, produced in these activities.

Hazardous wastes and mixed wastes are not expected to be produced during D&D activities.

It should be noted that normal accumulation of hazardous and mixed wastes will occur during the final months of CEC operation. The volume of these final wastes at decommissioning, not due to D&D activities, are estimated to be equivalent to the annual amounts listed in the CEC Environmental Report, Table 3.3-8. An increase of $0.1 million in the decommissioning cost has been added to cover disposal costs of these wastes.

8.0 BENEFIT-COST ANALYSIS

1. What are the weighted cost of capital, the fixed charge rate, and the depreciable plant investment projected by LES?

This information and related information were provided in LES' May 1, 1992 letter to the NRC regarding Financial Information. Specifically, Attachment E of the May 1, 1992 submittal "LES Project Financial Plan (Proprietary Version)" page 15, Financing provides the requested information.

3.6 GEOLOGY AND SEISMOLOGY

Question 2:

The response did not address the issue of the potential for an earthquake of design basis severity, or worse, to occur in the next 50 years.

In paragraph 1 of the response, a statement is made that "...there must be sufficient magnitude of stress in the proper orientation to reactivate old faults or create new ones. There is no evidence for favorable stress orientation or magnitude as demonstrated by the lower levels of historical and recorded seismicity." Information contained in the other paragraphs point to a need for additional information to support the response.

The low levels of historical and recorded seismicity is not an adequate demonstration that no evidence exists for sufficient magnitude or orientation of stress to reactivate old faults or create new ones. Have in-situ stress measurements been made in the immediate area of the site, in the basement rocks, or in the overlying sediments? Do in-situ stress measurements exist in the area to indicate any stress orientation, favorable or unfavorable? Although principal seismogenic zones are commonly associated with geologic structures (basement faults, basement rifts, uplifts, or basins), not all similar geologic structures are seismic source zones. In areas where seismic activity is relatively low, a tendency is to assume that one-to-one spatial correlation exists between these major geologic structures and earthquakes. In northern Louisiana or the Texas-Louisiana border, the earthquakes are not related to obvious geologic structures and have yet to be adequately explained. In areas where seismogenic structures do not exist or are subtle, for example at the subject site, efforts should be made to examine earthquake focal mechanisms and in-situ stress measurements.

Response

The design basis earthquakes (DBEs) for the mid-field and far-field were earthquakes with 500 year return-periods (10% probability of exceedence in 50 years) in their associated source zones, and they were located at the closest points of those source zones.

The near-field DBE was an earthquake of magnitude (m_b) 4.3 located at a distance of 15 kilometers and a depth of 5 kilometers. The magnitude of m_b = 4.3 was for an earthquake with a 500 year return period in the Interior Salt Basin source zone. The distance of the near-field DBE was calculated by positioning the site at the center of a circle with an area equal to the

average area for a magnitude 4.3 event over 2,374 years, assuming an average event distribution. Within the circle, the event was located at a distance such that half of the area of the circle was closer and half was farther. The depth of the event was fixed at the top of the crystalline basement. The basis for this location approach for the near-field DBE is that seismicity within the Interior Salt Basin cannot, in general, be associated with identifiable geologic structures (i.e., the seismicity of the region must be treated as random). The only constraint is that damaging tectonic earthquakes are limited to the crystalline basement.

Question 4:

Paragraph 3 indicates that the interface locations at the test boring locations were determined from a combination of "...C)...comparison of the test boring data with nearby cone penetration tests (CPT)..." Results from CPTs provided data on soil properties for engineering applications, and are not directly related to a specific lithology of lithologic changes. Similarly, for changes in drilling behavior (line 2) provides information only on drilling behavior and is not directly related to a specific type of lithology. The response indicates that the interface locations are "imprecise" (line 9) and further specifies that interfaces are estimated to be within ±5 feet of the actual location (line 15). What is the basis for stipulating ±5 feet?

The response indicates that the interfaces "...may actually be transitional... (line 13)." This sentence implies that the respondents are not certain if the interfaces are or are not interfaces.

If the interface locations between the boring locations on Figures 3.6-29 through 3.6-35 are schematic (lines 20 and 21), then how reliable are the cross-sections? The statement that "This does not diminish the usefulness of the...figures...to depict a geotechnical model of the site..." With the many weaknesses in the data set, the assumptions used to correlate lithology with CPT and drilling change data, the uncertainty of locating interfaces (What is the basis for 5 foot error?), the "transitional interfaces" and locations of borings not drawn to scale, all combine to suggest that the usefulness of the figures is markedly diminished for the intended purpose of the figures. Show that the information is adequate for the use to which it is put.

Response

The boring data are based on geotechnical samples, generally 1.5 feet in length, collected every 5 feet. The stated ±5 foot error in the geotechnical unit interfaces shown on the generalized geotechnical cross section is incorrect. The response should have stated that the interfaces are estimated to be within ±2 feet of the actual location.

The reference boring data were used to develop a geotechnical model of the site. Geologic information was collected from the same sources and by field reconnaissance at the surface. An evaluation of marker beds identified in soil borings drilled at the site shows no structural faulting across the site; however, a slight westerly to southwesterly dip of approximately one degree

does exist. This interpretation is based on the marker bed represented by the interface between soil stratum V and VI which is approximately 40 feet below the contact between the Cockfield and Cook Mountain formations. Stratum V is characterized by a silty fine sand and Stratum VI a silty clay laminated with silty fine sand. These soil strata along with diagrammatic cross sections are further discussed in the Geotechnical Exploration Report.

Variations to the planar dip of this interface of 5 to 10 feet vertically is attributed to facies changes as a result of lateral variation in depositional conditions and/or the interface not being encountered in the sampled interval. These variations are not linear and cannot be traced across significant portions of the site.

The following table SG-1 presents the elevations of this interface as encountered in the soil borings.

TABLE SG-1
SUMMARY OF STRATUM V/VI INTERFACE ELEVATIONS

SOIL BORING NUMBERS	INTERFACE ELEVATIONS (feet, MSL)
B-2	288.8
B-3	289.5
B-4	INE
B-6	282.0
B-7	281.6
B-8	285.0
B-9	INE
B-10	289.4
B-15	278.3
B-16	277.6
B-17	286.8
B-18	292.0
B-19	288.0
B-24	INE
B-25	INE
B-26	284.4
B-27	INE
B-28	289.6
B-33	291.0
B-34	299.3

SOIL BORING NUMBERS	INTERFACE ELEVATIONS (feet, MSL)
B-35	INE
B-36	275.3
B-37	INE
B-38	INE
B-39	INE
B-40	286.9
B-41	INE
B-42	INE
B-43	INE
B-44	INE
B-45	INE
B-46	INE
B-47	INE
B-48	283.2
B-49	INE
B-50	274.1
B-51	266.8
B-52	267.0
B-53	274.1
B-54	274.8

NOTE: INE = INTERFACE NOT ENCOUNTERED

Question 5:

Information in the last sentence in the response appears to be based on a literature search. The search is a first approximation and provides some direction for obtaining data. The response does not satisfactorily answer the question. Does the literature report the results of field work been conducted in the area of the Mexia-Talco Fault Zone to assess (1) the age of last movement along the fault and (2) the potential for future fault movement?

Response

The subject of fault movement in the region surrounding the CEC site has primarily been investigated through review of prior work by other authors. This has been done for the following reasons:

1. The region of the site has undergone detailed geologic investigation as part of hydrocarbon and mineral exploration. These investigations paid particular interest in evaluating the nature and age of faults.

2. Most of the fault zones are a considerable distance from the site.

Work conducted by Jackson (1982) to evaluate the seismic potential of the East Texas Basin considered the following fault zones: Mexia-Talco Fault Zone, faults in the central part of the East Texas Basin, and the Mt. Enterprise Fault Zone. This author concluded that all of these fault zones are of low seismic risk. All of the faults studied were normal-slip, moved steadily during the Mesozoic and Early Tertiary, and appear to be related to salt mobilization. Faulting ceased before the Quaternary except for potential movement at the western end of the Mt. Enterprise fault zone and small faults to the south of the Mt. Enterprise Fault Zone.

Prior work by Law Engineering to evaluate the potential of using salt domes in the East Texas, North Louisiana, and Mississippi Salt Basins as nuclear waste repositories, evaluated faults in the region for their seismogenic potential and found that the potential was low (Law Engineering, 1981c; SAR Reference 8).

Question 6:

Response paragraph on Border faults, last sentence. "This earthquake, however, may have been related to fluid withdrawal in a hydrocarbon production region (references cited)" (lines 5 to 7) but (lines 4 and 5) refers to "the inconclusive evidence...possibly suggesting movement was the Mexia earthquake of 1932" indicates that this issue is not resolved. The concept of fluid withdrawal as a possible mechanism, although appealing, is speculative. The issue should be addressed with more in depth analysis.

The response paragraph on Mt. Enterprise Fault Zone, suggests that the fault zone is related to salt creep. Such a relation is speculative and to accept the relation of the fault zone to salt creep. The indication of low seismic potential, based on the speculative relation, is not being responsive to the question. Do the respondents have data or evidence to support the relation or are they relying on Jackson (1982)? What information is provided by the microseismic activity, its distribution, the cumulative focal mechanism, trends, and so forth? If Pleistocene movement has been detected, what are the field conditions - could there be Holocene movement?

Paragraph on Growth Faults. The question relates more to surface rupture at the site and the resulting surface offset at the facility's foundation and tilting, not to earthquake shaking. The response does not clarify this point. An analysis should be done to assess (1) growth faults at the surface in the area of the site, (2) growth faults beneath the site or nearby, (3) growth rate, (4) potential for any and all of the above to occur within the life expectancy of the facility. Data may have to come from detailed field work and other methods of determining growth faulting at the site.

If the respondents interpretation is "that most or all of the tectonic earthquakes...occur in the pre-salt basement" (next to last sentence), then the respondent should place the tectonic earthquake beneath site, in the basement (see reply to respondent's comment, Item 2, this memorandum). The association of fluid withdrawal related to hydrocarbon production is an interesting model, and mathematical arguments can be presented to support it. The model may not be applicable.

Response

Fluid withdrawal or injection has been postulated to be a cause of a number of earthquakes within the region surrounding the site. A summary of this phenomenon was provided by Davis et al (1989; SAR Reference 15). They cite numerous examples of fluid

withdrawal with associated subsidence, in some cases with accompanying earthquakes. Table 2 (see p. S-11) summarizes the information they provide. Sharp et al (1991) evaluated widespread subsidence in the Trinity Bay - Port Arthur region of the Texas coast of the Gulf of Mexico. They concluded that depressurization of petroleum reservoirs is likely to be a major cause of the subsidence.

The Mexia, Texas earthquake of 1932 has been associated with oil withdrawal in the Mexia and Wortham Fields. By 1932, 112 MMbbl of oil had been removed from these fields. Production at the time of the earthquake was high. Evidence for the induced nature of this earthquake includes association of the highest Modified Mercalli Intensity with the area of highest hydrocarbon production (SAR Reference 15). Evidence against an induced event includes the occurrence of other earthquakes in the vicinity of the Mexia fault system which are not associated with oil fields.

Fluid injection has been cited as the potential cause of earthquakes in central and western Texas (Davis et al, 1989; SAR Reference 15) as well. A series of small earthquakes near the end of one segment of the South Arkansas Fault Zone has been associated with El Dorado South brine disposal field by Cox (1991). Based on the lack of prior seismicity, the correlation of seismicity with known disposal rates, and location of hypocenters in the basement beneath the well field, the author contends that there is a strong case for induced seismicity.

In summary, seismicity has been associated with the injection and withdrawal of fluids in numerous locations in the Gulf Coast region. In addition, subsidence has been associated with fluid removal. All of the seismicity and subsidence effects have occurred in the immediate vicinity of the pumping activity. The potentially induced earthquakes are all low in magnitude (< 5.0) and are interpreted to occur in the basement below the production horizons. The cause of both subsidence and earthquakes has been hypothesized to be the rapid changes in fluid pressure generated by human activity. While there is a potential for induced earthquakes to occur near the site, the probability is small since there is currently no significant pumping near the site. For more information on wells in the vicinity, see SAR Section 2.1.2.5.2.

Mt. Enterprise Fault Zone: The Mt. Enterprise Fault Zone is one of the few fault zones in the Gulf Coast Region which potentially has seismicity spatially associated with it. The most notable earthquake was the MMI VII 1891 Rusk, Texas earthquake. This fault zone was described by Jackson (1982) as being potentially related to movement in the Louann Salt. According to Collins et al. (1980; SAR Reference 10), the faults in the Mt. Enterprise system may represent hingeline effects between the East Texas

Basin and the subsiding Gulf Basin. While fault activity began during the Cretaceous, most movement occurred during the Eocene. Fault movement reduced as sedimentation slowed and the basal salt reached equilibrium.

Growth Faults: Growth faults have been and are formed in the Gulf Coast region at active depo-centers. Movement on the growth faults subsides as sedimentation terminates and the locus of deposition proceeds coastward. Movement along the growth faults occurs aseismically. Currently active growth faults are limited to the region of the current deposition (i.e., offshore area of the continental shelf).

TABLE 2
SUBSIDENCE AND EARTHQUAKES ASSOCIATED WITH FLUID WITHDRAWAL

FIELD	LOCATION	DATE	PREVIOUS PRODUCTION	MAXIMUM SUBSIDENCE	MAGNITUDE OF ASSOCIATED EARTHQUAKES	REFERENCES
Sour Lake	W. of Beaumont, TX	1929	73 MMbbl	50m	----	Sellards (1930), National Oil Scouts of America (1931), Sheets (1947)
----	Houston, TX	1943-1974	ground water	2m	----	Gustavson and Kreitler (1976), Verbeek and Clanton (1981)
Chocolate Bayou	S. of Houston, TX	1944-1974	----	0.5m	----	Grimsrud, et al. (1978)
Goose Creek	E. of Houston, TX	1944-1974	----	----	small	Pratt and Johnson (1926), Yerkes and Castle (1976)
Mexia; Wortham	Mexia, TX	1932	112 MMbbl	----	3.8[a]	Sellards (1933), Yerkes and Castle (1976)
East Texas	Gladewater, TX	1957	3.5 Bbbl	----	4.0,2.5,2.5,2.5[a]	Docekal (1970), Yerkes and Castle (1976)
Imogene (gas); Flashing (oil)	Flashing and Pleasanton, TX	1973-1984	----	----	3.9,3.4	Pennington et al. (1986)

Source: Davis et al. (1989)
a Magnitudes calculated using relationships developed by Sibol et al. (1987).

Question 7:

Although the Interior Salt Basin shows a short history of low seismic activity, such activity does not address the issue. In Louisiana, earthquakes do not appear to be associated with tectonic seismogenic zones. The local extension of the non-association is that the earthquakes, regardless of their frequency of occurrence, can be assumed to occur anywhere in the Basin.

The response describes several interesting and important relations. Paragraph 1 discusses the Rusk, Texas earthquake of 1891 as a shallow earthquake that had a low magnitude and a relatively high modified mercalli intensities. This relation has been recognized in other parts of the central United States where tectonic seismogenic zones have not been clearly recognized (Table 23, NUREG/CR-1577), and clearly demonstrates that very shallow but small magnitude earthquakes can generate large intensities and are not necessarily restricted to known seismogenic zones.

The response, paragraph 2, states that the Interior Salt Basin is stable in the modern tectonic environment, that the Lou Ann Salt has stopped moving, that the basins are isostatically stable, that minor movement is related to tilting and evidenced by flexure, that transmission of stress from the North America plate to the crust in the Gulf Coast is not well coupled because "of the faulted nature of the basement, and that growth faults occur toward the coast and move aseismically."

None of these statements address the question regarding the basis for the magnitude 4.9 as the maximum magnitude for the Interior Salt Basin and do not address the important issue of small magnitude shallow earthquakes and associated large intensities. The response statements are not internally consistent in that (1) the Interior Salt Basin is stable in the modern tectonic environment and (2) transmission of stress from the North American Plate to the crust in the Gulf Coast is not well coupled because of the faulted nature of the basement. Firstly, the comment that stress transmission across faults in the basement ignores the stress transmission and potential accumulation in the blocks between the faults. Secondly, that stress accumulation has the potential for release and producing earthquakes. Thirdly, the respondent's comment regarding stress transmission is not defensible unless in-situ stress measurements are presented. Growth faults, although reported to move aseismically, have the potential for ground surface rupture and/or tilting that could affect the proposed facility.

The response states that the maximum magnitude of 4.9 is based on seismicity and tectonic considerations. The tectonic considerations cannot be fully defended (see previous paragraph). The use of historic seismicity in a region is generally accepted as one method for assigning a maximum magnitude and the value of 4.9 may be reasonable. Commonly, a 1000-year return period earthquake is chosen as the maximum event or on the basis that the largest historic earthquake represents the maximum event. From the information contained in the response (paragraph 1) the largest earthquake in the general area was the 1891 Rusk, Texas, event, magnitude 4.1 (although the revised Law Engineering report, March 1992, p. 3.6-25 gives the January 7, 1981, event as an approximately value of magnitude 3.8).

Response

The basis for assigning a maximum likely earthquake of $m_b = 4.9$ to the Interior Salt Basin Source Zone is rooted in previous work by Law Engineering on reevaluating seismic hazard in the central and eastern United States (Law Engineering, 1986, SAR Reference 17). Evaluation of maximum earthquake is based on regional seismicity and nature of faulting.

Seismicity: The rate of earthquake activity in the region is among the lowest east of the Rockies. In general, the rate of modern seismicity is considered to be a reflection on seismicity in the recent past and the near future. The computation of a 1000-year event is generally appropriate in areas of relatively high seismicity (Nuttli and Herrmann, 1978). Nuttli and Herrmann (1978) found that, in relatively active areas, the 1000-year event was roughly equivalent to the maximum earthquake. The Gulf Coast is not such an area.

The largest recorded earthquake within the Interior Salt Basin had an $m_b = 4.1$ which is 0.8 magnitude units below the assigned maximum.

Faulting: The date of last movement of most faults in the region is interpreted to be Eocene. In addition, most of the fault movement was normal slip which is inferred to be related to salt movement or sedimentation. In any case, such movement does not coincide with the compressive stress field dominant in the eastern United States. One composite focal mechanism from earthquake first-motion data is available from earthquakes located in the vicinity of the Mt. Enterprise Fault Zone (Pennington and Carlson, 1984; SAR Reference 51). This mechanism shows normal faulting with a small strike-slip component with a fault plane oriented either N15E or N75E and dipping 62 deg. southeast. Another focal mechanism is available for the 1983 Lake Charles, Louisiana earthquake near the Louisiana coast (Stevenson and Agnew, 1988; SAR Reference 55). The mechanism shows predominantly norma-slip faulting on either a N55E dipping

40 deg. southeast or N80W dipping 64 deg. northeast. In the Gulf Coast, major normal faults dip in a southerly direction.

The nature of faulting in the region favors a low maximum magnitude. In-situ stress measurements, other than the few composite focal mechanisms, are generally not available for the region. Those that are available are from geothermal fields or are from above the basement. In either case, these do not represent the stress conditions at seismogenic depths.

The depth of the near-field DBE was selected to be the top of the crystalline basement. This is considered to be the shallowest possible depth for a damaging earthquake. While many of the earthquakes in the region are considered to be shallow, none are believed to be shallower than the top of the basement. Even the events possibly caused by fluid injection or withdrawal are considered to have occurred within the basement (Cox, 1991; Collins et al. 1989; SAR Reference 10). It is conceivable that shallower earthquakes can occur but the strength of the subsurface materials would likely limit the magnitude of the event to non-damaging size.

Question 8:

Because the source(s) of earthquake(s) in the area have not been related to some seismogenic structure, the location of the next event cannot be reasonable (statistically?) located. In such cases, the general assumption is to consider the events to be randomly distributed. The conservative approach is to locate the maximum event under the site. And, because the respondent appears to be implying some of these events are shallow, the near-field event should be located under the site and shallow. The design spectra, etc., should be based on this proposed revision.

The response probability analysis indicates (last paragraph) that the near-field DBE occurrence directly beneath the site is "minuscule" and the 500-year return period is "insignificant." Statistical analyses of the available earthquake data in the area are based on very few earthquakes, and they all have small magnitudes and therefore tend to cluster. Consequently, the results of the analyses cannot be especially reliable for extrapolation. The respondent should identify the limitations of the analyses and errors based on a small population and a short history of earthquakes. The terms "minuscule" and "insignificant" are subjective.

Response

The near-field DBE is based on a probabilistic approach. The magnitude of the event and the distance at which it occurs are tied to the return period. As the near-field DBE is moved closer to the site, the magnitude of the event must be reduced in order to maintain the same probability of occurrence.

In actuality, the near-field DBE was calculated to have a 2374 year return period. This longer return period was used to obtain an event large enough to cause potentially damaging ground motion and to circumvent uncertainties in the seismicity, due to the low number of earthquakes on which the return period calculation is based.

Question 9:

A common approach in seismic hazard analysis is to use estimates from several different workers. The references supplied in the original question all provided estimates of return periods or maximum earthquake different than those proposed in the SAR. The estimates of the other workers either had high magnitudes or shorter return periods. The response does not provide adequate data or analysis to support the selection of a magnitude 6.7 rather than a magnitude 7.4 as the CEC design basis. The last paragraph refers to Johnson and Nava (1984). The response does not provide an adequate support the selected value of 6.7 is more appropriate than a 7.4 and that the shorter return period provided by others is not appropriate.

Response to Question 9:

The calculation for determining the magnitude of a 500 year return period event is dependent on the area of the source zone being considered. For this reason, the return periods calculated by different workers can only be compared when the source zones are exactly the same. Johnston and Nava (1984; SAR Reference 47, 1985) calculate the return periods for New Madrid source zone configurations larger than the one used in this study; consequently, the magnitude for a 500 year return period is larger. Johnston and Nava (1984, 1985) did not actually report the return period for the maximum New Madrid event.

Question 10:

The comments provided that "more modern interpretations" is not referenced in the response. Are these interpretations by the respondent or others? If data and analyses are available, the references should be cited. The last sentence in the next to last paragraph indicates that "It is our interpretation that if a town had existed on the CEC site in 1811, the reported MMI would be VI." What is the basis for this interpretation?

Response:

The isoseismal maps frequently cited from the literature for the 1811-1812 New Madrid earthquake sequence (Algermissen, 1983 and Bolt, 1978) are based on widely separated intensity reports from towns in deep alluvial river valleys. Isoseismal maps based on these data will tend to overestimate the intensity encountered in non-alluvial locations such as the site. In addition, the isoseismal maps cited above are based on older intensity estimates for the region. More recent interpretations (Street and Nuttli, 1984; SAR Reference 18) use lower intensity estimates. Based on these intensities and the location of the site on high ground, it is estimated that the MMI encountered at the site during the New Madrid earthquake sequence did not exceed VI.

It should also be noted that the maximum site intensity for the New Madrid earthquake sequence is reported for comparison and reference. The site intensity was not directly used in the probabilistic analyses.

Questions 11, 12, 13, and 14:

The response does not:

- address uncertainties or estimated errors,
- include accelerograms,
- provide estimates of probabilities of exceedance, and
- provide a discussion of earthquake characteristics (acceleration, velocity, displacement, duration of strong ground shaking, frequency or period).

Response to Questions 11, 12, 13, and 14:

The DBEs used in this study were defined as events with a 10% probability of exceedence in 50 years (a 500 year return period); therefore, the uncertainty in the occurrence of the DBE is fixed by definition. The location of the near-field DBE was calculated using a 2374 year return period.

Accelerograms were included with the earlier response, and are shown on SAR Figures 3.6-42 through 3.6-45. Their characteristics are listed in SAR Table 3.6-16. The velocities and displacements have been added to Table 3.6-16 (see Table 3.6-16, page S-20). The duration of strong shaking is discussed in Section 3.6.2.2.1. The frequency content of the time histories is demonstrated by the response spectra in Figures 3.6-26 and 3.6-27.

Acceleration time histories recorded from naturally occurring earthquakes were used to model the design spectra for the near-field and mid-field DBE. Because the designer specified an interest in frequencies higher than normally used, earthquake time histories which recorded these higher frequencies were selected. In addition, it was desirable to use time histories from eastern United States (mid-plate) earthquakes because of potential differences in the source spectra for events in this region when compared to events in the western United States (plate-margin events).

Near-field DBE: The near-field DBE was modeled using acceleration time histories from the shallow, March 31, 1982 aftershock of the New Brunswick, Canada earthquake. This time history was provided at 200 samples per second. This allows useful spectral information up to 100 Hz. No appropriate acceleration time histories are available for the region of the site (Louisiana, Texas, Oklahoma). Some time histories are available for the New Madrid area (Arkansas, Missouri, Tennessee) but these are not located on free-field hard bedrock sites (Herrmann, 1977).

Mid-field DBE: The mid-field DBE was modeled using acceleration time histories from the 25 km depth 1988 Saguenay, Quebec, Canada earthquake. The time histories used were from approximately the same magnitude and distance as the mid-field DBE; so, only minor scaling was required.

Far-field DBE: No appropriate time history was available to model the m_b 6.7 far-field event. All time histories available for this magnitude and 'istance combination are too low in amplitude. Using a time history recorded closer to the source but scaled to the acceleration for the appropriate distance would likely generate a response spectrum deficient at some frequencies. A synthetic acceleration time history was used.

TABLE 3.6-16
EARTHQUAKE RECORDS USED FOR
SITE RESPONSE ANALYSES

EARTHQUAKE	RECORDING SITE	GEOLOGY	MAGNITUDE	DURATION (SEC)	DISTANCE FROM ZONE OF ENERGY RELEASE (miles)	SCALED A_{max} g (PEAK)	Scaled V_{max} (cm/s)	Scaled Disp (cm)	COMPUTED CEC SITE SURFACE A_{max} g (PEAK)
Near-Field New Brunswick 3-31-82	Mitchell Road	Bedrock	m_N = 4.8	1	2.5	.045 hor .033 ver	0.54 0.17	0.015 0.004	0.021 hor 0.0098 ver
Mid-Field Saguenay 11-25-88	Riviere-Ouelle, Quebec	Bedrock	m_b = 5.9	4.5	71	.040 hor .028 ver	2.20 1.57	0.28 0.14	0.047 hor 0.041 ver
Far-Field (Synthetic)	NA	NA	m_b = 6.7	26	227	.022 hor .0157 ver	-- --	-- --	0.024 hor 0.027 ver

<u>Attachment A</u>

Question 15:

The use of RVT (Random Vibration Theory) modelling may have value in providing information that may be used in providing information for design.

An actual time history of acceleration is a preferable source of data for analysis compared to Seed's method using data based on SPT (Standard Penetration Test). The data from Seeds method is questionable for application to developing ground response spectra.

Response

After the magnitude and distance of the 500 year return period DBEs were determined, the RVT method was used to estimate the ground motion (acceleration, velocity, displacement, duration) at the site. Ground motion was modeled using frequency dependent attenuation appropriate to the central United States. More rapidly attenuating Gulf Coast attenuation rates were not used since the bulk of the hazard originated in the central United States (New Madrid Fault Zone, Reelfoot Rift, and Ouachita source zones). The input time histories used in the SHAKE program were scaled using the accelerations determined by the RVT method.

[Question 17]:

If Seed's method is to be used for assessing the likelihood of liquefaction at the site, then the respondent should provide information (analyses, etc.) demonstrating that Seeds curves are applicable to the test from the site. If the curves are not appropriate, how do the differences affect the respondent's results?

Response

It is recognized that Seed's data are largely from the west coast and other very active, high strain-rate, generally interplate or plate-marginal seismic zones. The site is a low activity, interplate zone. Thus, some differences in seismic source characteristics (mechanisms, focal depths, stress drops) can be expected. It is unknown as to the effects of these differences upon the results obtained using Seed's method for the site environment. However, Seed's methods were shown to work well in predicting liquefaction and ground failures associated with the 1988 Saguenay, Quebec earthquake (Tuttle et al, 1990). This suggests that Seed's methods can be successfully applied to liquefaction problems associated with east coast earthquakes.

An alternative to Seed's method is the cyclic strain method. A cyclic strain approach to the liquefaction problem (Dobry, et al, 1982) is based on the premise that pore water pressure buildup during cyclic shear loading of sand is controlled mainly by the magnitude of the cyclic shear strain. This premise leads to the conclusion that shear modulus is the main parameter controlling pore water pressure buildup in the field. An important practical consequence is that measurements of in-situ modulus at small strains, which can be obtained from geophysical measurements of shear wave velocity, should be used for predicting pore pressures.

The method requires estimating both the seismic strain induced in the sand layer and the effective shear modulus of the layer during the earthquake. The method is based on measuring the shear modulus (computed from the shear wave velocity) in-situ at small strains, G_{max}, using geophysical techniques, and on performing cyclic strain-controlled tests in the laboratory to determine: (i) the modulus reduction values, G/G_{max}, (ii) the value of threshold strain at which pore pressure increases begin, and (iii) the pore water pressure buildup versus cyclic shear strain and number of cycles.

Dobry, et al (1982) state that the modulus reduction curve for sand given by Seed and Idriss (1970) and used in the SHAKE computations for this project has been confirmed by other investigators. Thus, the Seed and Idriss modulus reduction curve can be used for calculations of the induced cyclic strain.

The computer program SHAKE was used to compute the soil's response to the various earthquakes for generating the response spectra. This calculation also produces the cyclic shear strains throughout the soil profile.

The threshold strain, according to Dobry, et.al., (1982) is 0.01 percent for a wide variety of soils. This strain, if not exceeded, means that the cyclic loading does not generate pore pressures in the soil. The effective cyclic shear strains determined from SHAKE analyses were compared with this value. The effective cyclic shear strains in the stratum IV and stratum V sands in the analyses by SHAKE did not exceed .01 percent.

Thus, the cyclic strain approach predicts no pore pressures will be induced by the earthquakes. This indicates that the liquefaction from the earthquake loading is not a risk, which is the same conclusion reached from application of Seed's empirical "stress-based" procedure.

Thus, two independent methods predict no liquefaction of the stratum IV and stratum V sands.

4.3 Facilities Design Criteria

Questions 1, 2, 3, 4, and 5:

The questions request documentation which demonstrates function of Class I and II components and systems under design basis loads. It is anticipated that the analysis (structural, hydraulic, etc.) utilizes mass, force, moment and energy balances to demonstrate that the response of the component to the enforced load does not contribute to a release in excess of the NUREG-1391 guidelines. While walkdowns of the facility do play an important role in the installation and review process, they are not an acceptable substitute for engineering analysis of the components and systems at the preconstruction phase. Please provide the requested analysis. The following paragraph provides an example of the type of issues which need to be addressed. Although the example refers to the feed autoclave, similar analyses would need to be presented for the product blending and sampling systems if the meteorological analysis presented in responses under Section 4.6 identifies a threshold release quantity in the range of possible releases from these systems.

Example 1: The loads considered in this case are forces and moments produced by the design basis earthquake and the system under analysis is the feed autoclave and associated valves and piping. Structural analysis of the response of the autoclave mounts, containment, and interior and exterior valves, piping, instrumentation and controls should be presented. If all components sustain the loads without failure the analysis is complete. If selected components fail under the projected loads, additional analysis is required to demonstrate conformance to the NUREG-1391 guidelines. For example, if the primary autoclave exit pipe fails, then structural and control analysis is needed to demonstrate that the heater, heater fans, and outlet valve PV-135 are shut-off or closed. If it is not possible to demonstrate that the valves close, then energy and/or transport analysis is required to demonstrate that the released quantity is less than the allowable threshold. Such calculations would include consideration of the quantity of sensible heat present and estimation of the maximum amount of UF_6 which could be evaporated under the given conditions.

Example 2: The SAR reports that the feed purification desublimers have design capacities of 2000 kg. If the threshold release quantity referenced above is less than 2000 kg, provide structural analysis to demonstrate that the desublimers and associated inlet and outlet piping and valves do not fail under design basis loads. If it is not possible to demonstrate the integrity of these systems under design basis loads, provide analysis (i.e., thermal, transport, etc.) which demonstrates that NUREG-1391 limits are not exceeded in the event of such potential failures. If evacuation is not proposed in the Site Emergency Plan, do not take credit for length of time to evaporate/sublime. If the actual quantity of UF_6 is cited as less than 2000 kg, provide analysis to demonstrate that the feed autoclave or purification station load cells and the associated instrumentation and control systems maintain function under design basis loads. If inability to supply heat of evaporation/sublimation is cited, provide analysis of the function of FREON supply control system instrumentation, valves, and logic to support the position.

