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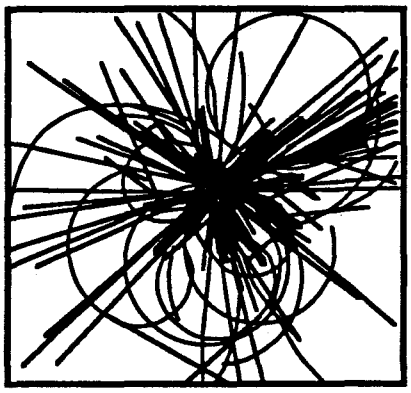
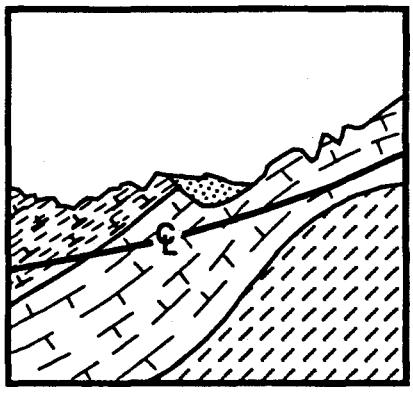
Preliminary Geotechnical Parameters for the Superconducting Super Collider Site





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PROJECT NO. 87-888-0017
DECEMBER 1990

REPORT NO. SSC-GR-69

PRELIMINARY GEOTECHNICAL PARAMETERS FOR THE SUPERCONDUCTING SUPER COLLIDER SITE

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1.0 INTRODUCTION

This report evaluates the geotechnical parameters of the three rock units in the construction zone at the Superconducting Super Collider (SSC) site in Ellis County, Texas. The objective is to provide a preliminary evaluation of relevant geotechnical parameters to support the site specific conceptual design of the SSC facilities. The evaluation is based on synthesis of available project data (geologic, geohydrologic, geophysical and geotechnical engineering data), engineering interpretation, and engineering judgment. Geotechnical parameters are presented in terms of range and best estimated values (Section 3). Potential uncertainties are noted and recommendations for future work are also made (Section 4).

Figure 1 shows the planned locations of various SSC facilities. The SSC underground facilities include:

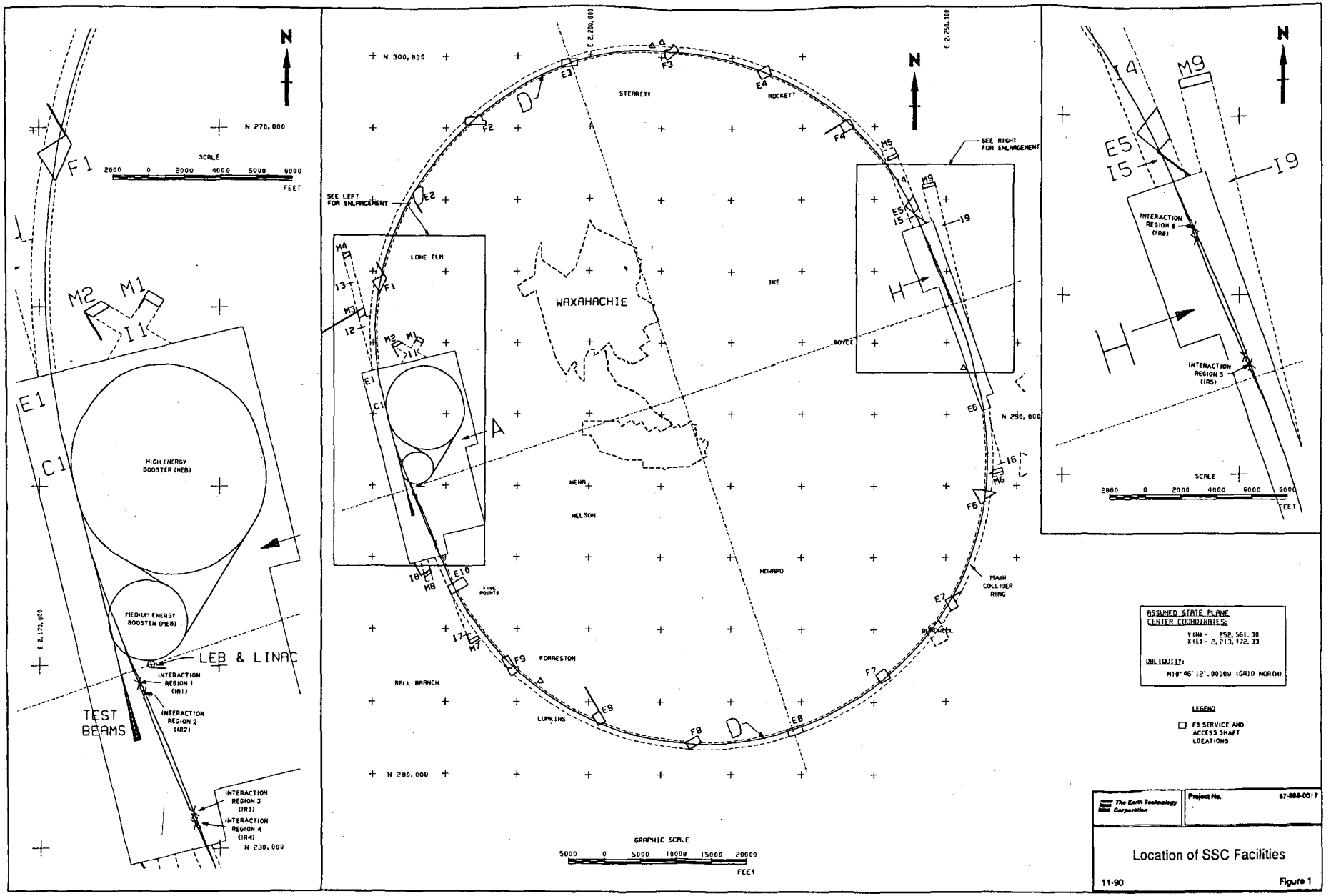
- the main collider ring tunnel, about 54 miles long with a minimum finished internal diameter of about 12 feet
- an injection and beam test facility consisting of a series of particle accelerators and energy boosters culminating in a 6-mile-long High Energy Booster (HEB) (for injecting particles into the main ring)
- six experimental halls (interaction regions) and associated connecting tunnels and shafts, and a series of service and access shafts (SSC Laboratory, 1989).

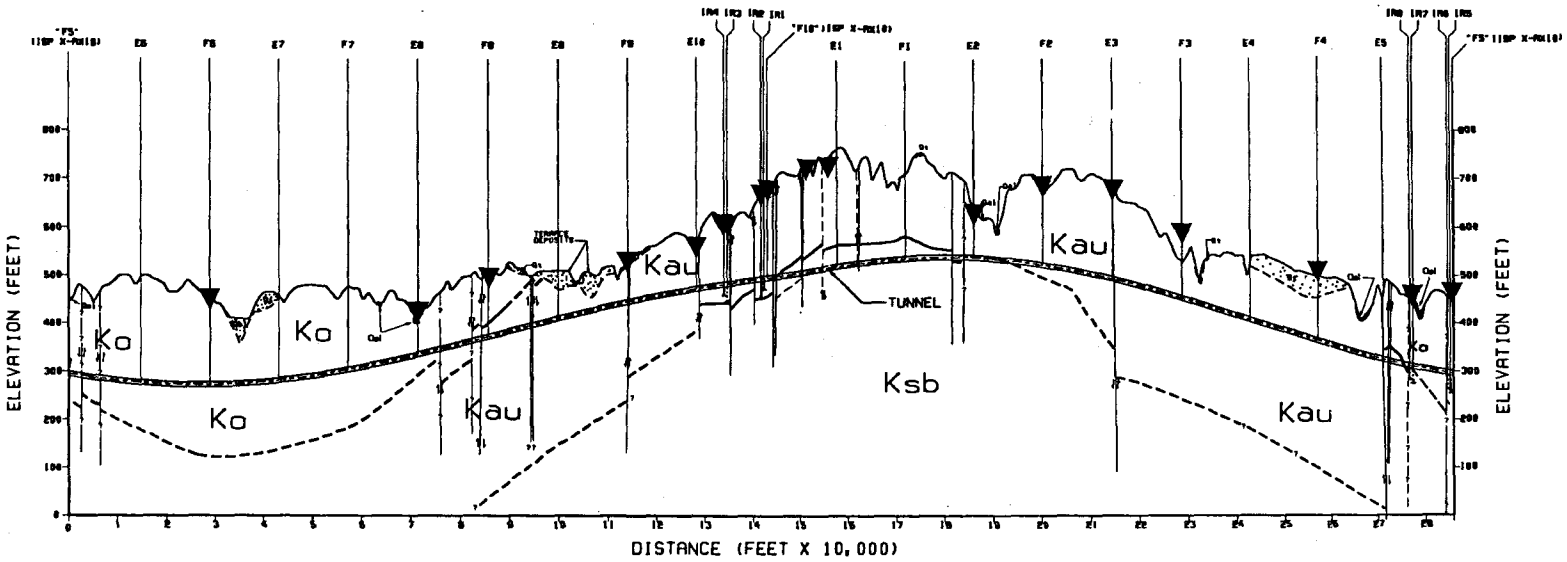
It is anticipated that a major portion of the tunnel will be excavated using tunnel boring machines (TBM). The shafts may be excavated by blasting and mucking or by mechanical shaft drilling machines. Some experimental halls may be constructed by a cut-and-cover method whereas others may be underground excavations.

1.1 SITE GEOLOGIC SETTING

The subsurface materials at the SSC site consist, in descending order, of locally present Quaternary alluvial and terrace deposits (generally along streams and on terraces) overlying the Upper Cretaceous Gulf series, which, in turn, overlies the Lower Cretaceous Comanche Series over the Paleozoic basement rocks. Data on engineering properties of Quaternary deposits at the SSC Site are limited and these deposits are not considered in this report. The SSC underground facilities are located within the Gulf Series, which includes the Taylor Marl (TM), Austin Chalk (AC), and Eagle Ford Shale (EFS) and Woodbine groups. The TM, AC, and EFS groups are generally subdivided into several formations (e.g., Allen and Flanigan, 1986), but the more traditional names (TM, AC, EFS) are used in the SSC project reports. The Woodbine group is located far below the planned SSC underground facilities and should have no effect on their design and construction. In general, the rocks dip gently southwest (i.e., gulfward) with AC and TM cropping out at the SSC site.

A general geologic profile along the main tunnel alignment at the site is shown in Figure 2. Some data on hydrologic conditions are also shown in this figure. About 17, 29, and 8 miles of the main tunnel will be in TM, AC, and EFS, respectively. Figure 2 shows that TM crops out at the surface on the eastern third of the site whereas AC is exposed in the western two-thirds of the site. Several faults are known to cross the tunnel alignment. Details and locations of the fault zones with respect to the overall SSC facilities are presented in Report No. SSC-GR-65.


N
N 270,000
SCALE
2000 0 2000 4000 6000 8000
FEET
F1
M2
M1
I1
E1
C1
HIGH ENERGY BOOSTER (HEB)
MEDIUM ENERGY BOOSTER (MEB)
LEB & LINAC
INTERACTION REGION 1 (IR1)
INTERACTION REGION 2 (IR2)
TEST BEAMS
E 2,170,000
INTERACTION REGION 3 (IR3)
INTERACTION REGION 4 (IR4)
N 230,000
N 300,000
E 2,250,000
E 2,350,000
STERRETT
ROCKETT
LONE ELM
WAXAHACHIE
IKE
BOYCE
NEWA
NELSON
HOWARD
FIVE POINTS
FORRESTON
BELL BRANCH
LUMKINS
BENGEL
MAIN COLLIDER RING
SEE LEFT FOR ENLARGEMENT
SEE RIGHT FOR ENLARGEMENT
N 280,000
N 290,000
A
D
H
GRAPHIC SCALE
5000 0 5000 10000 15000 20000
FEET
INTERACTION REGION 6 (IR6)
INTERACTION REGION 5 (IR5)
ASSUMED STATE PLANE CENTER COORDINATES:
Y(N) - 252,561.30
X(E) - 2,213,172.33
OBLIQUITY:
N18°46'12".0000W (GRID NORTH)
LEGEND
F8 SERVICE AND ACCESS SHAFT LOCATIONS
The Earth Technology Corporation
Project No.
87-868-0017
Location of SSC Facilities
11-90
Figure 1



LEGEND

Qal/Qt	ALLUVIUM/TERRACE DEPOSITS
Ko	TAYLOR MARL - OZAN FORMATION
Kau	AUSTIN CHALK
Ksb	SOUTH BOSQUE FORMATION - "EAGLE FORD SHALE"
▼	STATIC WATER LEVEL

CONTACT

INFERRED CONTACT

FAULT

GROUND ELEVATION DETERMINED FROM USGS TOPOGRAPHIC MAPS

TUNNEL PROFILE - SEE KEY PLAN FOR STRIKE, OBLIQUITY, AND CENTER LOCATION

APPROXIMATE BASE OF EXPERIMENTAL HALL

The Earth Technology Corporation

Project No. 87-888-0017

Geologic Profile

12-90

Figure 2

The groundwater level at the SSC site generally ranges from about 40 feet below to about 30 feet above the ground surface (Report No. SSC-GR-63). As shown in Figure 2, the static groundwater levels range from about 60 feet to about 200 feet above the crown levels of the main tunnel. AC and EFS strata appear to be mostly below the piezometric level and are considered to be saturated; the portion of TM below the groundwater level also appears to be saturated.

1.2 GEOTECHNICAL PARAMETER NEEDS

Geotechnical parameters of both in situ rock mass and discontinuities are needed to address various design and construction-related issues. Key issues may include, but are not necessarily limited to, the following:

- In-situ subsurface geologic, geotechnical, and hydrologic conditions
- Feasibility of planned excavation and construction methods in these geologic units
- Both short-term (during construction) and long-term (during underground facility operation) deformation and stability characteristics of the in situ rock masses
- Design of primary support systems and final linings for shaft, interaction halls, and tunnel facilities.