The only credible UF6 release scenario which could occur at the CEC that exposes the public to values of uranium and/or hydrogen fluoride (HF) beyond those stated in NUREG-1391 is one in which multiple cylinders of liquified UF6 and their companion autoclaves fail simultaneously. A discussion of the failures analyzed and the results of these analyses are presented in SAR section 9.1 and 9.2. The analyses include failure of UF6 piping inside and outside an autoclave (reference SAR sections 9.1.3 and 9.1.6), and failure of a desublimer (reference SAR section 9.1.2).

SAR Table 9.2-2 provides a summary of the exposures predicted under worse case meteorological conditions and release conditions for three different release cases.

4.6 SUMMARY OF STRUCTURES, COMPONENTS, AND SYSTEMS CRITERIA

Question 1:

Provide a copy of the FDI report on identification of structures, components and systems important to safety.

SAR section 4.6 contains the information regarding the identification of structures, systems, and components (SSC) important to safety (i.e., safety-related). Since the only credible UF6 release scenario which could occur at the CEC that exposes the public to values of uranium and/or hydrogen fluoride (HF) beyond those stated in NUREG-1391 is one in which multiple cylinders of liquified UF6 and their companion autoclaves fail simultaneously, the autoclave instruments for air temperature and air pressure have been designated as safety related. Since there are three instruments for temperature and pressure for each autoclave, this ensures a redundant and diverse method for preventing an accidental release from cylinders containing liquid UF6.

Question 2:

The response to this question indicates that the threshold quantity of UF_6 which, if released through the stack, would exceed NUREG-1391 limits is 3700 kg. NRC staff analysis, conducted using the methods of Regulatory Guide 1.145 and the revised meteorological data, indicates that this threshold quantity may be as low as 1800 kg. The 95 percent over-all concentration per unit release factor (X/Q) was estimated as approximately 1.6×10^{-5} s/m^3. Provide detailed documentation supporting the proposed threshold quantity, including description of the dispersion analysis and cumulative distribution for X/Q. If method used in calculation of the X/Q differs from that of Regulatory Guide 1.145, provide a justification for use of the alternative method.

SAR section 9.2 provides the detailed description of the UF6 release scenarios including the amount of UF6 released, the features of and justification for the dispersion model used, and the Chi/Q factors used. Specifically, SAR section 9.2.4 provides a detailed description of the atmospheric dispersion analysis used, the exposures predicted by the analyses, and the uncertanties associated with the analyses.

6.4.10 Control Systems

Question 1:

The question requests upper level descriptions of the operating procedures which would provide an understanding of the integrated operation of the feed, enrichment and take-off systems. The response states that a separate letter of March 31, 1992, transmits the information. The response supplies useful information of system operation at the local level and on some equipment interactions (e.g., feed autoclave and desublimer). Is there an integrated, over-all control system/strategy implemented at the control room level? If so, please provide a description and diagram representing this control scheme.

CONTROL ROOM FUNCTIONS

Focal Point

The control room acts as a focal point for plant operations. It is manned 24 hours per day.

Communications Channel

The control room is a major communications channel and is equipped with telephones and radio equipment.

Data Management

Almost all parameters, measured by the instrumentation installed in the plant, are transmitted to the control room. The control room is provided with data processing equipment to enable the management of this transmitted data. Facilities are typical of modern data processing equipment and include:

- electronic screen mimic displays of plant flow sheets showing valve positions, parameter values, and equipment states in real time.

- event logging which enables a chronological record of plant state changes to be obtained. A plant state change is defined as any discreet alteration (e.g., valve switching from "open" to "close", an analog value of a parameter moving through an alarm, trip or reset level). This facility is particularly useful in reconstructing a series of events which has occurred quickly.

- data trending, statistical treatment, graphing, and hard copy facility are available.

- retrievable historical data is available for defined periods of time. This facility is particularly useful in investigating if a trend had developed, leading up to a plant alarm level being reached.

- alarm management provides for every parameter on the plant which reaches an alarm level, information is transmitted to the control room in a manageable fashion.

In line with all other plant personnel, the control room operator has a series of routine tasks per shift to perform. These may, for example, range from trivial (e.g., changing paper in hard copy devices) to more substantial. A more substantial task, for which additional personnel may be drafted into the control room, may be the execution/supervision of plant trip testing.

A further important feature of the control room operator's work is communication with the shift supervisor. The shift supervisor visits the control room throughout the shift, to gain information from the data processing equipment, to obtain updates from the control room operator and to use the communications equipment to contact other members of the shift team.

Limited Control

As explained in response to a previous question on SAR Section 6.4.10, most operator initiated control (e.g., state switch manipulation) and operator intervention in the process is conducted locally. For some equipment which is rarely switched (e.g., cascades and UF_6 pumps) the state switches are in the control room and are altered by the control room operator as and when required.

Disturbance Overview

In the event of a system or plant-wide disturbance, the control room is the place where an overview of the state of all systems can be obtained. A typical disturbance might be an irregularity in the local electricity supply. In such events the shift supervisor together with a small team gather in the control room where they use the data processing equipment to monitor the plant state. The plant is designed to be auto-tripping, autoprotecting and auto-resetting. Consequently the control room team need only monitor that, following the passage of the disturbance, the plant has reset satisfactorily. In the event of a piece of equipment not resetting satisfactorily, the control room operator directs a local operator to intervene directly.

Equipment Switching

For equipment which runs continuously and rarely experiences state changes, the state switch controls are located in the control room (e.g., cascades and UF_6 pumps).

CONTROL ROOM OPERATOR FUNCTIONS

Communications Channel/Operations Coordination

The control room operator performs a key role in the day-to-day operation of the plant.

The control room is manned 24 hours per day by a series of operators, usually two per shift are assigned the task of sequential manning. The control room operators maintain an ongoing log of significant activities and ensure turnover briefings are conducted at each control room operator change.

The control room operator periodically reviews the plant state as given by the data processing equipment. One of his main tasks is to maintain an up-to-date overview of the plant. Such an overview would, for example, comprise information on which auxiliary equipment is online, which is on standby and which is on maintenance. A particularly important overview task is awareness of vessel state (i.e., which vessels are filling, which are emptying, which are on standby available for use).

A further important task is alarm management. All alarms which are generated on the plant are annunciated locally and in the control room. The control room is always manned but the local alarms out in the plant may not be. Consequently, one of the control room operator's task is, for each alarm, to confirm that a local operator is aware of the situation and is dealing with it. This local operator/control room operator communication is also required to confirm details of the alarm (i.e., what has caused the alarm and what remedial action is being taken).

In support of the overview activity, described above, the control room operators also keep in communication with the local operators such that they are up-to-date with equipment changes and vessel state changes.

CONTROL ROOM STRATEGY SUMMARY

The above information demonstrates that in the LES plant, the control room functions primarily as an information management and communications center. Very little direct control, in the sense of commanding plant state changes by control room switching, actually occurs.

Question 2:

The question requested Failure, Modes and Effects Analyses (FMEA) referenced in the SAR. The response states that a copy of a autoclave FMEA was submitted by letter dated March 31, 1992. The letter has not been received. Please provide an additional copy of the report.

A copy of the FMEA for a Contaminated Autoclave is enclosed.

New Question:

A complete understanding of control system logic is required for those systems which are potentially important to safety. Provide the following:

- For the feed, sampling and blending autoclaves, logic diagrams which represent the function of the control systems for the heating elements and fans. For example, in the mechanical flow diagram for the feed autoclave these would be in the diamond immediately above JC-125 and immediately below HS-155.

- For the feed and blending autoclave pressure control valves (PV-135), logic diagrams representing the function of the control system shown in the mechanical flow diagrams immediately below PX-135.

- For the feed, sampling and blending autoclaves, logic diagrams for the door control systems. For example, this would be represented by the diamond immediately above element HS-152 in the feed autoclave mechanical flow diagram.

PRESSURE CONTROL PHILOSOPHY FOR AUTOCLAVE CHANGEOVER

For the LES plant the configuration of autoclaves per plant unit are to be any one of the four to be online, with one other being on hot standby ready to replace the online autoclave should its container become empty. The remaining two of the four being in any one of the remaining state conditions.

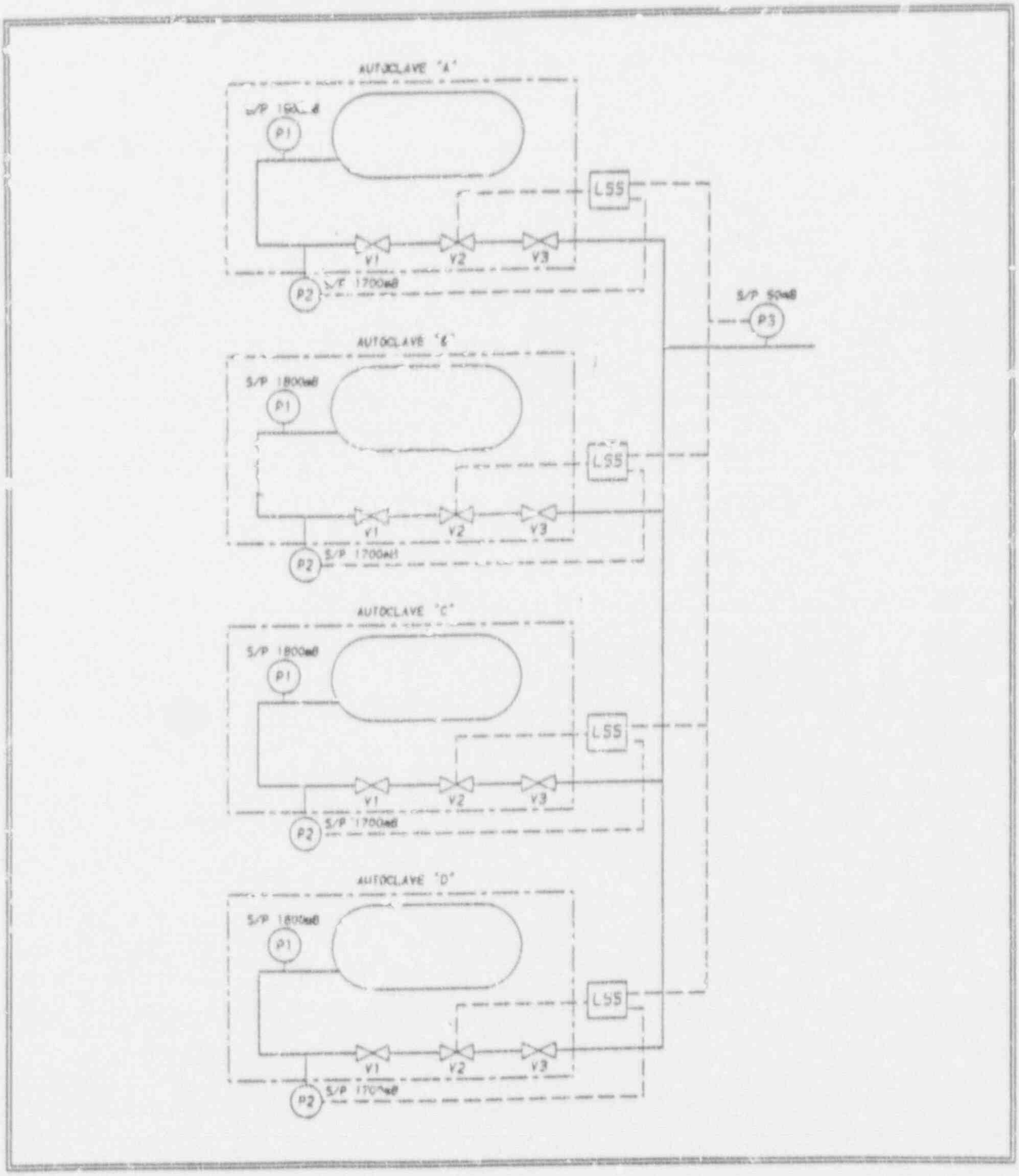
The following is a description of operational philosophy of how pressure control of the feed autoclaves is achieved during online, emptying and standby conditions. The diagram on the next page (Scheme of Pressure Control Philosophy for Autoclave Changeover) provides a schematic to assist in the understanding of the control system logic.

Initial conditions:

- Autoclave A online and feeding an operating unit (cascade)
- Autoclave B on hot standby
- Autoclaves C and D in some other mode

The feed container contents of autoclave A are being controlled at a pressure of 1800 mBars by a cascaded pressure/temperature control loop AP1. A pressure control loop AP2 measures the pressure upstream of the autoclave pressure control valve AV2 and has a setpoint value set at 1700 mBars. The feed header pressure downstream of the autoclave is being measured by a pressure control loop P3. The output signal from loops AP2 and P3 are fed to a low select circuit that will only pass one of the signals, the one that is relatively lower through to the pressure control valve AV2. Since the container at this moment is being maintained at a pressure of 1800 mBars, the output of pressure loop AP2 which is set to control at 1700 mBars is configured to be high with respect to the output of pressure loop P3 and therefore the low select circuit passes the P3 signal to modulate valve AV2.

Scheme of Pressure Control Philosophy for Autoclave Changeover



Attachment A

The feed container contents of autoclave B are being controlled at a pressure of 1800 mBars by a cascaded pressure/temperature control loop BP1. A pressure control loop BP2 measures the pressure upstream of the autoclave pressure control valve BV2 and has a setpoint value set at 1700 mBars. As with autoclave A above, the feed header pressure downstream of the autoclave is being measured by a pressure control loop P3. The output signal from loops BP2 and P3 are fed to a low select circuit that will only pass one of the signals, the one that is relatively lower through to the pressure control valve BV2. Since the container at this moment is being maintained at a pressure of 1800 mBars, the output of pressure loop BP2 which is set to control at 1700 mBars is configured to be high with respect to the output of pressure loop P3 and therefore the low select circuit passes the P3 signal to modulate valve BV2. But since autoclave B is on hot standby the isolation valves on either side of valve BV2 are closed.

As the container in autoclave A becomes exhausted of feed, loop AP1 is unable to maintain a container pressure of 1800 mBars. The container pressure starts to drop and as it approaches 1700 mBars the output signal of loop AP2 becomes less than that of loop P3, at which point the low select circuit selects control of valve AV2 to loop P2. In this mode autoclave A is in empty mode and cannot provide the required flow to the plant. This transition of autoclave A from online to emptying triggers the selection of autoclave B from hot standby to online. Selection to online opens the isolation valves on either side of control valve BV2 enabling autoclave B to feed the operating unit. The pressure downstream of the autoclave still needs to be maintained at 50 mBars by P3 but this is achieved now by controlling autoclave B, with the feed from autoclave A being accounted for.

When the container in autoclave A is truly exhausted, that autoclave is isolated from the operating unit for container replacement, and some other autoclave may be put in in standby mode.

Note during normal running the output signal of loop P3 is less than the output signal of loop xP2 since,

i) loop xP2 setpoint is 1700 mBars

ii) loop xP2 measured variable or process variable is 1800 mBars

iii) control action is direct acting therefore output signal is maximum (e.g., 20 mA)

iv) loop P3 setpoint is 50 mBars

v) loop P3 measured or process variable is nominally 50 mBars

vi) output signal of P3 will be approximately mid scale (e.g., 12 mA)

Therefore, P3<xP2.

As the container pressure though drops to 1700 mBars

i) loop xP2 setpoint is 1700 mBars

ii) loop xP2 measured or process variable nominally 1700 mBars

iii) output signal of xP2 will be mid-scale (e.g., 12 mA)

iv) loop P3 setpoint is 50 mBars

v) loop P3 measured or process variable is falling below 50 mBars

vi) loop P3 output signal is increasing (e.g., > 12 mA)

Therefore xP2<P3.

During this transitional stage, loop P3 may be exercising control over both autoclave valves simultaneously.

9.2 Staff Qualifications

New Question:

Explain why a person in charge of the technical management of various aspects of plant operation, e.g., managers for the Claiborne Enrichment Center, for Quality Assurance, for Emergency Preparedness, for Health Physics, for Chemistry, for Industrial Safety, etc., should not be required to have a diploma in a related field of technical training (SAR 11.1.4). In our view, such training is desirable to enable a person to understand those disciplines under his responsibilities.

SAR section 11.1.4.1 has been revised to require a BS in engineering, science or related field for the CEC Manager, Superintendents, Projects Manager, and Criticality Specialist within the Projects group. This requirement has not been specified for other individuals (e.g., Quality Assurance) because it is too restrictive. The personnel assigned the responsibilities specified in SAR section 11.1.3.1 will be well qualified and will have the necessary training, background, and experience to operate the CEC safely and efficiently. The specific requirement for formal education may needlessly exclude certain individuals from these positions. These revisions also satisfy similar concerns concerning formal education for managing the criticality safety program for the CEC.

Wind and Tornado Design

Question 2:

The response does not explicitly state that the reinforced concrete thickness will satisfy the thickness and ACI requirements. The formula used for scabbing thickness is for integral reinforced concrete. The prefabricated T-sections do not have the necessary thickness alone. Either the concrete above the T-section must alone satisfy the thickness requirement or it must be so bonded as to respond as though integral with the T-section. Any concrete credited toward the thickness to preclude scabbing must fully satisfy the requirements of ACI 349 for reinforced concrete. This requirement includes areas such as minimum reinforcement, steel cover and others.

NOTE: This question was marked #2, however, it appears that the text is addressing the original Question 3.

The topping over the precast roof members will be designed to satisfy all applicable requirements of ACI-349. Some applicable ACI 349-85 requirements include section 7.7.1 (cover) and section 7.12 (Minimum Reinforcement). The topping will also be bonded with the precast member to ensure response as an integral section. Conformance with these requirements will provide a section which will act as a unit to satisfy the total thickness requirements, including scabbing, for tornado missile impact. As an alternate, an unbonded, reinforced, topping could be used, provided the topping alone satisfies the total thickness required.

Question 7:

The response does not appear to address the NRC concern. Differential pressures resulting from the worst case tornado and tornado locations should be presumed for all interior walls/partitions until shown not to exist or to be negligible.

The interior walls are designed for the worst case loading from the Design Basis Tornado, unless it can be shown positively that the interior walls are protected by walls and doors which are properly designed for DBT pressures and missiles. This covers the case of an exterior wall or roof failure in the case of a DBT. The design considers the worst case tornado (pressure, direction, missiles) and location (around and over the building).

Question 10:

The response indicated as acceptable action but the calculations implementing the action were not provided with the response. Submittal of the revised calculation is required.

A copy of revised calculation DC-SE-0001-SD is enclosed.

Question 12:

The "welded metal stops" referenced in the response to be on either side of a roof member and capable of transmitting the shear from the roof to the wall do not satisfy the concern. The stops would transfer shear force from the roof to the wall perpendicular to the member but would not serve to transfer force from a wall to members perpendicular to the wall. The roof is apparently a one-way structural system, thus members would not be supported on all four sides of a rectangular area. The descriptions of distribution of windload (sheets 47 and 48) include cases where lateral loads on walls which are on opposite sides of a rectangular roof section are to be transferred by the same roof section, which is incompatible with expansion provisions for the roof support members (e.g., distribution of pressure loads on walls 9 & 10, 1 & 11, 2 & 12, 3 & 13, 14 & 32/38, etc.).

In order to clarify this issue on the transfer of force with an expansion joint at the roof, it is helpful to explain the transfer of forces on one example area of the building. For this example, consider the center high bay roof area, which is bounded by columns 6 and 11 on the west and east side, and columns A and J on the north and south sides, respectively (refer to the roof plan on sheet 12 of calculation DC-SE-0003-SD). The expansion joint detail is shown on sheet 18 of this calculation. As can be seen from the detail, the west side of the wall is the expansion side, which allows the roof and the wall to move independently in the east and west direction, but still transfers shear forces in the north-south direction.

First we will look at north-south forces, whether wind or earthquake. Forces from the top of the walls at J and A will be transferred to the roof diaphragm by means of solid connections between the roof and the walls. This solid connection is indicated in the building section on sheet 19. The north-south forces on the walls at 6 and 11 will be taken directly to the foundation by those walls in shear. The north-south forces from the diaphragm will also be taken by the walls at 6 and 11 in shear. The connection at wall 6 is a solid connection and the connection at 11 is the special expansion joint. At this connection the force will be transferred by bearing on the welded metal stops on the north and south side of the roof beam web.

For the east-west forces, the distribution is slightly different. The force from the wall at column 6 is directly taken by the roof diaphragm between column lines 6 and 11. The east-west force on the wall at 11 is taken by the adjacent diaphragm, between 11 and 16. The force travels to this diaphragm due to the expansion joint which prevents east-west forces from being transferred on

the west side of the wall at 11. The forces in the diaphragm will then be transferred to the north and south walls by solid connections.

The behavior of the other diaphragm units in the building will be similar to this one. The end result being a positive transfer of forces while still allowing expansion movement to take place.

Question 13:

It is not clear from the response that the maximum dynamic pressure (due to wind velocity) shall not be reduced due to atmospheric pressure change. The statement that "APC pressure always acts outward" is accepted for a perfectly sealed space, however, when there is some venting, the interior of a structure may have static interior pressure less than that outside at the time of maximum wind pressure normal to a wall. The interior pressure may have been reduced and then be increasing as the tornado position changes.

Accepted practice for design of tornado effects is to consider that the atmospheric pressure change will only act in an outward direction. This applies to a sealed building, as is stated on pages 37 and 38 of the Mehta report "Tornado and straight wind speed study for LES uranium Enrichment Plant Site." For an unsealed building, the APC is assumed to equalize on the inside and the outside such that the differential is negligible.

Question 16:

Stiffness of the lateral support at 45 ft height relative to the stack in the different directions is not established to validate assumption of a rigid support (in XX and YY directions only, freedom to rotate about XX and YY at the point of support should be assumed unless demonstrated otherwise).

This question is referring to the calculation of equivalent static missile load which is shown on sheet 25 of calculation number DC-SE-007-SD. It is recognized that the stack will rotate about either horizontal axis at the roof support point. However, the stack at this point has a significant rotational restraint from the lower portion of the stack. For purposes of calculating the equivalent missile load, the maximum load is obtained if the stack is assumed fixed at this upper support, which is what was done in the calculation. The fixed support condition gives the highest effective frequency of the stack, fe, and thus the highest, and most conservative, effective load.

Seismic Design

Question 2:

The safety class of an enclosing building is based on the class of its contents, unless it is shown that the contents are fully protected by inner structures or inherent resistance such that nothing that could happen to the building would impact the safety of the contents. Therefore, the building should be designed as a Safety Class I building.

The CEC Separation Building is designed to the System Class I criteria including Design Basis Tornado and Design Basis Earthquake, but is not designated a SC I structure. The only function of the building is to remain standing during design basis events. No credit has been taken for the building to mitigate the consequences of a possible release of UF6. Therefore, the Separations Building is not an active structure that must function to mitigate the effects of a design basis event and is designated as a non-safety related structure. It should be noted that as stated in SAR section 5.2.1.1.2, the Separations Building is designated Quality Assurance II and is designed and constructed in accordance with the criteria described in SAR section 10.19.

Question 3:

During an earthquake the seismic dissipation of energy within a structure depends on a number of factors. Acceptable critical damping values for type of structure or component are tabulated in Regulatory Guide 1.61. The 5% damping value used would be acceptable for riveted or bolted stacks but is not conservative for a welded stack without further evidence that it is appropriate.

This value is conservative when compared with the LES criteria. The 5% value is also consistent with Reg Guide 1.61. The stack is bolted at the base and also the support structure at the roof level is a bolted structure, therefore a value close to the bolted-steel-structure damping (4%) is appropriate.

Question 6:

Service loads for pre-stressed concrete should be in accordance with ACI 349, p. 18.1.4.

Design of prestress elements is in accordance with ACI 349 Chapter 18, including the service load conditions as defined in section 18.1.4.

Question 10:

The response does not refer to the specific design in question. The response states that analyses shall be done. Results of the stated analyses should be submitted. Quantitative evidence should be submitted to verify any assertions of non-significance.

Analysis performed demonstrate that the structural concept is adequate for the design basis loads. The final design, which will include all details of design of the Separations Building (e.g., exact location of equipment), will consider the traveling wave effect.

Question 14:

An engineering basis for the amplification factor of 3 is not provided. The seismic spectrum of the support at 45 ft should be determined and it and the spectrum for the base of the stack used for worst case combinations of forces on the stack. There is no basis given to assume that a factor of 3 envelopes the possible situations.

In the course of analysis, estimates are used to approximate conditions which will be determined in final design. As a point of reference, ASCE 4-86, provides a maximum amplification factor of 2.71 for 5% damping in Section 2.2.2.1, Table 2200-1. As was mentioned in the previous response, tornado loading governed the stack, therefore significant changes in the stack size are not expected due to a more refined seismic analysis.

QUESTIONS AND REQUESTS FOR ADDITIONAL INFORMATION:
QUALITY ASSURANCE

1. SAR Section 10.19.3 states that some QA Level 2 activities, because of their ease and repetitive nature, do not require written procedures. Provide examples of such activities as this approach appears to be in contradiction to SAR Section 11.4.1.

For example, to implement the Physical Security Plan one of the activities of the security force personnel will be to hand out badges to personnel as they enter the Controlled Access Area. This activity is sufficiently uncomplicated and repetitive as to not warrant a specific procedure. Likewise, operator activities in the Control Room for monitoring cylinder filling and emptying, and the monitoring of centrifuge operating characteristics are activities sufficiently uncomplicated and repetitive as to not warrant a specific procedure for implementation.

2. The fifth paragraph of SAR Section 10.1.4 states that LES QA personnel will audit the contractor's QA plans and procedures to ensure they meet the requirements of the LES QA plan. Clarify whether the audits by LES also verify that the plans and procedures are implemented such that they ensure an effective QA program.

The following clarifying statement has been added to 10.1.4 (e):

"This includes verifying plans and procedures are implemented such that they ensure an effective QA program."

3. The commitment in the sixth paragraph of SAR Section 10.5 is still not clear where it states that procedures will be reviewed "on a frequency determined by the age and use of the procedure." Clarify.

The commitment "on a frequency determined by the age and use of the procedure" was intended to focus review of procedures on those that are used most frequently. To ensure that all procedures will be reviewed at least every two years, a commitment to review procedures at least every two years has been added to SAR section 10.5.

4. The third paragraph of SAR Section 4.6.3 states that certain Class II items "e.g., the Separations Building," are controlled by QA requirements detailed in SAR Section 10.19. Similar to the way SAR Table 4.6-1 identifies Class I items, identify Class II items that are so controlled.

Class II items that are controlled in accordance with Quality Assurance Level 2 requirements (refer to SAR 10.19) are listed in Table 10.1-4. Class II activities that are controlled in accordance with Quality Assurance Level 2 requirements are listed in SAR section 10.1.4 on page 10.1-3.

5. Some of the commitments that were in the second paragraph of SAR Section 11.3.1 have been moved to SAR Section 11.1.5 in Revision 3. Two commitments regarding personnel qualification appear to have been deleted. They are that personnel qualification is verified by (a) demonstrating the ability to perform assigned tasks and (b), where required by regulation, maintaining a current and valid license. Similarly, the commitment that the use of procedures will be included in the General Employee Training has been deleted from SAR Section 11.3.1.1. Finally, the commitment that continuing training will include "Quality awareness" has been deleted from the Continuing Training program described in SAR Section 11.3.1.2.2. Replace these commitments or justify their deletion.

The two commitments regarding personnel qualifications have been added to SAR section 11.3. SAR section 11.3.1 has been clarified to indicate that "General administrative controls and <u>procedure use</u>" are General Employee Training topics. Quality Awareness has been added as a continuing training topic to SAR section 11.3.1.2.3.J).

6. SAR Revision 3 added procedures for surveillances, inspections, and audits to the list of procedures Section 11.4.1. Consider replacing the commitment in item 1), "Procedures for surveillances and inspections," and item n), "Procedures for audits," with a commitment which says: "Procedures for implementing the Quality Assurance Program" (similar to items e through h).

The recommended changes have been made to SAR section 11.4.1.

7. Section 11.4.1.1 of the SAR includes a commitment that the QA Manager must approve all safety-related procedures. Unless the procedure involves direct QA involvement and the involvement, we believe that requiring a QA signature may give the impression that the QA organization is responsible for the quality of the procedure. Thus, we no longer have this requirement for nuclear power plants. The responsibility for the quality of a procedure should be the originator's and the originator's management. Where line personnel are properly trained, qualified, and motivated and the line organization performs an "independent" review, the QA approval is superfluous. Performance-based oversight by QA of procedure development, review, and use is a better approach to ensure high quality procedures. Consider the use of this approach to ensure procedure quality.

The commitment that QA must approve all safety-related procedures has been revised to indicate that QA must approve a procedure only if it directly involves QA.

8. The list of records retained for three years and the list of records retained for the life of the license that were in Revision 0 of SAR Section 11.4.2 are no longer in Revision 3. Replace these lists or justify their deletion.

The lists have been replaced in SAR section 11.4.2.

9. Some commitments in SAR Chapter 10 (QA) are virtually duplicated in Chapter 11. To prevent possible contradictions between the two chapters, consider referencing rather than duplicating. Two areas where this approach might be beneficial are organization (10.1 and 11.1) and audits (10.18 and 11.3).

Although there is some benefit to referencing as suggested, at this time, in order to ensure Quality Assurance and Operational aspects of the CEC organization are addressed, no changes to reduce duplicity will be made.

REVISED PAGES FROM SECTIONS:

- ❑ Environmental Report 4.4
  - ❑ Environmental Report 8.1
- ❑ Safety Analysis Report 11.8
  - ❑ License Application Exhibit I

REFERENCE RIA:

- ❑ Environmental 2.1.3

  Decontamination and Decommissioning

4.4.4.1 Decommissioning Costs

Table 4.4-2, Estimated Decommissioning Costs and Duration, provides a summary listing of the costs of the major decommissioning activities described above in Section 4.4.2. All costs are in 1996 dollars. As shown in the table, the estimated total cost is $663.9 million. Costs and salvage values are anticipated to change between the time of license application and decommissioning. The cost estimate will be adjusted periodically consistent with the requirements of 10 CFR Part 70.25 (e) and the guidance in Regulatory Guide 1.159.

Louisiana Energy Services' evaluation of decommissioning costs included an evaluation of current experience by one of the general partners in the project, Urenco, Ltd., at similar facilities in Europe. Appropriate adjustments have been made to account for cost differences associated with the performance of specific activities in the United States. The experience and adjustments are documented in the Urenco paper "Decommissioning and Decontamination of a USJVC Plant", USPDC(89)07, 27 April, 1989. Cost figures selected from this paper were escalated to 1996 dollars; otherwise, the selected figures were unchanged. The selected figures include:

Decon Facility Capital Cost	$6.8 million
System Cleaning	$1.1 million
Dismantling	$6.8 million
Decontamination	$13.7 million
Salvage (aluminum only)	($7.9) million
Sub-total (from Urenco figures)	$ 20.5 million

In addition to the figures supplied by Urenco, the following costs were estimated directly for LES:

Decon Facility Labor Cost $ 1.4 million

This was taken to be 20% of capital cost above.

Decontamination of Decon Facility $ 0.5 million

An independent estimate was provided by Naylor Industrial Services, Inc., transmitted by letter dated September 11, 1990.

Radioactive Waste Disposal $ 1.4 million

This assumes 100 m^3 @ $350 per ft^3, in 1992 dollars, escalated to 1996 dollars. This cost of disposal is estimated specifically for radioactive waste disposal in the Central States Compact. (References 5 and 6)

FOR INFORMATION ONLY

(The Urenco estimate of 200 m^3 of low-level waste in the cited reference was reduced by half due to a closer look at solid arisings from the decontamination process. A facsimile from Urenco's Almelo facility, 23 August, 1990, provides an estimate of 2 m^3 of "citric cake" arisings. This "citric cake" was considered in the Urenco cost estimate as a major portion of the low-level solid wastes from decommissioning; thus, it was concluded the estimate of 200 m^3 was high.)