The above issues can be preliminarily addressed based on the following geotechnical parameters:

- Rock mass quality
- In-situ stress conditions
- Hydrologic conductivity of the rock mass
- Intact rock index properties such as density, moisture content, specific gravity, degree of saturation, slake durability, swell potential, etc.
- Strength and deformation properties of the rock masses
- Strength and deformation properties of discontinuities
- Seismic wave velocities and dynamic modulus of rock mass
- Time dependent response (e.g., swell and consolidation) characteristics of rock mass.

It should be noted that specific geotechnical parameter needs are dependent on specifics of facility design/construction that are typically interactive. In addition, the geotechnical parameters are generally further refined as the design of facilities progresses. In this report, the geotechnical parameters are developed based mostly on our understanding of the general engineering characteristics of the three major bedrock units. Thus, no attempt was made to specifically match the parameter needs with any specific design details of the SSC facilities.

1.3 GEOTECHNICAL CHARACTERIZATION INVESTIGATIONS

Field and laboratory investigations have been done to evaluate the subsurface geologic, geotechnical, and hydrogeologic conditions at the SSC site. These investigations included the following:

- Drilling and coring 116 borings to various depths in all three rock formations to provide an overall geologic/geotechnical characterization of the site environs
- Geophysical surveys and wireline logging of borings
- Hydrogeologic testings including groundwater piezometric level monitoring, slug tests, and packer tests

- A limited field testing program including pressuremeter testing in two borings and hydraulic fracturing testing in one boring for a preliminary estimate of in-situ moduli and stress conditions
- A laboratory test program to (1) determine index properties and strength and deformation characteristics of the intact cores, and (2) estimate strength characteristics of discontinuities.

All investigations were done by or under the direction of The Earth Technology Corporation (TETC). Pressuremeter testing was done by STS Consultants, Ltd. and the hydraulic fracturing testing was performed by RE/SPEC Inc. Most of the laboratory tests were done by Mason Johnston and Associates (MJA) and Southwestern Laboratories (SWL), who subcontracted mineralogy tests to Southern Methodist University (SMU) and Core Laboratories (CL), respectively. Some tests were also conducted by Professor Roy E. Olson at the University of Texas (Austin) and Professors G. Mesri and A. Nieto at the University of Illinois (Champaign-Urbana).

The available results of geologic, geophysical, geotechnical, and hydrogeologic investigations are documented in a series of SSC geotechnical reports prepared by TETC (see reference list in Section 5.0).

2.0 FIELD AND LABORATORY GEOTECHNICAL CHARACTERIZATION

2.1 FIELD GEOTECHNICAL CONDITIONS

2.1.1 Bedrock at SSC Site

Details of bedrock geology of the SSC site, including lithology, stratigraphy, and geologic faults, and their distribution with respect to the planned facilities are included in Report No. SSC-GR-65. The primary characteristics of the three major bedrock units are summarized below:

Taylor Marl

Taylor Marl is underlain by AC and is exposed at the surface on the eastern third of the site. The thickness of TM varies from 0 to a maximum of about 360 feet at the site. The following characteristics are typical of TM:

- Medium-gray to bluish-black shale cemented with calcite
- Soft (very low strength) rock with uniaxial compressive strength generally less than 1000 psi
- Occasional fossil fragments and thin calcite layers
- Moderate carbonate content of about 13 to 34 percent with an average value of about 24 percent.

Austin Chalk

Austin Chalk is exposed at the surface on the western two-thirds of the site and is underlain by EFS. In the remaining one-third of the site, AC is sandwiched between the overlying TM and underlying EFS. Characteristics of the AC vary somewhat depending on stratigraphic location in the unit, as follows:

Top 147 feet

- Primarily light to medium gray chalk (microgranular calcite)
- Thick chalk beds, ranging from 0.1 to 8 feet but generally greater than 1 foot thick
- Moderately argillaceous to shaly chalk interbeds usually less than 1 foot thick but generally ranging from 0.1 to 5.3 feet
- Infrequent thin bentonitic shale interbeds
- Saturated.

Middle 187 feet

- Primarily light to medium gray chalk (microgranular calcite)
- More frequent interbeds of argillaceous chalk and shaly chalk (to calcareous shale) than in the overlying 147 feet
- Common bentonitic layers, generally 1 to 2 inches thick
- Common pyrite nodules, 1 to 3 inches in diameter

- A marker bentonite bed (MB), 0.75 to 1.0 foot thick, widely traceable laterally, occurs at the base of this section. This marker bed is nominally 70 feet above the AC/EFS contact zone
- Saturated.

Bottom 69 feet

- Primarily light to medium gray chalk (microgranular calcite)
- Thick chalk beds with thin interbeds of moderate to very argillaceous chalk and shaly chalk
- Lithologically similar to the top 147 feet
- Small (generally less than 0.25 inch) pyrite nodules
- A fossiliferous argillaceous and arenaceous zone at the base known as the Fish Beds
- Saturated.

Eagle Ford Shale

Eagle Ford Shale at the site is completely overlain by AC or both AC and TM. The general features of EFS are as follows:

- Dark, gray-black soft shale
- Including some very thin limestone laminae, particularly in a zone 40 to 75 feet below the top of the unit
- Calcite concretions and pyrite nodules
- Slight to moderate carbonate content (1 to about 54 percent) averaging about 8.6 percent
- Saturated
- On the order of 300 feet thick at the site
- No rock outcrop in the site area.

2.1.2 Geotechnical Divisions of Bedrock

Based on the wireline logs, the three major bedrock units can be further subdivided into several subunits (Report No. SSC-GR-23). Each of the subunits within the three rock units is believed to belong to a different geologic age group and may exhibit different geomechanical behavior. However, analyses of available data (Report Nos. SSC-GR-66, -67, and -68) with respect to these wireline units indicating the following:

- The geotechnical characteristics of TM are variable and do not appear to correlate with stratigraphic position in the unit
- Top AC contains less argillaceous material and is generally stiffer and stronger than middle AC and bottom AC
- EFS appears to be stiffer and stronger both with increasing depth below ground surface and with relative location of stratigraphic units (i.e., depth below AC/EFS contact). Thus, the deeper parts of EFS are generally stiffer and stronger than the upper parts of EFS.

Based upon these findings, TM is discussed herein as a single unit. AC and EFS, on the other hand, are addressed both as single units and divided into three and two subunits, respectively, for geomechanical characterization. The corresponding approximate thicknesses of the subunits of AC and EFS are as follows:

Subunits	Approximate Thickness (ft.)	Approximate Interval or Stratigraphic Distance Above/Below Reference Contact (ft.)	Corresponding Wireline Units (Report No. SSC-GR-23)
Austin Chalk			
Top AC	147	0-147 below TM	A-15, A-16, A-17, A-18
Middle AC	187	147–334 below TM or 0-187 above MB	A-19, A-20, A-21, A-22, A-23, A-24, A-25
Bottom AC	69	334-403 below TM or 0-69 below MB	A-26, A-27
Eagle Ford Shale			
Top EFS	42	0-42 below AC	E-20
Bottom EFS	>64	>42 below AC	E-21, E-22

2.1.3 Discontinuity Characteristics and Rock Quality

Discontinuities in the rock mass include faults, fractures and bedding planes. A number of mapped faults (with tens of feet of displacement) and suspected faults have been interpreted to date (Table 1). These are principally in the eastern and western areas of the site, which are the most intensely explored parts of the site. Additional faults will probably be found as exploration continues in other areas. The three bedrock units also contain near-horizontal bedding planes and joints/fractures at various strike/dip orientations. The phenomenon of core disking in which rock cores fragment into disk-like short cylinders upon coring, during shipping and storage, or after being exposed to air has also been observed during field exploration and laboratory testing programs.

Similar disking phenonmena have been observed in borings in the Dallas area (Henley, 1990). An examination of various core disking segments indicates that causes of disking may be one or a combination of the following:

- Weakening of weak horizontal bedding planes due to mechanical disturbance during drilling, stress relief upon core recovery, or environmental deterioration (TM and EFS have low slake durability).
- Effect of drilling procedure or worn drill bit causing rock fragments to jam between core and drill, which grinds the core circumferentially causing the core to shear at relatively weaker horizontal bedding planes. This is evidenced by the observations of reduced diameter in disking cores, lack of drill ridges on the core and numerous fragmented areas near horizontal separation.

TABLE 1. MAPPED FAULTS NEAR THE SSC SITE[1]

Fault	Strike	Dip	Displacement	Rock Type Faulted (thickness of faulted zone)	RQD of Fault Zones (percent)	Fracture Spacing (ft) Range	Fracture Spacing (ft) Average	Hydraulic Conductivity (cm/sec)	Estimated RMR [2] (Rock Class)
Italy Graben	N15°E-N5°W	Steeply W	~10 ft. down to the west	AC					
SF 8.3	N5°-15°E	Steeply SE	~25 ft. down to the southeast	TM AC					
SIR 3 Graben	N40°E to N50°E	Southern fault 70-75° NW	Graben; Southern fault has 73 ft. down to north; northern fault has 64 ft. down to the south	AC (5'±) EFS (0.6'±)	96 85	0.2 to 0.4 0.1 to 3.3	0.25 1	$<4.6 \times 10^{-7}$	51 (III) 30 (IV)
SE 10.7	Due north to N5°E	65°-90° West	20 ft. down to the west	AC					
SE 10.9 Fault	Average trend N28°E	~65°SE	Graben; 35 ft. on southern fault, 80 ft. on northern fault	AC (4.5'±) EFS (11.9'±)	54 24	0.1 to 3.2 0.1 to 0.3	1.2 0.12	3.3×10^{-7}	57 (III) 20 (V)
SF 9	N50°E	50° to 70°SE	55 ft. down to the south	AC					
SF 9.8	N85°E	60°N	> 18 ft. down to the north	AC					
SE 10	N50°E	59° to 65°SE	54 ft. down to the south	AC					
SF 10.1 Graben	N50°E	Northern fault steeply SE; southern fault steeply NW	~25 ft. on northern fault; 60 ft. on southern fault	AC (0.3 to 1.1') EFS (2.3'±)	99 78	0.1 to 6.1	1.8 0.6	8.5×10^{-7} 4.1×10^{-6}	59 (III) 29 (IV)
SF 10.8	E-W	Steeply N	8 ft. down to the north	AC					
SI 2 Fault	N40°E	Steeply SE	75-80 ft. down to the southeast	AC (8.7±)	89	–	–	2.9×10^{-5}	
SF 10.6 Fault	N10°-20°E	Steeply to the NW	10 ft. down to the west	AC					
SE1 Graben	N10° to 30°	Northern fault ~44SE; southern fault steep to the NW	Graben, 50-90 ft. on both faults	AC (1.25) EFS	86	–	–	3.4×10^{-5}	
SE 1.5 Fault	N10°E	Moderately steep SE	4 ft. down to the east	AC EFS					
SF 1.8	N35°W	N65°SW	21 ft. down to the southwest	AC					
SE 3	N10°-20°N	Steeply to E	64 ft. down to the east	AC					
SE 5	N80°W	Steeply to S	70 ft. down to the south	TM AC					
Sardis Fault	N65°E	Steeply to NW ~60°	~90 ft. down to the northwest	AC					
Sterrett Fault	N50°E	Steeply NW	> 100 ft. down to the northwest	AC					

TABLE 1. MAPPED FAULTS NEAR THE SSC SITE[1]
Page 2 of 2

Fault	Strike	Dip	Displacement	Rock Type Faulted (thickness of faulted zone)	RQD of Fault Zones (percent)	Fracture Spacing (ft) Range	Fracture Spacing (ft) Average	Hydraulic Conductivity (cm/sec)	Estimated RMR [2] (Rock Class)
Bear Creek Fault	N40°E	~63°NW	> 90 ft. of offset down to the northwest	TM AC					
Rockett Graben	N30°E	North fault steeply SE, south fault steeply NW	> 100 ft. of offset in the graben	TM AC					
SE 5.2 Fault	E-W	~55° N	25 ft. down to north	TM (3.2′±) AC (2.4′±)	93 99	0.12 to 5.6 0.3 to 3.25	1.6 1.7	- 4.2×10^{-7}	36 (IV) 59 (III)
SF 5.8 Fault	E-W	-	Tenuous correlation suggests 8 ft. down to the south	TM AC					
Unnamed Fault	N15°-20°E	-	Down to the west	TM					
Lake Waxahachie Graben	N70°E	Northern fault dips steeply SE; Southern fault dips 50-70° NW	> 60ft.	TM AC					

Notes: (1) Refer to Report No. SSC-GR-65 for more details.

(2) Refer to Table 7 for Rock Mechanics Rating (RMR) details. Class III (Fair rock), IV (Poor rock) and V (Very poor rock) correspond to RMR ranges of 41 to 60, 21 to 40, 0 to 20, respectively.

Statistics of fracture zones, depth of weathering, and core-disking zones are summarized in Table 2 (see also Report No. SSC-GR-83). The statistics on occurrence of core disking shown in Table 2 may be less than the actual values because core disking was observed to occur even after core photographs were taken and was not included in the statistics. Statistics of joint spacing, conditions and orientation were evaluated and summarized in Table 3. These statistics are based on the data from the following:

- 13 inclined borings
- Surficial mapping in the Interaction Regions 1 to 5 and 8A.

Boring logs and rock cores provide a quantitative classification of in-situ rock masses in terms of rock quality designation (RQD), which is defined as the ratio (in percent) of the total length of core pieces of size equal to or longer than 4 inches to the total length of core attempted. Rock quality in terms of RQD is defined as follows (Deere et al., 1966):

RQD Percent	Rock Quality
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

RQD from 74 borings (including 6 vertical and 13 inclined borings) is summarized in Figure 3. The results indicate that the rock quality of the three bedrock units is excellent with RQD mostly in the range of 90-100 percent, except in the weathered zones and fault zones. However, it should be noted that RQD alone is not a good indicator for rock mass quality for thinly bedded, low-strength materials with high swelling, slaking potential, such as TM and EFS. RQD statistics for TM and EFS are presented here as one of many rock mass rating factors (Refer to Section 3).

2.1.4 Hydrogeologic Characteristics

Details and results of hydrogeologic investigations at the SSC site are documented in the Report No. SSC-GR-63. The studies included constructing and monitoring wells and conducting packer and slug tests to assess hydrogeologic conditions and hydraulic conductivities of the three bedrock units. Figure 4 summarizes hydraulic conductivity values with respect to stratigraphic units (Report No. SSC-GR-23). The results of the hydrogeologic investigations are summarized as follows:

- As shown in Figure 2, the piezometric level at the SSC site construction zone mostly ranges from about 40 feet below to 30 feet above the ground surface.
- AC and EFS formations appear to be mostly below the piezometric surface and are considered to be saturated. At least the portion of TM below groundwater level also appears to be saturated.
- Measured hydraulic conductivity data cannot be clearly correlated with lithology, wireline stratigraphic units, or RQD values.
- Measured hydraulic conductivity (Figure 4) for the in situ rock mass, except the fault/graben zones (Table 1), ranges from less than 1.2×10^{-8} to 3.1×10^{-5} cm/sec with most values in the range of 2×10^{-8} to 4×10^{-6} cm/sec. A median value is about 2×10^{-7} cm/sec.

TABLE 2. SUMMARY STATISTICS OF FRACTURE ZONE, DISKING ZONE AND WEATHERING THICKNESS

Item	Taylor Marl (TM)	Austin Chalk (AC)				Eagle Ford Shale (EFS)
	Overall	Overall	Top	Middle	Bottom	Overall
Sample Basis[1] (feet drilled)	4,270	11,429	2,034	6,105	3,290	1,919
Percent Fractured[2] (% of total interval)	0.7	0.9	1.9	0.8	0.6	4.3
Width of Fracture Zone (ft)						
Max	5.6		14.6	9.8	7.0	5.9
Mean	1.8		1.9	2.5	1.1	1.1
Std. Dev.	1.7		3.1	3.1	1.5	1.1
Number of Disking Zones	15	48	13	17	18	8
Width of Disking Zone (ft)						
Max	25.0	14.0				6
Mean	6.2	2.9				2.6
Std. Dev.	6.6	3.2				2.2
Thickness of Weathering[3] (ft)						
Max	44	32.5				
Min	0	0				
Mean	13.1	8.9				
Std. Dev.	12.6	6.5				

Notes: (1) This summary is based on logs and core photographs of 172 fracture zones from 78 vertical borings and 71 fracture zones from 13 inclined borings (refer to Report No. SSC-GR-83) and does not include the omnipresence of near-horizontal bedding planes in TM and EFS.

(2) Fracture zone is defined as a depth range containing two or more fractures per foot.

(3) Total sample sizes are 25 and 53 boring locations for TM and AC, respectively.

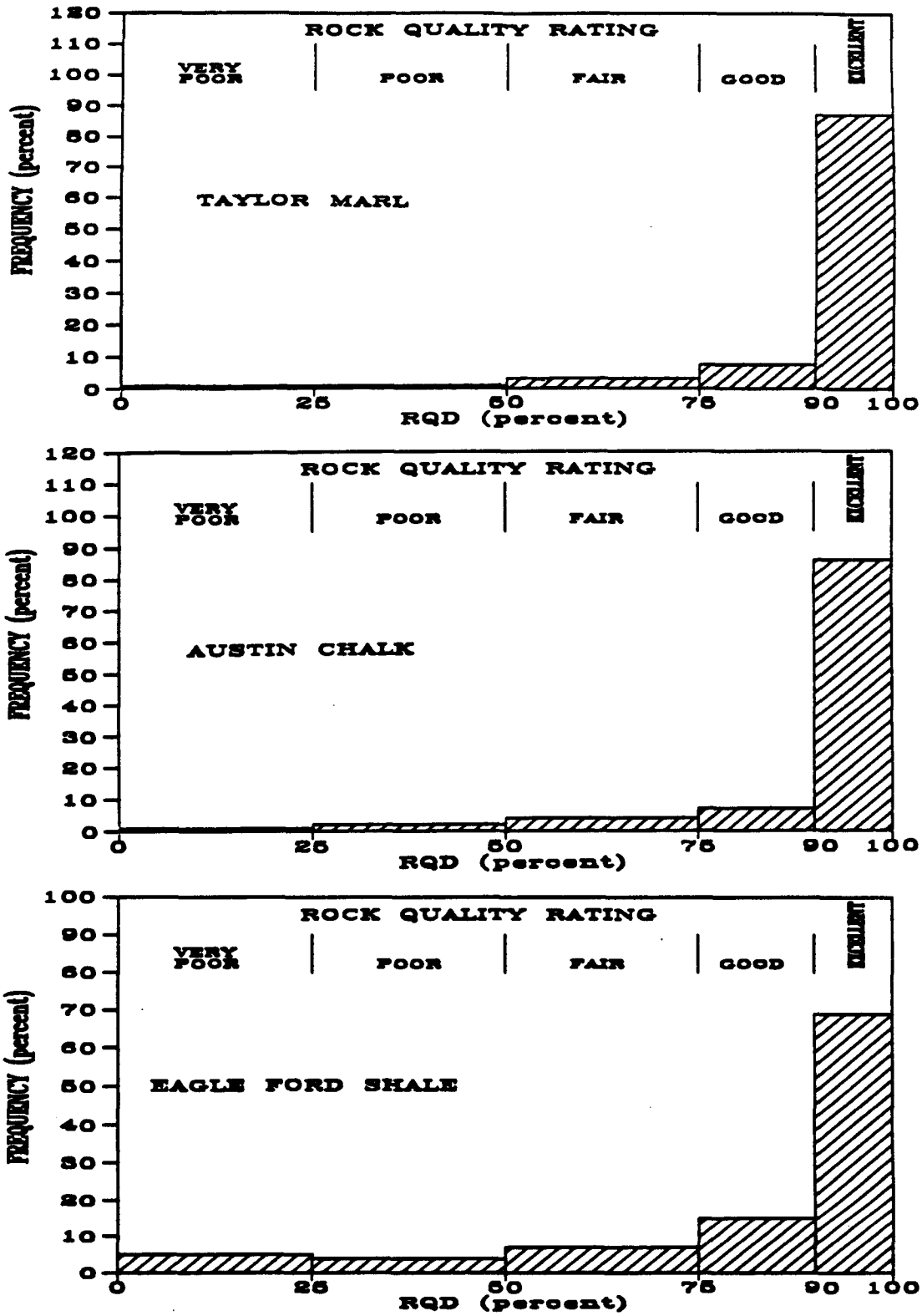
TABLE 3. STATISTICS OF SPACING, NUMBERS, AND CONDITIONS OF JOINTS/FRACTURES

BASED ON 13 INCLINED BORINGS					
Rock Mass	Sample Basis[1] (ft)	Number of Joints	Average Joint Spacing[2] (ft)	Overall Spacing Including Unbroken Bedding (ft)	Approximate Conditions of Joints
Taylor Marl	607	78	7.8	<0.2	60% smooth 40% rough
Austin Chalk	2,774	219	12.7	3.1	40% smooth 60% rough
Eagle Ford Shale	516	78	6.6	<0.2	75% smooth 25% rough

BASED ON SURFICIAL MAPPING AT INTERACTION REGIONS 1 TO 5 AND 8A				
Rock Mass	Bedding or Joint Set	Average Spacing (ft)	Predominant Dip	Approximate Joint Conditions
Taylor Marl	Bedding	0.16	Subhorizontal	Smooth, planar to rough undulating
Taylor Marl	Joint set	2.03	Subvertical 64 to 90°	Smooth, planar to undulating
Austin Chalk	Bedding	0.45	Subhorizontal	Smooth, undulating to rough undulating
Austin Chalk	Joint set	5.1	Subvertical 60 to 90°	Smooth, planar to rough undulating

Notes: (1) Based on a total of 13 inclined borings used in the geomechanical characterization of TM, AC, and EFS documented in reports GR-66, GR-67, and GR-68, respectively.

(2) Including joints, shears and broken bedding planes and excluding closely-spaced bedding planes.



Note: The histograms are based on core data from 74 borings including 61 vertical and 13 inclined borings.

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RQD Histograms

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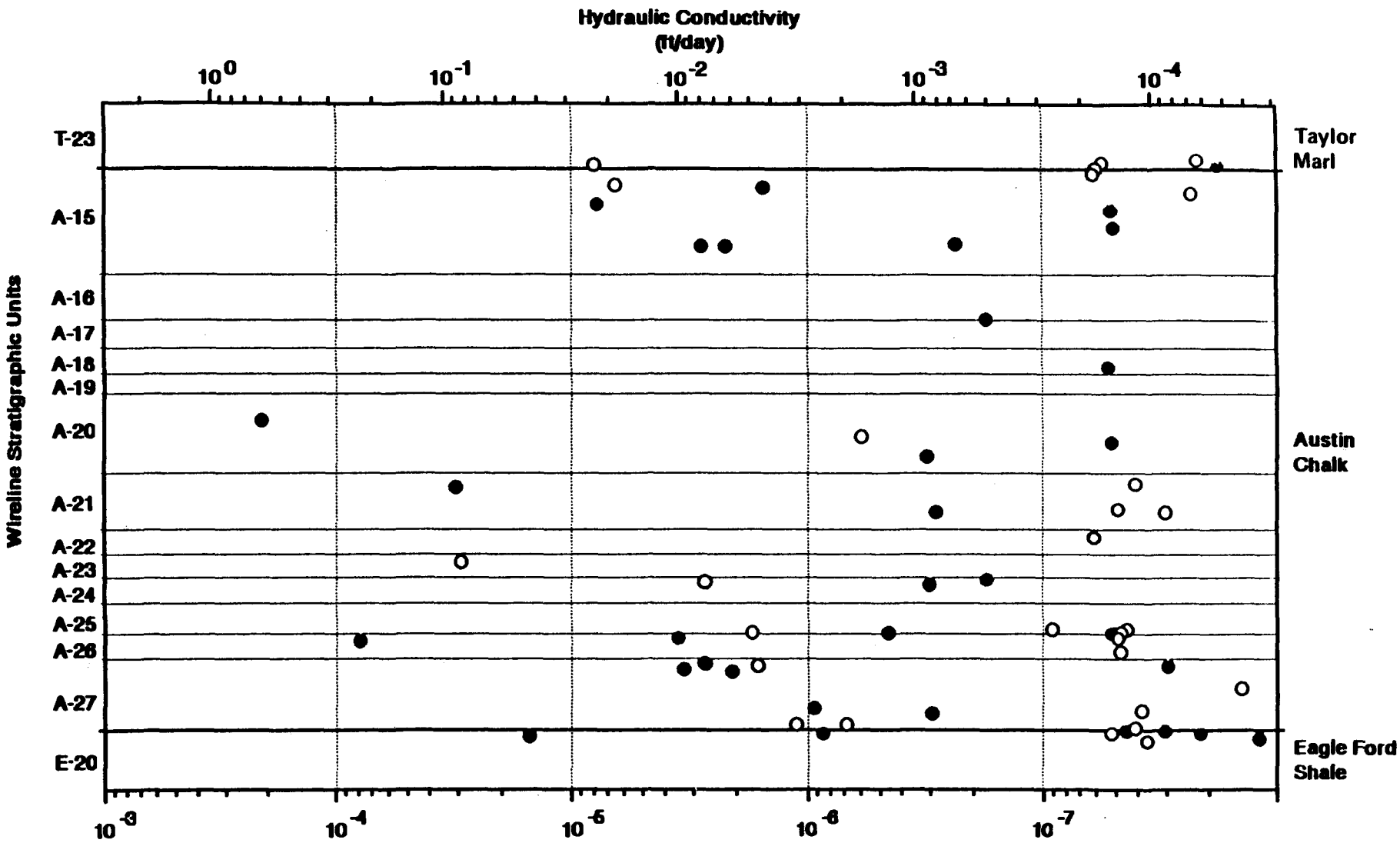
Figure 3



Explanation

○ Unfractured

● Fractured

Note: Only packer test data are depicted.

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Measured Hydraulic Conductivity by Stratigraphic Unit

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Figure 4

- Measured hydraulic conductivity at fault zones ranges from less than 4×10^{-8} to 2.1×10^{-4} cm/sec, with most values in the range of 3×10^{-7} to 4×10^{-6} cm/sec and a median value of about 2×10^{-6} cm/sec. This is one order of magnitude higher than the median value for the in-situ rock mass in general.
- As shown in Figure 4, hydraulic conductivities for unfractured and fractured rock masses are similar in magnitude. However, as shown in Figure 4 and Table 1, some of the fracture zones in the fault zones have significantly higher hydraulic conductivities (up to about 4×10^{-5} cm/sec) than the rock mass. These high hydraulic conductivity fault zones may provide hydrologic connection with the surficial aquifer; as a result, facility design may need to consider high hydrostatic pressure or some method of providing effective drainage to alleviate high pressure.