Hazardous/Mixed Waste Disposal $ 0.1 million

Decontamination and decommissioning processes, as described in this section, do not result in the production of hazardous or mixed wastes for disposal. Normal accumulation of hazardous and mixed wastes will occur during the final months of CEC operation. The volume of these final wastes, not due to D&D activities, are estimated to be approximately equivalent to the annual amounts listed in the CEC Environmental Report, Table 3.3-8.

Tails Disposal $ 639 million

The annual tails disposal cost is estimated to be $21.3 million. This is multiplied by 30 years to arrive at the $639 million figure. Costs are based on converting UF6 to UF4 with subsequent UF4 burial at a low level waste disposal site. Estimates vary over a defined range depending on vendor charges for UF6 conversion services. The cost of conversion can be reduced by 20% - 50% if LES enters into a long-term contract, which LES intends to do. The $21.3 million per year value is therefore based on such a long-term arrangement. Details of the estimate are provided in two studies entitled "UF6 Tails Disposition", submitted to the NRC by LES letter dated April 18, 1991, and "Depleted Uranium Hexafluoride Management Study", submitted to the NRC by LES letter dated October 1, 1991. Disposal costs of UF4 are based on estimates for low-level radioactive waste disposal in the Central States Compact, escalated to 1996.

The disposition of tails from the CEC, including potential disposition at the end of facility operation, is an element of authorized normal operating activities. It involves neither decommissioning waste nor is it a part of decommissioning activities. The disposal of these tails is analogous to the disposal of radioactive materials generated in the course of normal operations (even including spent fuel in the case of a power reactor), which is authorized by the operating license and subject to separate disposition requirements (i.e., requirements such as reflected in 10 CFR Part 20). Such costs are not appropriately included in decommissioning costs (this principle (in the Part 50 context) is discussed in Regulatory Guide 1.159, Section 1.4.2, page 1.159-8). Further, the "tails" products from the CEC are not mill tailings, as regulated pursuant to the Uranium Mill Tailings Radiation Control Act, as amended (42 USC 7901, et seq) and 10 CFR Part 40, Appendix A, and are not subject to the financial requirements applicable to mill tailings.

Nevertheless, LES intends to provide during facility life for expected tails disposition costs (even assuming ultimate disposal as waste). Funds to cover these costs, estimated at $21.3 million per equivalent years of tails production, will be set aside during the operating life of the CEC. Accordingly, tails disposition costs are now explicitly reflected in the funding table (SAR Table 11.8-2, ER Table 4.4-2), which reflect both decommissioning funding and the separate matter of contingent end-of-life tails disposition funding.

Final Radiation Survey $ 1.0 million

This figure was estimated by two methods, as follows:

1) The first method is by extrapolation from "Technology and Cost of Termination Surveys Associated With Decommissioning of Nuclear Facilities", NUREG 2241, February, 198?. The 1980 costs of decommissioning a fuel fabrication facility and a UF6 production facility were escalated at 5% per year to 1990. The higher of the two costs, (calculated for a 1 mrem and a 5 mrem dose to the public), were selected and then averaged, for a total of $750,000. Further escalation brings the cost to $950,000.

2) The second estimate was roughly approximated at $725,000 in 1990 dollars, and is escalated to 1996 dollars. The estimate was based on experience, using the following assumptions:

12,000 hours for grid of property and gamma count
$23,000 for soil sampling
150 core holes for depth profile
Building size of 750' x 380'
Workhour rate, including per diem, $60/hour
Extensive use of swipes
Final analyses and report included

Subtotal (from non-Urenco sources)	$ 643.4 million
Total Estimate	$ 663.9 million

4.4.4.2 Funding Arrangements

The funds for decommissioning the facility will be provided in the form of a surety method, insurance, or other guarantee method as required by 10 CFR Part 40.36 (e) and 10 CFR Part 70.25 (f). The selected guarantee method is described in the decommissioning funding plan which is presented in the CEC License Application. As a part of this plan, methods are described for periodic adjustments in the cost estimate, and resulting necessary adjustments to the funding method.

FOR INFORMATION ONLY

REFERENCES FOR SECTION 4.4

1. LES CEC Depleted UF6 Disposition Study, September, 1990, prepared by Duke Engineering and Services, Inc.

2. Depleted Uranium Hexafluoride Management Study, October 1, 1991, prepared by Duke Engineering and Services, Inc.

3. Decommissioning and Decontamination of a USJVC Plant, USPDC (89)97, 27 April, 1989, prepared by Urenco.

4. Minerals Yearbook, Volume I, "Metals and Minerals," U. S. Department of the Interior, Bureau of Mines. Published annually.

5. Duke Engineering & Services, Inc., Telephone Conversation Report, John Etheridge of Entergy, June 17, 1992, DE&S File No. 6046-00-1901.00.

6. Duke Engineering & Services, Inc., Telephone Conversation Report, Rich Patton of US Ecology, June 18, 1992, DE&S File No. 6046-00-1901.00.

Table 4.4-2
Estimated Decommissioning Costs and Duration

Activity		Cost (Millions, 1996 $s)	Time (Yrs)
Decontamination Facility Installation	Capital	$6.8	< 1
	Labor	1.4	
System Cleaning		1.1	1/4
Dismantling		6.8	3
Decontamination		14.2	
Sale/Salvage		(7.9)	(a)
Radioactive Waste Disposal		1.4	(a)
Hazardous/Mixed Waste Disposal		0.1	(a)
Tails Disposal		(b) 639.0	(a)
Final Radiation Survey		1.0	1
TOTALS		$ 663.9	5 yrs

For related information, reference also the decommissioning funding plan contained in the CEC License Application.

(a) To be performed along with dismantling and decontamination.

(b) Tails disposal costs are estimated to be $21.3 million per year of tails production.

A major portion of the skilled labor force needed for and operating the facility is expected to be drawn from unskilled workers hired locally and trained by LES in on-the-job training programs.

About 180 full-time employees will be needed to operate the LES uranium enrichment facility. The estimated total annual operational payroll for the CEC in 1990 dollars will be approximately $8,000,000. This figure includes all costs including benefits. It is projected that the majority of this money will be spent in Claiborne and surrounding Parishes.

Expenditures for materials, equipment, and services associated with the construction and operation of the LES facility will represent a substantial addition to local as well as regional incomes. While major components of the facility including the centrifuge units are not manufactured locally, much of the other equipment and materials required for facility construction and operation will be purchased from qualified local and regional vendors.

In addition to direct construction and operating payroll costs, project monies are expended on services and supplies, much of which is available locally. Examples of such services and supplies include water treating chemicals, vehicle maintenance and fuel, miscellaneous hardware, food and clothing, janitorial supplies, pumps, motors, instruments and electrical equipment.

8.1.1.4 Capital costs of Land Acquisition

Purchase costs of the LES property tract was approximately $538,000.

8.1.1.5 Capital Costs of Plant Facility Construction

Direct capital cost of the LES plant facility construction including interest and property tax and input transmission facilities is projected to be approximately $800 million. This cost does not include escalation, capitalized interest, contingency or replacement centrifuges.

8.1.1.6 Facility Decommissioning Costs

A decommissioning cost study for the LES facility assuming a 1.5 million separative work unit (SWU)/year production rate for 30 years of operation has been made. Projected cost for the facility decommissioning is approximately $663.9 million (1996 dollars). This amount includes an estimated $21.3 million per each year of operation for disposal of UF6 tails. Detailed information pertaining to this study and projected costs are presented in Section 4.4.

TABLE 8.1-1

Quantitative Benefits/Costs of Socioeconomic Factors Associated With Plant Construction and Operation

<u>One Time Benefit</u>	
Claiborne Parish School Board Tax	$5,000,000.
<u>Annual Benefits</u>	
Value of enriched uranium enrichment services	$165,000,000.
Operating Payroll	8,000,000.
Tax Revenues (local/State/Federal)	
- Years 1990-2001	5,400.
- Year 2002 to end of facility life	7,900,000.
Personnel/business income(a)	21,000,000.
<u>One Time Costs</u>	
Land acquisition	$ 538,000.
Site selection, community relations and licensing	3,000,000.
Plant decommissioning	24,900,000.
Plant engineering & construction	800,000,000.
<u>Annual Costs</u>	
Operating and maintenance	$ 16,000,000.
Depleted Uranium Disposal	21,300,000.

(a) Based on 2.65 multiplier of primary dollars (i.e., payroll) for the Shreveport Economic Area which includes Claiborne Parish.

FOR INFORMATION ONLY

Louisiana Energy Services' evaluation of decommissioning costs included an evaluation of current experience by one of the general partners in the project, Urenco, Ltd., at similar facilities in Europe. Appropriate adjustments have been made to account for cost differences associated with the performance of specific activities in the United States. The experience and adjustments are documented in the Urenco paper "Decommissioning and Decontamination of a USJVC Plant", USPDC(89)07, 27 April, 1989. Cost figures selected from this paper were escalated to 1996 dollars; otherwise, the selected figures were unchanged. The escalated figures include:

Decon Facility Capital Cost	$6.8 million
System Cleaning	$1.1 million
Dismantling	$6.8 million
Decontamination	$13.7 million
Salvage (aluminum only)	($7.9) million
Sub-total (from Urenco figures)	$ 20.5 million

In addition to the figures supplied by Urenco, the following costs were estimated directly for LES:

Decon Facility Labor Cost $ 1.4 million

This was taken to be 20% of capital cost above.

Decontamination of Decon Facility $ 0.5 million

An independent estimate was provided by Naylor Industrial Services, Inc., transmitted by letter dated September 11, 1990.

Radioactive Waste Disposal $ 1.4 million

This assumes 100 m^3 @ $350 per ft^3, in 1992 dollars, escalated to 1996 dollars. This cost of disposal is estimated specifically for radioactive waste disposal in the Central States Compact. (References 5 and 6)

(The Urenco estimate of 200 m^3 of low-level waste in the cited reference was reduced by half due to a closer look at solid arisings from the decontamination process. A facsimile from Urenco's Almelo facility, 23 August, 1990, provides an estimate of 2 m^3 of "citric cake" arisings. This "citric cake" was considered in the Urenco cost estimate as a major portion of the low-level solid wastes from decommissioning; thus, it was concluded the estimate of 200 m^3 was high.)

FOR INFORMATION ONLY

Hazardous/Mixed Waste Disposal $ 0.1 million

Decontamination and decommissioning processes, as described in this section, do not result in the production of hazardous or mixed wastes for disposal. Normal accumulation of hazardous and mixed wastes will occur during the final months of CEC operation. The volume of these final wastes, not due to D&D activities, are estimated to be approximately equivalent to the annual amounts listed in the CEC Safety Analysis Report, Table 7.1-1.

Tails Disposal $ 639 million

The annual tails disposal cost is estimated to be $21.3 million. This is multiplied by 30 years to arrive at the $639 million figure. Costs are based on converting UF6 to UF4 with subsequent UF4 burial at a low level waste disposal site. Estimates vary depending on vendor charges for UF6 conversion services. The cost of conversion can be reduced by 20% - 50% if LES enters into a long-term contract, which LES intends to do. The $21.3 million per year value is therefore based on such a long-term arrangement. Details of the estimate are provided in two studies entitled "UF6 Tails Disposition", submitted to the NRC by LES letter dated April 18, 1991, and "Depleted Uranium Hexafluoride Management Study", submitted to the NRC by LES letter dated October 1, 1991. Disposal costs of UF4 are based on estimates for low-level radioactive waste disposal in the Central States Compact, escalated to 1996.

The disposition of tails from the CEC, including potential disposition at the end of facility operation, is an element of authorized normal operating activities. It involves neither decommissioning waste nor is it a part of decommissioning activities. The disposal of these tails is analogous to the disposal of radioactive materials generated in the course of normal operations (even including spent fuel in the case of a power reactor), which is authorized by the operating license and subject to separate disposition requirements (i.e., requirements such as reflected in 10 CFR Part 20). Such costs are not appropriately included in decommissioning costs (this principle (in the Part 50 context) is discussed in Regulatory Guide 1.159, Section 1.4.2, page 1.159-8). Further, the "tails" products from the CEC are <u>not</u> mill tailings, as regulated pursuant to the Uranium Mill Tailings Radiation Control Act, as amended (42 USC 7901, et seq) and 10 CFR Part 40, Appendix A, and are not subject to the financial requirements applicable to mill tailings.

Nevertheless, LES intends to provide during facility life for expected tails disposition costs (even assuming ultimate disposal as waste). Funds to cover these costs, estimated at $21.3 million per equivalent years of tails production, will be set aside during the operating life of the CEC. Accordingly, tails disposition costs are now explicitly reflected in the funding table (SAR Table 11.8-2, ER Table 4.4-2), which reflect both decommissioning funding and the separate matter of contingent end-of-life tails disposition funding.

Final Radiation Survey $ 1.0 million

This figure was estimated by two methods, as follows:

1) The first method is by extrapolation from "Technology and Cost of Termination Surveys Associated With Decommissioning of Nuclear Facilities", NUREG 2241, February, 1982. The 1980 costs of decommissioning a fuel fabrication facility and a UF6 production facility were escalated at 5% per year to 1990. The higher of the two costs, (calculated for a 1 mrem and a 5 mrem dose to the public), were selected and then averaged, for a total of $750,000. Further escalation brings the cost to $950,000.

2) The second estimate was roughly approximated at $725,000 in 1990 dollars, and is escalated to 1996 dollars. The estimate was based on experience, using the following assumptions:

12,000 hours for grid of property and gamma count
$23,000 for soil sampling
150 core holes for depth profile
Building size of 750' x 380'
Workhour rate, including per diem, $60/hour
Extensive use of swipes
Final analyses and report included

Subtotal (from non-Urenco sources) $ 643.4 million

Total Estimate $ 663.9 million

REFERENCES FOR SECTION 11.8

1. LES CEC Depleted UF6 Disposition Study, September, 1990, prepared by Duke Engineering and Services, Inc.

2. Depleted Uranium Hexafluoride Management Study, October 1, 1991, prepared by Duke Engineering and Services, Inc.

3. Decommissioning and Decontamination of a USJVC Plant, USPDC(89)07, 27 April, 1989, prepared by Urenco.

4. Minerals Yearbook, Volume I, "Metals and Minerals", U.S. Department of the Interior, Bureau of Mines. Published annually.

5. Duke Engineering & Services, Inc., Telephone Conversation Report, John Etheridge of Entergy, June 17, 1992, DE&S File No. 6046-00-1901.00.

6. Duke Engineering & Services, Inc., Telephone Conversation Report, Rich Patton of US Ecology, June 18, 1992, DE&S File No. 6046-00-1901.00.

Table 11.8-2
Estimated Decommissioning Costs and Duration

Activity		Cost (Millions, 1996 $s)	Time (Yrs)
Decontamination Facility Installation	Capital	$6.8	< 1
	Labor	1.4	
System Cleaning		1.1	1/4
Dismantling		6.8	3
Decontamination		14.2	
Sale/Salvage		(7.9)	(a)
Radioactive Waste Disposal		1.4	(a)
Hazardous/Mixed Waste Disposal		0.1	(a)
Tails Disposal		(b) 639.0	(a)
Final Radiation Survey		1.0	1
TOTALS		$ 663.9	5 yrs

For related information, reference also the decommissioning funding plan contained in the CEC License Application.

(a) To be performed along with dismantling and decontamination.

(b) Tails disposal costs are estimated to be $21.3 million per year of tails production.

Decommissioning Cost Estimate:

Pursuant to 10 CFR §§40.36(d) and 70.25(e), Louisiana Energy Services has evaluated the estimated costs of decommissioning the Claiborne Enrichment Center. The facility will be decommissioned such that the site and facilities may be released for unrestricted use. A summary of the estimated costs of decommissioning, arranged by principal activity, is set forth in the table below. The sources of the cost estimate data are also provided below.[3] As indicated, the total estimated cost of decommissioning the facility is $663.9 million ($1996).

Louisiana Energy Services' evaluation of decommissioning costs included an evaluation of current experience by one of the general partners in the project, Urenco, Ltd., at similar facilities in Europe. Appropriate adjustments have been made to account for cost differences associated with the performance of specific activities in the United States. The experience and adjustments are documented in the Urenco paper "Decommissioning and Decontamination of a USJVC Plant", USPDC(89)07, 27 April, 1989. Cost figures selected from this paper were escalated to 1996 dollars; otherwise, the selected figures were unchanged. The escalated figures include:

Decon Facility Capital Cost	$6.8 million
System Cleaning	$1.1 million
Dismantling	$6.8 million
Decontamination	$13.7 million
Salvage (aluminum only)	($7.9) million
Sub-total (from Urenco figures)	$ 20.5 million

In addition to the figures supplied by Urenco, the following costs were estimated directly for LES:

Decon Facility Labor Cost	$ 1.4 million

This was taken to be 20% of capital cost above.

3/ A detailed description of the activities associated with decommissioning is also set forth in Section .1.8 of the Louisiana Energy Services Claiborne Enrichment Center Safety Analysis Report.

Decontamination of Decon Facility $ 0.5 million

An independent estimate was provided by Naylor Industrial Services, Inc., transmitted by letter dated September 11, 1990.

Radioactive Waste Disposal $ 1.4 million

This assumes 100 m^3 @ $350 per ft^3, in 1992 dollars, escalated to 1996 dollars. This cost of disposal is estimated specifically for radioactive waste disposal in the Central States Compact. (References 5 and 6)

(The Urenco estimate of 200 m^3 of low-level waste in the cited reference was reduced by half due to a closer look at solid arisings from the decontamination process. A facsimile from Urenco's Almelo facility, 23 August, 1990, provides an estimate of 2 m^3 of "citric cake" arisings. This "citric cake" was considered in the Urenco cost estimate as a major portion of the low-level solid wastes from decommissioning; thus, it was concluded the estimate of 200 m^3 was high.)

Hazardous/Mixed Waste Disposal $ 0.1 million

Decontamination and decommissioning processes, as described in this section, do not result in the production of hazardous or mixed wastes for disposal. Normal accumulation of hazardous and mixed wastes will occur during the final months of CEC operation. The volume of these final wastes, not due to D&D activities, are estimated to be approximately equivalent to the annual amounts listed in the CEC Safety Analysis Report, Table 7.1-1.

Tails Disposal $ 639 million

The annual tails disposal cost is estimated to be $21.3 million. This is multiplied by 30 years to arrive at the $639 million figure. Costs are based on converting UF6 to UF4 with subsequent UF4 burial at a low level waste disposal site. Estimates vary depending on vendor charges for UF6 conversion services. The cost of conversion can be reduced by 20% - 50% if LES enters into a long-term contract, which LES intends to do. The $21.3 million per year value is therefore based on such a long-term arrangement. Details of the estimate are provided in two studies entitled "UF6 Tails

Disposition", submitted to the NRC by LES letter dated April 18, 1991, and "Depleted Uranium Hexafluoride Management Study", submitted to the NRC by LES letter dated October 1, 1991. Disposal costs of UF4 are based on estimates for low-level radioactive waste disposal in the Central States Compact, escalated to 1996.

The disposition of tails from the CEC, including potential disposition at the end of facility operation, is an element of authorized normal operating activities. It involves neither decommissioning waste nor is it a part of decommissioning activities. The disposal of these tails is analogous to the disposal of radioactive materials generated in the course of normal operations (even including spent fuel in the case of a power reactor), which is authorized by the operating license and subject to separate disposition requirements (i.e., requirements such as reflected in 10 CFR Part 20). Such costs are not appropriately included in decommissioning costs (this principle (in the Part 50 context) is discussed in Regulatory Guide 1.159, Section 1.4.2, page 1.159-8). Further, the "tails" products from the CEC are <u>not</u> mill tailings, as regulated pursuant to the Uranium Mill Tailings Radiation Control Act, as amended (42 USC 7901, et seq) and 10 CFR Part 40, Appendix A, and are not subject to the financial requirements applicable to mill tailings.

Nevertheless, LES intends to provide during facility life for expected tails disposition costs (even assuming ultimate disposal as waste). Funds to cover these costs, estimated at $21.3 million per equivalent years of tails production, will be set aside during the operating life of the CEC. Accordingly, tails disposition costs are now explicitly reflected in the funding table (SAR Table 11.8-2, ER Table 4.4-2), which reflect both decommissioning funding and the separate matter of contingent end-of-life tails disposition funding.

Final Radiation Survey $ 1.0 million

This figure was estimated by two methods, as follows:

1) The first method is by extrapolation from "Technology and Cost of Termination Surveys Associated With Decommissioning of Nuclear Facilities", NUREG 2241, February, 1982. The 1980 costs of decommissioning a fuel fabrication facility and a UF6

production facility were escalated at 5% per year to 1990. The higher of the two costs, (calculated for a 1 mrem and a 5 mrem dose to the public), were selected and then averaged, for a total of $750,000. Further escalation brings the cost to $950,000.

2) The second estimate was roughly approximated at $725,000 in 1990 dollars, and is escalated to 1996 dollars. The estimate was based on experience, using the following assumptions:

12,000 hours for grid of property and gamma count
$23,000 for soil sampling
150 core holes for depth profile
Building size of 750' x 380'
Workhour rate, including per diem, $60/hour
Extensive use of swipes
Final analyses and report included

Subtotal (from non-Urenco sources)	$ 643.4 million
Total Estimate	$ 663.9 million

SUMMARY OF DECOMMISSIONING COSTS

Activity	Estimated Cost ($1996)
Planning and Preparation	600,000
Decontamination Facility Installation	8,200,000
System Cleaning, Decontamination and Dismantling of Radioactive Facilities	21,500,000
Sale, Salvage	(7,900,000)
Packaging, Shipping, and Disposal of Wastes	1,500,000
Final Radiation Survey	1,000,000
Tails Disposal	639,000,000
Site Stabilization, and Long-Term Surveillance	N/A
Total	$663,900,000

Finally, Louisiana Energy Services recognizes the need to adjust cost estimates and funding levels periodically, pursuant to 10 CFR §§40.36(d) and 70.25(e). These measures are described below. Louisiana Energy Services also recognizes that, pursuant to 10 CFR §§40.42(c) (2) (iii) (d) and 70.38 (c) (2) (iii) (d), it must update its detailed cost estimate at the time of license termination and provide, if necessary, additional assurance of the availability of adequate funds for completion of decommissioning.

REVISED PAGES FROM SECTION:

- Safety Analysis Report 6.4

REFERENCE RIA:

- Environmental 3.2

  Plant Operation - Plant Features

Displays include discrete display devices, mimic panels, and VDUs. The arrangement of displays and controls promotes efficient use of task-related components, rapid location of any given component, and maximum awareness of plant conditions. Displays contain only the information needed by the operator to meet the task requirements in normal, non-routine, abnormal and accident situations. Status is displayed for important parameters. Displays are formatted and designed to ensure that they are readable, understandable and accessible. Consideration is given to letter size, font, contrast, viewing distance and angle, lighting, color and the task complexity. Displays for similar systems use consistent layouts and operator required responses. Mirror image arrangements of components are avoided. Component arrangement promotes easy association of related controls and displays or other related components. Displays are designed to present information to the operator on a system or subsystem basis and are readily accessible by single keystrokes where possible. Keyboard arrangement promotes easy association of related displays. The display takes full advantage of techniques of control display integration including component grouping techniques, system mimics, system demarcation, and hierarchical labeling. Removal or relocation of marginally useful data is used to avoid operator information overload.

System response to any operator query is made in less than two seconds. System feedback to control action is less than 0.2 seconds wherever possible.

Alarms are prioritized as to importance and presented to the operator in an efficient, timely manner. Keyboard lights and/or VDU displays guide the operator to the proper detail display upon which the alarming function can be found. First-out alarms are used when required to identify the first of several alarms which may occur almost simultaneously. Alarms are both audible and visual and provisions are made for silencing the audible alarm before acknowledging the visual display.

The design of equipment incorporates the objective of efficient maintainability. Equipment out of service is suitably identified to prevent attempted operation or operation of other dependent equipment.

6.4.11 STANDBY GENERATOR SYSTEM

The Standby Generator System comprises two generator package units, two aboveground fuel storage tanks, and associated equipment and controls to provide backup power to the CEC essential loads during a loss of power.

d. Operator Error - Operation of any pump in a blocked-in or dry condition may result in damage to the pump. The operational sequence of the standby generators is automatic. If the fuel pumps fail, it is necessary for operators to set up the proper valve positioning and manually operate a hand fuel pump.

6.4.11.3 Safety Considerations

Failure of this system will not endanger the health and safety of the public. Nevertheless, redundancy is provided in the major components for reliability.

6.4.11.4 Diesel Fuel Spill Control and Leak Detection

Diesel fuel will be stored on site in the two 10,000 gallon diesel fuel storage tanks and the two 600 gallon day tanks. Each of these four tanks will be provided with a secondary containment device to contain the contents of the tank in the event of a tank leak or spillage of oil. Leak detection will be accomplished by daily visual inspection of the containment area between the tank wall and the secondary containment device.

The two day tanks are located within the Diesel Generator Building and each tank is surrounded by curbing to contain a leak if one occurs. The curbing will be designed to contain the full contents of the tank until cleanup can be completed. Detection of a leak from one of the two day tanks will be accomplished by daily visual inspection of the containment area within the curbing.

The two diesel fuel storage tanks are located aboveground in the plant yard near the Standby Diesel Generator Building. Secondary containment for the two diesel fuel storage tanks will be provided by completely encapsulating each tank in a second steel shell. The secondary containment shell will be designed to contain any leak or oil spillage from the tank including the entire tank contents until the leak can be cleaned up. Leak detection will be accomplished by daily visual inspection of the containment area between the primary tank shell and the secondary containment shell. The secondary containment shell will be equipped with an inspection access lid to allow for inspection of the containment reservoir for evidence of a leak or spill. The daily inspection will also include a visual check of the soil surrounding the secondary containment shell for evidence of a leak or spill.

6.4.12 REFRIGERANT SUPPLY SYSTEM

A single Refrigerant Supply System, located in the Auxiliary Area in Plant Unit 1, supplies Freon to the Hot and Cold Refrigerant Systems in each of the three plant units.

REVISED PAGES FROM SECTION:

- ❑ **Safety Analysis Report 11.1**

REFERENCE RIA:

- ❑ **Safety 9.2**

  Staff Qualifications

A written report of each Radiation Safety Committee meeting shall be forwarded to all superintendents within 15 working days of the meeting. Records of the committee proceedings shall be maintained for two years.

The committee shall consist of managers or representatives from quality assurance, operations, integrated scheduling, maintenance, compliance and technical support.

11.1.3.4 Approval Authority For Personnel Selection

The assignment of individuals to the position of Superintendent and manager shall be approved by the CEC Manager.

Assignments to all other staff positions shall be made within the normal administrative practices of the CEC.

11.1.4 PERSONNEL QUALIFICATIONS REQUIREMENTS

11.1.4.1 Minimum Qualifications

The minimum qualification requirements for the facility positions that are directly responsible for its safe operation are outlined below. The experience of each individual is determined acceptable by the President of LES. Different experience requirements may be approved by the LES President. Substitution of additional experience by academic training may be made only with prior NRC approval. This is done in writing and only on a case by case basis.

a) CEC Manager

The CEC Manager shall hold a BS degree in an engineering or scientific field and have a minimum of ten years of appropriate, responsible nuclear experience. A maximum of four years of the ten years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

b) Quality Assurance Manager

The Quality Assurance (QA) Manager shall have a minimum of eight years of appropriate, responsible nuclear experience in the implementation of a quality assurance program. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

c) Operations Superintendent

The Operations Superintendent shall hold a BS degree in an engineering or scientific field and have a minimum of eight years of appropriate, responsible nuclear experience. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

d) Integrated Scheduling Superintendent

The Integrated Scheduling (IS) Superintendent shall have a minimum of eight years of appropriate, responsible nuclear experience. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

e) Maintenance Superintendent

The Maintenance Superintendent shall hold a BS degree in an engineering or scientific field and have a minimum of eight years of appropriate, responsible nuclear experience. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

f) Compliance Superintendent

The Compliance Superintendent shall hold a BS degree in an engineering or scientific field andhave a minimum of eight years of appropriate, responsible nuclear experience. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

g) Technical Support Superintendent

The Tecnnical Support (TS) Superintendent shall hold a BS degree in an engineering or scientific field and have a minimum of eight years of appropriate, responsible nuclear experience. A maximum of four years of the eight years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

h) Security Manager

The Security Manager shall have a minimum of five years of experience in the management of security at similar facilities.

i) Safeguards Manager

The Safeguards Manager shall have a minimum of five years of experience in the management of a safeguards program for special nuclear material.

j) Emergency Preparedness Manager

The Emergency Preparedness Manager shall have a minimum of five years of experience in the implementation of emergency plans and procedures at a nuclear facility.

k) Health Physics Manager

The Health Physics Manager shall have a minimum of five years of appropriate, responsible experience in the implementation of a health physics program at a nuclear facility. A maximum of two years of the five years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

l) Projects Manager

The Projects Manager shall hold a BS degree in an engineering or scientific field andhave a minimum of five years of appropriate, responsible nuclear experience. A maximum of two years of the five years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES. The Projects Manager shall also have at least one year of direct experience in the administration of criticality safety reviews. Within the Projects group shall be at least one individual with a minimum of five years experience in the implementation of a criticality safety program. This individual shall hold a BS degree in an engineering or scientific field. This individual shall hold a BS degree in an engineering or scientific field.

m) Chemistry Manager

The Chemistry Manager shall have a minimum of five years of appropriate, responsible nuclear experience. A maximum of two years of the five years may be fulfilled by academic training on a one-for-one time basis. To be acceptable this academic training shall be in engineering or scientific fields, unless specifically approved by the President of LES.

REVISED PAGES FROM SECTIONS:

- Safety Analysis Report 10.1
  - Safety Analysis Report 10.5
- Safety Analysis Report 11.3
  - Safety Analysis Report 11.4

REFERENCE RIA:

- Quality Assurance

(e) Verifying vendor QA programs, which includes development and approval of approved vendors lists, audit and surveillance of vendor QA programs, and review, approval and control of vendor and procurement QA records. This includes verifying plans and procedures are implemented such that they ensure an effective QA program.

(f) Development, maintenance, and issuance of QA manuals.

(g) Management of the QA Audit Program.

The above responsibilities include verification of the quality of actual hardware, software and documents.

QA activities, plans and programs occurring during the operation phase are described in implementing procedures and are as follows:

- QA Audits of vendors

- Internal audits, inspections of and/or providing technical support for the activities listed below. The following activities are considered to be QA Systems Level 2 activities. The implementation of these activities would follow the requirements for QA Systems Level 2 detailed in section 10.19:

  - UF6 enrichment, sampling and blending operations.
  - Radiological safety and implementation of ALARA practices.
  - Criticality safety.
  - Environmental protection.
  - Decontamination and waste disposal.
  - Maintenance.
  - Emergency Planning.
  - Material control and accounting.
  - Facility modifications.
  - Procedure preparation.
  - Security (both physical security and protection of classified information).
  - Industrial safety (including fire protection).
  - Quality Assurance
  - Interface with NRC and other regulatory agencies.

- Quality Assurance provides technical support for the following activities:

  - Procurement of Class I and Class II materials, equipment, and/or spare parts including specifying QA requirements.

10.5 INSTRUCTIONS, PROCEDURES, AND DRAWINGS

Written instructions and procedures, approved by authorized individuals shall address:

(a) Actions to be accomplished.

(b) Associated responsibilities.

(c) Methods or systems used.

(d) Appropriate quantitative (e.g., dimensions, tolerances, operating characteristics) and qualitative criteria

(e) Identification of interfacing procedures.

(f) Sequence of activities or operations.

To ensure that design requirements imposed by codes, standards, regulations, and site considerations have been considered, procedures provide for review, approval and documentation of activities which affect the quality of systems, structures, and components.

The QA Program requires procedures which specify that work performed shall be accomplished in accordance with the requirements and guidelines imposed by applicable specifications, drawings, codes, standards, regulations, quality assurance criteria and site characteristics.

Acceptance criteria established by the designer are incorporated in the instructions, procedures and drawings used to perform the work. Documentation, including test results, and inspection records, demonstrating that the work has been properly performed is maintained. Procedures also provide for review, audit, approval and documentation of activities affecting the quality of items to ensure that applicable criteria have been met.

Maintenance, modification, and inspection procedures are reviewed by qualified personnel knowledgeable in the quality assurance disciplines to determine:

(a) The need for inspection, identification of inspection personnel, and documentation of inspection results, and

(b) That the necessary inspection requirements, methods, and acceptance criteria have been identified.

Facility procedures shall be reviewed by an individual knowledgeable in the area affected by the procedure on a frequency determined by the age and use of the procedure to determine if changes are necessary or desirable, and at least every two years. Procedures are also reviewed to ensure all procedures are maintained up-to-date with facility configuration. These reviews are intended to ensure that any modifications to facility systems, structures or components are reflected in current maintenance, operations and other facility procedures.

11.3 TRAINING PROGRAM

The principle objective of the LES training program system is to ensure job proficiency of all facility personnel involved in work through effective training and qualification. The training program system is designed to accommodate future growth and meet commitments to comply with applicable established regulations and standards.

Qualification is indicated by successful completion of prescribed training, demonstration of the ability to perform assigned tasks and where required by regulation, maintaining a current and valid license issued by the agency establishing the requirements. Training is designed, developed and implemented according to a systematic approach. Employees are provided with formal training to establish the knowledge foundation and on-the-job training to develop work performance skills. Continuing training is provided, as required, to maintain proficiency in these knowledge and skill components, and to provide further employee development.

11.3.1 PROGRAM DESCRIPTION

The training program is designed to prepare initial and replacement personnel for safe, reliable and efficient operation of the facility. Appropriate training for personnel of various ability and experience backgrounds is provided. The level at which an employee initially enters the training program is determined by an evaluation of the employee's past experience, level of ability, and qualifications.

Facility personnel may be trained through participation in prescribed parts of the training program which consists of the following:

A) General Employee Training
B) Technical Training
C) Employee Development/Management-Supervisory Training

Training is made available to CEC personnel to initially develop and maintain minimum qualifications outlined in Section 11.1.4. The objective of the training shall ensure safe and efficient operation of the facility and compliance with applicable established regulations. Training requirements shall be applicable to, but not necessarily restricted to, those personnel within the plant organization who have a direct relationship to the operation, maintenance or other technical aspect of the CEC. Training courses are kept up-to-date to reflect plant modifications and changes to procedures when applicable.

Continuing or periodic retraining courses shall be established when applicable to ensure that personnel remain proficient.

FOR INFORMATION ONLY

Periodic retraining generally is conducted to ensure retention of knowledge and skills important to facility operations. The training may consist of periodic retraining exercises, instruction, and review of subjects as appropriate to maintain proficiency of all personnel assigned to the facility.

11.3.1.1 General Employee Training

General Employee Training (GET) encompasses those Quality Assurance, radiation protection, safety, emergency and administrative procedures established by CEC management and applicable regulations. Continuing training is conducted in these areas as necessary to maintain employee proficiency. All persons under the supervision of facility management must participate in General Employee Training; however, certain facility support personnel, depending on their normal work assignment, may not participate in all topics of the GET. Temporary maintenance and service personnel receive General Employee Training to the extent necessary to assure safe execution of their duties. Certain portions of General Employee Training may be included in a New Employee Orientation Program.

General Employee Training topics are listed below.

A) General administrative controls and procedure use

B) Quality Assurance policies and procedures

C) Facility systems and equipment

D) Nuclear safety (See Section 11.3.1.1.1 - includes the use of dosimetry, protective clothing and equipment)

E) Industrial safety, health and first aid

F) Emergency Plan and implementing procedures

G) Facility Security Programs (includes the protection of classified matter and information)

H) Fire Protection and Fire Brigade (See Section 11.3.1.1.2)

I) New Employee Orientation

11.3.1.1.1 Nuclear Safety Training

program is to assure the trainee's ability to perform job tasks as described in the task descriptions and the Training and Qualification Guides.

11.3.1.2.3 Continuing Training

Continuing Training is any training not provided as Initial Qualification and Basic Training which maintains and improves job-related knowledge and skills such as the following:

A) Facility Systems and Component Changes

B) OJT/Qualifications Program Retraining

C) Procedure and Directive Changes

D) Operating Experience Program Documents Review to include Industry and In-House Operating Experiences.

E) Continuing Training required by Regulation (e.g., Emergency Plan Training)

F) General Employee, Special, Administrative, Vendor, and/or Advanced Training topics supporting tasks which are elective in nature.

G) Training identified to resolve deficiencies (task-based) or to reinforce seldom used knowledge skills

H) Refresher training on initial training topics

I) Pre-job instruction, mock-up training, walk throughs, that are structured.

J) Quality Awareness

Continuing Training and Requalification Training may overlap to some degree in definition; however, Requalification or Retraining refers to specific training designed for proficiency maintenance.

Continuing Training consists of formal and informal components performed on a frequency needed to maintain proficiency on the job. Each Section's Continuing Training Program is developed from a systematic approach, using information from job performance and safe operation as a basis for determining the content of continuing training. Continuing training may be offered, as needed, on any of the topics or programs listed in Section 11.3.1.2.3 "Continuing Training."

Once the objectives for Continuing Training have been established, the methods for conducting the training may vary.

11.4 FACILITY OPERATIONS

11.4.1 FACILITY PROCEDURES

All safety-related (system class I) operations are conducted through the use of procedures. Before initial enrichment activities occur at the facility, a list of titles of procedures that clearly indicate their purpose and applicability are made available to the NRC for their inspection. As noted throughout the Safety Analysis Report procedures are used to control activities in order to ensure the activities are carried out in a safe manner. These activities would typically include:

a) Procedures for cylinder handling

b) Procedures for autoclave operation

c) Procedures for takeoff stations operation

d) Procedures for other production operations (e.g., blending)

e) Procedures for implementing the Fundamental Nuclear Material Control (FNMC) Plan

f) Procedures for implementing the Emergency Plan

g) Procedures for implementing the Physical Security Plan

h) Procedures for implementing the Security Plan for the Protection of Classified Matter and Information

i) Procedures for design changes to the facility

j) Procedures for maintenance of facility structures, systems and components

k) Procedures for construction and testing of facility structures systems and components

l) Procedures for implementing the Quality Assurance Program

m) Procedures for training

11.4.1.1 Preparation of Procedures

For operating, abnormal, maintenance, instrument, periodic test, chemistry, radioactive waste management, health physics, emergency preparedness, annunciator responses, and modification procedures, each procedure is assigned to a member of the facility staff for development. Initial procedure drafts are reviewed by members of the facility staff, by personnel from the supplier of centrifuges (Urenco), and other vendors, as appropriate. Following resolution of review comments, if any, a revised procedure is prepared and forwarded to a previously designated qualified reviewer for review and comment. This qualified reviewer also makes the determination whether or not any additional, cross-disciplinary review is required. After all required and appropriate reviews have been completed a final version of the procedure is prepared. Upon approval by the CEC Manager and all Superintendents, a procedure becomes available for use. If the procedure involves QA directly, the QA Manager also must approve the procedure.

11.4.1.2 Administrative Procedures

Facility administrative procedures (Department Directives) are written by each department as necessary to control facility testing, maintenance, and operating activities. Listed below are several areas for which administrative procedures are written, including principle features:

a) Operator's authority and responsibility: The operator is given the authority to manipulate controls which directly or indirectly affect the enrichment process, including a shut down of the process if deemed necessary by the Shift Supervisor. The operators are also assigned the responsibility for knowing the limits and set points associated with safety-related equipment and systems as specified in designated operating procedures.

b) Activities affecting facility operation or operating indications: All facility personnel performing functions which may affect unit operation or control room indications are required to notify the Control Room Operator (operator) prior to initiating such action. Removal of an instrument or component from service requires the permission of the Shift Supervisor or Unit Supervisor.

c) Manipulation of facility control: No one is permitted to manipulate the facility controls who is not an operator, except for operator trainees under the direction of an qualified operator.

d) Relief of Duties: This procedure provides a detailed checklist of applicable items for shift turnover.

e) Equipment control: Equipment control is maintained and documented through the use of tags, labels, stamps, status logs or other suitable means.

f) Master surveillance testing schedule: This procedure establishes a master surveillance testing schedule to ensure that required testing is performed and evaluated on a timely basis. Surveillance testing is scheduled such that the safety of the facility is not dependent on the performance of a structure,

a) Records of Facility Safety Review Committee meetings.

b) Surveys of equipment for release to unrestricted areas.

c) Instrument calibrations.

d) Classified/reportable incident reports.

e) Safety audits.

f) Personnel training and retraining.

g) Radiation work permits.

h) Surface contamination surveys.

i) Concentrations of airborne radioactive material in the facility.

j) Radiological safety analyses.

In addition, the following records shall be retained for at least the periods indicated:

The following shall be retained or at least 3 years:

a. Records of all Reportable Events;

b. Records of surveillance activities, inspections, and calibrations;

c. Records of changes made to procedures; and

d. Records of radioactive shipments.

The following records shall be retained for the duration of the facility license:

a. Records and drawing changes reflecting design modifications made to systems and equipment described in the Safety Analysis Report;

b. Records of radiation exposure for all individuals entering radiation control areas;

c. Records of gaseous and liquid radioactive material released to the environs;

d. Records of training and qualification for current and past members of the CEC staff;

e. Records of reviews performed for changes made to procedures or equipment or reviews of tests an experiments;

f. Records of analyses required by the Radiological Environmental Monitoring Program that would permit evaluation of the accuracy of the analyses at a later date. These should include procedures effective at specified times and QA records showing that these procedures were followed; and

g. Records of quality assurance activities required by the Quality Assurance Program. These shall be retained for a period of time as recommended by ANSI N.45.2.9-1974.

Other retention times are specified for other facility records as necessary to meet applicable regulatory requirements. These retention times are indicated in specific facility procedures.

11.4.3 REVIEW AND AUDIT ORGANIZATIONS

A review and audit program for operational quality assurance of the CEC is established, and periodically reviewed by management, to:

- verify that the facility is consistent with LES company policy, approved procedures and license provisions,
- review important proposed facility modifications, tests and procedures,
- verify that reportable occurrences are investigated and corrected in a manner which reduces the probability of recurrence of such events (reference section 11.4.5), and
- to detect trends which may not be apparent to a day-to-day observer.

The intent of this program is to ascertain that the facility is constructed and operated safely and in accordance with the license conditions.

a) The organizational structure for conducting the operational quality assurance review and audit program is as follows:

1) The Facility Safety Review Committee appointed by the CEC Manager.

2) The Radiation Safety Committee appointed by the CEC Manager.

3) Regular audits conducted by the Quality Assurance Department.

REVISED PAGES FROM SECTION:

- ❑ **Safety Analysis Report 3.6**

REFERENCE RIA:

- ❑ **Safety 3.6**

Geology and Seismology

TABLE OF CONTENTS

LIST OF TABLES

FOR INFORMATION ONLY

LIST OF FIGURES

LIST OF FIGURES

3.6 GEOLOGY AND SEISMOLOGY

3.6.1 BASIC GEOLOGY AND SEISMIC INFORMATION

The site for the Claiborne Enrichment Center (CEC) is located in an area of rolling hills in northern Louisiana. The site comprises 442 acres, of which approximately 70 acres will be developed for the facility. Elevations range from roughly 340 feet above mean sea level (MSL) in the central portion of the site to 280 feet above MSL in the southern portion. The site drainage is to the west and south where small creeks have formed at the base of the hills. Vegetation is thick and composed of pine forest with some oak. Trees in the areas to be developed have been cleared, leaving stumps typically six inches high.

A review of the geological history of the site as well as in-depth field exploration were performed in order to clearly define the regional and site specific geology (References 1 and 2). The results of the geological investigation of the site are discussed in this section.

3.6.1.1 <u>Regional Geology</u>

3.6.1.1.1 Geologic History

During the Triassic, the Gulf of Mexico region was a land area with Paleozoic sediments. In the mid-Triassic, the region was elevated and block faulted into a basin and range-type terrain. Areas of attenuated crust later became zones of sediment accumulation, and areas of thicker crust became basin margins, interbasin and interbasin arches or uplifts and carbonate platforms. This tectonism (uplift and elongation) is interpreted to relate to the breakup of Pangea (Reference 3), which resulted in the rifting of the Gulf of Mexico.

By late Triassic, much of the area was buried with only the larger horsts and interbasin blocks remaining as topographic prominences. In late Triassic or early Jurassic, the uplifting ceased and subsidence began. Marine waters entered large areas of the region and resulted in deposition of evaporites in the circulation-restricted basin and range topography. Subsidence continued and eventually led to open marine conditions. Most of the subsidence that occurred during the 100 million years following mid-Jurassic was probably due to the cooling of the thermal anomaly related to the Triassic rifting event.

In the late Jurassic and early Cretaceous, shorelines became evident in the stratigraphic record, while the central Gulf of Mexico became a small, deep-water, sediment-starved basin. By early to mid-Tertiary tectonic subsidence had ceased. In the peripheral basins, sedimentation had kept pace with subsidence. Sediments transported into the interior basin were carried through to the central Gulf where the sediments slowly prograded into the basin. Gulfward tilt of the interior basins was caused by the load-induced subsidence of the northern Gulf.

Activity of the Gulf region was proceeded by predominantly non-diastrophic events, including gravity slides, salt movement and dome formation including crestal and radial faults, and growth fault development. These activities proceed a seismically. In the northern interior salt basins sedimentation virtually kept pace with subsidence. With the cooling of the thermal anomaly and little additional sediment loading in the already filled basins, subsidence is virtually complete making active faults unlikely in the northern portions of the Gulf Region.

Gulfward of the interior salt dome basins, downdip movement of salt and active growth faults have followed the depocenters from the vicinity of the Commanchean Shelf Edge in the late Tertiary southward to the Sigsbee Escarpment today. The driving mechanism is differential loading and the resulting tectonic features; growth faults, salt domes and other salt structures, and the faulting associated with the salt structures are aseismic. The pre-salt basement is still subsiding at the depocenters and this and the associated flexure of the block-faulted Triassic and older basement rocks may be the cause of what little seismicity is present.

3.6.1.1.2 Physiography

North Louisiana is located in the north central portion of the Gulf Coastal Plain physiographic province and is bounded to the north by the Ouachita Mountains physiographic province. This area of Louisiana is in the Western Hills subprovince as the Mississippi River to the east forms the boundary between east and west subprovinces.

Physiographic and geomorphic features have been significantly influenced by Quaternary sea level changes which during sea level lowering caused streams and rivers to entrench themselves, thereby increasing erosion rates within their drainage basins. With the latest sea level rise since about 18,000 years before present, rivers have filled their valleys with sediments, thereby reducing erosion rates from those of lower sea level stands.

Major drainage features of north Louisiana are the Mississippi River to the east and the Red River to the west. The rivers and a large portion of their feeder streams flow southward toward the Gulf of Mexico. These major drainage features occupy broad steep sided, flat bottomed valleys. There Quaternary alluvial filled valleys range from a few miles wide for the Red River to several tens of miles wide for the Mississippi River. The average elevations in the north Louisiana upland area is 300 feet above mean sea level (MSL) with typical relief averaging 100 to 150 ft.

3.6.1.1.3 Structural Geologic Conditions

Subcrustal movement during the late Paleozoic signified the beginning of the formation of the Gulf Basin or Gulf Coast Geosyncline. Block faulting associated with what is believed to be rifting of the continental crust initiated subsidence in what was to be the Gulf Basin in late Triassic. The block faulting resulted in a similar terrain such as the Basin and Range province of the southwestern United States. The more positive elements remain positive throughout geologic history, and similarly the more negative elements remain negative features such as basins. During the late Triassic the positive elements were eroded and this erosion supplied large volumes of sediment which created distinctive red beds in the adjacent and subsiding basins. Differential regional subsidence is believed to have formed basins and uplifts within the Upper Gulf Basin which corresponds to the east Texas and north Louisiana and Mississippi interior salt basins as negative features and positive features such as the Sabine and Monroe uplifts (see Figure 3.6-1, Gulf Coast Regional Structural Features Map).

During late Triassic or early Jurassic, the process of infilling the basins isolated the basins from the open Gulf, and evaporites (Louann Salt) were deposited and subsequently buried by younger sediments. In north Louisiana the basin is referred to as the North Louisiana Salt Basin and also the North Louisiana Syncline. The Sabine Uplift which covers most of northwestern Louisiana and the Monroe Uplift in northeastern Louisiana were created during the Triassic by block faulting and have remained positive areas bounding the north Louisiana Salt Basin. North of the basin is a series of basin boundary faults referred to as the South Arkansas Fault Zone. These are attributable to basin subsidence activity and not related except by control to the basement structure. The Sabine Uplift and Monroe Uplift were further uplifted and exposed to erosion during the Cretaceous Period. As a result of the positiveness of the uplift, strata have a slight southwestern dip from the Monroe Uplift and southeasterly dip from the Sabine Uplift.

Between the Monroe Uplift and the North Louisiana Salt Basin is the Claiborne Platform (D'Arbonne Platform) which is an extension of the more stable Monroe Uplift and is a transitional structure to the subsiding North Louisiana Salt Basin. (The term "more stable" refers to the cessation of movement of the Monroe Uplift.) The Claiborne D'Arbonne Platform is the term applied to the north central Louisiana area as mapped on Eocene beds which crop out in the area (Reference 45, Durham, 1964). These beds are virtually flat lying. This platform defines a structurally stable area established during the Tertiary. The Claiborne (D'Arbonne) Platform bridges over the western portion of the Monroe Uplift and the North Louisiana Salt Basin, both older and deeper structural features present during the Mesozoic, indicating that the Monroe Uplift ceased to be a topographic high and indicating the lack of differential movement between the Monroe Uplift and the Northern Louisiana Salt Basin (part of the Interior Salt Basin region). The platform remained a shoal area from late Jurassic through late Cretaceous and therefore probably represents primarily a stratigraphic effect on the marine deposits of the basin and not a structural effect.

Deposition in the North Louisiana Salt Basin is characterized by cycles representing transgressions, inundation and regression which resulting in the deposition of large thickness of marine to deltaic sediments. Initial sediment loading initiated plastic flow of Louann Salt in the Upper Jurassic and perpetuated it throughout basin infilling. Salt movement continued throughout Cretaceous and into the middle Tertiary, creating large salt subbasins with piercement domes (salt diapirs) and their corresponding rim synclines. Normal faults and radial faults are present in the basin and are all related to salt movement at depth.

The influx of terrigenous sediment during the middle Tertiary signified the end of basin filling and thus the mechanism for salt dome growth and basin subsidence. The salt structures were left in various stages of development from what is termed pillows and turtle structures to the more mature piercement salt domes.

As the interior basins filled, sediment depocenters migrated further south out of the interior basins and into the lower Gulf Basin, which was previously starved of sediment. Sediment loading south of the interior basins produced significantly more subsidence than in the interior basins. This is primarily related to the differences in crustal composition and thickness between the interior basins and the lower Gulf Basin (transitional continental crust versus oceanic crust). As a result of the differential subsidence, tensional stresses resulted in the creation of boundary structures along the southern portions of the interior basins. These are sometimes thought of as hinge line flexures in response to the differential subsidence. On the south flank of the east Texas Basin, just west of the Sabine Uplift, is the Mt. Enterprise Fault Zone which occupies a portion of the hinge zone and is reported to have been active until the Miocene. The North Louisiana Salt Basin does not have a corresponding hinge line fault zone but does have the Angelina Flexure which may represent a counterpart to hinge line structure in Texas.

3.6.1.1.4 Boundary Fault Systems

Boundary fault systems consist of those fault zones which tend to define the limits of the basin (see Figure 3.6-2, Tectonic Features Map). These include the Mexia-Talco fault zone, South Arkansas fault zone, Pickens-Quitman-Gilbertown-Pollard fault zone, Mount Enterprise fault zone and the Rodessa fault zone. In most cases, boundary faults shown on Figure 3.6-2 actually represent fault zones, or a series of faults in the same general alignment, not individual faults.

3.6.1.1.4.1 Mexia-Talco, South Arkansas, and Pickens-Quitman-Gilbertown-Pollard Fault Zones

The Mexia-Talco fault zone is considered to be continuous with the South Arkansas fault zone in Arkansas, and continuous with the Pickens-Quitman-Gilbertown-Pollard fault zone in Mississippi. Each of these fault zones has similar characteristics. These fault zones form a nearly continuous series of en echelon normal faults and grabens. The fault zones parallel or are sub-parallel to regional strike and mark the updip limit of thick Jurassic and lower Cretaceous sediments in most places. The system is aligned along the regional trend of the Ouachita frontal zone and is located gulfward from the main zone of basement thrust-faults which make up the Ouachita frontal zone. The boundary faults are considered to represent a zone of major fracturing and graben formation associated with the upper portion of the Gulf Coast basin (Reference 4).

The zone is composed of complex grabens. At a few localities either up-to-the-basin or up-to-the-margin faults are dominant, and in these areas the complementary fault sets are poorly developed or absent. The faults are normal, and form an en echelon series of grabens up to five or eight miles across. The faults increase in displacement at varying rates with depth (Reference 4). Displacement along the faults ranges from none at the strike termination of individual fault segments to more than 2,500 feet at various locations. Surface displacements rarely exceed 300 feet and are usually in upper Cretaceous or Eocene beds. Dips of the fault planes range from 35 to 70 degrees and generally are steeper at the surface flattening out with depth. Stratigraphic sequences are commonly thicker on the down-thrown blocks, indicating that some fault movement was contemporaneous with deposition and documenting recurrent movement over long periods of time. Movement may have occurred on some of the faults as late as mid-Tertiary, but since displacement on these faults is known to decrease towards the surface, the amount of movement has progressively decreased with time. Pleistocene terraces extend across the fault systems, at places without interruption, indicating that movement probably ceased about the Eocene. Most reports state that recent movement has not occurred along these faults; however, others based on inconclusive evidence (i.e., Mexia earthquake of 1932) suggest that movement is still occurring. This earthquake, however, may have been related to fluid withdrawal in a hydrocarbon production region (Davis et al. (1989); Reference 15).

FOR INFORMATION ONLY

Seismic reflection evidence indicates the boundary fault system to be tectonic in origin because the system is underlain by a fault scarp in pre-salt rocks (Reference 3). This scarp is believed to predate salt deposition, as the Louann Salt shows an abrupt change in thickness across the scarp. The change in thickness equals the height of the scarp, indicating that the scarp existed at the time of Louann Salt deposition and that the fault in the basement has not experienced significant movement since Louann time. The basement fault scarp is significant in that it marks the boundary between continental crust of normal thickness marginal to the fault and thinner attenuated continental crust basinward of the fault.

Fault movement in the overlying sediments was probably due to two mechanisms:

a. Bending across a hinge line between the two crustal thicknesses

b. Down-dip salt flowage

Either of these mechanisms, or both, may have operated along any given segment of the fault system. It should be pointed out that the fault scarp in the basement does not necessarily cause the graben system above it, but controls the location of the graben fault system. When sedimentation occurred, the thinner crust beneath the basin subsided to greater depths than the adjacent thicker crust, and the differential subsidence was accommodated by bending along the hinge line and resulted in movement along the fault system as subsidence and sedimentation occurred. This bending is believed to stretch the overlying sediments creating a system of normal faults typically in the form of a graben. The boundary fault system also generally marks the up-dip limits of Louann Salt deposition. Mechanical experiments shows that salt will flow down-dip, and if applied to the area of the boundary fault systems suggest an alternative or an additional mechanism for fault movement. When the salt flows down-dip, it cannot be replaced by salt flowing from up-dip because of non-deposition of salt up-dip. The salt thickness adjacent to the scarp will diminish constantly until the salt has been completely evacuated from the area. As this occurs at the base of the scarp, the overlying beds are lowered to the pre-salt surface, disrupting the overlying beds and resulting in the formation of the fault system.

The youngest sediments which overlie the boundary zone faulting are Pleistocene terrace sediments and geomorphic surfaces, such as along the Red River, which traverses the nearest boundary fault zone, the South Arkansas fault zone. There are no references in the literature indicating displacement in these units. Reference 57, Walthall and Walper (1967), state the youngest displaced formations along the boundary faults (in

Arkansas) are of Eocene Age. Also, Jackson (1982) refers to Kehle (1978; Reference 3) stating "... future movement on the Mexia-Talco Fault zone is extremely unlikely because undeformed Pleistocene terraces cross them."

3.6.1.1.4.2 Mt. Enterprise Fault Zone

The Mount Enterprise fault zone is considered to be a boundary fault system although its origin is not similar to the Mexia-Talco fault system. The Mount Enterprise system is not considered to have had a basement scarp origin as did the Mexia-Talco fault system. The Mount Enterprise system is related to a hinge line or transitional zone between the attenuated continental crust and oceanic crust. The differential subsidence along this hinge line resulted in tensional stresses and faulting which was initiated in early Cretaceous time.

The Mount Enterprise Fault Zone consists of a parallel series of displacements which mark the southeastern border of the East Texas Salt Basin. The system apparently dies out before reaching the western border of the basin at the Mexia-Talco Fault Zone. The dip of the Mt. Enterprise fault planes at the surface range from about 30 to 60 degrees, with lower dips at depth. Stratigraphic displacements in Eocene Beds at the surface range up to 500 feet, increasing with depth to 700 feet in rocks of the upper Cretaceous (Reference 5).

Seismic reflection data indicate that the fault system originated during the deposition of the Cotton Valley and Trinity Groups (References 6 through 8). The fault system is a series of graben structures resulting from extensional forces generated by subsidence in the Northeast Texas Embayment to the north and the Lower Gulf Coast Basin to the south.

According to interpretation of seismic reflection and well control data, the Mt. Enterprise Fault Zone developed in response to two mechanisms which occurred during different periods. Initiation and growth of the fault system is postulated in the following sequence:

a. Initial differential loading of post-Louann sediments mobilized salt from the main center of deposition southward to the present position of the fault system, forming a linear salt ridge, possibly due to pillowing inferred from regional gravity data and the position of Elkhart and Slocum Domes (Reference 9).

b. Faulting, which apparently occurred during the upper Jurassic, was the principal structural adjustment to the continuing salt buildup.

c. Maximum movement occurred in the early Tertiary during Wilcox deposition as indicated by the appreciable thickening of sediments on the downthrown (south) side of the Mt. Enterprise Fault where early Tertiary centers of deposition were located.

d. Subsidence in the Northeast Texas Embayment to the north and the Gulf Coastal Basin to the south caused the fault zone to function as a hinge line and accelerated the movement of salt.

e. As centers of deposition migrated farther south, stresses lessened and movement along the fault zone diminished.

Cessation of significant movement was apparently no later than Miocene. Previous geologic mapping in the vicinity has shown no faulting in Quaternary age terrace deposits. However, displacement in the Quaternary has been reported to be attributed to minor faulting near the Trinity River (Reference 10) along the surface projection of the subsurface western extension of the Mt. Enterprise Fault Zone, and a microearthquake recording net has shown activity near the Mount Enterprise Fault Zone (Reference 11).

Microseismic activity has been monitored near the Mt. Enterprise fault zone (Pennington and Carlson (1984), Reference 11) and Pleistocene movement has been suggested (Collins et al., 1980, Reference 10). The Mount Enterprise Fault Zone, however is over 100 miles from the site. Jackson (1982) while stating that this fault zone is not well understood, suggests that the fault zone is related to salt creep (based on reflection data), indicating a low seismic potential.

The Mt. Enterprise Fault Zone is one of the few fault zones in the Gulf Coast Region which potentially has seismicity spatially associated with it. The most notable earthquake was the MMI VII 1891 Rusk, Texas earthquake. This fault zone was described by Jackson (1982) as being potentially related to movement in the Louann Salt. According to Collins et al. (1980; SAR Reference 10), the faults in the Mt. Enterprise system may represent hingeline effects between the East Texas Basin and the subsiding Gulf Basin. While fault activity began during the Cretaceous, most movement occurred during the Eocene. Fault movement reduced as sedimentation slowed and the basal salt reached equilibrium.

3.6.1.1.4.3 Rodessa and Hosston Fault Zone

The Rodessa Fault zone is a series of en echelon down-to-the-basin faults comprising a zone on the north flank of the Sabine uplift which extends partly into the Northeast Texas Salt Dome basin. The fault zone extends into northern Louisiana, making a total length for the fault zone of about 125 miles. The Rodessa fault zone may be related to the South Arkansas fault zone (Reference 5). The Hosston fault zone in northern Louisiana parallels the Rodessa zone about 10 to 15 miles further south, but is much shorter in length, and its occurrence appears to be limited to Louisiana. Faulting in the Rodessa and Hosston zones is believed to have been initiated in the early Cretaceous and to have ceased in the middle Tertiary. The Rodessa and Hosston fault zones are believed to be a part of or related to the Mexia-Talco - South Arkansas fault zone. These fault zones acted indirectly as compensating faults to the system but are complicated by their proximity to the Sabine Uplift. As with the other boundary faults, movement was probably not initiated along faults in the pre-salt basement rocks, but crustal controlled, modified somewhat by the presence of salt at depth.

3.6.1.1.4.4 Growth Faults (Coastal Faults)

Growth faults form a series of major down-to-the-coast fault zones that trend roughly parallel to the Gulf of Mexico. They are characterized by main periods of movement occurring simultaneously with main periods of sedimentation. For this reason they have been termed contemporaneous faults, growth faults, flexure faults, syndepositional faults and depositional faults. The fault zones with the oldest time of principal movement are located farther inland from the Gulf than the fault zones with the most recent movement. Thus, the time of principal movement corresponds with the thickest sequence of sediments in each sedimentary unit. These sequences occur seaward or downdip from the thickest sequence in the preceding older unit. This results in the lower Gulf basin having an inclined axial plane (four degrees gulfward). The following features are characteristic of these structures:

a. Fault planes commonly dip at approximately 45 degrees, but frequently steepen toward the surface.

b. Adjustment or compensating faults form narrow grabens within the trend.

c. Beds on the down-thrown block commonly dip back t. us the fault plane.

d. Beds on the up-thrown block commonly dip away from the fault plane.

e. Throw or vertical displacement increases greatly with depth.

f. The faults frequently die out quickly with no relative displacement of beds a mile or more away.

g. The amount of formation dip into the down-thrown side of the fault commonly decreases with depth.

Several theories have been presented to explain the mechanics of these faults. The theories include the subsiding basin hypothesis, gravity flow hypothesis and the salt ridge hypothesis. No general theory is completely accepted for their origin, but it is believed that most growth faults originate either by the gravity flow mechanism or salt ridge mechanism, resulting in initial shearing of unconsolidated sediments within a geologically short period after the sediments were deposited.

It is a generally agreed characteristic of these faults that movement takes place during deposition of sediments and is rather slow, unlike the rapid fault rupture generally associated with earthquake generating faults. The down-thrown side of the faults commonly receives much more sediment during an interval of time than the up-thrown side. Therefore, the increased sedimentation on the down-thrown side tends to perpetuate movement as the result of increased loading and possibly subsidence. Since deposition was the major driving force causing these faults, as sedimentation ends, so does faulting. Major growth activity on these faults apparently ceased in the Miocene as sediment depocenters moved further gulfward.

Recent movement of some of these faults has occurred, resulting in measurable fault scarps and movement rates. Most noted are those in the Houston area. The time of movement is indicated as Late Quaternary as sediments of this age are displaced. Recent fault movement has disrupted man-made structures over short-time intervals. This later movement is believed to be the direct result of the removal of ground water, or hydrocarbons resulting in the reduction of pore pressures in sands, increasing overburden pressure and consolidating the interbedded clays in the Quaternary sediments. Since the fault plane restricts or partly restricts the movement of fluids across the fault to replenish the extracted fluid supply, differential subsidence occurs, resulting in fault movement. Movement is believed to only take place in the portion of the strata undergoing consolidation and dies out with depth. All fault movement of this nature has occurred along preexisting faults which were initially the result of sediment loading. There has been no measurable movement along these faults attributed to subsidence of the lower Gulf basin; however, it has been inferred by some geologists to be a result of continued tilting and subsidence of the Gulf Coast.