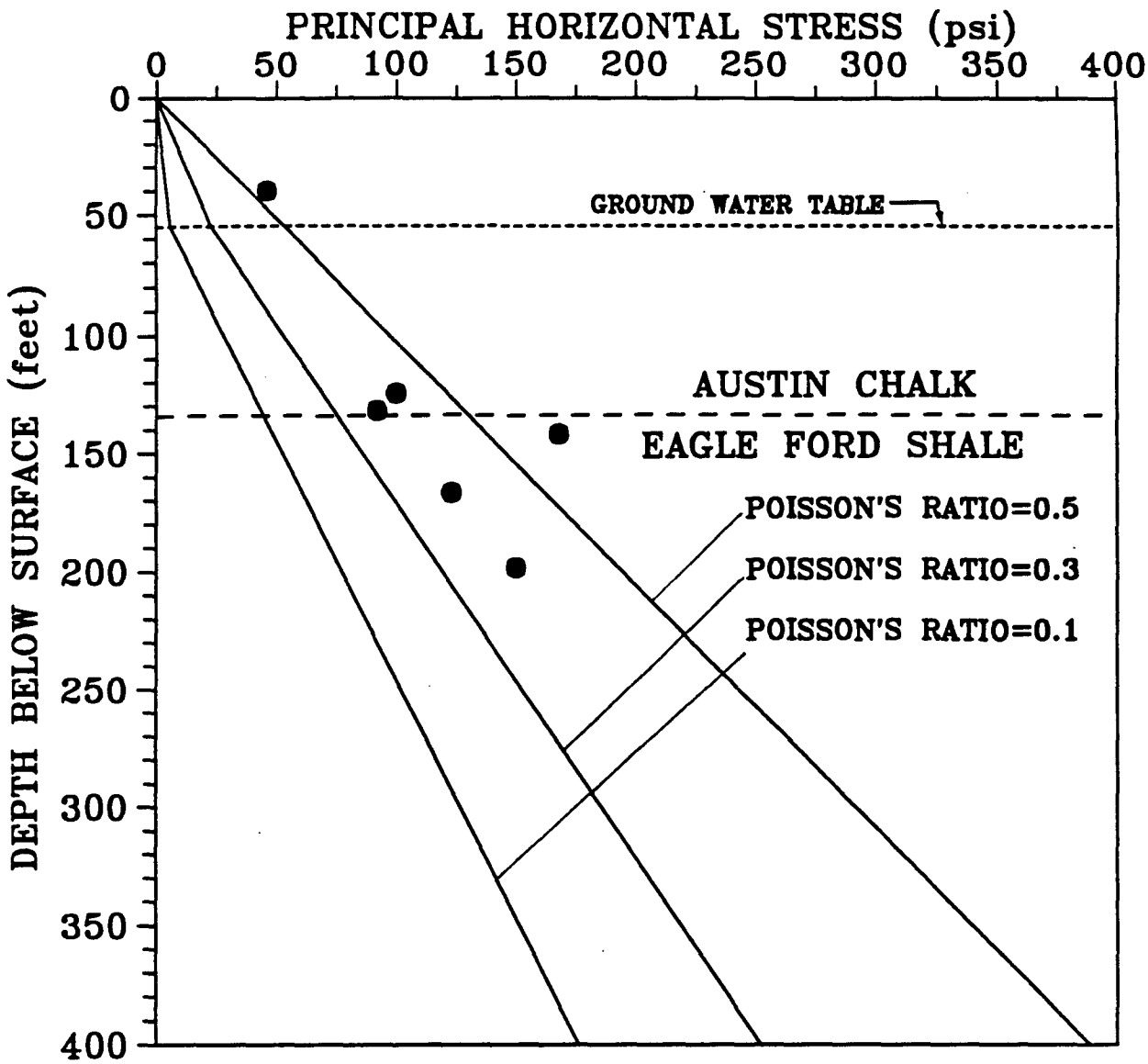
2.1.5 In Situ Stress Conditions

The design of the SSC underground facilities will require knowledge of the in situ state of stress. In areas of flat topography and assuming no tectonic-induced stresses/folding, total and effective vertical stresses at any specific location are equal to the overburden stress and can be closely estimated from the knowledge of densities of subsurface materials and groundwater levels. Two limited field experimental trials were conducted to estimate the horizontal stress components at the SSC site. These included pressuremeter tests in two borings (Boring BE 2 in AC and EFS and Boring BF 6 in TM and AC) and hydraulic fracturing tests in one boring (Boring BE 5 in TM and AC). The pressuremeter and hydraulic fracturing tests were done by STS and RE/SPEC, respectively and the results are documented in Report Nos. SSC-GR-74 and GR-75, respectively.

As shown in Figures 5a and b, the interpreted in-situ total horizontal stresses from the pressuremeter tests are erratic and range from about 50 to 125 percent of the corresponding total overburden stress. The horizontal stress estimates based on interpretation of the pressuremeter test results should be used with caution because there is no standardized method of interpretation available in the engineering literature.

The results of hydraulic fracturing tests are summarized in Figure 6. These results indicate that both horizontal stress components are larger than the corresponding total vertical stress. The maximum horizontal stress is found to be approximately twice as large as the vertical stress and the minimum horizontal stress is approximately 30 percent larger than the vertical stress. The results of hydraulic fracturing tests should be used with caution because interpretations may be affected by a number of uncertainties such as pressure leak off, induced stress concentration, pore pressure, rock tensile strength, and equipment limitation (Report No. SSC-GR-75). In addition, it should be noted that the boring (BE 5) used in hydraulic fracturing is located near a significant topographic relief and intersects a fault (SE5 fault - Table 1) which has an estimated displacement of about 70 feet. Thus, the seemingly high horizontal stress measurements from hydraulic fracturing tests may be partly due to effect of topography and fault and may not be representative of the general in situ conditions in the site areas. Further investigations will be needed to verify this postulation and to resolve the apparent differences in in situ stress-measurements between pressuremeter and hydraulic fracturing tests.

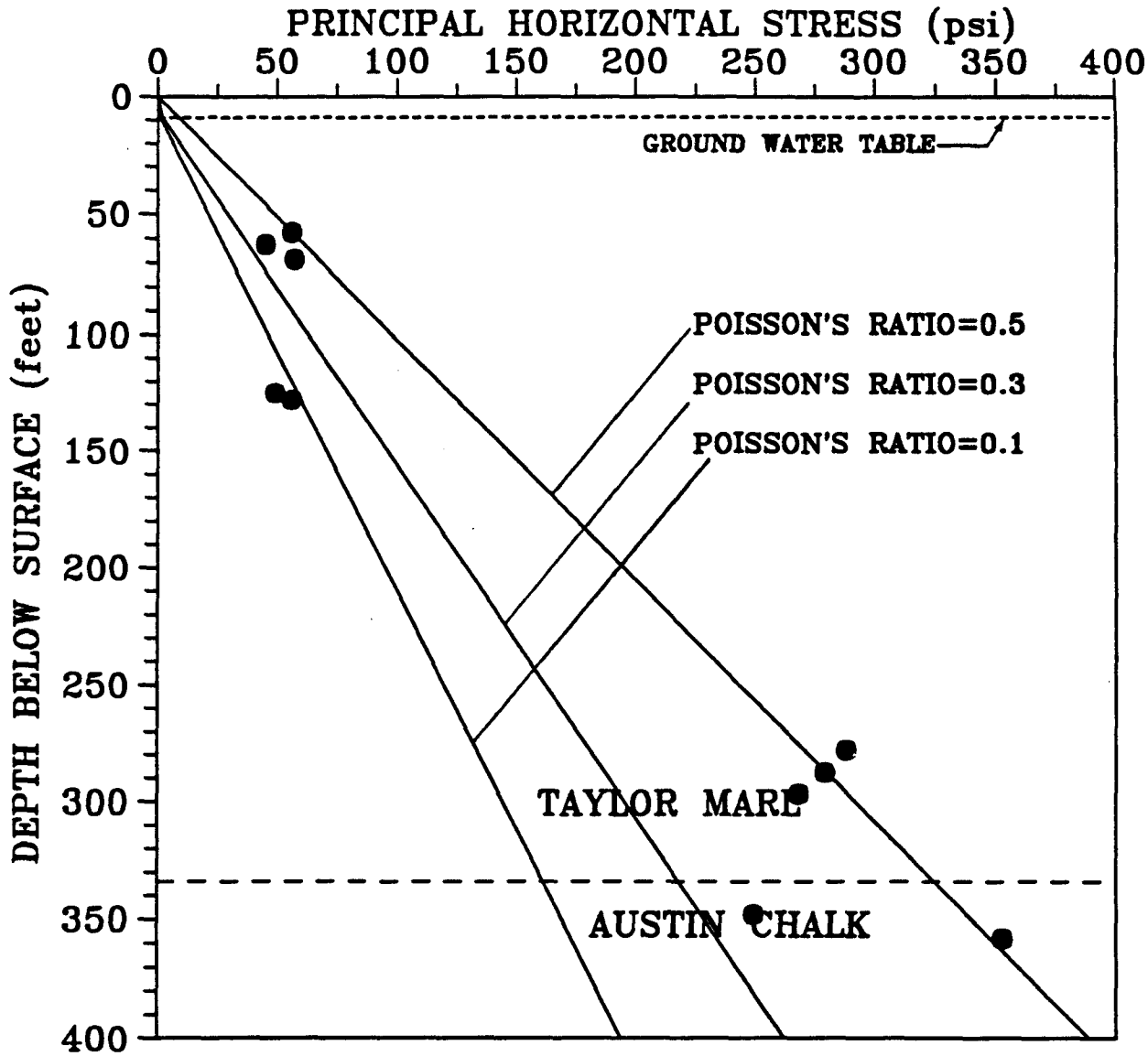
In lieu of further investigation and for site-specific conceptual design purposes, it is postulated that in situ stresses at the SSC site will be generally less than the corresponding vertical overburden stresses based on engineering judgment and available presuremeter test data. Since in situ stress are an important design parameter, further refinement will be necessary. In addition to borehole techniques (i.e., pressuremeter, hydraulic fracturing) other in situ techniques (i.e., overcoring) should also be attempted



●●●●● HORIZONTAL STRESS (TETC INTERPRETATION OF PRESSUREMETER TEST DATA, REPORT GR-74)

—— CALCULATED HORIZONTAL STRESSES (LINEAR-ELASTIC FOR VARIOUS VALUES OF POISSON'S RATIO ASSUMING TOTAL UNIT WEIGHT OF 140 pcf)

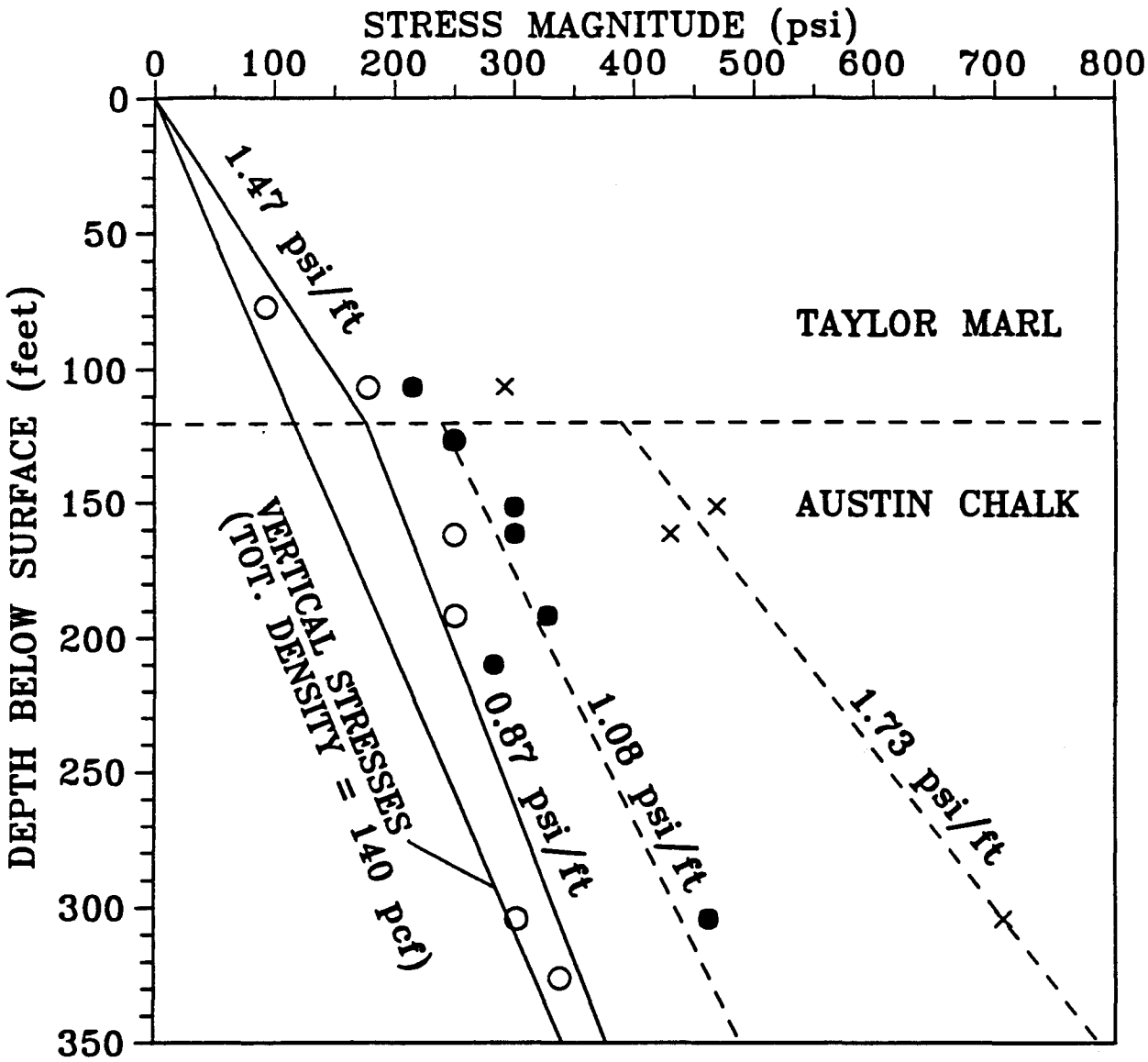
The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Results of Pressuremeter Tests in Boring BE 2	
12/90	Figure 5a



●●●●● HORIZONTAL STRESS (TETC INTERPRETATION OF PRESSUREMETER TEST DATA, REPORT GR-74)

—— CALCULATED HORIZONTAL STRESSES (LINEAR-ELASTIC FOR VARIOUS VALUES OF POISSON'S RATIO ASSUMING TOTAL UNIT WEIGHT OF 140 pcf)

The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Results of Pressuremeter Tests in Boring BF 6	
12/90	Figure 5b



○○○○○ VERTICAL STRESS (HYDRAULIC FRACTURING TESTS)
●●●●● MINIMUM HORIZONTAL STRESS (HYDRAULIC FRACTURING TESTS)
×××××× MAXIMUM HORIZONTAL STRESS (HYDRAULIC FRACTURING TESTS)

The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Results of Hydraulic Fracturing in Boring BE 5	
12/90	Figure 6

in the planned large diameter geotechnical instrumentation shaft and/or during the excavating of the prototype shaft facilities.