Movement of growth faults in the Gulf Coast Region during the Pleistocene and into Holocene is well documented (for example, Reference 48, McCulloh and Autin (1991) and Reference 49, McCulloh (1990). In metropolitan areas such as Houston, Texas and Baton Rouge, Louisiana movement has occurred along the near-surface planes of old growth faults associated with differential settlement caused by ground water withdrawal. Due to the poorly consolidated nature of the sediments involved, this movement is gradual and occurs as creep which has not produced earthquakes. Infrequent earthquakes appear to have occurred randomly in the region and are not known to be associated with any specific geologic structure. Maximum historical intensity is VI, recorded near Donaldsonville, Louisiana, over 200 miles southeast of the site, on October 19, 1930.

The 1983 Lake Charles earthquake is a relatively recent instrumentally recorded earthquake which has been interpreted to be associated with faulting in the pre-salt basement. The earthquake has a felt area of only 2600 km^2 and was not felt at the site (Stevenson and Agnew (1988)). Kehle (1978; Reference 3) states that growth faulting typically proceeds aseismically with localized ground tilting and ground disruption primarily along surface traces of active growth faults. It is our interpretation that most or all of the tectonic earthquakes in the Gulf Coast Region occur in the pre-salt basement.

3.6.1.1.4.5 Salt Related Faulting

A major structural agent influencing local basin development was the movement of the Louann Salt in response to the uneven distribution of the overlying sediments. As early as late Jurassic, during deposition of Smackover sediments, mobilization of the salt commenced with migration into linear ridges and continued as additional sedimentation occurred. Where the salt was sufficiently thick and adjacent sedimentation continued, pillowing and then diapirism occurred in the form of salt domes. Major salt migration apparently ceased after diapirism climaxed, and movement finally stabilized during the middle Tertiary in the Interior Salt Basins and during by Pleistocene in the Coastal Salt Basins; however, some domes may be moving near the present shoreline. Offshore doming still continues, associated with modern sedimentation offshore. The timing of salt migration and stabilization in the Interior Salt Basins is reflected in the gradual gulfward shift of the centers of clastic deposition from Cretaceous through Recent times.

Major concentration of faults occur above and adjacent to domes and salt ridge structures. The age and amount of displacement along the faults corresponds to the timing and magnitude of the salt movement. Faults related to shallow piercement salt domes usually extend to or near the surface, while faults caused by deep salt structures generally die out before reaching the surface. This condition indicates that faulting tends to stop soon after domal emplacement ceases.

Normal faulting is generally associated with shallow piercement salt domes because of the extensional forces exercised upon the overlying beds. These faults generally extend to the surface and commonly have a radial pattern. Graben structures also typically form over shallow domes (Reference 12).

Faults not directly related to shallow dome emplacement are also present. Major movement along these normal faults occurred

FOR INFORMATION ONLY

during the Cretaceous, coinciding with the time of maximum salt movement. Most of the faults trend roughly paralleling the strike of the Cretaceous beds.

Growth faults have been and are formed in the Gulf Coast region at active depo-centers. Movement on the growth faults subsides as sedimentation terminates and the locus of deposition proceeds coastward. Movement along the growth faults occurs a seismically. Currently active growth faults are limited to the region of the current deposition i.e., offshore area of the continental shelf.

3.6.1.1.5 Subsidence and Induced Earthquakes Related to Fluid Withdrawal and Injection

Fluid withdrawal or injection has been postulated to be a cause of a number of earthquakes within the region surrounding the site. A summary of this phenomenon was provided by Davis et al. (1989; SAR Reference 15). They cite numerous examples of fluid withdrawal with associated subsidence, in some cases with accompanying earthquakes. Table 2 summarizes the information they provide. Sharp et al. (1991; SAR Reference 61) evaluated widespread subsidence in the Trinity Bay - Port Arthur region of the Texas coast of the Gulf of Mexico. They concluded that depressurization of petroleum reservoirs is likely to be a major cause of the subsidence.

The Mexia, Texas earthquake of 1932 has been associated with oil withdrawal in the Mexia and Wortham Fields. By 1932, 112 MMbbl of oil had been removed from these fields. Production at the time of the earthquake was high. Evidence for the induced nature of this earthquake includes association of the highest Modified Mercalli Intensity with the area of highest hydrocarbon production (SAR Reference 15). Evidence against an induced event includes the occurrence of other earthquakes in the vicinity of the Mexia fault system which are not associated with oil fields.

Fluid injection has been cited as the potential cause of earthquakes in central and western Texas (Davis et al., 1989; SAR Reference 15) as well. A series of small earthquakes near the end of one segment of the South Arkansas Fault Zone has been associated with El Dorado South brine disposal field by Cox (1991; SAR Reference 59). Based on the lack of prior seismicity, the correlation of seismicity with known disposal rates, and location of hypocenters in the basement beneath the well field, the author contends that there is a strong case for induced seismicity.

In summary, seismicity has been associated with the injection and withdrawal of fluids in numerous locations in the Gulf Coast region. In addition, subsidence has been associated with fluid removal. All of the seismicity and subsidence effects have occurred in the immediate vicinity of the pumping activity. The potentially induced earthquakes are all low in magnitude (< 5.0) and are interpreted to occur in the basement below the production horizons. The cause of both subsidence and earthquakes has been hypothesized to be the rapid changes in fluid pressure generated by human activity. While there is a potential for induced earthquakes to occur near the site, the probability is small since there is currently no significant pumping near the site. For more information on wells in the vicinity, see SAR Section __.

3.6.1.1.6 Stratigraphy

The depositional history of north Louisiana is one of more or less continuous basin infilling with periods of marine inundation that formed limestones and an occasional evaporite deposit and regressive conditions where deltaic deposits of sands and shales predominated as the deltas prograded into the basin. The post Louann Salt inundation began in the late Jurassic with the deposition of Smackover carbonates. The southward regression of the sea was followed by a significant clastic influx from the north during the transition of the Jurassic to the Cretaceous, as was marked by Cotton Valley and Hosston deltaic deposits. A period of inundation followed Hosston deposition, as represented by the Sligo Formation. During the early Cretaceous an extensive barrier reef developed peripheral to the lower Gulf Basin and gulfward from the salt basin. Within the salt basin wide spread carbonate reef and lagoonal sediments predominate (see Figures 3.6-3, North Louisiana - Stratigraphic Column, and Figure 3.6-4, Index Map of the North Louisiana Salt Basin).

During the middle Cretaceous the continental emergence was most pronounced, as represented by areas around the Sabine and Monroe uplifts where lower Cretaceous sediments were partly eroded. The regressive phase of this uplift was least noticeable in the central portions of the interior salt basins where clastic deposition was almost uninterrupted. During the middle to late Cretaceous the Woodbine-Tuscaloosa clastic deposits were followed by extensive chalk and marl deposits of the Austin Group, and then marine clays predominated throughout the remainder of the Cretaceous Period.

A general regressive sequence of terrigenous sands and shales were deposited throughout the Tertiary as basin infilling was ending, as represented by the last major deposition of deltaic sediment of the Wilcox Group. The succeeding deposition of the Claiborne Group is characterized by interbedded sands and shales representing cyclic marine and nonmarine conditions. A regressive pattern was again initiated by deltaic building during the Miocene, but associated with the developing down warping of the southern Gulf Basin where the Miocene depocenters predominated.

Pleistocene terrace and rece.. deposits lie unconformably upon the Tertiary sediments which had been undergoing erosion since the Miocene. Pleistocene and Pecent deposits are primarily a product of sea level fluctuations resulting in deposition and erosional processes which were locally controlled by existing structure and lithology of preexisting sediments.

3.6.1.1.7 Mireral Resources

Mineral resources of the North Louisiana Salt Basin are oil, gas, salt, sulphur, lignite, iron ore and construction materials. Only oil and gas are under widespread production at the present time and are mainly from Jurassic through Tertiary age strata. Production has been ongoing since the early 1930s, and activity continues to increase with the development of enhanced recovery techniques. Oil and gas production is from structural, stratigraphic, hydrodynamic and any combination of these trapping mechanisms.

Associated with the numerous piercement salt domes is a significant resource of salt and associated sulphur in the overlying salt domes caprock. Some production of salt and sulphur has been attempted at various times over the past 60 years. At the present, no production of salt or sulphur is ongoing in the North Louisiana Salt Basin.

Lignite occurs in commercial quantities in the Tertiary Wilcox Group. The lignite deposits are of deltaic origin and are characteristically thin and discontinuous deposits. Because of the low BTU quality of lignite and the expense of its production, it is mainly utilized at mine mouth power generating stations where transportation is minimized. Although there are significant quantities of lignite in north Louisiana, the demand for power has not created the need for extensive mining of lignite.

Iron ore in the form of glauconitic strata is abundant in the north Louisiana area, but the ore is at a depth which would make its exploitation uneconomic. However, weathered surficial deposits in the form of iron ore gravel are used as base material and surface material for secondary roads. Construction materials such as sand and gravel (including limestone) are fairly common within the area. Its primary use is also as road base material or aggregate for pavement wearing surfaces.

3.6.1.2 <u>Site Geology</u>

3.6.1.2.1 Physiography

Topographically the 442-acre site is characterized by gently rolling hills with elevations that range from approximately 340 feet above MSL in the central portion to 280 above MSL in the southern portion of the site (see Figure 3.6-5, Site Topographic Map). The change in elevations across the site (relief) is approximately 60 feet. Vegetation is thick and composed of pine forest with some oaks in the uncleared areas. Site drainage is to the west and south where small unnamed creeks have formed at the base of the hills. Topography flattens adjacent to these streams, creating a definite drainage plain. These unnamed creeks join in the southwestern portion of the site where the flow is into a larger drainage feature which flows into Cypress Creek farther to the west and eventually into Lake Claiborne. Drainage for the eastern portion of the property is directed into a small lake, Lake Avalyn, which empties into an eastern unnamed stream and then into McCasland Creek. The entire site lies within the Red River drainage basin (see Figure 3.6-6, Red River Watershed Map).

3.6.1.2.2 Structural Geologic Conditions

Structurally, the site lies on the Claiborne Platform which bridges between the north flank of the North Louisiana Salt Basin and southwest flank of the Monroe Uplift. The proximity of the site to the Monroe Uplift and the flank of the North Louisiana Salt Basin results in a slight southwesterly dip to the nearly horizontal strata. Faulting in Claiborne Parish is related to regional subsidence of the salt basin and salt intrusion. Since the middle Tertiary, faulting has not been active, as sedimentation has ceased and thus the mechanism for basin subsidence and dome growth has been removed.

No salt domes are located at the site. The closest are the Minden Dome to the west in Webster Parish and Gibsland Dome to the south in the Bienville Parish. The site does, however, overlie the northeastern flank of the Homer salt pillar (early mounding stage of salt dome development) which developed some time during the late Jurassic to early Cretaceous and ceased its structural development during the middle to late Cretaceous. The Homer salt pillar is also known as the Darley High (see Figure 3.6-4).

An evaluation of marker beds identified in soil borings shows no structural faulting across the site; however, a slight southwesterly dip does exist. Based on data gathered from previous investigations of the area by Law Engineering, the Darley High extends from the top of the Louann Salt up to the Cretaceous Hosston-Sligo-Pine Island sequence. In the lowest stratigraphic unit mapped (base of the Ferry Lake anhydrite), a northeast-southwest trending fault was identified with approximately 100 feet of throw. However, by deposition of the Pine Island, faulting had died out and the structure is very planar from the top of the Ferry Lake through the Tertiary Cook Mountain Formation.

3.6.1.2.3 Stratigraphy

Site stratigraphy is based on the interpretation of deep oil and gas well logs, seismic data in the site area and shallow (100 feet) site stratigraphic borings. The table below presents the elevation (referenced to MSL) to the top of each formation below the site starting with the Cockfield Formation at the surface.

Cockfield Formation	+350 feet
Cook Mountain Formation	+310 feet
Carrizzo Formation	-740 feet
Wilcox Group	-950 feet
Midway Group	-1250 feet
Nacatoch Formation	-2250 feet
Saratoga Formation	-2430 feet
Eagle Ford Formation	-2890 feet
Tuscaloosa Formation	-2930 feet
Rusk Formation (Mooringsport Member)	-3370 feet
Ferry-Lake Formation	-4400 feet
Rodessa Formation	-4800 feet
James Formation	-5250 feet
Pine Island Formation	-5580 feet
Sligo Formation	-5700 feet
Hosston Formation	-5820 feet
Cotton Valley Group	-8950 feet

The depth to the top of the Jurassic Louann Salt is estimated at approximately -16,000 feet MSL as inferred by seismic data. Thickness of the salt in the Homer area is approximately 2000 - 2500 feet, establishing the Triassic basement at -18,000 to -18,500 feet MSL. Stratigraphically, all geologic units are present from the middle Tertiary Cockfield Formation through the Jurassic Louann Salt. These units thicken as they dip south-southwesterly or basinward. The upper 2500 feet of sediments at the site are consolidated marine and deltaic sediments with major fresh water production coming from the upper 300 feet.

Sediments encountered within the upper 100 feet at the site are identified in the Tertiary Cockfield and Cook Mountain Formations along with recent deposits of alluvium in and adjacent to drainages. The Tertiary deposits exhibited planar deposition and southwesterly dip similar to those of the deeper sediments previously discussed.

Surface soils encountered at the site are of the Gilead and Shubuta Soil associations and developed from the weathering of the Cockfield Formation. These soils are grayish-brown fine sandy loams on the more strongly sloping areas and are well drained. The Cockfield Formation is divided into a marine and a non-marine unit. The non-marine unit, which is primarily composed of light brown fine-grained sands with some silts and clays, is exposed at higher elevations in the northern areas of the site. This unit is underlain by the marine unit of the Cockfield. Layers of siderite were observed in the lower unit as exposure on hillsides in the northern portion of the site. These layers created a perched water table on the hilltops as exhibited by some springs percolating from the sides of hills above the siderite layer. Thick outcrops of siderite were also observed along ridges on the southern portion of the property (outside of the boring exploration area). These siderite outcrops are not continuous throughout the area. Below the siderite layer are characteristically massive crossbedded sands and glauconitic sands which are dark green but weather to red and brown as exhibited by the soil cover. Total thickness of the lower unit is up to 50 feet thick and lies conformably over the Cook Mountain Formation.

The Cook Mountain Formation, which can be up to 300 feet thick, is divided into five lithologic units. The lowermost unit (lower sand) consists of up to 20 feet of glauconitic sand, siderite ledges and an abundance of marine fossils. The lower unit is overlain by a calcareous fossiliferous clay up to 60 feet thick with silt and clay near the base. The overlying silt and clay unit, about 60 feet thick, consists of alternating beds and almost pure silt in the upper 20 feet. The middle sand unit averages about 80 feet thick and is typically composed of fine-grained crossbedded, slightly glauconitic sand with some clay and silt stringers. The uppermost unit is composed of silt and clay about 40 feet thick and grades upward into 20 feet of glauconitic sand with iron (siderite) ledges and marine fossil casts. This in turn grades upward into about 20 feet of alternating thin beds of silt and clay which become sandy in the upper 2 to 3 feet before grading into the Cockfield Formation.

Pleistocene terrace deposits lie unconformably upon the erosional surface of the Tertiary formations. At the site however, these terrace deposits do not overlay the Cockfield and are thought to have been eroded and incorporated into Recent alluvial sediments.

Recent alluvial sediments and colluvial sediments are present in downslope and low lying areas adjacent to and in the drainages. These sediments are composed of light brown to gray sand with some silt, clay and chert gravel. The main criterion for distinguishing recent sediments from Tertiary sediments is the lack of bedding, unconsolidated nature and presence of chert gravel. Quaternary terrace deposits have the same characteristics but may be slightly cemented in whole or part.

3.6.1.2.4 Mineral Resources

Mineral resources of the North Louisiana Salt Basin are discussed in Section 3.6.1.1.6. Mineral resources currently being developed in the site vicinity are oil and gas and a minor amount of construction materials in the form of sand, limestone gravel and siderite gravel.

As shown in Figure 3.6-7, North Louisiana Oil and Gas Field Map, several oil and gas fields are located in the vicinity of the site. Activity in some of these fields has been good over the past 40-50 years but has declined, along with production and exploration all over North Louisiana, in the last 10-15 years.

Sand, limestone gravel and siderite gravel are abundant in the vicinity of the site. Commercial marketing of this resource is almost non-existent due to the lack of market demand in the regional construction industry.

3.6.1.3 Geotechnical Exploration

An in-depth field investigation and laboratory testing program was performed to explore subsurface conditions at the site. The purpose of the investigation is to identify and characterize the subsurface soil, groundwater and geologic conditions. The field investigation and laboratory testing program are presented in Sections 3.6.1.3.1 and 3.6.1.3.2, respectively.

3.6.1.3.1 Field Investigation

The following subsections describe the details of the field investigation program performed at the site.

3.6.1.3.1.1 Soil Test Borings and Sampling

A series of 40 soil test borings, together with 15 electric cone penetrometer tests (CPT) and 13 test pits, were performed at the site. Locations of test borings, CPT's and test pits are shown in Figure 3.6-8. Table 3.6-1 indicates the boring type, depth, ground elevation and location.

Soil test borings were accomplished utilizing two All-Terrain Vehicle (ATV) drill rigs and rotary wash boring techniques. All drilling and sampling operations were performed in general accordance with current ASTM Specifications and the Site Subsurface Investigation Specification (Reference 13). Soil samples were obtained continuously (2-foot intervals) in the upper 10 feet and at intervals of 5 feet thereafter. Standard penetration test (ASTM D 1586) and thin-wall tube sampling (ASTM D 1587) methods were used to obtain soil samples. When soils in the lower stratum became too hard to push thin-wall tubes, relatively undisturbed samples were obtained using a pitcher sampler. All soil samples were returned to the laboratory for further classification and testing. Copies of soil test boring logs are contained in the Geotechnical Exploration Report (Reference 1).

3.6.1.3.1.2 Groundwater Measurements

Temporary piezometers were installed in Test Borings B-10, B-24, B-40 and B-48 following the completion of soil sampling. These piezometers were installed to determine the stabilized groundwater level at representative locations across the site. Groundwater levels in the piezometers were measured daily during the field exploration program and are listed in Table 3.6-2.

3.6.1.3.1.3 Electric Cone Penetrometer Tests

CPTs were performed at 15 locations within the site Process Area (i.e., Separations Building and Centrifuge Assembly Building). Twelve of the locations were between soil test boring locations, and three were located adjacent to borings for direct correlation of test data. Figures 3.6-9, 3.6-10 and 3.6-11 exhibits the test data comparison for the CPTs located adjacent to the soil test borings.

CPTs were accomplished utilizing a 20-ton CPT truck with electronic data collection capability. The truck-mounted cone penetrometer testing equipment was selected in order to provide sufficient thrust resistance to achieve the maximum possible penetration at each location.

Electric CPTs were made in general accordance with ASTM D 3441. Continuous data was obtained to the maximum depth possible without damaging the electrical strain gauges in the probe. Continuous logs (plots) of point resistance, side friction, and friction ratio are presented in the Geotechnical Exploration Report (Reference 1). Computer interpreted cone data, including soil type, equivalent blow count, friction angle and undrained shear strength, are also included with the logs.

3.6.1.3.1.4 Test Pits

A total of 13 test pits were excavated to obtain disturbed bag samples to evaluate the suitability of soils for yard fills and backfills. Test pits were excavated to depths of 10 to 11 feet using a backhoe. Samples were obtained in general accordance with ASTM D 1452 at representative depths and at changes in the stratigraphy.

Test pit records are contained in the Geotechnical Exploration Report (Reference 1). Test pit locations are identified in Figure 3.6-8.

3.6.1.3.1.5 Geophysical Surveys

The geophysical surveys consisted of down-hole seismic testing at soil test boring locations B-15 and B-17, and crosshole seismic testing at locations B-17 and B-27. Crosshole and downhole seismic tests were performed to obtain insitu seismic velocities of the subsurface soil.

All testing was performed in general accordance with ASTM D 4428. Each seismic test borehole was measured for verticality using an inclinometer. This enabled the data to be corrected for any deviations in alignment.

3.6.1.3.2 Laboratory Testing

In the laboratory, soil samples from the field investigation were further examined and visually classified in accordance with the Unified Soil Classification System (USCS). During this review process, laboratory tests were assigned to selected soil samples representative of the site soil stratigraphy.

Laboratory tests were conducted to further classify the soils, in addition to measuring the strength, consolidation, swell potential, corrosion potential and dynamic properties of the soil. Additional laboratory tests were assigned to bulk samples from the test pits to determine permeability and compaction properties. The types of laboratory tests performed included Atterberg limits, moisture content and unit weight, grain size analysis, triaxial compression, consolidation, permeability, shrink-swell, pH and resistivity, expansion, standard Proctor, CBR and resonant column tests. Laboratory test results, together with a review of appropriate test procedures are presented in Appendix 3.6-1, Laboratory Testing.

Resonant column tests were performed for three soil samples obtained from the upper soil stratigraphy in the Process Area. Test results and discussion are presented in Appendix E of the Geotechnical Exploration Report (Reference 1).

3.6.1.3.2.1 Crosshole and Downhole Seismic Testing

The seismic crosshole and downhole investigation produced P- and S-wave velocities at the three locations. In general, both P- and S-wave velocities are low at the surface and tended to increase to a depth of about 40-50 feet. One or two lower velocity zones were detected in the upper 50 feet. From 50 to 100 feet, the P- and S-wave velocities were relatively consistent. P-wave velocity varied from approximately 800 to 6,000 ft/s in the upper 40 feet. Below 40 feet, P-wave velocity was approximately 5,000 to 7,000 ft/s. Shear velocities ranged from approximately 300 to 1,100 ft/s in the upper 40-50 feet. Below 50 feet, the shear wave velocity is approximately 1,200 to 1,800 ft/s.

Tables 3.6-3, 3.6-4, 3.6-5 and 3.6-6 present the interpreted P- and S-wave velocities with depth at the test locations. In Section 3.6.2.2.4, these results are combined with other geotechnical parameters to produce models of the subsurface elastic properties used in computing the site-specific response spectra.

For comparison, velocities from the crosshole and downhole testing at B-17 are shown in Figure 3.6-12. Below about 35 feet, the two test methods yield similar results for both P- and S-wave velocities.

Above 35 feet, the crosshole tests infer higher P-wave velocities than the downhole tests. There are several possible causes for the difference in velocities.

a. The crosshole method measures velocities in the horizontal direction while the downhole method measures a smaller sampling area in the vertical direction.

b. Thin competent layers would tend to affect the crosshole data more than the downhole data.

c. Interference from the grouted borehole can make the P-wave arrivals difficult to interpret in the downhole method.

d. The saturation of the soil from rains which occurred after the downhole testing but before the crosshole testing may have resulted in the P-wave velocity differences.

Some difference in shear wave velocity measured by the two techniques at B-17 is normal (Reference 14). The differences between the two techniques reflect the different sampling volumes, the wave propagation direction of each technique and anisotropic velocity distribution.

Measured seismic velocities from the geophysical surveys are shown in Figure 3.6-13 and Figure 3.6-14. Both crosshole nests indicate similar velocity with depth relationships (see Figure 3.6-13). At a depth of approximately 30 feet, both nests indicate a lower P-wave velocity zone exists.

Both downhole tests indicate increasing velocity with depth with some low velocity layers (see Figure 3.6-14).

Dynamic shear modulus, Young's modulus and Poisson's ratio values based on shear and compression wave velocities measured in the crosshole seismic survey are presented in Table 3.6-7, Dynamic Soil Properties (Before Site Grading). These values represent the existing conditions prior to site grading. Estimated dynamic soil parameters corrected for stress changes resulting from site grading are presented in Table 3.6-8. Dynamic Soil Properties (Estimated After Site Grading).

FOR INFORMATION ONLY

3.6.2 ANALYSIS OF GEOLOGIC STABILITY

Most earthquakes which occur in the United States are located in the tectonically active western portion of the country. However, areas of the central and eastern United States may also experience seismic activity, although at a lower rate. Earthquake activity in the central and eastern United States has included such large events as the 1811-1812 New Madrid earthquakes which occurred in Missouri and Arkansas, and the 1886 Charleston, South Carolina earthquake.

3.6.2.1 Seismic History of Region and Vicinity

The historical record of earthquakes in the south central United States began with the settlement of the area in the early nineteenth century. This section discusses the general level of activity and individual important earthquakes in the portions of the southern states which may affect the north-central Louisiana area.

Figure 3.6-15 indicates locations of earthquakes which have occurred within the region. All earthquake intensities in this report use the Modified Mercalli Intensity scale presented in Appendix 3.6-2. Associated ground motion parameters are not listed in Appendix 3.6-2 because the intensity registered at a site depends upon a number of dependent factors, which are difficult to separate. Felt reports and building damage depend upon duration of shaking, frequency content of shaking, the responses of structures and soils, as well as upon peak acceleration and velocity.

Examination of available literature and earthquake catalogs revealed that the nearest intensity reports for earthquakes are reported for Shreveport, Louisiana, approximately 50 miles west of the site.

The earthquake data used in the seismic study came from several sources (References 15, 16 and 17) and a catalog of earthquakes from the Electric Power Research Institute (EPRI) study on seismic hazard methodology east of the Rocky Mountains.

3.6.2.1.1 Regional Earthquakes

Historical data indicate that several large, distant earthquakes may have been felt in Claiborne Parish, Louisiana. These distant earthquakes are responsible for the highest intensity shaking reported or interpreted for north-central Louisiana.

3.6.2.1.1.1 1811-1812 New Madrid Earthquakes

This series of earthquakes which occurred during the winter of 1811-1812 consisted of four major earthquakes (m_{bLg} = 7.2, 7.0, 7.1, 7.3) and numerous smaller events (Reference 18). The earthquakes occurred in an area extending from southeastern Missouri into northeastern Arkansas. These earthquakes were the largest to occur in the lower 48 states.

In our assigning MMI = VI to the site for the December 16, 1811 earthquake, we evaluated the location and geology of the site as well as the various sources of intensity information.

Algermissen, 1983 and Bolt, 1978 both reference, as a source on the December 16, 1811 earthquake, work done by Otto Nuttli prior to 1978. We have used more recent studies by this author and by Ron Street (Street and Nuttli, 1984, Reference 18; Street, 1982, Reference 56). Their interpretation of the intensity maps has changed with time. In the Louisiana area, the overall effect has been a lowering of the reported MMI values. The most recent interpretations estimate the intensity felt in northern Louisiana at I=VI. A recent tendency has been to omit the isoseismal lines from the interpretation maps leading investigators to evaluate sites by considering epicentral distance and site geology.

The intensity of shaking experienced at the site from the New Madrid earthquake sequence can only be estimated since there are no intensity reports for the vicinity. The isoseismal maps referenced show generalized isoseismal lines. Specifically, there is a I=VII line passing through north central Louisiana. Detailed analysis of available intensity reports and more modern interpretations, shows that I=VII was typical for alluvial valleys which tend to amplify shaking, while no reports were available for high ground sites. It is our interpretation that if a town had existed on the CEC site in 1811, the reported MMI would be VI.

Using Modified Mercalli Intensity to directly estimate shaking for the New Madrid Earthquakes at this site is not appropriate because the site intensity can only be guessed at and there are better methods for modeling strong ground motion.

Intensity data for Louisiana resulting from the large New Madrid earthquakes is scarce. Intensities, in or near what is now the state of Louisiana, range from VII at Vicksburg, Mississippi, to III-V reported in New Orleans, Louisiana (Reference 19). Examination of intensity data along the Mississippi River for the December 16, 1811 (8:15) earthquake may give a Modified Mercalli Intensity of VII as the highest for the State, based on data from sites in alluvial valleys which tend to amplify the shaking. A similar level of intensity was interpreted by Davis, *et al.*, (Reference 15) for northeast Texas. The highest verifiable intensity report for Louisiana was VI, reported for the Town of Washington in south-central Louisiana. Based on the above reports, intensity VI shaking was probably produced in the site region by the four major New Madrid earthquakes.

3.6.2.1.1.2 1843 Mississippi Valley Earthquake

This event occurred on January 4, 1843, in the Mississippi River valley near Memphis, Tennessee. The epicentral intensity is estimated to have been VIII with an m_b 6.1. This earthquake was felt over a large area including northern Louisiana. Nuttli (Reference 20) places the intensity IV isoseismal near the site in Claiborne Parish, Louisiana.

3.6.2.1.1.3 1886 Charleston, South Carolina Earthquake

This large earthquake (m_b 6.8) occurred in coastal South Carolina on September 1, 1886, and produced shaking throughout the eastern United States, including the region surrounding the site (Reference 21 and 43). Intensity II shaking was detected at Shreveport, Natchitoches and Alexandria, Louisiana and at Hampton and Monticello, Arkansas (Reference 19). Intensity II shaking probably occurred at the site from this earthquake.

3.6.2.1.1.4 1895 Charleston, Missouri Earthquake

This large earthquake (m_b = 6.2) occurred on October 31, 1895, at the northern end of the New Madrid Fault Zone near Charleston, Missouri. The effects of this earthquake were felt as far as New Mexico (Reference 20).

Intensity III shaking was reported for Louisiana (Reference 19) and it is assumed that similar shaking occurred at the site.

3.6.2.1.2 Near Earthquakes

This section discusses significant historical earthquakes located within 320 km (200 mi) of the Claiborne Parish, Louisiana site. Table 3.6-9 lists earthquakes which have occurred near the site through 1985. The table includes the time of occurrence, location of epicenter, distance from Homer, Louisiana, magnitude, felt area and epicentral Modified Mercalli Intensity (I_o).

3.6.2.1.2.1 Near Earthquakes - Ouachita Region

3.6.2.1.2.1.1 1882 Ft. Gibson, Oklahoma Earthquake

Occurring on October 22, 1882, this earthquake was originally reported to be located near Paris, Texas (Reference 22). Reevaluation of newspaper accounts and work by earlier researchers led to relocating the earthquake approximately 240 km (150 mi) farther north near Fort Gibson, Oklahoma (Reference 15). This relocation places the event far outside the 320 km (200 mi) radius of the site.

The earthquake was assigned a magnitude, $m_b = 5.5$. The CEC site is located at the southeastern edge of the felt area for this earthquake.

3.6.2.1.2.1.2 1911 Rison, Arkansas Earthquakes

An earthquake and aftershock occurred on March 31, 1911, near the south central Arkansas towns of Rison and Warren. At Rison, houses swayed and articles were thrown from shelves. The main shock was felt throughout southeastern Arkansas, northeastern Louisiana and along the Mississippi River from Memphis to Vicksburg, an area roughly 200 miles north-south by 100 miles east-west (Reference 23). The main event was given a magnitude $m_b = 4.6$ and an epicentral intensity of VII. The aftershock was less strongly felt (I_o = IV-V).

Neither earthquake was reported to be felt in the Homer, Louisiana area (Reference 19).

3.6.2.1.2.2 Near Earthquakes - Interior Salt Basin Region

3.6.2.1.2.2.1 1891 Rusk, Texas Earthquake

This earthquake occurred on January 7, 1891, near Rusk, Texas. The epicentral intensity is given as either VI or VII, depending on the interpretation of actual earthquake structural damage. Other intensity information for the area suggests I_o = VI (Reference 15). The nature of the damage supports a shallow focus earthquake of approximate magnitude $m_b = 3.8$.

Although this event was not felt in Claiborne Parish, it occurred in the same tectonic province. The 1891 Rusk, Texas earthquake was located near the Mount Enterprise Fault System.

3.6.2.1.2.2.2 1940 Rodessa, Louisiana Earthquake

This earthquake, occurring on December 2, 1940, is the nearest event to the site. The earthquake was located 96 km (60 mi) from the site. The epicentral intensity of the earthquake was assigned an intensity of IV and an estimated magnitude of $m_b = 3.1$. No damage near the site was reported from this earthquake.

3.6.2.1.2.2.3 1957 Gladewater, Texas Earthquakes

A series of four earthquakes was felt near the Texas Town of Gladewater on March 19, 1957. The largest of these earthquakes had a maximum intensity of V. The magnitude was estimated to be $m_b = 4.0$ for the main shock and $m_b = 2.5$ for the three aftershocks.

These events may also have been related to fluid withdrawal, because they were located in the area of greatest well density in the northern portion of the East Texas oil field. In the 27 years preceding these earthquakes, 3.5 billion barrels of oil had been extracted from the field (Reference 15).