2.1.6 In Situ Static Stiffness

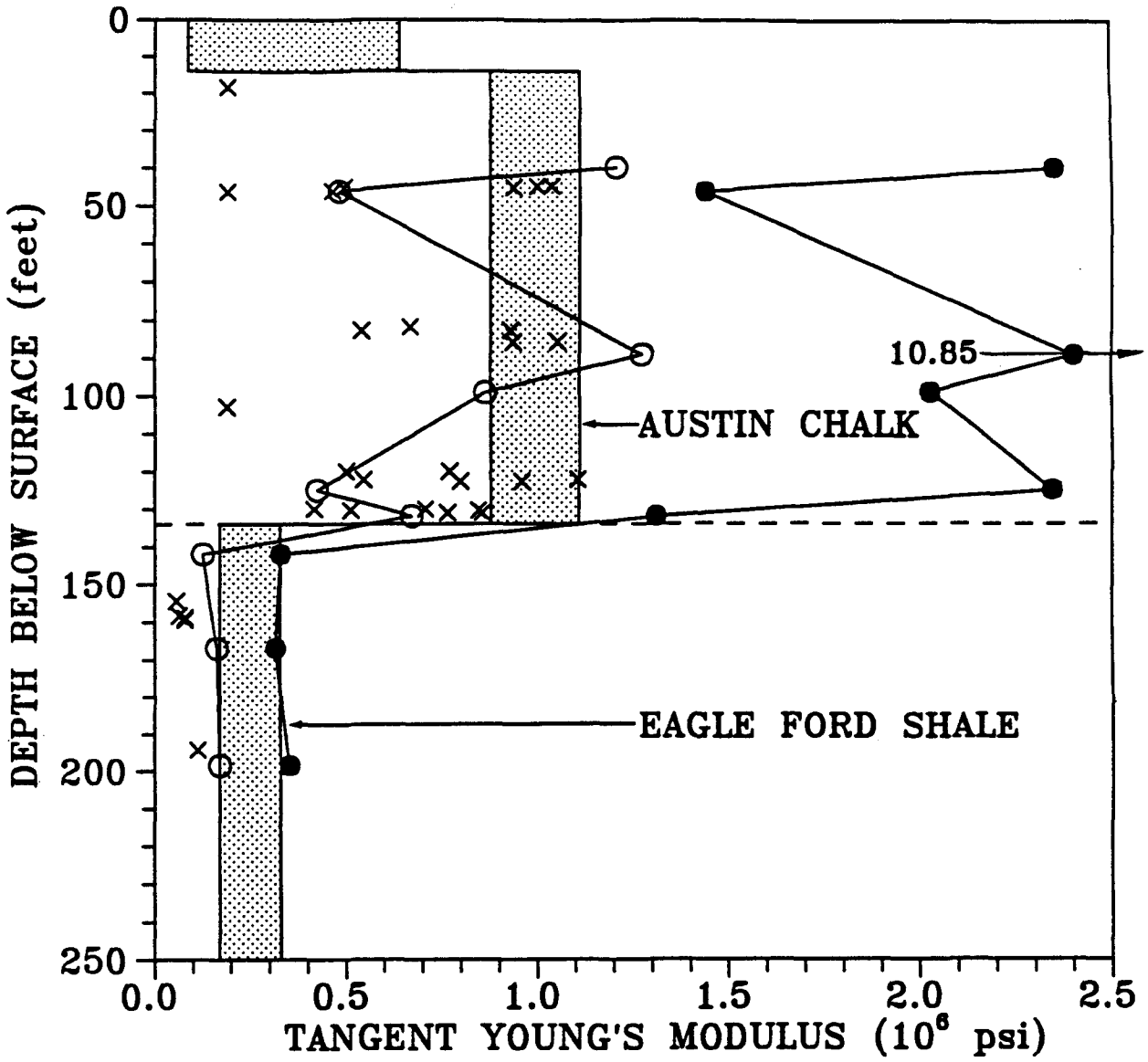
Knowledge of the in-situ stiffness of the rock mass is critical for predicting the performance of various SSC underground facilities during construction and under operational loads. Two limited field test programs were conducted to estimate the stiffness parameters: (1) geophysical surveys, including sonic velocity logging (P-wave), downhole seismic surveys (P- and S-waves), and refraction surveys; and (2) pressuremeter tests. Detailed information regarding sonic velocity logging, downhole seismic surveys, and refraction data are presented in individual corehole data reports. Detailed information on pressuremeter test data is presented in Report No. SSC-GR-74.

The procedures for interpreting stiffness from pressuremeter tests are well established (e.g., ASTM D4719). Figures 7a and b show the profile of interpreted initial and unload-reload moduli for two borings. These figures indicate that the modulus of TM, as determined by the pressuremeter tests, can apparently be divided into a top zone of low stiffness and a bottom zone of higher stiffness with corresponding stiffness values similar to and about three times as much as that of EFS, respectively. Measured modulus values for AC are about six times those for EFS. It should be noted that the data shown in Figures 7a and b are limited; further in-situ testing to determine the deformation modulus of in-situ rock masses is necessary.

2.1.7 In Situ Dynamic Modulus and Wave Velocities

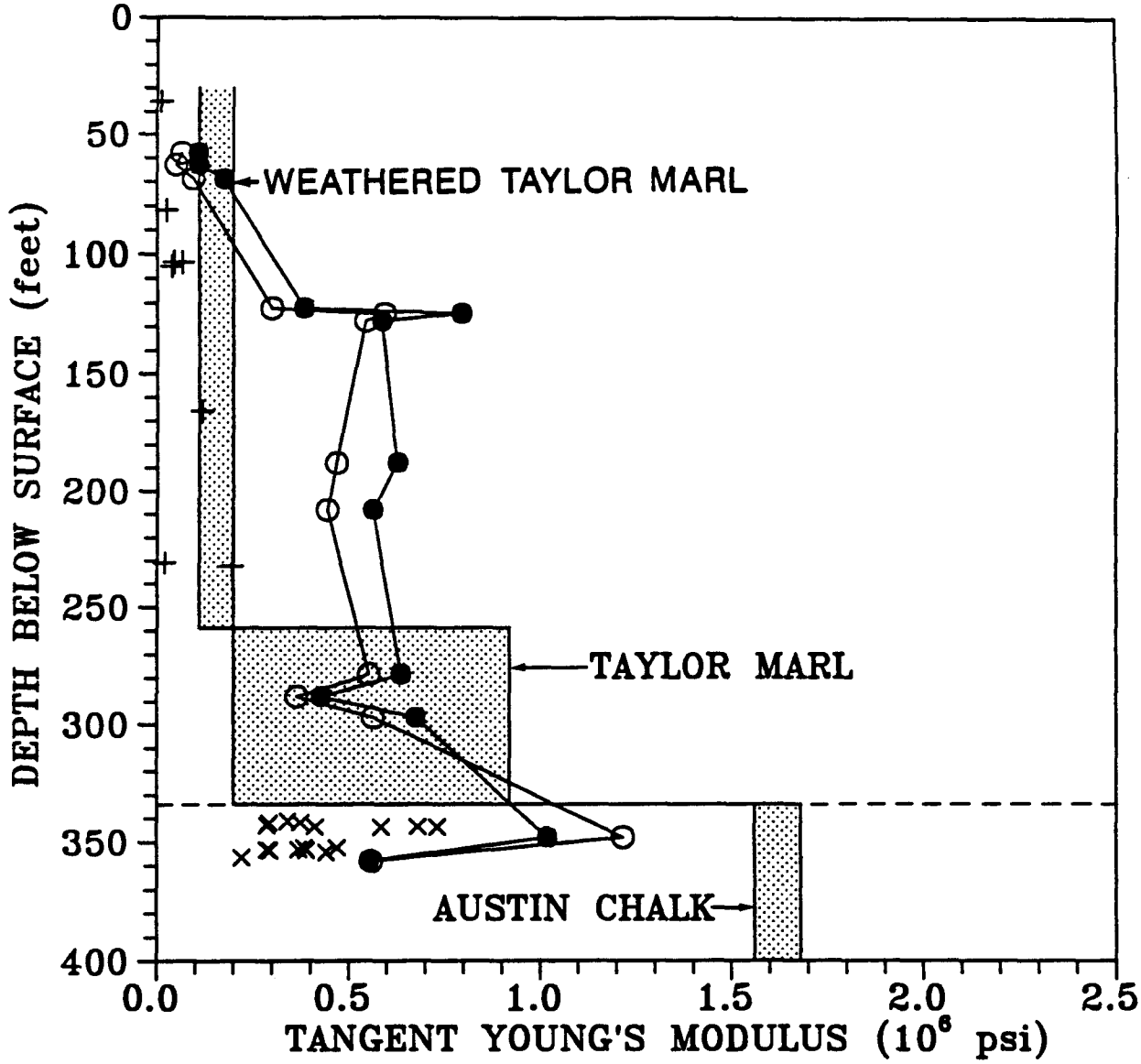
The results of refraction, sonic velocity logging, and downhole seismic surveys are summarized in Table 4. Calculated seismic Young's modulus (Esels) values and Poisson's ratios are also shown in this table. A statistical summary for the three rock units is presented in Table 5. Based on the geophysical investigations, the following observations can be made:

- Among the three in-situ bedrock units in general, EFS has the lowest stiffness and fresh AC has the highest stiffness. TM appears to have intermediate stiffness.
- Seismic wave velocity in weathered AC is significantly lower than that in fresh AC.
- Among the three bedrock formations, TM yielded has the widest scatter for shear modulus and shear wave velocities. The largest values of shear modulus for TM are about two to four times higher than those for EFS, whereas the lower values for TM are even lower than those for EFS. This broad range is very similar to that revealed by the pressuremeter test data for TM (Figures 7 a and b). This scatter may reflect in situ variation and indicate the nonhomogeneous nature of the TM formation. However, the compressional wave velocity in TM is relatively consistent (5,700 to 6,800 ft/sec). The reason for such variation in shear and compressional wave velocities is not clear.
- A comparison of Young's modulus profiles as determined by pressuremeter tests and geophysical data at similar subsurface conditions is shown in Figures 7 a and b. This comparison shows good agreement by the two methods. However, this comparison is based on very limited data and further work will be necessary.



E_0–PRESSUREMETER TEST DATA
E_{U-R}–PRESSUREMETER TEST DATA
E_{50}–LABORATORY TEST DATA
RANGE OF VALUES MEASURED BY GEOPHYSICAL TESTING IN SIMILAR SUBSURFACE PROFILES (BIR 11, BIR 21, BIR 31 & BIR 41)

The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Modulus Profile: Boring BE 2	
12/90	Figure 7a



INITIAL DEFORMATION MODULUS, E_0–PRESSUREMETER TEST DATA
RELOAD DEFORMATION MODULUS, E_{U-R}–PRESSUREMETER TEST DATA
TANGENT YOUNG'S MODULUS (BF 6), E_{50}–LAB. TEST DATA
TANGENT YOUNG'S MODULUS IN A SIMILAR SUBSURFACE PROFILE (BE 7A), E_{50}-LAB. TEST DATA
RANGE OF VALUES MEASURED BY GEOPHYSICAL TESTING IN SIMILAR SUBSURFACE PROFILES (BIR 81 & BIR 54)

The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Modulus Profile: Boring BF 6	
12/90	Figure 7b