3.6.2.1.2.2.4 1964 Hemphill-Pineland, Texas Earthquakes

From April 24 to August 19, 1964, a series of earthquakes occurred near the east Texas towns of Hemphill and Pineland. At least thirteen events were felt, with the largest having an epicentral intensity of VI and magnitude m_{bLg} = 3.6 (Reference 16). After seismographs were installed in the area in July 1964, seventy shallow focus earthquakes were recorded before the swarm ended on August 19 (Reference 15).

Although they occurred between two large reservoirs, these earthquakes were not caused by the infilling or water level fluctuations, since that did not begin until 1965. One possible explanation for this seismic activity is movement along the nearby Angelina-Caldwell Flexure (a hinge line parallel to the coastline) due to sediment loading in the Gulf of Mexico. These events may also have been related to nearby oil and gas fields.

3.6.2.1.3 Seismic Hazard

The following sections describe the evolution of the state of the art in seismic hazard analysis used in the Gulf Coast region. In addition, the sections provide a description of the methodology used both for the probabilistic hazard analysis and for the determination of the design earthquakes for the facility. The results of the probabilistic hazard analysis are presented in the form of a table of accelerations for 100, 500 and 1000 year return periods, and a graph of acceleration versus probability. The computed accelerations are modified to represent effective acceleration values on soil. Near and far field design earthquakes are presented. In Section 3.6.2.2 appropriate response spectra are developed.

3.6.2.1.3.1 Development of Seismic Hazard Analysis

Prior to the licensing of nuclear power plants, the question of the degree of seismic hazard in areas of the United States with low seismicity, such as northern Louisiana, was not of much interest to the designers or owners of industrial or public service facilities.

The seismic zone maps used in building codes reflected the influence of practitioners from the far west. Those maps are based primarily on the pattern of historic seismicity. The Uniform Building Code Seismic Zonation Map places the CEC site in Seismic Zone 1 which is characterized by minor damage from distant earthquakes.

The causes of earthquakes in central and eastern United States are not well known. In this intraplate region, the causative faults are not exposed at the surface and, with the exception of the New Madrid Fault Zone, the occurrence rate of earthquakes is not great enough to delineate the causative tectonic structures. Also, until recently there were few seismic recording stations in areas of low seismicity such as northern Louisiana. Lack of recording stations results in a lack of detection of small earthquakes and imprecise location of earthquakes that were detected.

In the midwest and east, the concept of "tectonic provinces" is relied upon to deal with the uncertainty regarding the causes and location of future earthquakes. A tectonic province, as defined in Appendix A to 10 CFR 100, is a region "characterized by a relative consistency of geologic structures contained therein". For nuclear power plants, regulations require that seismic design be based on the largest historical earthquake within the tectonic province hosting the facility. As a minimum, the facility must be designed for a peak horizontal acceleration of at least 0.1 g and a NRC specified design spectrum. In the Gulf Coast area, site-specific spectra were occasionally proposed when the applicant considered the NRC spectra inappropriate.

In 1984 the Electric Power Research Institute (EPRI) began a comprehensive study titled "An Evaluation of Seismic Source Zones in the Eastern United States East of 105 degrees". This study was in response to considerable research into the causes of earthquakes in the central and eastern United States.

The study contributed significantly to the understanding of intraplate seismicity in the United States. Specific results of the study included:

a. A catalog of central and eastern earthquakes with uniform magnitude estimates.

b. A rationale for estimating the maximum earthquake for a region rather than using the maximum historic earthquake.

c. Advances in strong ground motion modeling.

d. Better understanding of seismic source zones.

A similar, parallel study was also performed by the Lawrence Livermore National Laboratory (LLNL) (Reference 24). The current state-of-the-art for seismic hazard analysis was developed from both the EPRI study and the LLNL study.

3.6.2.1.3.2 Probabilistic Hazard Analysis

The object of the probabilistic seismic hazard analysis is to develop a realistic curve of maximum ground acceleration versus probability (or return period). In Section 3.6.2.2, the same methodology is extended to probabilistically determined response spectra characteristics. The probabilistic hazard analysis approach is to use discrete seismic source zones representing areas of similar geology at seismogenic depths, geologic history, stress conditions and historic seismicity. Each zone is assigned a maximum earthquake magnitude as described in Section 3.6.2.1.3.5. These source zones are referred to as seismotectonic regions.

Within each source zone, the temporal occurrence of earthquakes is assumed to follow the Poisson statistical process, and the spatial seismic activity is assumed to be homogeneous (i.e., earthquakes are assumed to occur at all points within the source zone with equal likelihood). Each source zone is divided into a large number of small grid areas, with each grid area having a representative portion of the total source zone's activity. Ground motion at the CEC site is estimated from each of the grid points by use of an appropriate ground motion attenuation model with an associated scatter distribution. The frequency of exceeding a specified acceleration level is calculated by integrating the contributions of earthquakes of various sizes that have some probability of occurring in each of the earthquake source zones. The end product is a curve showing ground motion versus annual probability of exceedence (see Figure 3.6-16).

3.6.2.1.3.3 Estimation of Ground Motion

The ground motion parameters were calculated using the random vibration theory (RVT) method. The RVT method uses results from random vibration analysis to make estimates of peak ground motion and spectral velocity from Fourier amplitude spectra (References 25, 26, 27 and 28). The RVT method provides a way for direct computation of theoretical response spectra in addition to providing values of peak acceleration, velocity and displacement. RVT models were an important part of the ground motion input for both the EPRI and the LLNL studies.

Determining the input parameters for the RVT model involves defining how the earthquake Fourier source spectrum scales with earthquake magnitude, as well as defining the effects of geometric and anelastic attenuation. Source scaling and attenuation parameters appropriate to the eastern and central United States is utilized for the analysis (References 29 and 30). Since considerable hazard comes from the New Madrid Fault Zone, anelastic attenuation appropriate to the central United States is used rather than the more attenuating Coastal Plain values.

The RVT modeling used source scaling and attenuation properties appropriate to the eastern United States. The use of RVT modeling to derive ground motion parameters is becoming more widespread (see Sections 3.5.21.3.3 and 3.6.2.2.4.3.3). The method has been shown to correlate well with recorded data depending on the source scaling and attenuation factors used. Uncertainties arise in regions where source characterization and seismic wave attenuation properties are poorly known.

Attenuation properties have been shown to be relatively uniform throughout eastern North America with slight variations from region to region (Singh, 1981, Dwyer, et al., 1983; References 54 and 46, respectively). Uncertainties in attenuation are not likely to affect estimates of probabilistic ground motion in the SAR.

The uncertainty in source scaling may have a greater effect than variations in attenuation. The RVT source scaling used in this study tends toward events richer in frequency and longer in duration than other RVT source scaling models.

The RVT modeling was used to calculate ground motion parameters. It was also used to help synthesize the far-field DBE. Basic Newmark-Hall spectra for the far-field DBE were developed using RVT scaling and then modified using amplification factors developed using site-specific modeling (SHAKE).

3.6.2.1.3.4 Design Earthquake

The design earthquake is specified as having a 500-year return period. The maximum acceleration for that return period is taken directly from the probabilistic hazard results. Since the design acceleration can be the result of a large distant event, a modest close event, or many combinations in between, three design earthquakes are investigated.

a. Local near-field earthquake

b. Mid-field earthquake centered 100 km (62 mi) from the site

c. Far-field earthquake from the New Madrid area

3.6.2.1.3.5 Seismotectonic Regions

As discussed in Section 3.6.2.1.3.2, a seismotectonic region approach to seismic risk analysis is used for the seismic hazard analysis. The seismotectonic regions within 320 km (200 miles) are shown in Figure 3.6-17. The regions include:

a. Interior Salt Basin Region

b. Gulf Coast Region

c. Central Texas Region

d. Ouachita Region

e. Wichita - Arbuckle Region

f. Reelfoot Rift

g. New Madrid Fault Zone

h. Central Stable Region

i. Mississippi Embayment

The individual seismotectonic regions defined within the south-central United States are briefly discussed in the following paragraphs. For each source zone, a maximum magnitude (m_b) is assigned which is typically greater than the maximum historic earthquake magnitude for that source zone.

a. m_b 7.4 represents events associated with well developed rifts that can support fault lengths greater than 30 km (19 mi), that have evidence of reactivation in the last 100 million years and that open on one end to oceanic or extensional crust (e.g. 1811-1812 New Madrid).

b. m_b 6.8 represents events associated with rift structures surrounded by continental crust and with poorly defined features; considered unable to support m_b 7.4 events. If activity is low (1000 year event << 6.8) then use 1000 year event.

c. m_b 6.8 represents events associated with failure of a significant thickness of brittle crust but in areas where inhomogeneities in the crustal structure and stress field prevent development of an extended length of faulting (e.g. Charleston).

d. m_b 5.7 represents events associated with crystalline rock areas where depths of focus are typically less than 10 km (6 mi) (e.g. New Brunswick).

e. m_b 5.7 is used for source zones that have moderate seismicity with some events in the m_b 4.0 to 5.7 range, but do not display tectonics or seismicity evidence of large throughgoing discontinuities in the brittle crust.

f. m_b 5.5 is used for source zones that capture little seismicity, have a historic maximum earthquake less than m_b 4.0. Because of insufficient evidence, the probabilistic analysis limits the maximum magnitude to less than m_b 5.5.

g. m_b 4.9 represents upper limit event, where enough evidence is available considering both historic seismicity and causative mechanism, to imply that maximum $m_b < 5.0$.

3.6.2.1.3.5.1 Interior Salt Basin Region

The CEC site is located within the Interior Salt Basin Region. The Interior Salt Basin seismotectonic region is the area south of the Gulf Coast Basin boundary fault system and north of the Angelina-Caldwell Flexure in Texas and Louisiana and the Wiggins Uplift in Mississippi. The Interior Salt Basin Region contains three major salt dome basins (Mississippi, North Louisiana, and East Texas). Other major structures include the Sabine and Monroe uplifts. The region is tectonically stable and is currently in an emergent, erosional cycle. Some earthquakes have been spatially and temporally associated with hydrocarbon production.

The seismicity of the region is low. Only six earthquakes with magnitudes exceeding 3.5 have been reported in the area. The largest event to occur was a magnitude 4.1 earthquake which was located more than 200 miles from the site. The immediate site area has lower seismicity than the Region as a whole. A magnitude of 4.9 is used as the maximum magnitude for this region in the hazard analysis.

The use of a $m_b = 4.9$ maximum magnitude is based on seismicity and tectonic considerations. The basis for assigning a maximum likely earthquake of $m_b = 4.9$ to the Interior Salt Basin Source Zone is rooted in previous work by Law Engineering on reevaluating seismic hazard in the central and eastern United States (Law Engineering, 1986, SAR Reference 17). Evaluation of maximum earthquake is based on regional seismicity and nature of faulting.

Seismicity: The rate of earthquake activity in the region is among the lowest east of the Rockies. In general, the rate of modern seismicity is considered to be a reflection on seismicity in the recent past and the near future. The computation of a 1000-year event is generally appropriate in areas of relatively high seismicity (Nuttli and Herrmann, 1978). Nuttli and Herrmann (1978) found that, in relatively active areas, the 1000-year event was roughly equivalent to the maximum earthquake. The Gulf Coast is not such an area.

The largest recorded earthquake within the Interior Salt Basin had an $m_b = 4.1$ which is 0.8 magnitude units below the assigned maximum.

Faulting: The date of last movement of most faults in the region is interpreted to be Eocene. In addition, most of the fault movement was normal slip which is inferred to be related to salt movement or sedimentation. In any case, such movement does not coincide with the compressive stress field dominant in the eastern United States. One composite focal mechanism from earthquake first-motion data is available from earthquakes located in the vicinity of the Mt. Enterprise Fault Zone (Pennington and Carlson, 1984; SAR Reference 11). This mechanism shows normal faulting with a small strike-slip component with a fault plane oriented either N15E or N75E and dipping 62 deg. southeast. Another focal mechanism is available for the 1983 Lake Charles, Louisiana earthquake near the Louisiana coast (Stevenson and Agnew, 1988; SAR Reference 55). The mechanism shows predominantly norma-slip faulting on either a N55E dipping 40 deg. southeast or N80W dipping 64 deg. northeast. In the Gulf Coast, major normal faults dip in a southerly direction.

The nature of faulting in the region favors a low maximum magnitude. In-situ stress measurements, other than the few composite focal mechanisms, are generally not available for the region. Those that are available are from geothermal fields or are from above the basement. In either case, these do not represent the stress conditions at seismogenic depths.

The depth of the near-field DBE was selected to be the top of the crystalline basement. This is considered to be the shallowest possible depth for a damaging earthquake. While many of the earthquakes in the region are considered to be shallow, none are believed to be shallower than the top of the basement. Even the events possibly caused by fluid injection or withdrawal are considered to have occurred within the basement (Cox, 1991, SAR Reference 59; Collins et al. 1989, SAR Reference 10). It is conceivable that shallower earthquakes can occur but the strength of the subsurface materials would likely limit the magnitude of the event to non-damaging size.

At the time of the SAR, the highest recorded magnitude in the Interior Salt Basin was m_b = 4.1 for an earthquake over 200 miles from the site. The closer 1891 Rusk, Texas earthquake had a Modified Mercalli Intensity of VI. Studies of intensity reports indicate that this event had a relatively shallow focus. Davis et al. (1989, SAR Reference 15) estimated that the magnitude is m_b = 4.0 based on felt effects. These two events are the largest recorded in the Interior Salt Basin. It has long been postulated that the maximum magnitude for a tectonic region was related to the seismic activity of the region. The low seismicity rate in the Interior Salt Basin implies a low maximum magnitude.

The Interior Salt Basin is stable in the modern tectonic environment. Movement in the LouAnn Salt within the basin has stopped and the interior salt basins are isostatically stable. What minor movement exists is related to tilting toward sediment loading at the coastal depocenters. This movement is evidenced by flexure. Transmission of stress from the North American plate to the crust in the Gulf Coast is not well coupled because of the faulted nature of the basement. Growth faulting occurs toward the coast above the basement but these faults move aseismically.

The pre-salt basement rocks located within the Interior Salt Basin are composed of faulted blocks. These basement rocks were faulted to a considerable degree by the extensional stress and thermal uplift associated with continental rifting as late as early Jurassic. In the current stress regime, the Interior Salt Basin experiences very low levels of seismicity (see Figure 3.6-17 and Table 3.6-9). In order for significant levels of seismicity to occur (including the occurrence of strong earthquakes), there must be sufficient magnitude of stress in the proper orientation to reactivate old faults or create new ones. There is no evidence for favorable stress orientation or magnitude as demonstrated by the low levels of historical and recorded seismicity.

Geologic evidence for the lack of modern faulting in the vicinity of the site is the planar nature of the sediments in the Claiborne Platform indicating no significant movement since at least the Eocene (Durham, 1964, SAR Reference 45).

The seismicity which occurs in the Interior Salt Basin is spatially nonuniform. The site is located in an area of relatively low seismicity compared to the rest of the Interior Salt Basin. The seismic hazard calculations assume that seismicity within a zone is uniform, therefore the calculations of seismic hazard at the site may be conservative. In addition, the choice of maximum magnitude (m_b = 4.9) may be conservative as well. The highest recorded magnitude in the Interior Salt Basin, was about m_b = 4.1 which was for an earthquake located over 200 miles from the site in the East Texas Salt Basin.

3.6.2.1.3.5.2 Gulf Coast Region

The Gulf Coast seismotectonic region is the area north of the Sigsbee Deep in the central Gulf of Mexico. Major structures include the South Louisiana, South Texas and Houston embayments. The offshore portion of the region is undergoing slow subsidence due to sedimentation and is characterized by active growth faults. The growth faults do not extend to the basement rock, nor do they generate earthquakes, since the displacement occurs as creep which affords a continual release of stress.

The seismicity of the Gulf Coast Region is low, and no correlations can be made between epicenters and individual geologic structure. Four earthquakes exhibiting magnitudes between 4.2 and 4.4 have occurred in the historical record. A magnitude of 4.9 is used as the maximum magnitude for this region in the hazard analysis.

3.6.2.1.3.5.3 Central Texas Region

The Central Texas seismotectonic region is the area south of the Wichita-Arbuckle Uplift, east of the Rio Grande Rift Zone and west of the Mexia-Talco (boundary) Fault Zone. Major structures include the Llano Uplift, the Permian Basin and several other broad basins and arches. The Central Texas Region is a tectonically stable platform area. The surface faults within the region are considered inactive.

Seismicity of the region is low. Only eight earthquakes exhibiting magnitudes in excess of 4.0 have occurred in the historical record. The largest historic earthquake of the Central Texas seismotectonic region was a magnitude 4.6. A magnitude of 5.7 is used as the maximum magnitude for this region in the hazard analysis.

3.6.2.1.3.5.4 Ouachita Region

The Ouachita seismotectonic region is a belt of deformed rocks south of the Central Stable Region, north of the boundary fault system and east of the Arbuckle Mountains. The Ouachita Region is an orogenic belt composed of several over-thrust sheets of tightly folded, metamorphosed Paleozoic sediments. The Ouachita Trend is thought to extend eastward beneath the Coastal Plain or the Mississippi Embayment, into the folded Appalachian Mountains. This regional trend has numerous surface and subsurface faults; however, the faults have not been directly correlated to earthquakes. The comparison of earthquake epicenters reveals that earthquakes have been widely distributed throughout the region. Since the causative mechanisms of earthquakes in this region are not well understood, all the folded areas within the Ouachita Region are arbitrarily considered to have equal potential for earthquake occurrence. The largest historic earthquake had a magnitude of 4.8. A magnitude of 5.7 is used as the maximum magnitude for this region in the hazard analysis.

3.6.2.1.3.5.5 Wichita-Arbuckle Region

The Wichita-Arbuckle seismotectonic region is the area south of the Central Stable Region, north of the Central Texas Region and west of the Ouachita Region. Major structures within the region include the Wichita Uplift, the Arbuckle Mountains, the Anadarko Basin and the Muenster Arch. The region includes a failed Eocambrian aulacogen.

The Meers Fault lies within this source zone (more than 200 miles from the site). The fault was discovered in the 1930s and is evidenced at the surface by a long scarp. Quaternary displacement has been inferred by some researchers. The maximum magnitude range is estimated to be between 6.1 and 6.6.

The seismicity of the Wichita-Arbuckle seismotectonic region is similar to that of the Ouachita seismotectonic region to the east and is higher than the seismicity in the bordering regions to the south. Five earthquakes with magnitudes exceeding 4.0 have occurred in the region. The largest historic earthquake within the region was a magnitude 4.8. A magnitude of 6.8 is used as the maximum magnitude for this area in the hazard analysis.

3.6.2.1.3.5.6 The Reelfoot Rift and New Madrid Fault Zone

The Reelfoot Rift is interpreted to be a failed arm of a late Precambrian triple junction which has been reactivated during the late Mesozoic. Seismic reflection, magnetic and gravity data confirm the extent of the Reelfoot Rift. Two parallel, linear northeast trending magnetic and gravity anomalies are interpreted as the signatures of plutons associated with the boundaries of the central graben. Seismic data indicate that a zone of disturbed basement reflections correlates with the present area of active seismicity. The disturbed zone is interpreted to disappear south of Marked Tree, Arkansas. The Reelfoot Rift has been interpreted to extend as far south as the Ouachita Region.

During the historical period, over one hundred earthquakes exceeding epicentral intensity IV have been reported in the New Madrid Fault Zone. The largest earthquakes in the south-central United States occurred near New Madrid, Missouri during 1811-1812 and exhibited epicentral intensities of XII.

In forming seismic source zones, the above mentioned disturbed zone containing the faults responsible for the large historic New Madrid earthquakes is used as a separate source zone. The zone is referred to as the New Madrid Fault Zone. The Reelfoot Rift's central trunk (exclusive of the Saint Louis Arm and Wabash Arm) is used in conjunction with the New Madrid Fault Zone cut out.

The Reelfoot Rift, with the New Madrid Fault Zone removed, had four earthquakes with magnitude above 4.5 in the last century. The largest historic earthquake was a magnitude 4.9 in 1903. A magnitude of 6.8 is used as the maximum magnitude for this region in the hazard analysis.

The New Madrid Fault Zone is responsible for the large New Madrid Earthquakes of 1811 and 1812. The historic maximum event was a m_b 7.4 which is also the maximum earthquake used in the hazard analysis.

3.6.2.1.3.5.7 Central Stable Region

The Central Stable seismotectonic region includes much of the area east of the Rocky Mountains, north of the Wichita-Arbuckle and Ouachita Mountains and west of the Ozark Plateau. Major structures include the Anadarko Basin and the southern half of the Nemaha Ridge. The region is tectonically stable, seismicity is low and none of the fault zones show surface displacements during historic earthquakes.

During the historical record, four earthquakes exceeding magnitude 4.0 have occurred within the Central Stable Region. The largest earthquake occurred in 1882 near Ft. Gibson, Oklahoma with an estimated magnitude of 5.5. A magnitude of 5.7 is used as the maximum magnitude for this region in the hazard analysis.

3.6.2.1.3.5.8 The Mississippi Embayment

This source zone borders the Reelfoot Rift to the north and east. The Embayment sediments overlie relatively horizontal Paleozoic sediments similar to those in the Central Stable Zone. The zone includes the Saint Louis and Wabash Arms of the Reelfoot Rift. The zone has had thirteen historic earthquakes with magnitudes above 4.5. The maximum historic earthquake had a magnitude of 5.4. A magnitude of 5.7 is used for the maximum magnitude for this region in the hazard analysis.

3.6.2.2 Vibratory Ground Motion

3.6.2.2.1 Duration of Shaking

From a seismological point of view, the duration of shaking is a function of magnitude and distance. The duration increases with increased magnitude and decreases with distance (mostly through geometrical and anelastic attenuation). The near-field DBE has a duration of strong shaking of about 1 second. The mid-field DBE has a duration of about 4 to 5 seconds. The far-field DBE was assigned a duration of about 26 seconds using a relationship developed for RVT modeling by Herrmann (1985; Reference 26).

Professor H. Bolton Seed (1975), Reference 53, and his co-workers developed the concept of the "equivalent number of cycles" or the "equivalent number of uniform cycles" to express the duration of an earthquake with regard to its impact on analyses of soil liquefaction by his empirical approach. The irregular shear stress time history computed in the sand at borings B-17 and B-27 created from application of two of the three design basis earthquakes was analyzed for the equivalent number of uniform stress cycles using Professor Seed's procedure with the following results:

Earthquake	No. of Equivalent Uniform (at 65% max. stress) Cycles
Far-field	26 (See Text)
Mid-field	7.5
Near-field	3.5

FOR INFORMATION ONLY

The number of cycles associated with the far-field earthquake was not determined from a calculation herein, as was the number of cycles for the mid-field and near-field earthquakes. This is because actual recorded time histories from eastern north American earthquakes were used to represent the latter two events while no natural acceleration time-history is available to represent the far-field event. Instead, a synthetic time history scaled to the appropriate peak acceleration and of the appropriate total duration (see earlier discussion above) was used to represent the far-field event. The number of equivalent cycles for earthquakes like the far-field event but recorded in western north America and elsewhere has been determined by Prof. Seed to be 26 cycles. This was the number of cycles used in the analysis of soil liquefaction, which far exceeds the 7.5 and 3.5 cycles calculated for the mid-and near-field earthquakes, respectively.

Thus, it is concluded that the impact of the duration of the design basis earthquakes has been conservatively accounted for in the analysis of soil liquefaction.

3.6.2.2.2 Results of Probabilistic Seismic Hazard Analysis

The probabilistic hazard analysis described in Section 3.6.2.1.3.2, was performed for the CEC site. Each Seismotectonic Region described in Section 3.6.2.1.3.5 was used as a source zone. The EPRI earthquake catalog was used to determine the earthquakes which occurred in each zone. As mentioned in Section 3.6.2.1, instrumental magnitudes were used when available, otherwise relations for the central United States from Sibol (Reference 44) are used to convert epicentral intensity and/or felt area to magnitude. Half-magnitude intervals are used to determine annual seismic activity rates for each zone (i.e., number of earthquakes having a magnitude ranging between 3.0 and 3.5, 3.5 and 4.0, 4.0 and 4.5, etc.). Catalog completeness for the various magnitude intervals was determined using results from the EPRI study.

The attenuation model is discussed in Section 3.6.2.1.3.2. Figure 3.6-18 shows peak horizontal acceleration versus distance curves generated by the model for m_b magnitudes of 5.0 and 7.0. A log-normal distribution of scatter was assumed for the calculations, using a natural logarithm value of 0.4 for the standard deviation (i.e., a multiplicative factor of 1.5 on acceleration).

Figure 3.6-16 shows the curve of peak horizontal acceleration in rock versus the annual probability of exceedence (P_a) that resulted from the seismic hazard analysis. The return period, R_t, of a given value of acceleration is given by $R_t = 1/P_a$, and is defined as the period of time in which the given acceleration has a 63% probability of being exceeded. The values of acceleration associated with the return periods of 100, 500 and 1000 years are shown in Figure 3.6-16 and given in Table 3.6-10.

Using the RVT model, horizontal ground motion was computed. A conversion was then made to determine the vertical ground motion. Toro and McGuire (Reference 28) used central and eastern ground motion data to compute a ratio of horizontal-to-vertical (H/V) ratio. They defined the H/V ratio as the ratio of acceleration from a horizontal component to the vertical component. Toro and McGuire obtained an H/V ratio for acceleration of 1.4. This ratio was also used for the EPRI study (Reference 42). For the EPRI study, a H/V ratio of 1.4 was used to compute vertical acceleration at bedrock. The computed values for vertical acceleration at bedrock are shown in Table 3.6-10.

The results of the hazard analysis are in terms of peak acceleration. This represents the highest peak in an acceleration time history. However, damage from earthquakes is generally associated with a value of acceleration sustained over a period of time, not with a specific peak value. This effective peak acceleration has been estimated by using some fraction of the peak acceleration value. Nuttli (Reference 31) investigated the relation between sustained maximum horizontal acceleration and peak acceleration. Nuttli defines the sustained horizontal acceleration as the third largest value of acceleration observed on a given accelerogram. Nuttli found that the sustained maximum acceleration is 0.7 times the peak acceleration. For this analysis, the effective peak acceleration is taken as 70% of the peak acceleration.

In addition, an examination was made to determine the contributions of the individual source zones to the total seismic hazard. For the return period of 500 years, over 90% of the hazard was contributed by four source zones:

a. New Madrid Fault Zone - 28%

b. Ouachita Region - 23%

c. Interior Salt Basin (region containing site) - 22%

d. Reelfoot Rift - 19%

Similar results are obtained for the 100 and 1000-year return periods. These results indicate that the greatest hazard to the site is represented by moderate to large earthquakes occurring at distances of 100 km (62 mi) or more from the site.

The curve shown in Figure 3.6-16 represents peak horizontal acceleration in rock. The CEC site is located in an area where a deep column of soil overlays bedrock. A correction to the peak horizontal ground motion is made to account for the effect of the overlying soils. For this purpose, the analysis directly computes the surface response based on the site bedrock accelerations and the actual site soil conditions. These results are presented in Section 3.6.2.2.4.

3.6.2.2.3 Design Earthquake

The Design Basis Earthquake (DBE) represents the level of earthquake shaking at the CEC site with a return period of 500 years. As seen in Table 3.6-10, the peak horizontal acceleration in rock associated with a 500 year return period is 0.046 g with a corresponding effective acceleration of 0.033 g. The probability for this level of acceleration includes input from both small near-field events and large far-field events. Since the response spectra and duration of these types of earthquakes are significantly different, three design earthquakes are considered:

a. Near-field earthquake - small magnitude, short duration event

b. Mid-field earthquake - medium sized event approximately 100 km (62 mi) away

c. Far-field earthquake - large magnitude, long duration event

The near-field design basis earthquake (DBE) is defined as an earthquake having a 500 year return period. No single earthquake can be used to express the full spectrum of ground motion expected at the site in 500 years. Therefore, three DBEs were selected in order to provide the designer with appropriate input to model the shaking. The near-field DBE represents a short duration, relatively high vibration frequency event. The mid-field DBE represents a moderate-duration, broad frequency event. The far-field DBE represents a long duration, moderate to low frequency event. In order to satisfy the 500 year return period, the magnitude of the event must decrease as the site is approached.

The location and distance of the three DBEs was based on magnitude and seismicity. For the mid and far-field DBEs, the nearest part of two of the higher seismicity regions (Ouachita Region and New Madrid Fault Zone) were selected.

For the near-field DBE, the 500 year event in the Interior Salt Basin was selected. The 500 year event had a magnitude of m_b = 4.3.

The earthquake is located at a basement depth of 5 km or 3 mi. The probability of the DBE event occurring as close as 15 km is very low. The likelihood of the near-field DBE occurring directly under the site is minuscule. If the magnitude is allowed to vary, the size (magnitude and duration) of a 500 year return period earthquake under the site is insignificant. This event produces a peak horizontal acceleration at the site seismic basement of 0.045 g and has a return period of approximately 500 years. For the purposes of calculating surface response spectra, the seismic basement is defined as material which consistently has shear wave velocities greater than 2,500 ft/s.

For the mid-field event, the design earthquake is a m_b 5.7 earthquake occurring 105 km (65 mi) from the CEC site in the Ouachita Region. The m_b 5.7 earthquake represents the maximum earthquake for this region. The distance 105 km (65 mi) represents the distance to the closest point of the Ouchita Seismic Source Zone (see Figure 3.6-17). This event produces a peak horizontal acceleration at the site seismic basement of 0.04 g and has a return period of approximately 500 years.

For the far-field event, the design earthquake is a m_b 6.7 earthquake located at the closest point to the New Madrid Fault Zone, a distance of 365 km (227 mi) from the CEC site. An earthquake of this size is chosen because it represents the 500-year return period event for the source zone. This earthquake produces a peak horizontal acceleration of 0.022 g at the seismic basement of the site.

Estimates for the recurrence rate for earthquakes in the New Madrid Fault Region are per various methods and references. The methods can be broken into two types of study: statistical analyses based on historical and instrumental earthquakes, and paleoseismological studies using radiocarbon age dating to date paleoliquefaction events.

The earthquake recurrence rates referenced by Reference 58, Hamilton and Johnston (1990) are based on previous probabilistic estimates by Reference 47, Johnston and Nava (1984). The m_b > 7.0 recurrence rate of 550± 125 years is based on the seismicity of a huge source zone (4° latitude by 3.8° longitude). This use of a New Madrid Fault Zone source zone is much smaller, and should be compared to the Johnston and Nava small source zone. Johnston and Nava (1984) summarized various early recurrence rates based on magnitude (Table 3.6-15). They warn that the "results are not directly comparable because of difference in magnitude, area normalization, cumulative versus non-cumulative number of events, and varying time windows".

The work by Reference 31, Nuttli and Herrmann (1978), used weighted least squares on a list of historical earthquakes to develop recurrence rates. Their result have somewhat lower rates than more recent statistical studies (see Table 3.6-15 and Johnston and Nava (1984)) and results from paleoliquefaction studies.

References 50, 51, and 52, Russ (1978, 1979), has used trenching to identify liquefaction events in the New Madrid meizoseismal area prior to the 1811-1812 earthquake sequences. From an age dating analysis of liquefaction events, Russ (1979) states that there were at least three m_b > 6.2 earthquakes in about 2000 years, (or, about a 500 year return period).

Using the small source zone, Johnston and Nava (1984), Reference 47, give the probability of m_b > 7.0 as 0.2 to 1.0% in the next 50 years. In light of these studies, our use of m_b = 6.7 far-field DBE at the nearest point of the New Madrid Fault Zone is appropriate.

3.6.2.2.4 Uniform Building Code (UBC) Hazard Estimation

The CEC site is located in UBC Seismic Zone 1 (Reference 32) as shown on Figure 3.6-19. Zone 1 has a Seismic Zone Factor of 0.075, which corresponds to the effective peak horizontal acceleration in % g. This value represents a 10% probability of exceedence in 50 years which is equivalent to a 475-year return period.