TABLE 4. SUMMARY OF SEISMIC VELOCITY AND CALCULATED ELASTIC MODULI

Boring No. or Location	Depth Below Ground Surface (ft)	Geologic Unit (1)	Seismic Wave Velocity (ft/sec)		Calculated Poisson's Ratio	Calculated Modulus (ksi)	
			P-Wave	S-Wave		Young's Modulus	Shear Modulus
BIR 11	0-20	Qa, AC-W, AC	9,200	2,200	0.46	415	144
	20-218	AC	9,200	3,700	0.37	1,112	406
	218-250	EFS	6,100	1,700	0.44	245	85
BIR 21	0-15	Qa, AC-W, AC	9,400	2,700	0.46	640	220
	15-206	AC	9,400	3,400	0.43	993	349
	206-310	EFS	5,900	1,700	0.46	250	86
BIR 31	0-9	Qa and AC-W	2,100	1,100	0.31	96	36
	9-157	AC and AC-W	9,000	3,200	0.43	878	307
	157-280	EFS	6,200	1,970	0.44	328	114
BIR 41	0-23	Qa, AC-W	3,000	1,000	0.44	87	30
	23-184	AC	9,600	3,400	0.43	998	349
	184-220	EFS	7,000	1,400	0.48	171	58
BIR 54	0-19	QT, TM-W	2,100	840	0.41	52	18
	19-184	TM	6,100	1,530	0.47	200	68
	184-410	AC	9,500	4,250	0.38	1,555	570
BIR 81	0-12	QT	1,750	600	0.43	29	10
	12-62	TM-W, TM	6,000	1,150	0.48	110	37
	62-137	TM	6,000	3,600	0.22	922	378
	137-390	AC	9,100	4,450	0.34	1,675	625
BC1	0-7	QT, AC-W, AC	4,300	2,000	0.36	317	116
	7-164	AC	9,000	3,100	0.43	818	285
	164-236	EFS	6,500	1,850	0.46	293	101
B 1637	0-7	AC-W	4,300	2,000	0.36	328	120
	7-167	AC	9,600	3,300	0.43	924	322
	167-240	EFS	5,000	1,450	0.45	182	62
BF 1A	0-14	AC-W	4,100	2,400	0.24	409	165
	14-140	AC	8,800	3,400	0.41	966	342
	140-200	EFS	6,000	1,650	0.46	234	80
VF 1.7	0-13	QT	1,450	500	0.43	20	7
	13-45	AC	3,600	2,050	0.26	308	122
	45-65	AC	5,700	3,300	0.25	791	317
VE 3.5	0-7	QT	1,200	750	0.18	37	16
	7-31	AC-W, AC	4,600	2,900	0.17	573	245
	31-78	AC	8,900	5,100	0.26	1,900	757
VF 5.4	0-23	QT	2,100	500	0.47	21	7
	23-110	TM	5,700	2,500	0.38	519	188
	110-260	TM	6,800	3,700	0.29	1,062	411
VE 9.3A	0-12	QT	1,320	420	0.44	14	5
	12-89	AC-W, AC	4,900	2,300	0.36	418	154
	89-165	AC	8,300	4,000	0.35	1,256	466
Near Lumkins (Refraction Survey) Report GR-62		Dry Qa	1,000-1,500	-	-	-	-
		Dry Qt	2,100-2,500	-	-	-	-
		Saturated Qt or AC-W	5,000-6,100	-	-	-	-
		AC	8,300-9,600	-	-	-	-

Note: (1) QT = Quaternary Deposit (Qa or Qt)
Qa = Quaternary Alluvium
Qt = Quaternary Terrace Deposit
AC-W = Weathered AC
AC = Fresh AC
TM-W = Weathered TM
TM = Fresh TM
EFS = Fresh EFS

TABLE 5. STATISTICAL SUMMARY OF GEOPHYSICAL TEST RESULTS

Geologic Unit	P-Wave Velocity (ft/sec)			S-Wave Velocity (ft/sec)			Young's Modulus (ksi)			Shear Modulus (ksi)			Poisson's Ratio		
	Range	Mean	SD[1]	Range	Mean	SD[1]	Range	Mean	SD(1)	Range	Mean	SD(1)	Range	Mean	SD(1)
Quarternary Deposit and Weathered TM[2]		2100			840			51			18			0.41	
Fresh Taylor Marl	5700 -6800	6120	366	1150 -3700	2496	1041	110 -1062	563	379	37-411	216	154	0.22 -0.48	.37	0.10
Weathered Austin Chalk	2100 -4900	3863	864	1000 -2900	1969	598	87 -573	327	153	30-245	123	65	0.17 -0.44	0.31	0.08
Fresh Austin Chalk	5700 -9600	8807	960	2200 -5100	3536	706	415 -1900	1065	392	143-757	390	159	0.25 -0.46	0.39	0.07
Eagle Ford Shale	5000 -7000	6100	566	1400 -1970	1674	187	171 -328	243	52	58 -114	84	18	0.44 -0.48	0.46	0.01

Notes: (1) SD =Standard Deviation
(2) The data base is too limited to have adequate statistical representation

wp/V159A/TABLE-5

2.2 LABORATORY GEOTECHNICAL CHARACTERIZATION

2.2.1 Test Results from Commercial Laboratories

Laboratory tests on intact cores and samples of discontinuities from the three bedrock units were done principally by MJA and SWL. The test program included tests to determine index properties as well as strength and deformation characteristics. Index properties included moisture content, bulk and dry densities, specific gravity, Atterberg limits, carbonate content, swell pressure index, slake durability, Tabor hardness, tensile strength (Brazilian), and bulk and clay mineralogy. Laboratory tests to determine strength and deformation characteristics included uniaxial compression (UC), undrained triaxial compression (UU), and direct shear.

The laboratory test results from several phases of drilling (excluding those from the Prototype Installation Facilities investigation) were used for an initial characterization of geomechanical properties of the three bedrock units. Details of the laboratory test program and individual test results for EFS, AC, and TM are documented in Report Nos. SSC-GR-66, GR-67, and GR-68, respectively.

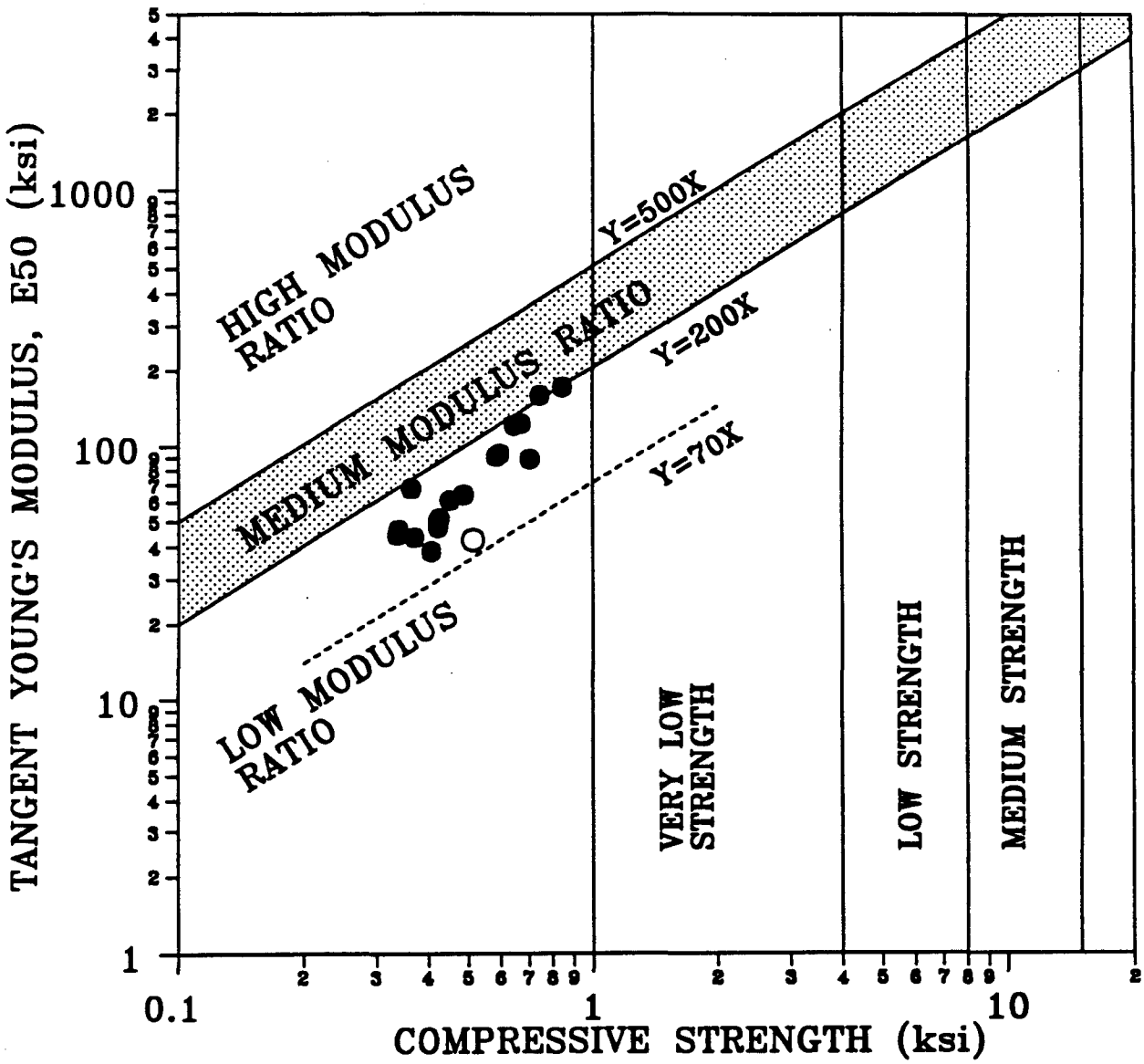
All of the laboratory test results in terms of range, mean, and standard deviation are summarized in the Appendix. Data on compressive strength and tangent moduli for TM, AC, and EFS are summarized in Figures 8a through c, respectively. Also shown in these figures is a rock classification based on the ratios of strength modulus (modulus/strength) as introduced by Deere and Miller (1966). A review of these figures indicates the following:

- As shown in Figures 8a and 8c, all the data for TM and EFS fall below the zone corresponding to very low strength rocks with mean values of 340 and 360 psi, respectively. TM appears to be only slightly stronger than EFS.
- Figure 8b shows that most of the data for AC fall in the very low strength zone with a few points below this zone. The average value of compressive strength for AC is about 2,500 psi.
- The modulus data for EFS plot in a zone of low- to medium-modulus ratio, whereas data for TM fall predominantly in the low-modulus ratio zone. This is contrast to in situ tests indicating a stiffness for TM about 2 to 3 times higher than that for EFS (see Figure 7 and Table 5). The data for AC fall primarily in the region of low to medium modulus ratio.

Based on the above observations, the three rock groups can be classified as very low strength rocks. However, as shown in Figure 9, the strength and modulus values for EFS and TM are higher than those for overconsolidated clays and lower than those for very low strength, argillaceous, sedimentary rocks (Deere and Miller, 1966). Thus, EFS and TM can be defined as either "soil-like" rock or "rock-like" soil. These materials are difficult to characterize.

In addition, based on test results documented in Report Nos. SSC-GR-66 to GR-68, the following observations and conclusions can be made:

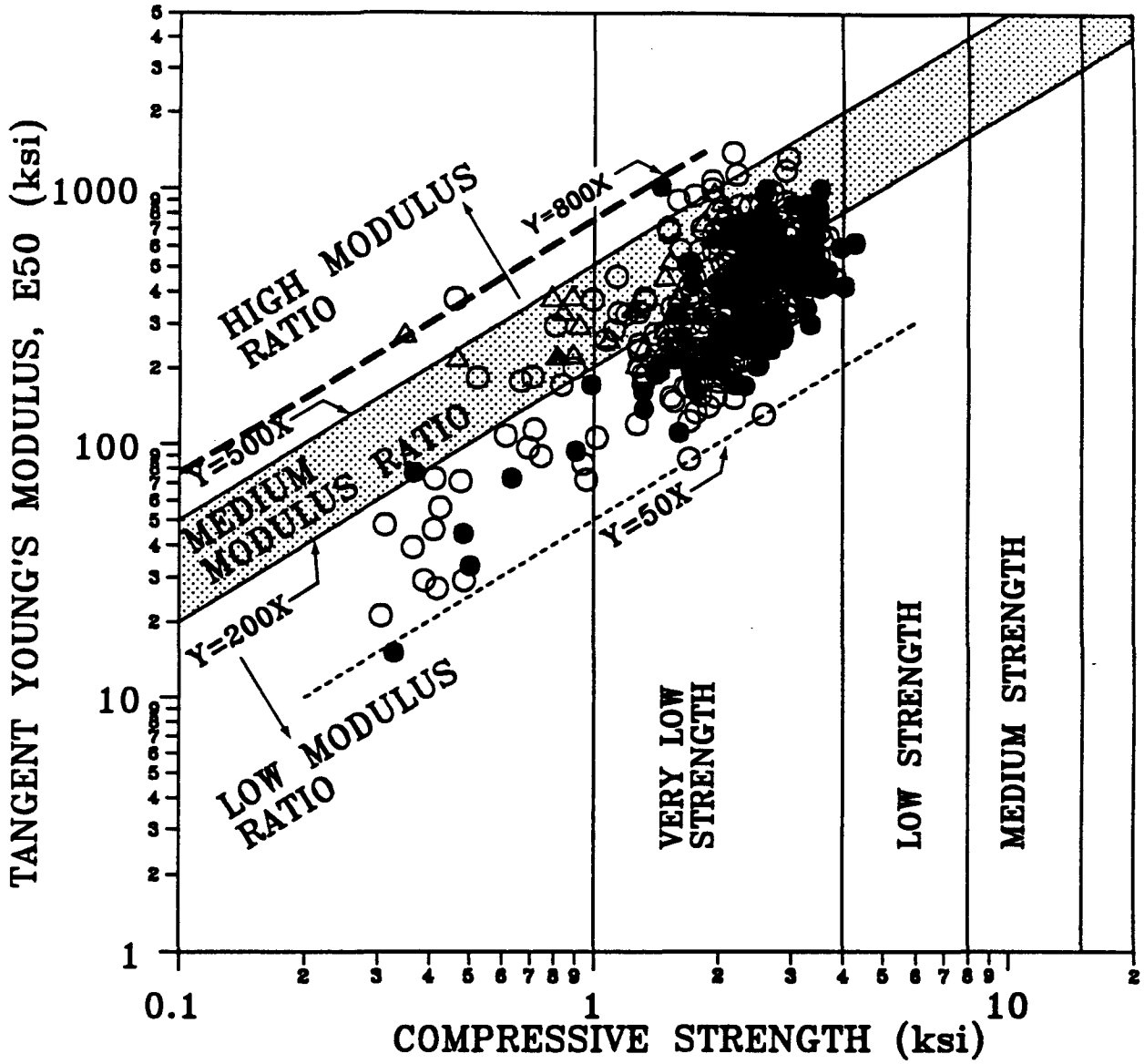
- Strength and deformation characteristics determined in the laboratory test program, show significant data scatter. In addition to possibly reflecting the in-situ variation of the rock masses, it is postulated that this wide scatter may also be caused by sample disturbance and/or potential sample size effects. The very low strength of the three rock formations, especially TM and EFS, indicates that they are easily susceptible to mechanical and physical disturbance during coring and subsequent sample handling. Also, for fissured and fissile materials such as TM and EFS, measured strength and



UC VERTICAL POST-FOOTPRINT BORINGS
UU VERTICAL POST-FOOTPRINT BORINGS

Note: Rock classification system is after Deere and Miller, 1966.

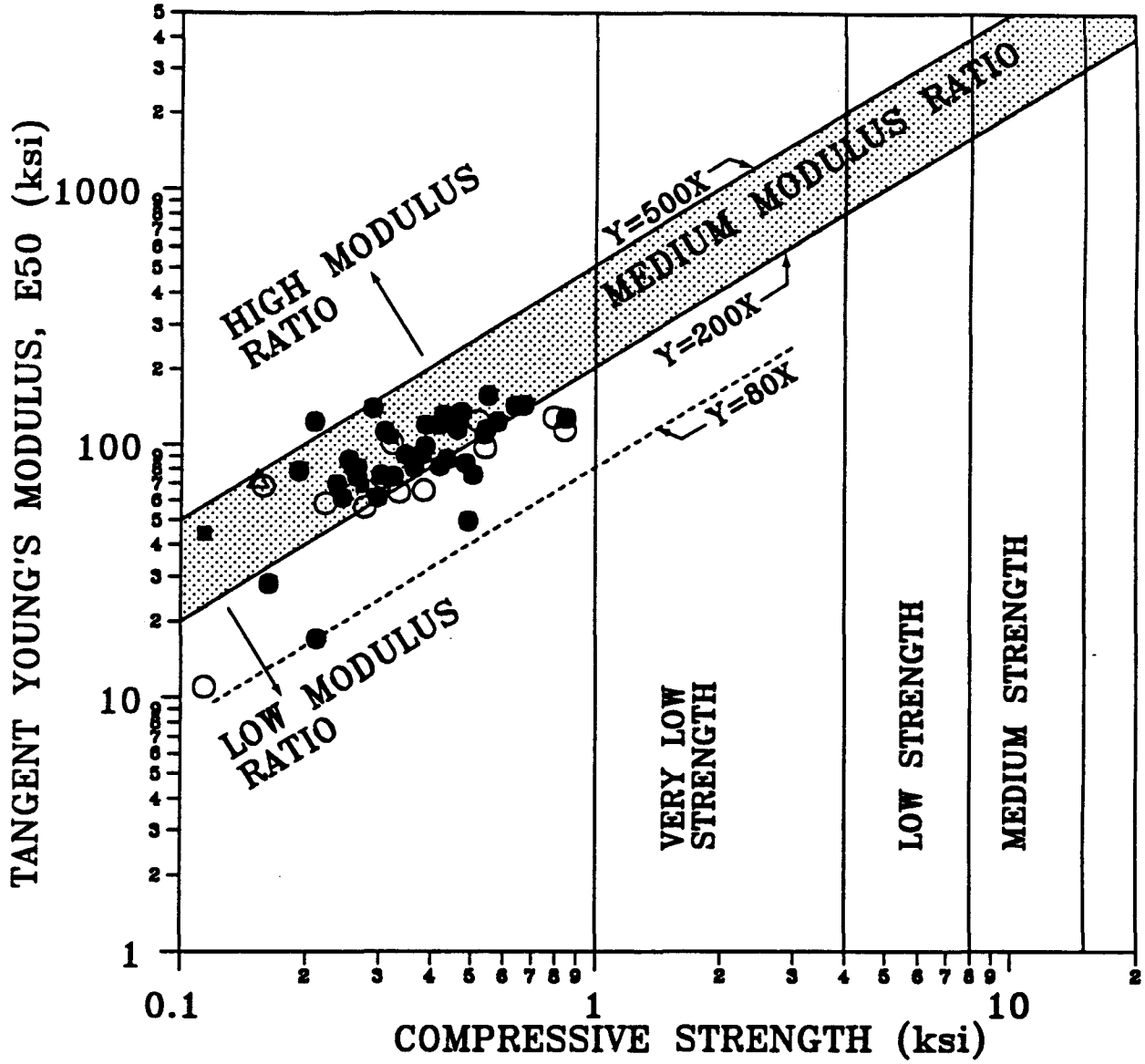
The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Rock Classification System -Taylor Marl	
12/90	Figure 8a



UC VERTICAL POST-FOOTPRINT BORINGS
UU VERTICAL POST-FOOTPRINT BORINGS
UC INCLINED POST-FOOTPRINT BORINGS
UU INCLINED POST-FOOTPRINT BORINGS

Note: Rock classification system is after Deere and Miller, 1966.

The Earth Technology Corporation	Project No.: 87-888-0017 RTK Joint Venture
Rock Classification System - Austin Chalk	
12/90	Figure 8b



UC VERTICAL POST-FOOTPRINT BORINGS
UU VERTICAL POST-FOOTPRINT BORINGS
UC INCLINED POST-FOOTPRINT BORINGS
UU VERTICAL FOOTPRINT BORINGS

Note: Rock classification system is after Deere and Miller, 1966.

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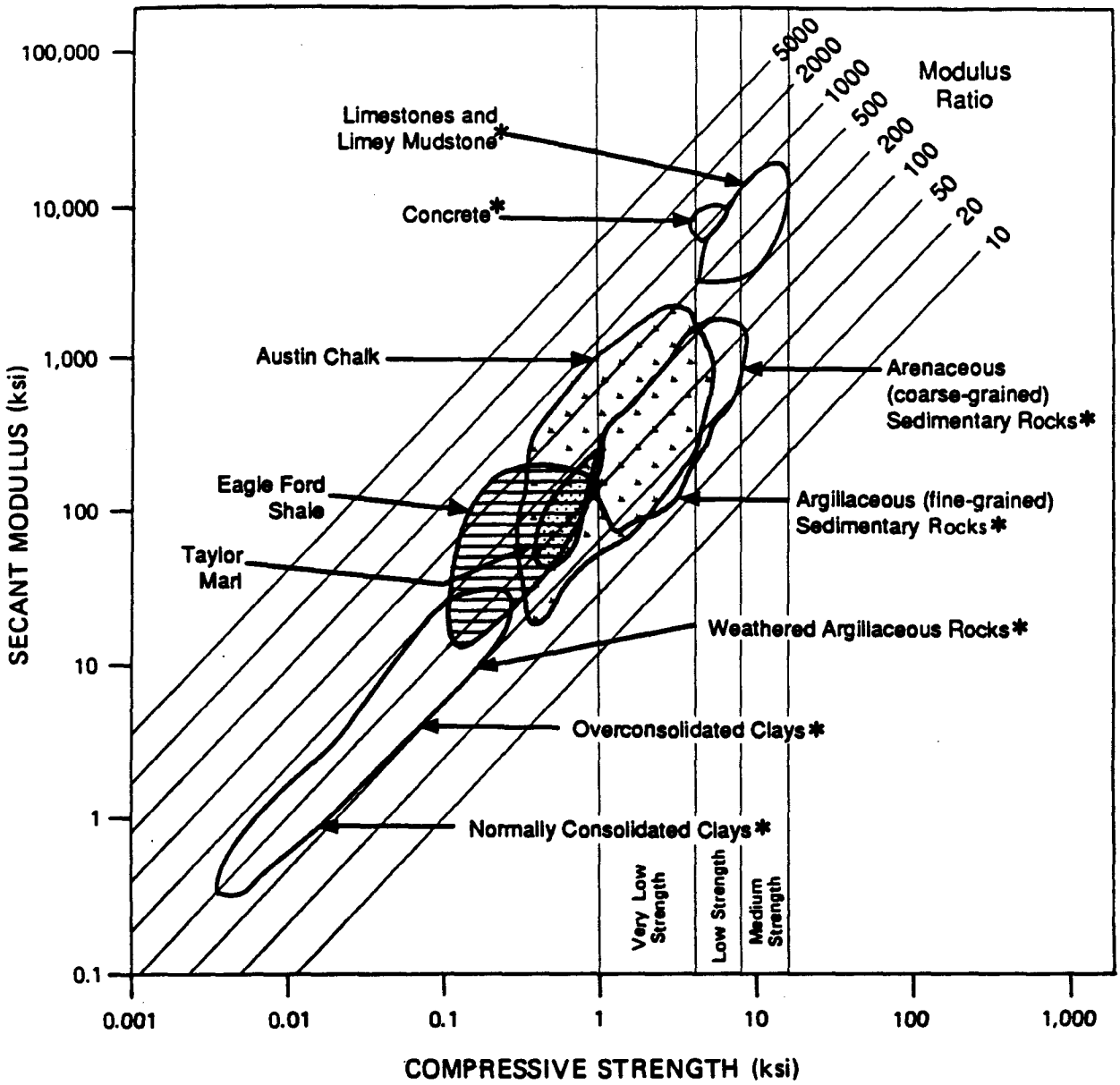
Rock Classification System - Eagle Ford Shale

12/90

Figure 8c



*LITERATURE DATA AFTER DEERE AND MILLER, 1966 AND HOBBS, 1974.

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Rock Classification According to Uniaxial Compressive Strength and Secant Modulus

12/90

Figure 9

deformation values of small specimens tend to have a wider scatter because some specimens may have few or no fissures but others may have major fissures aligned with potential failure planes.

- Despite significant data scatter, index, strength, and deformation properties of intact cores show general trends of variation with respect to depth, density, and moisture content. In general, density, strength, and stiffness increase and moisture content decreases with depth below ground surface. The materials within each rock type generally become stronger, stiffer, and less susceptible to swell or slake/alteration as depth increases.
- AC and EFS are at or close to complete saturation (100 percent saturation) and the degree of saturation in TM is generally greater than 90 percent.
- Uniaxial and triaxial compression tests on intact cores from inclined borings in EFS and TM indicate failure along pre-existing shear planes or bedding in most of the cases. Hence, the bedding or pre-existing fracture or shear is potentially the weakest orientation in EFS and TM.
- The average second cycle slake durability indices for TM, AC, and EFS are about 53, 96, and 21 percent, respectively. This indicates that EFS and TM are very susceptible to slaking/alteration and require immediate protection as excavation progresses during construction.
- Index properties (such as Atterberg limits, grain size and mineralogic composition) indicate that the AC (primarily chalk and argillaceous chalk) except bentonite marker (MB) and thin bentonite shale beds within the AC should exhibit minimal swell potential and negligible creep deformation. They also indicate that EFS and TM, as well as MB and bentonitic shale within AC, should have moderately high swell potential. The swell pressure index test data show that the degree of swell potential may vary depending on the material, as shown below.

Rock Type	Swell Pressure Index, psi		Number of Tests
	Range	Mean	
TM	6.5 to 57.9	19.9	49
EFS	7.7 to 115	35.8	149
MB	4.8 to 57.6	17.8	10

- The swell potential for EFS appears to be much higher than that for TM and MB. It is postulated that TM may have a lower swell potential than EFS for the following reasons:
  - TM has a higher carbonate content (about three times as much as EFS), suggesting higher cementation, which may inhibit swelling.
  - As has been observed in the field and laboratory, TM may be more easily disturbed than EFS during sample handling and testing. This disturbance would induce additional fissures and cracks, which increases initial void ratio and inhibits full swell pressure from developing.
  - TM contains less swell-susceptible clay minerals (montmorillonite and illite/smectite).

2.2.2 Test Results from University of Texas at Austin

Under the direction and supervision of Professor Roy E. Olson of University of Texas (UT), a series of laboratory tests were performed on core samples of TM, EFS, and MB within the AC formation. These tests included consolidation, swell pressure, hydraulic conductivity, "drained" direct shear, "drained" triaxial compression, and index tests consisting of grain size analyses, specific gravity, moisture, density, and Atterberg limits. It should be noted that most tests were performed on EFS samples while no consolidation or strength tests were performed on TM or MB. These tests were performed primarily as an attempt to develop test procedures for these "soil-like" rocks. The results are documented in Report No. SSC-GR-79. Statistics of the test results are also included in the appendix to this report. Relevant conclusions and observations are as follows:

- Index properties are generally within the ranges of test results obtained by MJA and SWL, except for specific gravity values, which are significantly higher in UT data. UT attributed these results to the pyrite content in EFS, TM, and bentonite marker.
- Consolidation tests indicate that the compression and recompression indices correlate with liquid limits and vertical stress. Measured recompression index (all tests were still in recompression range under a maximum consolidation stress of up to 178,000 psf) values for EFS range from 0.001 to 0.123, with most of the values within the range of 0.02 to 0.09.
- The coefficients of consolidation for EFS are low and often are less than 0.01 ft^2/day. Measured hydraulic conductivity (in the vertical direction) for EFS ranges from about 10^{-9} to 10^{-10} cm/sec, which is one to four orders of magnitude smaller than those measured from the field tests (3.1×10^{-5} to 1.2×10^{-8} cm/sec). Because of the low hydraulic conductivity, it was not certain whether the samples used in the UT consolidation and strength tests were saturated. The consolidation data, although serving as an approximate indication for consolidation settlement estimate, may not be confidently used for interpretation of other time-dependent behavior (such as creep behavior).
- "Drained" direct shear tests yield a broad range of results. High shear strength values may be, at least partially caused by shearing across beddings.
- Swell pressures were indirectly estimated by UT based on the consolidation test data (void-ratio-settlement curve) and by adjusting the assumed initial void ratio to account for sampling disturbance (i.e., compensating for opening of fissures or microfissures). These estimates are significantly higher than those obtained by MJA and SWL (Section 2.2.1). This high swell pressure (Table 6) is probably caused by over-adjustment of the initial void ratio to account for sample disturbance effects and may not be representative of the TM and EFS behavior, as evidenced by a significant amount of data showing calculated swell pressures significantly higher than the corresponding effective overburden stresses.
- Significant scatter is observed in the "drained" triaxial test data primarily as a result of apparently random distribution of surfaces of weakness in samples. Based on four best tests (out of a total of ten), UT found that peak drained shear strength of EFS has an effective friction angle of 29 degrees and a cohesion intercept of 29 psi.