Figure 3.6-19 is based on the ATC-3 seismic zone map developed in the late 1970s and early 1980s, but values of effective peak acceleration conform with the 1985 UBC seismic zone factors. In 1988, the National Earthquake Hazards Reduction Program (NEHRP) presented an updated version of the recommended provisions for the development of seismic regulations for new buildings. Two sets of seismic zone maps were included for use. The first set is similar to the 1988 UBC map, but has more detail in the definition of effective peak acceleration. In fact, one of these maps, showing contours of effective peak velocity-related acceleration, is used by the 1988 Edition of the Standard Building Code. From this map, the site has an effective peak acceleration of 0.06 g. The second set of NEHRP maps, based on a more recent study by the United States Geologic Survey, gives a maximum acceleration for the site of approximately .04g for a 475-year return period and .06 g for a 2300-year return period.

3.6.2.2.5 Site Response Spectra

Site response spectra are developed to estimate ground motions resulting from near-field, mid-field and far-field earthquakes. The ground motion parameters for these design earthquakes are developed in Section 3.6.2.1.3.3. Subsurface elastic parameters, based on crosshole and downhole seismic testing, are combined with elastic parameters estimated from geology information contained in Section 3.6.1.2 to determine the ground motion parameters. The downhole soil profiles are used in modeling the site-specific response spectra since downhole velocity measurement more closely approximates the velocity encountered by vertically propagating shear waves.

The horizontal and vertical response spectra for the near-field, mid-field and far-field earthquakes for damping values of 0.2, 0.5, 2.0, 5.0 and 10.0 % are shown in Figures 3.6-20 through 3.6-25. The response spectra represent actual earthquakes and are not considered "envelope" or composite spectra.

Figures 3.6-26 and 3.6-27 compare the horizontal and vertical components of the three design earthquakes at 5.0 % damping. As can be seen, the near-field earthquake response spectrum is below the mid-field earthquake and far-field spectra at all frequencies. The mid-field spectrum yields the highest spectral amplitude at frequencies higher than approximately 1.5 Hz. The far-field spectrum yields higher spectral amplitudes at frequencies less than 1.5 Hz.

The following sections describe the generation of the response spectra.

3.6.2.2.5.1 Generation of Response Spectra

The near-field and mid-field response spectra are derived using time histories of actual earthquakes which occurred in eastern North America. For the far-field earthquake, which is an m_b 6.7 earthquake located in the New Madrid Fault Zone, no appropriate time history of acceleration exists. An approach combining artificially generated seismic traces and random vibration theory is used to generate the response spectrum for the far-field design earthquake.

Horizontal response spectra are generated using the site specific subsurface model described in Section 3.6.2.2.4.2; peak ground accelerations for near-field, mid-field and far-field earthquakes from the seismic hazard analysis; and the earth response analysis computer program SHAKE (Reference 33).

Vertical response spect. are also generated for the near, mid and far field earthquakes. The procedure for generating vertical response spectrum involves modeling the horizontal response spectrum first, and then using the resulting shear velocities, with the Poisson's Ratios calculated from the geophysical surveys, to generate appropriate compression wave velocities at the effective strain levels. The compression wave velocities were used in place of the shear velocities and a vertical component time history was used as input to SHAKE.

3.6.2.2.5.2 Subsurface Model

The seismic parameters for subsurface soil columns at three locations at the CEC site are described in Section 3.6.1.3.2.1. These soil columns are used to develop three subsurface models for input to the SHAKE program.

The interpreted layered models are based on measured compression and shear wave velocity, unit weights and composition of the soils. The models extend from grade level (elevation 324+6) to a depth of approximately 1,500 feet (depth of interpreted "seismic basement"). For the purpose of calculating response spectra, the "seismic basement" is defined as material which consistently has shear wave velocities greater than 2,500 ft/s.

The site subsurface models are integrated from seismic crosshole data, seismic downhole data, boring logs, geotechnical measurements and geophysical logs from nearby oil fields. Each model layer has constant properties with depth as well as simplified geology.

Tests of the subsurface models using different earthquake time histories demonstrated that the different models resulted in very little difference in the generated response spectra. Based on this finding, a single subsurface model is used to represent the entire CEC site. The subsurface model is described in Table 3.6-11.

3.6.2.2.5.3 Selection and Generation of Time Histories

The DBEs used in this study were defined as events with a 10% probability of exceedence in 50 years (a 500 year return period); therefore, the uncertainty in the occurrence of the DBE is fixed by definition. The location of the near-field DBE was calculated using a 2374 year return period.

The SHAKE program uses time histories recorded by strong ground motion instruments during actual earthquakes. The time histories are scaled to appropriate bedrock accelerations by the probabilistic methods described in Section 3.6.2.1.3.2 .

The response shape from eastern North America earthquakes is significantly different from western North America earthquakes. The response shape difference is due to differences in earthquake source shapes and attenuation rates. Based on these differences, it is important that time histories and ground motion relations used in the generation of the site response spectra are specific to eastern North America. Since 1982, a variety of appropriate earthquakes have been recorded in Canada and the eastern United States. Time traces from several of these events are used in the development of site response spectra.

The design basis earthquake (DBEs) were defined as those events which occur with a return period of 500 years. The bedrock acceleration for other return periods is shown in Figure 3.6-16. Because the probabilistic seismic ground motion at the site is a sum of all the probabilities of earthquakes from many source zones, no single DBE can represent shaking at the site. For this reason, multiple DBEs were considered. For the same reason, the ground motion from each of the DBEs must be equal to or less than the probabilistic ground motion.

Three DBEs were chosen: a near-field, mid-field, and far-field event. These events originate from the Interior Salt Basin, Ouachita Region and New Madrid Fault Zone, respectively. Other regions, such as the Reelfoot Rift, Wichita-Arbuckle Region, and Central Texas Region were considered (Sections 3.6.2.1.3.5). The choice of source region for each DBE was based on a 500 year return period event for that zone. The seismicity and maximum magnitude determined the choice of zone. For example, both the Central Texas Region and Ouachita Region are about the same distance from the site and have the same maximum magnitude (pages 3.6-26 and 3.6-27). For the 500 year event, the Ouachita Region has a much larger event (m_b = 5.7) than the lower seismicity Central Texas Region.

Compared to the relatively well studied New Madrid fault area, the seismicity of the Interior Salt Basin and Ouachita source zones is based on less certain data. To reflect the uncertainty in the sparse seismicity data, our choice of maximum magnitude was conservative. The design basis earthquakes selected for use each have a computed return period of 500 years. Because of the conservativeness of our approach, the near-field and mid-field events may realistically have a return period longer than 500 years. The overall effect is that a spectrum enveloping the three design basis earthquakes is more conservative on the high frequency side. Thus, the actual probability of earthquakes shaking greater than the design basis is somewhat less than that given as a result of our analysis.

Table 3.6-16 shows information on the input acceleration time histories and the scaling of peak accelerations for the three DBEs.

After the magnitude and distance of the 500 year return period DBEs were determined, the RVT method was used to estimate the ground motion (acceleration, velocity, displacement, duration) at the site. Ground motion was modeled using frequency dependent attenuation appropriate to the central United States. More rapidly attenuating Gulf Coast attenuation rates were not used since the bulk of the hazard originated in the central United States (New Madrid Fault Zone, Reelfoot Rift, and Ouachita source zones). The input time histories used in the SHAKE program were scaled using the accelerations determined by the RVT method.

3.6.2.2.5.3.1 Near-field Time History Selection

The near-field DBE was from within the Interior Salt Basin. At a depth of 5 km and a distance of 14 km the 500 year return period has a magnitude of about 4.3. This event has sufficient strength and duration of shaking to be significant at the site. It also can be modeled using available bedrock acceleration time histories. The acceleration time histories (Figures 3.6-42 and 3.6-43) used for the near field DBE was from a magnitude 4.8 aftershock of the March 31, 1982 New Brunswick earthquake. These time histories are particularly rich in high frequency energy. The station selected was located on bedrock approximately 4 km (2.5 mi) from the epicenter (Reference 34). The transverse and vertical components of acceleration were used as input to SHAKE.

The near-field horizontal time history was scaled to a peak horizontal bedrock acceleration of 0.045 g in the SHAKE program. The vertical time history was scaled to a peak bedrock acceleration of 0.032 g.

For damping values less than 10%, the response spectra generated by SHAKE were smoothed. This was done by dividing the given response spectrum by the 10% damped spectrum, to remove the general spectral shape, and then applying a three-point moving average three times to the resulting trace. Multiplication of the result by the 10% damped response spectrum then gives the smoothed spectrum.

3.6.2.2.5.3.2 Mid-field Time History Selection

The mid-field DBE was placed at the near edge of the Ouachita Region at a distance of about 105 km. In order to effectively simulate the frequency content and duration of shaking from an event of magnitude 5.7 originating from the Ouachita Region, acceleration time histories from the 1988 Saguenay, Quebec earthquake (m_b = 5.9) were used (Figures 3.6-44 and 3.6-45).

Time histories from strong motion records recorded during the m_b 5.9 earthquake in Saguenay, Province of Quebec, is used for the mid-field earthquake. Recordings from six stations with epicentral distances varying from 64 to 149 km (40 to 93 miles) were examined. The stations were mostly surface sites located on bedrock. The station at Riviere-Oelle, Quebec, approximately 114 km (71 mi) from the epicenter was selected to represent the mid-field earthquake (Reference 35).

The horizontal time history was scaled to a peak bedrock acceleration of 0.040 g in the SHAKE program. The vertical time history was scaled to a peak acceleration of 0.028 g. The resulting response spectra were then smoothed as described for the near-field spectra.

3.6.2.2.5.3.3 Far-field Time History Generation

The far field DBE was placed at the near edge of the New Madrid Fault Zone. Since no acceleration time histories for an earthquake of this magnitude (m_b = 6.7) at about 365 km in a region with low anelastic attenuation rate are available, a

FOR INFORMATION ONLY

synthetic earthquake was used to develop the amplification factors for the site ground motion. The amplification factors were used to scale Newmark Spectra at various frequencies to produce the far-field response spectra.

The far-field design earthquake is represented by a m_b 6.7 earthquake located approximately 365 km (227 mi) from the site. In contrast to the near-field and mid-field design earthquakes, no event of this magnitude has been recorded in eastern North America. An alternate method was therefore used to produce the far-field response spectra.

The random vibration theory (RVT) model (see Section 3.6.2.1.3.3) is used to compute the peak acceleration, velocity and displacement for this earthquake. The ground motion values are used to construct far-field bedrock response spectra, using the method described by Newmark and Hall (Reference 36).

The RVT model is also used to determine the power spectral density function for the far-field earthquake. The power spectral density function is used as input in the SIMQKE program (Reference 37) to generate artificial time traces. Duration of these time traces is determined by magnitude and epicentral distance (Reference 26).

The time traces generated by SIMQKE are scaled to the appropriate vertical or horizontal acceleration and used as input to SHAKE. An average subsurface pseudo-relative velocity amplification function is computed for each damping value. To achieve final far-field response spectrum, the Newmark-Hall response spectrum is multiplied by the appropriate subsurface amplification function.

Limitations to the method of using a synthetic earthquake are primarily due to the lack of an appropriate time history for use in liquefaction analysis and design. (The lack of an actual acceleration time-history is not a limitation to liquefaction analysis by Seed's SPT-based methodology, which was used for the CEC). The far-field DBE spectra were obtained by scaling a basic Newmark-Hall spectrum using peak acceleration, velocity, and displacement for RVT modeling.

3.6.2.2.5.4 Adjustments for Time History Filtering

Time histories used for both the near-field and mid-field earthquakes were high-pass filtered. Adjustments were therefore made to the response spectra at low frequencies. Newmark-Hall bedrock spectra were constructed for both the near-field and mid-field events, and surface Newmark-Hall spectra were computed using subsurface amplification functions. The surface Newmark-Hall spectra were then scaled to match the earthquake response spectra at the appropriate frequency (2 Hz for the near-field earthquake and 0.44 Hz for the mid-field earthquake). The presented near-field and mid-field response spectra are comprised of the earthquake response spectra above the appropriate frequency and the adjusted surface Newmark-Hall spectra below that frequency.

3.6.2.2.5.5 High Frequency Pseudo-Relative Velocity Adjustment

At high frequencies, the pseudo-relative velocity response is determined directly from peak acceleration, regardless of the damping level (i.e., the peak response acceleration equals the peak ground acceleration). Using events from western North America, Newmark and Hall (Reference 36) determined that this relation holds at frequencies higher than 33 Hz. However, earthquakes generated in eastern North America generally have source spectra richer in higher frequencies, with lower rates of attenuation. Therefore, the frequency at which peak response acceleration equals peak ground acceleration is higher in eastern North America. Based on recommendations from Bernreuter (Reference 24), the response spectra used a frequency of 100 Hz.

3.6.2.3 <u>Surface Faulting</u>

As discussed in Section 3.6.1.2.2, faulting has not been active in Claiborne Parish since the Middle Tertiary. Therefore, a design basis for surface faulting is not applicable for the site.

3.6.2.4 <u>Subsurface Stability</u>

Subsurface soil data from the soil test borings, CPTs and laboratory test data were carefully assessed in order to understand the soil stratigraphy and properties. Information obtained in this assessment is discussed in this section.

3.6.2.4.1 Geologic Features

Several subsurface soil profiles are developed to help visualize the soil stratigraphy at various sections crossing the site. Figure 3.6-28 shows the location of the subsurface soil profiles. Individual subsurface soil profiles are presented in Figures 3.6-29 through Figure 3.6-35.

The cross sections of Figures 3.6-29 through 3.6-35 were all constructed by schematically placing the location of the test borings on the line of the sections, whose locations are generally depicted on Figure 3.6-28. (Please note that the location of Section F-F (Figure 3.6-34) is not shown on Figure 3.6-28 but is shown on Figure 3.6-37).

The test boring horizontal locations on the sections are not plotted according to a scale, but the horizontal distance between the borings stated on the figures is the actual distance measured parallel to the section line.

The interface locations at the test boring locations were determined from a combination of: a) observations of changes in drilling behavior in the field; b) engineering and geologic inspection of the samples obtained at maximum vertical intervals of 5 ft, and c) in some cases comparison of the test boring data with nearby cone penetration tests (CPT) as shown on Figures 3.6-9 through 3.6-11. Thus, although considerable care has been exercised in identifying the interface locations at the test borings shown on the profiles, they are nevertheless imprecise due to the limitations inherent in identifying such interfaces from boring and CPT data. Furthermore, even though the interfaces are shown as being abrupt on the test boring records and sections, they may actually be transitional in nature. At the boring locations, the interfaces are estimated to be within ±2 ft of the actual location. Between the borings, no actual data exists as to the locations of the various interfaces (except at the CPT tests where located between the test borings, which were not considered in preparing the referenced cross-sections but which were considered in the geotechnical analyses of the site). Thus, the interface locations between the boring locations on Figures 3.6-29 through 3.6-35 are schematic, and actual conditions between the borings could differ from those shown. This does not diminish the usefulness of the referenced figures, which were prepared to depict a geotechnical model of the site and to understand the soil stratigraphy and properties.

Subsurface soil conditions and soil properties are described in the following subsections.

3.6.2.4.1.1 Process Area

Subsurface soil conditions in the Process Area (i.e., Separations Building and Centrifuge Assembly Building) are summarized below:

Stratum No.	Description	Average Depth (Ft)
I	TOPSOIL: Very Loose to Firm Light Brown Silty Fine SAND (SM)* With Trace Organics	0 - 4
II	Stiff to Very Stiff Light Gray, Yellowish Brown and Reddish Brown Silty to Sandy CLAY (CL)	4 - 10
IIIA	Firm to Very Firm Gray Silty Fine SAND (SM) With Alternate Seams of Stiff to Very Stiff Brown Clayey SILT (ML)	10 - 15

Stratum No.	Description	Average Depth (Ft)
IIIB	Stiff to Hard Dark Brown Clayey SILT (ML) with Light Gray Silty Fine Sand partings	15 - 25
IV	Firm to Dense Light Gray, Yellowish Brown and Reddish Brown Silty Fine SAND (SM) With Light Gray Silty Clay Partings and Seams	25 - 55
V	Dense to Very Dense Greenish Gray Silty Fine SAND (SM) With Silty Clay Partings and Seams	25 - 55
VI	Hard Dark Greenish Gray Silty CLAY (CL) Laminated With Greenish Gray Silty Fine Sand Partings	55 - TOB

* Unified Soil Classification System Designation
** Termination of Boring

Figure 3.6-36 locates soil test borings, CPTs and test pits relative to the Separations Building and Centrifuge Assembly Building (CAB). Figures 3.6-29, 3.6-30 and 3.6-33 present subsurface soil profiles along the lines designated as A-A, B-B and E-E on Figure 3.6-36. Profiles A-A and B-B illustrate subsurface conditions from west to east, and Profile E-E depicts conditions from north to south.

Variations within individual soil strata and transitions between strata are evaluated using the CPT records which provide continuous data with depth. Figures 3.6-9, 3.6-10 and 3.6-11 present a comparison of CPT data with adjacent soil test borings B-6, B-18 and B-36.

The properties of the individual soil strata are discussed below with reference to the above soil strata summary, soil test boring records and the results of laboratory tests.

Stratum I, surficial topsoil consisting of very loose to very firm light brown silty fine sand with a trace of organics, was encountered in the upper 2 to 10 feet. The average thickness of this stratum is 4 feet. This stratum has standard penetration test (SPT) blow counts ranging from 3 to 28 blows per foot (bpf). The percentage of soil finer than the No. 200 sieve is between 15 to 30 percent. A standard Proctor maximum dry density of 110 pounds per cubic foot (pcf) at 12 percent optimum moisture and a CBR of 11 were determined for a bulk sample from Test Pit TP-3.

Stratum II, stiff to very stiff light gray, yellowish brown and reddish brown silty to sandy clay, was encountered between 4 to

10 feet below the surface. Atterberg limits tests indicate this soil is a medium plasticity clay. Liquid limits are in the range of 32 to 41 percent, plastic limits range from 17 to 26 percent and plasticity index is 13 to 23 percent. Percent fines vary more, ranging from 54 to 89 percent. Percentages of sand, silt

and clay portions are very close, varying by less than 10 percent.

Undrained shear strength, measured by unconsolidated-undrained triaxial shear strength tests, ranges from 1400 to 3100 pounds per square foot (psf) and averages 2500 psf. Dry density and natural moisture content average 106 pcf and 21 percent, respectively.

The results of two swell tests indicate a low swell pressure of approximately 500 psf and a percent swell of less than 1 percent at a confining pressure of 100 psf. Consolidation tests show the clay to be over consolidated with a preconsolidation pressure of 9500 psf and an overconsolidation ratio (OCR) of 6.1 to 8.6.

Two pH and resistivity tests indicate a pH range from 2.9 to 3.8 and a resistivity value from 16,029 to 28,972 ohm-cm.

Standard Proctor maximum dry density is 105 pcf at an optimum moisture content of 18 percent for a bulk sample from Test Pit TP-4. A CBR value of 5 was determined for the recompacted sample.

Soil samples of Stratum II soils, obtained from Test Pit TP-12 and Soil Test Boring B-24, were recompacted at optimum moisture content to 95 percent of the standard Proctor maximum dry density. Permeability tests on these recompacted samples resulted in coefficients of permeability between 2.13×10^{-7} and 3.89×10^{-8} centimeters per second (cm/sec). The Stratum II soils can therefore be considered to have relative permeabilities which are very low to impermeable.

An undrained shear strength of 1250 to 1500 psf and a friction angle of 22 to 25 degrees will be used for analysis of long term slope stability for the recompacted Stratum II fill soil.

Stratum III consists primarily of stiff to hard brown and dark brown clayey silt between depths of 10 to 25 feet. In some locations, there are alternate seams of firm to very firm light gray silty fine sand in the upper portion of Stratum III. This upper Stratum IIIA is approximately 6 feet thick and tends to grade into the lower Stratum IIIB, which is darker brown and contains only thin partings of silty fine sand. Laterally, Stratum III thins to the east and south, eventually disappearing outside the Process Area.

Properties of the clayey silt portion of Stratum III tend to vary. Atterberg limits are in the range of 33 to 48 percent for liquid limit, 24 to 41 percent for plastic limit, and 2 to 17 percent for plasticity index. The degree of plasticity varies from slight to medium. The percent fines in the test samples are typically 85 percent, but were as low as 68 percent.

Undrained shear strength measured by unconsolidated-undrained triaxial tests ranges from 1300 to 4700 psf and averages 2800 psf. The lower test value of shear strength, 1300 psf, was obtained in the upper Stratum IIIA. There is a trend towards increased shear strength with depth. Dry density and natural moisture content average 94 pcf and 26 percent, respectively.

The results of two swell tests showed opposite extremes at the two depths tested. Near the middle of the stratum, swell pressure and percent swell were low at 400 psf and one half percent. Test results at the lower depth near the bottom of the stratum indicate 2000 psf of swell pressure and 6 percent swell at 100 psf confining pressure. Consolidation tests show the soil to be over-consolidated with a preconsolidation pressure of 8000 to 9000 psf and an OCR of 2.8 to 3.7.

Four pH and resistivity tests indicate low pH values in the range of 2.9 to 3.8. Resistivity values were split between a low of 1000 ohm-cm to a high near 12,500 ohm-cm. The low pH and resistivity values combine to result in a high corrosion potential.

Bulk samples from TP-2 were recompacted and tested for standard Proctor maximum dry density, optimum moisture content and CBR value. The test results indicate a maximum dry density of 100 pcf at 18 percent optimum moisture and a CBR value of 6.

Stratum IV, firm to dense light gray, yellowish brown and reddish brown silty fine sand with light gray silty clay partings and seams, was encountered between 25 to 55 feet below the surface. This stratum is the result of weathering of the underlying Stratum V. Both strata occupy the same horizon, with Stratum IV becoming thicker to the east and thinning to the west.

SPT blow counts within Stratum IV vary from 4 to greater than 50 bpf. In general, SPT blow counts were lower in the weathered Stratum IV compared with the unweathered Stratum V. Grain size analysis show the gradation to be very uniform fine grained sand with percent fines ranging from 12 to 47 percent. Typically, the percent fines should be 15 to 25 percent. The higher test values are believed to be a result of the silty clay seams and samples obtained in transition zones grading from the overlying Stratum II and Stratum III soils.

The results of four consolidated-undrained triaxial compression tests indicate an effective cohesion and friction angle of 500 psf and 32 degrees. Dry density and natural moisture content are 103 pcf and 24 percent, respectively.

Stratum V, dense to very dense greenish gray silty fine sand with silty clay partings and seams, occupies the same soil horizon as Stratum IV, but is predominantly to the west and disappears to the east side of the Process Area. SPT blow counts varied from 20 to greater than 50 bpf. This unweathered stratum has notably higher SPT values compared with Stratum IV. Grain size analyses show a similarly uniform fine sand, and percent fines ranging from 12 to 45 percent.

Two consolidated-undrained triaxial compression tests indicate the influence of the silty clay. One test was similar to Stratum IV with an effective cohesion and friction angle of 700 psf and 29 degrees. The second test resulted in an effective cohesion and friction angle of 2500 psf and 16 degrees. Dry density and natural moisture content are 93 pcf and 26 percent, respectively.

Stratum VI, hard to very hard greenish gray silty clay laminated with greenish gray silty fine sand partings, was typically encountered between elevation 280 and 290 feet above MSL, roughly 55 feet below the surface. No soil strength or classification tests were performed on this strata because of its high strength and the difficulty in preparing the laminated samples without damaging them. In addition to high strength, the Stratum VI soil has very low compressibility. SPT blow counts typically are greater than 50 bpf with pocket penetrometer readings greater than 4.5 tsf, indicating shear strengths above 4500 psf.

3.6.2.4.1.2 Support Facilities

Subsurface soil conditions in the area of the Support Facilities (i.e., Office Building, Cylinder Receipt and Dispatch Building, etc) are summarized below:

Stratum No.	Description	Average Depth (Ft)
I	TOPSOIL: Very Loose to Firm Light Brown Silty SAND (SM) With Trace Organics	0 - 4
II	Stiff to Very Stiff Light Gray, Yellowish and Reddish Brown Silty to Sandy CLAY (CL)	4 - 16
IV	Firm to Dense Light Gray, Yellowish Brown and Reddish Brown Silty Fine SAND (SM) With Light Gray Silty Partings and Seams	16 - 33

Stratum No.	Description	Average Depth (Ft)
VI	Hard Dark Green Gray Silty CLAY (CL) Laminated With Greenish Gray Silty Fine Sand Partings	33 - TOB

Figure 3.6-37 shows the location of various Support Facilities relative to the soil test borings. Figure 3.6-34 presents a Subsurface Soil Profile along the line designated as F-F in Figure 3.6-37. Profile F-F illustrates the subsurface conditions from north to south.

The properties of the individual strata in the area of the Support Facilities are the same as described earlier in Subsection 3.6.2.4.1.1 for the Process Area. The stratigraphy in the area of the Support Facilities most noticeably includes only four strata. Stratum III and Stratum V are discontinuous in this area. Also evident is the decreasing overburden thickness above the lower Stratum VI. The stratigraphy tends to dip to the southwest as discussed previously in Section 3.6.1.1.5.

3.6.2.4.1.3 Tail Storage Area

Subsurface soil conditions in the Tail Storage Area are summarized below:

Stratum No.	Description	Average Depth (Ft)
I	TOPSOIL: Very Loose Light Brown Silty Fine SAND (SM) With Trace Organics	0 - 1
II	Stiff to Very Stiff Light Gray, Yellowish Brown and Reddish Brown Silty to Sandy CLAY (CL)	1 - 10
IV	Firm to Dense Light Gray, Yellowish Brown and Reddish Brown Silty Fine SAND (SM) With Light Gray Silty Clay Partings and Seams	10 - 30
V	Dense to Very Dense Greenish Gray Silty Fine SAND (SM) With Silty Clay Partings and Seams	30 - 45
VI	Hard Dark Greenish Gray Silty CLAY (CL) Laminated With Greenish Gray Silty Fine Sand Partings	45 - TOB

Figure 3.6-38 shows the perimeter roadways, tail storage, and product and feed storage areas relative to the soil test borings and test pit. Figures 3.6-31, 3.6-32 and 3.6-33 present subsurface soil profiles along the lines designated as C-C, D-D and E-E in Figure 3.6-38. Profiles C-C and D-D illustrate subsurface conditions west to east, and Profile E-E depicts conditions from north to south.

The properties of the individual soil strata in the Tail Storage Area are the same as described for the Process Area in Section 3.6.2.4.1.1. The sequence of strata and the thickness of the individual strata varies from the previous discussed subsurface conditions. The most dramatic change in the subsurface conditions in the Tail Storage Area is the absence of Stratum III, except for the extreme north and northwest corner adjacent to the Process Area. Also changing significantly is the ground surface elevation that drops off from Elevation 340+0 to 300+0.

3.6.2.4.1.4 Hold-Up Basin

Subsurface soil conditions in the Hold-up Basin Area are summarized below:

Stratum No.	Description	Average Depth (Ft)
I	ALLUVIUM: Very Loose to Loose Light Brown Silty Fine SAND (SM) With Trace Organics	0 - 4
II	Stiff to Very Stiff Reddish Brown, Yellowish Brown and Light Gray Silty CLAY (CL)	4 - 8
VII	Firm to Stiff Light Gray and Yellowish Brown Clayey SILT (ML) With Silty Fine Sand Seams and Partings	8 - 13
IV	Dense Dark Greenish Gray Silty Fine SAND (SM) With Silty Clay Partings and Seams	13 - 18
V	Very Stiff to Hard Dark Greenish Gray Silty CLAY (CL) Laminated With Greenish Gray Silty Fine Sand Partings	13 - TOB

Figure 3.6-39 shows the topography and the relative locations of the soil test borings and test pits to the proposed earthen dam. Figure 3.6-35 presents a subsurface soil profile along the line designated as G-G in Figure 3.6-39. Profile G-G illustrates the subsurface conditions along the centerline of the proposed earthen dam.

The properties of the individual strata in the Hold-Up Basin are the same as described earlier in Subsection 3.6.2.4.1.1, except for the new Stratum VII.

Stratum VII, firm to stiff light gray and yellowish brown clayey silt with silty fine sand seams and partings, was encountered between 8 and 13 feet below the surface. Atterberg limits indicate medium plasticity with liquid limit of 45 percent, plastic limit of 30 percent and plasticity index of 15 percent. Grain size analysis shows approximately 85 percent fines with 25 percent clay size and 60 percent silt size. The upper portion of the stratum tends to become sandy grading into silty fine sand.

Undrained shear strength measured by unconsolidated-undrained triaxial compression tests ranges from 1100 to 1900 psf. Dry density and natural moisture content are 87 pcf and 37 percent, respectively.

Standard Proctor maximum dry density is 91 pcf at an optimum moisture content of 22 percent for a bulk sample from Test Pit TP-11. A CBR value of 5 was determined for the recompacted sample.

Soil samples of Stratum VII soils, obtained from Test Pit TP-12 and Soil Test Boring B-53, were recompacted at optimum moisture content to 95 percent of the standard Proctor maximum dry density. Permeability tests on these recompacted samples resulted in coefficients of permeability between 3.42×10^{-7} and 1.53×10^{-7} cm/sec. These values are considered to be very low.

An undrained shear strength of 1000 psf and a friction angle of 22 degrees is used for the analysis of long term slope stability of the Hold-Up Basin earthen dam.

3.6.2.4.1.5 Groundwater

Water level readings were recorded at the site between March 31, 1990 and April 12, 1990. Measurements were recorded daily from temporary piezometers installed in Test Borings B-10, B-24, B-40 and B-48. The locations of the piezometers were selected to obtain groundwater levels that would be representative of conditions in each area of the developed site. Additional groundwater data were obtained from the existing subsurface Soil Test Borings A-1 through A-13 drilled between July 28, 1989 and August 8, 1989.

Water level readings are summarized in Table 3.6-2. Groundwater levels were found to be influenced by the surface drainage features and existing ground elevation. In the Process Area and north end of the Support Facilities Area, groundwater elevation is approximately 305 to 315 feet above MSL. Groundwater elevations drop to between 285 to 300 feet above MSL moving south into the Tail Storage Area and south end of the Support Facilities Area. The lowest groundwater elevations were reported in the southwest corner of the site where the Hold-up Basin will be constructed. The groundwater elevation is 270 feet above MSL in this area or approximately 10 feet below the ground surface. Groundwater levels measured during the field exploration program were at depths of between 10 to 40 feet below the proposed final site grade of elevation 324+6.

Observations at the site during the field exploration program and, in particular, during the test pit excavation, indicate that a perched groundwater condition exists. Groundwater was observed flowing into the test pits at the top of the Stratum II silty to sandy clay and from silty sand seams within the clay. Other observations of percolating springs were made along the sides of the hills in the Stratum IV and V silty fine sands. These springs are associated with groundwater perched above the siderite (ironstone) layers within the strata.

Fluctuations in groundwater level may occur seasonally, and can be due to variations in rainfall, construction activity, surface runoff and other factors.

3.6.2.4.2 Suitability of Subsurface Characteristics

3.6.2.4.2.1 Arrangement of Facility

The planned development of the CEC site is particularly suited to the site conditions. The settlement sensitive Separations Building is appropriately situated in the location that will be excavated of overburden soil, thereby significantly reducing the amount of settlement for foundations bearing in the exposed soils. Furthermore, the exposed bearing soil is stiff to very stiff and will provide ample bearing capacity with a low potential for shrinkage or swell.

The Support Facilities are also located appropriately within the site. Although fill placement will be required in this area to bring the site to grade, the depth of fill is limited. Structures planned for this area are also relatively lightly loaded and are not unusually settlement sensitive.

The largest quantities of fill will be required in the Tail Storage Area. This will not create a problem in this area since the underlying Stratum II is approximately 10 feet thick. Most settlement related to the fill surcharge will occur in the Stratum IV and V silty fine sand during construction. Long-term consolidation settlements will be small due to the thin Stratum II clay layer. In addition, the tail storage operation is not unusually settlement sensitive.

Location of the Hold-up Basin in the southwest corner of the site is strategically located to collect runoff which drains to the southwest through drainages south and west of the planned facilities. The topography of the natural drainage is also particularly suited to construction of the earthen embankment, as is evident by the many similarly constructed ponds in the area.

3.6.2.4.2.2 Site Grading and Fill Material

Soils which are classified as Stratum I soils, namely the topsoil, will be excavated and removed from the construction area, prior to the start of cut and fill operations. The depth of this stratum encountered in the soil test borings varied from 0 to 10 feet below the natural grade. This stratum is generally high in silt and organic content.

The site will then be graded to its final elevation (324+6) utilizing the Stratum II soils at higher elevations for fill in the lower areas.

Existing site grades in the Process Area and north end of the Support Facilities Area generally average elevation 325+0 to 340+0. Removal of approximately 0 to 14 feet of overburden soils will be required to obtain a rough design site grade at elevation 324+6.

All other areas of the site will require additional fill. Approximately 0 to 30 feet of fill will be required in the Tail Storage Area and roughly 0 to 20 feet of fill in the Support Facilities Area. Additional fill will also be required to construct the earthen dam for the Hold-Up Basin.

All compacted fill is constructed by spreading acceptable soils in loose layers not more than 8 inches in thickness. The soils used within the proposed structure areas and for the earthen dam is compacted to a minimum of 95 percent of the maximum dry density as determined by the Standard Proctor Method (ASTM D 698) and to within +2 or -2 percentage points of the optimum moisture content. Soils placed outside the proposed structure areas are compacted to a minimum of 90 percent of the maximum Standard Proctor dry density within +2 or -2 percentage points of optimum moisture content.

Prior to fill placement, the subgrade will be scarified and recompacted to ensure good bonding between the subgrade and the compacted fill material. Proofrolling of the entire site is not required; however, if rutted or loose areas develop during the construction, these areas will be proofrolled.

A representative sample of the Stratum II silty to sandy clay soil was tested for use as structural fill and was found to be suitable. Additional representative samples of soil will be tested during construction to ensure the continued suitability of the fill material.

3.6.2.4.2.3 Uniform Building Code (UBC) Soil Profile Type

The UBC soil profile type is S_2 based on the geotechnical data. UBC (Reference 32) Table No. 23-J recommends use of an S Factor of 1.2.

3.6.2.4.3 Foundation Requirements

The foundation for the Separations Building consists of isolated footings for columns and sensitive equipment (i.e., centrifuges and autoclaves), and grade beams or strip footings along building walls. The foundations for ancillary (support) facilities are a combination of isolated footings, strip footings, combined footings and mat foundations.

3.6.2.4.3.1 Foundation Design

The minimum bearing depth for shallow foundations is approximately 4 feet below the proposed final grade of elevation 324+6. Special provisions to mitigate swelling or frost related heave of the subgrade soil are not required. Swell potential measured in the laboratory tests is low, and the depth of frost in north Louisiana is zero. Shallow foundations in the Process Area will be supported primarily in the natural Stratum III, stiff to very stiff dark brown clayey silt. The shallow foundations for the CAB and the extreme east end of the Separations Building will obtain bearing in the natural Stratum II, stiff to very stiff silty to sandy clay. Buildings and structures in the Support Facilities Area will be supported on compacted Stratum II fill soil.

Footing dimensions, based on total loads, are designed for a maximum allowable net bearing pressure of 4000 psf when supported in the natural Stratum II or III soil. A 3000 psf maximum allowable net bearing pressure is used for the design of wall strip footings in the same natural soils. Footings supported in compacted Stratum II fill soil is designed for a maximum allowable net bearing pressure of 2000 psf. Maximum allowable net bearing pressures is increased by 20 percent for transient loading conditions such as wind or seismic. Minimum foundation widths are 24 inches for column footings and 18 inches for wall footings.

The recommended maximum allowable net bearing pressures are based on individual spread footings. Spread footings located adjacent to or near other footings have lower bearing capacities than those recommended above. Additional bearing capacity analyses is conducted if the individual spread footings are located such that the edge-to-edge distance is less than the width of the larger footing.

3.6.2.4.3.2 Settlement

In the Process Area, the foundations will experience negligible settlement primarily because 0 to 14 feet of overburden soil will be removed. The estimated total settlements is in the order of 0.24 inches and differential settlements of the order of 0.12 inches for footing sizes up to and including 15 by 15 feet in plan dimensions. Approximately 20 to 40 percent of the estimated settlement will occur during construction and the initial application of the sustained loads. The remaining settlement probably will occur slowly over a period of 10 to 20 years.

Table 3.6-12 summarizes the expected settlement for square spread footing foundations bearing in natural Stratum II soil at a depth of 4 feet below grade.

Recommended allowable net bearing pressures in the Support Facilities Area are lower than in the Process Area because footings are founded in fill soil. The recommended maximum allowable net bearing pressure of 2000 psf will control settlements within normally acceptable limits in the Support Facilities Area. The estimated total settlements are in the order of 0.45 inches and differential settlements of the order of 0.23 inches for footing sizes up to and including 15 by 15 feet in plan dimensions. Approximately 20 percent of the estimated settlement will occur during construction and initial application of sustained loads. The remaining settlement probably will occur slowly over a period of 10 to 20 years.

Table 3.6-13 summarizes the expected settlement for square spread footing foundations bearing in compacted Stratum II fill soil at a depth of 4 feet below grade.

Spread footings located adjacent to or near other footings will tend to have larger settlements than estimated above. Additional settlement analyses is conducted if the individual spread footings are located such that the edge-to-edge distance is less than the width of the larger footing.

3.6.2.4.3.3 Uplift Resistance

The uplift resistance of spread footings formed in open excavations is limited to the weight of the foundation concrete and the soil above it. The uplift resistance is computed using a unit weight of 115 pcf for the structural fill soil placed and compacted above the footing and a unit weight of 145 pcf for the concrete. As noted Section 3.6.2.4.1.5, the groundwater level is below the proposed bottom of the footings. A factor of safety of at least 1.5 is used in determining the allowable uplift resistance of spread footings.

3.6.2.4.3.4 Resistance to Lateral Loads

Soil resistance to horizontal forces is developed by earth pressure acting on the face of the footing and by friction on the footing base. The passive pressure for spread footings is determined based on the following equation.

$$Pp = 1500 + 50H \qquad \text{Eq. 3.6-1}$$

Pp is the passive soil pressure in pounds per square foot (psf) and H is the depth of the footing embedment in feet (ft). For interior footings, H is measured from the bottom of the floor slab (soil supported slab) or from the top of finished grade (structural slab). For exterior footings, H is measured from three feet below exterior finished grade. This equation also assumes that the adjacent soil is clay with a minimum cohesion of 750 psf.

Additional sliding resistance may be developed from the frictional resistance between the bottom of the footing and the sandy clay or clayey silt bearing soil. A coefficient of frictional resistance of u = 0.30 is used in determining frictional resistance.

A factor of safety of at least 2.0 is used when designing for horizontal load resistance.

3.6.2.4.3.5 Grade Slabs

Grade slabs are constructed on either a recompacted subgrade or on a dense graded crushed limestone base, depending on the support required from the subgrade. The assumed subgrade moduli for grade slab design is given below:

Structural Fill	=	150 pci
Crushed Limestone	=	400 pci

In either case, the minimum base thickness directly beneath the concrete slab is 8 inches. Grade slabs also have a polyethylene sheeting membrane immediately beneath the grade slab concrete to serve as a vapor barrier. Positive drainage is provided away from all structures to minimize the potential for infiltration of surface water run-off below the grade slabs.

3.6.2.4.3.6 Below-Grade Walls

Below-grade walls are designed to resist lateral earth pressures, as well as to provide sufficient drainage at the rear of the walls to minimize hydrostatic pressure. Similarly, the pit bottom slab are provided with an underdrain system such that hydrostatic uplift pressures are minimized.

Granular material (cohesionless and free draining) is used to backfill around below-grade walls. The granular backfill occupies a zone that extends at least 12 inches away from the wall at its base, and at least 12 inches plus one times the wall height away from the top of the wall.

Swell potential of the soils adjacent to the below-grade granular wall backfill materials is mitigated by placing a minimum thickness of two feet of low plasticity clay on top of the granular wall backfill material. The clay material minimizes the potential for infiltration of surface water run-off.

Table 3.6-14 presents earth pressure design parameters for potential backfill soils at the site. These parameters are applicable to the design of all below-grade walls at the site.

Factors of safety for lateral loads are given below:

Horizontal Load Resistance	2.0
Overturning	1.5

A waffle-type drainage system is placed immediately behind the exterior concrete wall. The plastic waffle construction material is applied directly to a cold-tar pitch wall waterproofing system before backfilling proceeds.

3.6.2.4.3.7 Underground Piping

Pipe bedding materials and construction requirements depend upon the pipe diameter and prevailing subsurface soil conditions. The test results of pH and resistivity indicate a significant potential for corrosion. All underground piping is afforded with appropriate corrosion protection (i.e., polyethylene pipe sleeving, cold-tar pitch coating, or cathodic protection).

3.6.2.4.4 Soil Behavior During Earthquake Loading

3.6.2.4.4.1 Liquefaction

The Stratum IV and Stratum V soils are of a type potentially susceptible to liquefaction or compaction by vibratory loading from earthquakes or other sources. This is a characteristic of cohesionless soils that are not compact (dense) enough to resist the vibratory loading.

Based on the boring data and local experience, it is judged that the possibility of liquefaction and compaction is confined to the Stratum IV and Stratum V soils. Some of these soils are above the water table, and therefore are not susceptible to liquefaction. However, "compaction" and resulting settlement could occur in these soils (as well as those below the water table) as the result of an earthquake.

The Stratum IV and Stratum V soils are analyzed for their sistance to liquefaction using the results of the borings and aboratory grain size tests in conjunction with empirical data on behavior of similar soils at locations where earthquakes and resulting liquefaction have occurred (References 38, 39 and 40). The process involves using the standard penetration test blow counts "standardized" to a vertical pressure of 2000 psf, and considering the percent "fines" (soil particles passing the No. 200 sieve), estimating the soil's strength against liquefaction by a given seismic loading from the "standardized" blow count and empirical curves prepared by Prof. Seed, calculation of the stress imposed on the soil by a given earthquake loading, and, finally, dividing the soil's strength against liquefaction by the stress imposed by the earthquake to obtain a "factor of safety" against liquefaction. If the strength is greater than the stress, then the calculated factor of safety is greater than 1 and thus liquefaction of the soil is not predicted. If the factor of safety is less than 1, then liquefaction is predicted.

Earthquake or shock loading of loose sandy soil results in a rise in the pressure of the water in the pore spaces between the grains of the soil; the pressure at liquefaction rises to become equal to the vertical pressure present in the soil before earthquake, resulting in a temporary complete loss of strength in a sandy soil which, if the liquefied soil is not too deep in the ground, results in deformations of the ground surface and structures supported thereon. Safety factors between 1 and 1.5 indicate that, although full liquefaction does not occur, there will be some increase in the pore water pressure in the soil resulting in a temporary condition of reduced soil strength and also subsequent settlement as the pore pressures dissipate. Safety factors greater than 1.5 indicate negligible effect on the soil from earthquake loading.

The above sequence of steps was performed for the Stratum IV and Stratum V sands. The earthquake loadings that might produce liquefaction and/or settlement due to compaction are discussed in Section 3.6.2.1.3 and 3.6.2.2. Section 3.6.2.2.2 summarizes the design earthquake loadings as follows: mb = 4.3, 0.045 g; mb = 5.7, 0.04 g, and mb = 6.7, 0.022 g. For the purpose of soil liquefaction assessment, a earthquake producing 0.046 g surface acceleration was assumed to be the controlling earthquake. This earthquake is more severe than any of the three design earthquakes discussed above. The following borings were chosen as representative of the loosest soil conditions encountered by the prese borings: B-10, B-18, B-19, B-27, and B-38. The "standardized" penetration resistance profile for the Stratum IV and Stratum V soil at the above borings is shown in Figure 3.6-40. Figure 3.6-40 indicates a standardized blow count that decreases with increasing depth to about 35 feet below the existing ground surface, then increases.

The soils at the referenced borings are evaluated for liquefaction resistance based on the standardized penetration resistances; the results are shown on Figure 3.6-41. All the computed safety factors against liquefaction exceed 1.5, and thus indicate negligible potential for liquefaction or induced pore pressures.

It is recognized that Seed's data are largely from the west coast and other very active, high strain-rate, generally interplate or plate-marginal seismic zones. The site is a low activity, interplate zone. Thus, some differences in seismic source characteristics (mechanisms, focal depths, stress drops) can be expected. It is unknown as to the effects of these differences upon the results obtained using Seed's method for the site environment. However, Seed's methods were shown to work well in predicting liquefaction and ground failures associated with the 1988 Saguenay, Quebec earthquake (Tuttle et.al., 1990, SAR Reference 62). This suggests that Seed's methods can be successfully applied to liquefaction problems associated with east coast earthquakes.

An alternative to Seed's method is the cyclic strain method. A cyclic strain approach to the liquefaction problem (Dobry, et. al., 1982, SAR Reference 60) is based on the premise that pore water pressure buildup during cyclic shear loading of sand is controlled mainly by the magnitude of the cyclic shear strain. This premise leads to the conclusion that shear modulus is the main parameter controlling pore water pressure buildup in the field. An important practical consequence is that measurements of in-situ modulus at small strains, which can be obtained from geophysical measurements of shear wave velocity, should be used for predicting pore pressures.

The method requires estimating both the seismic strain induced in the sand layer and the effective shear modulus of the layer during the earthquake. The method is based on measuring the shear modulus (computed from the shear wave velocity) in-situ at small strains, G_{max}, using geophysical techniques, and on performing cyclic strain-controlled tests in the laboratory to determine: (1) the modulus reduction values, G/G_{max}, (ii) the value of threshold strain at which pore pressure increases begin, and (iii) the pore water pressure buildup versus cyclic shear strain and number of cycles.

Dobry, et.al., (1982, SAR Reference 60) state that the modulus reduction curve for sand given by Seed and Idriss (1971, SAR Reference 38) and used in the SHAKE computations for this project has been confirmed by other investigators. Thus, the Seed and Idriss modulus reduction curve can be used for calculations of the induced cyclic strain.

The computer program SHAKE was used to compute the soil's response to the various earthquakes for generating the response spectra. This calculation also produces the cyclic shear strains throughout the soil profile.

The threshold strain, according to Dobry, et.al., (1982) is 0.01 percent for a wide variety of soils. This strain, if not exceeded, means that the cyclic loading does not generate pore pressures in the soil. The effective cyclic shear strains determined from SHAKE analyses were compared with this value. The effective cyclic shear strains in the stratum IV and stratum V sands in the analyses by SHAKE did not exceed .01 percent.

Thus, the cyclic strain approach predicts no pore pressures will be induced by the earthquakes. This indicates that the liquefaction from the earthquake loading is not a risk, which is the same conclusion reached from application of Seed's empirical "stress-based" procedure.

Thus, two independent methods predict no liquefaction of the stratum IV and stratum V sands.

3.6.2 1.4.2 Compaction (Earthquake) Settlement

Figure 3.6-41 indicates a safety factor of 1.5 against liquefaction by an earthquake producing 0.046 g site surface acceleration. This earthquake is more severe than any of the three design earthquakes developed in the Seismic Hazard Analysis (Section 3.6.2.1.3). A safety factor of 1.5 indicates negligible effect on the saturated part of the sand by the 0.046 g earthquake. The estimated ground surface settlement resulting from the shaking of the "dry" part of the sand (above the water table) by such an earthquake is made using the methodology recommended by Tokimatsu and Seed (Reference 41). The computed ground surface settlements are tabulated in the following Table.

Boring Number	Computed Earthquake-Induced Settlement (Stratum IV & V)
B-10	None
B-18	0.09
B-19	0.12
B-27	0.04
B-38	0.04

These computed settlements are less than 0.125 inch, and are considered to be low to negligible.

3.6.2.5 Slope Stability

There are no Safety Class I structures located adjacent to fill slopes. The fill slopes at the facility are located along the southern edge of the plant yard (Tail Storage Area) and the embankment for the Hold-Up Basin.

Analysis of both these slopes is performed using the computer program PC-SLOPE prepared by Geo-Slope Programming Ltd. of Alberta, Canada. The program is based on the simplified Bishop Method. The analysis assumes a seismic coefficient of 0.046 and soil conditions from Soil Test Borings B-47 and A-7. The soil profiles and design strength parameters are presented on the individual slope stability summary sheets in the Geotechnical Exploration Report (Reference 1).

Results of a preliminary slope stability analysis indicate a 2:1 (horizontal:vertical) fill slopes are acceptable for the plant yard fill slopes and the Hold-Up Basin embankment.

A factor of safety of at least 2.0 is used for soil slopes under normal operating conditions.

FOR INFORMATION ONLY

REFERENCES FOR SECTION 3.6

1. Law Engineering, Inc., Geotechnical Exploration Report, Louisiana Energy Services, Project No. Ht-3815-90G, Task 90, Dallas, Tx., Dated September 15, 1990.

2. Law Engineering, Inc. , Geologic and Seismic Report, Louisiana Energy Services, Project No. Ht-3815-90G, Task 90, Dallas, Tx., Dated May 17, 1990.

3. Kehle, R. O., 1978, Tectonic Framework and History, Gulf of Mexico Region, Unpublished report for Law Engineering Testing Company, Marietta, Georgia.

4. Murray, G. E., 1961, Geology of the Atlantic and Gulf Coastal Province of North America: New York, Harper, 692 p.

5. Fisher, W. L., 1965, Rock and Mineral Resources of East Texas, Bureau of Economic Geology, University of Texas, Austin, Report of Investigations No. 54, 430 p.

6. Law Engineering Testing Company, Geology and Hydrology of the Hainesville Salt Dome (Final Report), Job No. MV9767.25, 1981a.

7. Law Engineering Testing Company, An Investigation of an Inferred Fault in a Portion of the North Louisiana Salt Basin, Gulf Coast Salt Domes Project, Job No. MV9767.55, 1981b.

8. Law Engineering Testing Company, Geologic Area Characterization, Gulf Coast Salt Dome Project for Battelle Memorial Institute - Office of Nuclear Waste Isolation, 1981c.

9. Kreitler, S. W., and others, Texas Bureau of Economic Geology, Annual Report for 1979; Prepared for Department of Energy, Office of Nuclear Waste Isolation, 1980, in press.

10. Collins, E. W., Hobday, D. K., and Kreitler, C. W., Quaternary Faulting in East Texas; The University of Texas at Austin, Bureau of Economic Geology Geological Circular 80-1, 20 p., 1980.

11. Pennington, W.D., and Carlson, S.M., Observations from the East Texas Seismic Network (June 1981-August 1982); The University of Texas at Austin, Bureau of Economic Geology Geological Circular 84-3, 48 p., 1984.

12. Halbouty, M. T., Salt Domes, Gulf Region, United States and Mexico (2d ed.): Houston, Gulf Publishing, 561 p.

FOR INFORMATION ONLY

13. Duke Engineering & Services, Inc., Specification for Subsurface Investigation, No. MTS-6046-03-1609.20-0001.

14. Sykora, D. W., Examination of Existing Shear Wave Velocity and Shear Modulus Correlations in Soils, U.S. Army Corps of Engineers Miscellaneous Paper GL-87-22, 101pp., 1987.

15. Davis, S. O., Pennington, W. O., and Carlson, S. M., A Compendium of Earthquake Activity in Texas. Texas Bureau of Economic Geology, Geological Circular 89-3, 27 p., 1989.

16. Nuttli, O. W., and Brill, K. G., "Earthquake Source Zones in the Central United States Determined from Historical Seismicity", 1981, in Barstow, N. L., Brill, K. G., Nuttli, O. W., and Pomeroy, P. W., An Approach to Seismic Zonation for Siting Nuclear Electric Power Generating Facilities in the Eastern United States: U.S. Nuclear Regulatory Commission, NUREG/CR-1577.

17. Law Engineering Testing Company, Seismic Hazard Methodology for the Central and Eastern United States: Electric Power Research Institute, NP-4726, Volume 7, Research Project P101-21, 1986.

18. Street, R., and Nuttli, O. W., 1984, The Central Mississippi Valley Earthquakes of 1811-1812, in Gori, P. L. and Hays, W.H., Proceedings of the symposium on "The New Madrid Seismic Zone", U.S. geological Survey, Open File Report 84-770.

19. National Geophysical Data Center, Earthquake Intensity Data for the United States through 1983, Boulder Colorado (data tape), 1989.

20. Nuttli, O. W., 1974, "Magnitude-Recurrence Relation for Central Mississippi Valley Earthquakes", *Bulletin of the Seismological Society of America*, Vol. 64, 1189-1207.

21. Bollinger, G.A., "Reinterpretation of the Intensity Data for the 1886 Charleston, South Carolina Earthquake", in Rankin, D.W., Studies Related to the Charleston, South Carolina Earthquake of 1886 - A Preliminary Report: U.S. Geological Survey Professional Paper 1028.p. 32-42, 1977.

22. Docekal, J., Earthquakes of the Stable Interior, University of Nebraska, Ph.D. dissertation, 1970, volume 1, 169 p., volume 2, 332 p.

23. Coffman, J. L. and Von Hake, C. A., Earthquake History of the United States: U.S. Department of Commerce, Publication 41-1 (Revised Edition), 1-208, 1973.

24. Bernreuter, D. L., Savy, J.7 B., Mansing, R. W., and Chan, J. C., Seismic Hazard Characterization of 69 Nuclear Plant Sites East of the Rocky Mountains, U.S. Nuclear Regulatory Commission, NUREG/CR-5250 VCID-21517, 1989.

25. Boore, D. M., Stochastic Simulation of High Frequency Ground Motions Based on Seismological Models of the Radiated Spectra, Bulletin Seismological Society of America, 73, 1865 - 1894, 1983.

26. Herrmann, R.B., An Extension of Random Vibration Theory Estimates of Strong Ground Motion to Large Distances, Bulletin of Seismological Society of America, vol. 75, 1447-1453, 1985.

27. Boore, D. M. and Atkinson, G.M., Stochastic Prediction of Ground Motion and Spectral Response Parameters at Hard Rock Sites in Eastern North America, Bulletin of the Seismological Society of America, 77, 440-467, 1987.

28. Toro, G. R. and McGuire, R. K., An Investigation Into Earthquake Ground Motion Characteristics in Eastern North America, Bulletin of the Seismological Society of America, 77, p.468-489, 1987.

29. Nuttli, O. W., Rodriquez, R. and Herrmann, R. B., Strong Ground Motion Studies for South Carolina Earthquakes, U.S. Nuclear Regulatory Commission, NUREG/CR-3755 (UCRL-15594), 1984.

30. Nuttli, O. W., 1986, Letter to J. B. Savy dated September 19, 1986, Source: J. B. Savy, personal communication.

31. Nuttli, O. W., "State-of-the-Art for Assessing Earthquake Hazards in the United States; The Relation of Sustained Maximum Ground Acceleration and Velocity to Earthquake Intensity and Magnitude", Miscellaneous Paper S-73-1, Report 16, 1979, U.S. Army Engineer Waterways Experiment Station, C.E., Vicksburg, Mississippi.

32. Uniform Building Code, 1988 Edition, International Conference of Building Officials, Whittier, California.

33. Schnabel, P.B., Lysmer, U., SEED, M.B., SHAKE, A Computer Program for Earthquake Response Analysis of Horizontal Layered Sites, University of California, Berkeley, Report No. EERC 72-12, 1972.

34. Weichert, D. H., Pomeroy, P. W., Munro, P. S., and Mork, P. N., Strong Motion Records from Miramichi, New Burnswick, 1982 Aftershocks, Earth Physics Branch Open File Report 82-31, 1982, Ottawa, Canada.

FOR INFORMATION ONLY

35. Friberg, P. A., and Jacob, K.H., "Peak Horizontal Accelerations and Pseudo Velocity Response for the Saguenay Earthquake", in K. H. Jacob (Ed.), The Saguenay Earthquake of November 25, 1988, Quebec, Canada, Strong Motion Data, Ground Failure Observations, and Preliminary Interpretations, NCEER, Buffalo, New York, 1989.

36. Newmark, N. M., and Hall, W. J., Earthquake Spectra and Design, Earthquake Engineering Research Institute, 103pp, 1982.

37. Gasparini, D. A., and Vanmarcke, E. H., Simulated Earthquake Motions Compatible with Prescribed Response Spectra, Massachusetts Institute of Technology, Publication No. R76-4,65pp.

38. Seed, H.B., and Idriss, I.M. "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9, Proc. Paper 8371, September, 1971.

39. Seed, H. Bolton, "Influence of SPT Procedures In Soil Liquefaction Resistance Evaluations", ASCE, Journal of the Geotechnical Engineering Division, Vol. 111, No. 12, December, 1985.

40. Seed, H. Bolton, "Design Problems in Soil Liquefaction", ASCE, Journal of the Geotechnical Engineering Division, Vol. 113, No. 8, August, 1987, p. 827.

41. Tokimatsu and Seed, "Evaluation of Settlements in Sand Due to Earthquake Shaking", ASCE, Journal of the Geotechnical Engineering Division, Vol. 113, No. 8, August, 1987.

42. Toro, G.R. and McGuire, R.K., Ground Motion Estimates for the Central and Eastern United States: Response to Request for Additional Information, Risk Engineering, Inc., June 1988.

43. Bollinger, G. A., 1983, "Speculations On the Nature of Seismicity at Charleston South Carolina", in Gohn, G. S., Studies Related to the Charleston, South Carolina Earthquake of 1886 - Tectonics and Seismicity; U. S. Geological Survey Professional Paper 1313, pp. J1-J29.

44. Sibol, M. S., Bollinger, G. A., Birch, J. B., "Estimation of Magnitude in Central and Eastern North America Using Intensity and Felt Area", Bulletin of Seismological Society of America, Vol. 77, No. 5, pp 1635-1654, October 1987.

FOR INFORMATION ONLY

45. Durham, Clarence O., Jr., 1964, "Iron Ore of Central North Louisiana," Geological Bulletin No. 41, State of Louisiana Department of Conservation, Baton Rouge, Louisiana, pp. 11-12.

46. Dwyer, J.J., Herrmann, R.B. and Nuttli, O.W. (1983). Spatial attenuation of the Lg wave - the Central United States Bulletin of the Seismological Society of America, vol. 73, No. 3, 781-796.

47. Johnston, A. and Nava, S.J. (1984). Recurrence rates and probability estimates for the New Madrid Seismic Zone. in Gori, P.L. and W.W. Hays, (eds.). Proceedings of the Symposium on the New Madrid Seismic Zone", U.S. Geological Survey, Open File Report 84-770, 279-329.

48. McColloh, Richard P., and Autin, Whitney J., 1991, "Revised Mapping of Surface Faults, East Baton Rouge Parish, Louisiana, " Transactions of the Forty-First Annual Meeting of the Gulf Coast Association of Geological Societies AAPG Regional Meeting, Vol. XLI 1991, p. 473.

49. McCulloh, R.P., and Autin, Whitney J., 1990, "Surface Faults in East Baton Rouge Parish," Louisiana Geological Survey Project No. 90-MS-CP-0024.

50. Russ, D.P., 1978, Recent Surface Faulting and Earthquake Recurrence in the Reelfoot Lake Area, Northwestern Tennessee (abs): Midwest Annual Meeting, American Geophysical Union, St. Louis, MO.

51. Russ, D.P., Late Holocene Faulting and Earthquake Recurrence in the Reelfoot Lake Area, Northwestern Tennessee: Geol. Soc. Amer. Bull., V 90, pp 1013-1018.

52. Russ, D.P. (197?). Late Holocene faulting and earthquake recurrence in the Reelfoot Lake area, northwestern Tennessee. Bulletin of the Geological Society of America. vol. 90, 1013-1018.

53. Seed, H. Bolton, et. al. (1975), "Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses" Report No. EERC 75-29, University of California, Berkeley, October.

54. Singh, S. (1981). Regionalization of crustal Q in the continental United States, Ph.D. Dissertation, Saint Louis University, Saint Louis, Missouri.

55. Stevenson, Donald A., and Agnew, James D., 1988, "Lake Charles, Louisiana, Earthquake of 16 October 1983," Bulletin of the Seismological Society of America, Vol. 78, No. 4, pp. 1463-1474.

56. Street, R. (1982). A Contribution to the Documentation of the 1811-1812 Mississippi Valley Earthquake Sequence Earthquake Notes. vol. 53, No. 2, 39-52.

57. Walthall, Bennie H., and Walper, Jack L., 1967, "Peripheral Gulf Rifting in Northeast Texas," The American Association of Petroleum Geologists Bulletin, Vol. 51, No. 1, p. 102-110.

58. Hamilton, R.M. and Johnston, A.C., 1990, Techumseh's Prophecy: Preparing For the Next New madrid Earthquake: U.S. Geological Survey Circular 1066, 30pp

59. Cox, R. T. Possible Triggering of Earthquakes by Underground Waste Disposal in the El Dorado, Arkansas Area. Seismological Research Letters. Vol. 62, No. 2, 1991.

60. Dobry, R.; Ladd, R.S.; Yokel, F.Y.; Chung R.M.; and Powell, D. (1982) "Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by The Cyclic Strain Method," Building Science Series 138, National Bureau of Standards, U.S. Department of Commerce, U.S. Government Printing Office, Washington, D.C.

61. Sharp, J. M., Raymond, R. H., Germiat, S. J., and Paine, J. G. Re-evaluation of the Causes of Subsidence Along the Texas Gulf of Mexico Coast and Some Extrapolation of Future Trends. Land Subsidence (Proceedings of the Fourth International Symposium on Land Subsidence, May 1991), IAHS Publ. no. 200, 1991.

62. Tuttle, M., Law, T., Seeber, L., and Jacob, K. (1990), "Liquefaction and Ground Failure Induced by the 1988 Saguenay, Quebec Earthquake, " Canadian Geotechnical Journal, Vol. 27, pp. 580-589.

TABLE 3.6-16
EARTHQUAKE RECORDS USED FOR SITE RESPONSE ANALYSES

EARTHQUAKE	RECORDING SITE	GEOLOGY	MAGNITUDE	DURATION (SEC)	DISTANCE FROM ZONE OF ENERGY RELEASE (miles)	SCALED A_{MAX} g (PEAK)	SCALED V_{max} (cm/s)	SCALED DISP (cm)	COMPUTER CEC SITE SURFACE A_{max} g (PEAK)
Near-Field	Mitchell Road	Bedrock	$m_N = 4.8$	1	2.5	.045 hor	0.54	0.015	0.021 hor
New Brunswick						.033 ver	0.17	0.004	0.0098 ver
3-31-82									
Mid-Field	Riviere-Ouelle,	Bedrock	$m_b = 5.9$	4.5	71	.040 hor	2.20	0.28	0.046 hor
Saguenay	Quebec					.028 ver	1.57	0.14	0.041 ver
11-25-88									
Far-Field	NA	NA	$m_b = 6.7$	26	227	.022 hor	–	–	0.024 hor
(Synthetic)						.0157 ver	–	–	0.027 ver

TABLE 3.6-17
SUBSIDENCE AND EARTHQUAKES ASSOCIATED WITH FLUID WITHDRAWAL

FIELD	LOCATION	DATE	PREVIOUS PRODUCTION	MAXIMUM SUBSIDENCE	MAGNITUDE OF ASSOCIATED EARTHQUAKES	REFERENCES
Sour Lake	W. of Beaumont, TX	1929	73 MMbbl	50m	----	Sellards (1930), National Oil Scouts of America (1931), Sheets (1947)
----	Houston, TX	1943-1974	ground water	2m	----	Gustavson and Kreitler (1976), Verbeek and Clanton (1981)
Chocolate Bayou	S. of Houston, TX	1944-1974	----	0.5m	----	Grimsrud, et al. (1978)
Goose Creek	E. of Houston, TX	1944-1974	----	----	small	Pratt and Johnson (1926), Yerkes and Castle (1976)
Mexia; Wortham	Mexia, TX	1932	112 MMbbl	----	3.8[a]	Sellards (1933), Yerkes and Castle (1976)
East Texas	Gladewater, TX	1957	3.5 Bbbl	----	4.0,2.5,2.5,2.5[a]	Docekal (1970), Yerkes and Castle (1976)
Imogene (gas); Flashing (oil)	Flashing and Pleasanton, TX	1973-1984	----	----	3.9,3.4	Pennington et al. (1986)

Source: Davis et al. (1989)

[a] Magnitudes calculated using relationships developed by Sibol et al. (1987).