Although some uncertainties exist in the data, the tests performed by UT represent an initial attempt to understand the consolidation/swell behavior as well as stress/deformation behavior of TM, EFS, and MB under potentially drained conditions. Further laboratory refinement in terms of test procedures as well as large-scale field testing will be necessary to further understand the long-term engineering behavior of TM and EFS.

TABLE 6. COMPARISON OF MEASURED SWELL PRESSURES BY VARIOUS LABORATORIES

	COMMERCIAL LABORATORIES (MJA and SWL)			UNIVERSITY OF TEXAS [1]			UNIVERSITY OF ILLINOIS		
	No. of Tests	Swell Pressure (psi)		No. of Tests	Swell Pressure (psi)		No. of Tests	Swell Pressure (psi)	
		Range	Mean		Range	Mean		Range	Mean
Taylor Marl	49	6.5 to 57.9	19.9	3	21 to 118	71.8	11	12.5 to 56	36.2
Austin Chalk	--	--	--	--	--	--	--	--	--
Bentonite Marker within Austin Chalk	10	4.8 to 57.6	17.8	1	--	76.4	--	--	--
Eagle Ford Shale	149	7.7 to 115.0	35.8	7	62.5 to 243.1	129	18	11 to 111.5[2] (11 to 278)	41[2] (59)

Notes: (1) Swell pressure was calculated by adjusting assumed void ratio to account for closure of fissures.

(2) All the data ranged between 11 and 111.5 psi except for two anomalously high test results at 156 and 278 psi; range and mean values including all data are given in parentheses.

wp/V158A/TABLE-6

2.2.3 Laboratory Results from University of Illinois

Another suite of laboratory tests on intact cores and discontinuities was performed at the University of Illinois (UI) under the supervision of Professors G. Mesri (Report No. SSC-GR-80) and A. Nieto (1990), respectively. The laboratory test data on intact cores of TM, EFS, and MB include bulk and clay mineralogy, natural water content, total and dry densities, Atterberg limits, undrained shear strength and modulus, swell pressure, recompression index, swelling index, and humidity controlled deterioration tests. Laboratory test data on discontinuities within TM, AC, and EFS include direct shear tests to evaluate peak and residual shear strengths of discontinuities. Statistics of these test results are also included in the appendix. Upon examination of the test data, the following conclusions and observations can be made:

- Similar to the strength test results by MJA, SWL, and UT, the results from UI also exhibit significant scatter.
- Undrained shear strength and modulus values for both EFS and TM are of the same order of magnitude as those determined by MJA and SWL.
- Recompression index values as determined by UI are similar to those determined by UT.
- The swell pressure indices determined by UI range from 11 to 165 psi (except two values at 156 and 278 psi) for EFS, and from 12 to 56 psi for TM (Table 6). These widely scattered swell pressure values have no apparent correlation with moisture content, density, or depth. Close examination of the data and corresponding samples indicates that the higher swell pressures can be associated with samples that have few or no fissures, whereas the low values are generally from samples with visible fissures. It is not clear whether the fissures were caused by sampling disturbances or are natural. Hence, it is difficult to draw any conclusions regarding in-situ swell pressures based solely on the above data.
- Deterioration test results reveal that both EFS and TM are susceptible to alteration when exposed to air and water. They are expected to deteriorate when they are subject to an environment with relative humidity less than 95 percent, and they are likely to soften if exposed to environments with relative humidity larger than 95 percent. Similarly, marker bentonite samples develop partings parallel and normal to bedding as soon as they are subjected to drying.

3.0 PRELIMINARY ASSESSMENT OF GEOTECHNICAL PARAMETERS

As described in Section 2, a large geotechnical data base for the SSC site has been generated through a number of field and laboratory investigations. Although probably sufficient for site-specific conceptual design purposes and significantly contributing to the understanding of site conditions and geomechanical behavior of the three bedrock units, this data base is not yet sufficient to provide adequately definitive geotechnical parameters to confidently support final design and construction. This deficiency is the result of a variety of reasons, notably the following:

1. The laboratory test data, especially strength and deformation modulus data, exhibit large scatter. As postulated previously, some of the scatter may reflect in-situ variation, but some is likely the result of sample size effects as well as sample disturbance effects, especially in TM and EFS. Size effects and sample disturbance effects can be minimized only if laboratory tests on large size samples and large-scale in-situ tests/measurements are done.

2. Up-to-date locations of borings, sampling intervals and laboratory testing programs were selected for global characterization purposes without placing emphasis on any specific aspects which may have significant effects on site-specific final design of the planned facilities. Thus, some relevant data have yet to be determined. For example, most modulus and strength tests were done under simple stress conditions (uniaxial compression, undrained triaxial, and isotropically consolidated drained triaxial test conditions) to determine strength and linear elastic modulus parameters. The fissile and fissured nature (bedding planes, joints/fractures, shears, etc.), the relatively high slaking and swelling potential, and the low strength of the three bedrock units (especially TM and EFS) may indicate that tests under other stress paths and time-dependent environments may be necessary to model anisotropic, nonlinear, and time-dependent behavior of the bedrock.

3. The data base contains some inconsistent geotechnical property information. For example:
   - Specific gravity values measured by UT for TM, EFS, and MB are significantly higher than those measured by MJA and SWL. Differences in assessment of such a fundamental property will result in significant differences in assumptions about degree of saturation and initial void ratio which, in turn, affect the interpretations of consolidation, swell pressure, and drained strength tests.
   - TM contains carbonates -- on the average about three times as much carbonate as the EFS, while containing less clay minerals that may promote compressibility and deterioration, than those of EFS. It would be expected that TM would be stronger and less susceptible to slake/alteration or disturbance than EFS. However, laboratory tests indicate that TM and EFS have similar strength and modulus characteristics. Field and laboratory observations also indicate that TM is at least as susceptible to disturbance as EFS, if not more so. The reasons for such inconsistent behavior are not known.
   - Significant disagreement is observed between in-situ stresses predicted by pressuremeter and hydraulic fracturing tests.
   - Significant differences are observed among the swell pressures predicted by MJA/SWL, and UT and UI (Table 6).
   - Low swell pressure for the MB is inconsistent with its high montmorillinite content.

4. There is a general lack of in-situ tests/measurements to calibrate/verify the geotechnical parameters determined in the laboratories.

5. Most of the available engineering classification systems, including correlations between laboratory and field testing on rock masses, and correlation between field performance (such as standup, rock load, primary support needs, etc.) and rock types have been developed based on case histories of much stronger rocks with less slake/swell potential than the three rock units at the SSC site. These systems and associated practice-oriented correlations for use at the SSC site cannot be confidentally applied unless they are confirmed by site-specific (and material-specific) field testing and monitoring, and field construction experience (such as PIF), which are not available at the present time.

In addition, it should again be noted that (1) specifics of the planned SSC facilities are either tentative or lacking at the present time and (2) selection of geotechnical parameters for facility design is typically interactive. In other words, geotechnical parameters from sophisticated types of tests (in terms of types, anisotropy, nonlinearity, time dependency, etc.) may be needed as dictated by feedback from the design process.

In the remaining part of this section, geotechnical parameters are provided in terms of recommended range and best estimated values. Associated rationales and/or assumptions are also described. As in any civil engineering project, recommended geotechnical parameters depend on the nature of the facilities as well as specific problems or concerns. Thus, in some cases, lower bound values may be appropriate but, for other cases, upper bound or average values may be more reasonable. It should be emphasized that the developed geotechnical parameters are preliminary in nature and are subject to revision/refinement as more data become available (especially in situ measurements and field testing data) and as design of the SSC facilities progresses.

Geotechnical parameters for site-specific conceptual design (SSCD) are presented in terms of rock mass rating, density, moisture content, slake durability index, swell pressure, consolidation parameters, hydraulic conductivity, deformation, and strength properties of intact rocks and rock masses. Other properties such as Atterberg limits, percent clay, activity ratios, mineralogical content, carbonate content and Tabor abrasion index are presented in Report Nos. SSC-GR-66, GR-67, and GR-68; they are also summarized in the appendix. These properties, in general, are not directly used in SSCD analysis and are not affected by the effects of sample disturbances or the presence of discontinuities. Should these properties be needed for SSCD or other purposes, statistical range and mean values summarized in the appendix and in Report Nos. SSC-GR-66, GR-67, and GR-68 are considered appropriate and representative.

3.1 ROCK QUALITY

3.1.1 Intact Rock

Based on the rock classification system of Deere and Miller (1966), the intact rocks of the three major bedrock units can be classified as rock with very low strength and low modulus ratio (TM and EFS) and very low strength and medium modulus ratio (AC), as shown in Figures 8a-c. Among the three major rock units, AC has the highest strength and stiffness, while the laboratory test data indicate that TM and EFS have compressive strength and stiffness that are similar to one another but lower than AC. As described previously, the strengths of core samples of TM and EFS are higher than those of overconsolidated clays, but are unusually low when compared with those of most argillaceous sedimentary rocks (Figure 9). Thus, TM and EFS may be described either as "soil-like" rock or "rock-like" soil. They are anticipated to be neither completely similar to soils nor completely similar to rocks. Further site-specific testing and construction experience will be needed to refine the geotechnical characterization of such materials.

3.1.2 Rock Mass

Histograms of RQD values, shown in Figure 3, indicate that the rock masses of all three rock units can be classified as having RQD mostly between 90 and 100 percent. Although RQD can provide a good guide to the condition of the rock mass immediately after excavation, it has very limited application in high-swell and slake/alteration prone rocks because it does not take into account a number of parameters and factors that are important in the design and construction of underground facilities. These include rock strength, joint/fracture characteristics, groundwater or flow rates, swell potential, slake durability, etc.

Various rock mass classification systems exist in the literature for underground excavation. These include, but are not limited to, the geomechanics classification system (Bieniawski, 1973, 1974, 1975, and 1979), rock structure rating (RSR - Wickham and Tiedemann, 1972 and 1974), and rock mass quality (Q-System-Barton et al., 1974). The RSR and Q classification systems are more appropriate for medium strength and hard and highly jointed rocks, respectively, and are less suitable for the very low strength rocks at the SSC site. Bieniawski's geomechanics classification is used here. Descriptions of ratings and classifications of this system, and their associated implications with respect to strength parameters, are shown in Table 7 (Bieniawski, 1979).

The Bieniawski geomechanics classification uses the following factors for ratings:

- Uniaxial compressive strength or point-load strength of intact rock
- RQD
- Spacing of joints
- Conditions of joints
- Groundwater in terms of inflow or joint pore water pressure or general groundwater conditions
- Strike and dip of joints with respect to tunnel, or foundation or cut slope.

Average statistics of the above parameters were used to classify the in situ rock masses at the SSC site other than the fault zones. The results, shown in Table 8, indicate that the ratings for TM, AC, and EFS rock masses based on average parameters are 46, 67, and 36, respectively. AC at the SSC site can be classified as Class II (good rock) whereas TM and EFS can be classified as Class III (fair rock) and Class IV (poor rock), respectively.

Bieniawski's original classification system does not account for the swelling and slake/deterioration of TM and EFS when exposed to air and water during construction. The slaking potential is taken into account by applying a slake adjustment multiplier in accordance with the method developed by Newman and Bieniawski (1985). If this adjustment is made, as shown in Table 8, the rock mass ratings for TM, AC, and EFS become 41 (lowest possible rating for Class III - fair rock), 76 (Class II - good rock), and 29 (Class IV - poor rock), respectively. Figure 10 provides a general guideline for estimating of standup time for various rock classes (Bieniawski, 1979). This guide was developed for tunnels constructed by drilling and blasting methods and is anticipated to be somewhat conservative for the planned excavation by TBM. Figure 10 shows the relation between the geomechanics rock mass rating (RMR) value and standup time of unsupported underground excavation span, as developed by Bieniawski (1979).

TABLE 7. GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES[1]

A. Classification Parameters and their Ratings

			Ranges of Values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6-2 m	200-600 mm	60-200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surface Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1mm Slightly weathered walls	Slightly rough surfaces Separation <1mm Highly weathered walls	Slickensided surfaces OR Gouge <5mm thick OR Separation 1-5mm Continuous	Soft gouge >5mm thick OR Separation >5mm Continuous		
	Rating		30	25	20	10	0		
5	Ground Water	Inflow per 10 m tunnel length	None	<10 Liters/min	10-25 liters/min	25-125 liters/min	>125 liters/min		
			OR _____	OR _____	OR _____	OR _____	OR _____		
		Ratio joint water pressure and major principal stress	0	0.0-0.1	0.1-0.2	0.2-0.5	>0.5		
			OR_____	OR_____	OR_____	OR_____	OR_____		
		General conditione	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. Rating Adjustment for Joint Orientationa

Strike and dip orientations of joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. Rock Mass Classes Determined from Total Ratings

Rating	100-81	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. Meaning of Rock Mass Classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	>400 kPa	300-400 kPa	200-300 kPa	100-200 kPa	<100 kPa
Friction angle of the rock mass	>45°	35°-45°	25°-35°	15°-25°	<15°

E. The Effect of Discontinuity Strike and Dip Orientations in Tunneling

Strike perpendicular to tunnel axis				Strike parallel in tunnel axis		Dip 0°-20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20o-45°	Dip 45°-90°	Dip 20o-45°	Dip 45o-90°	Dip 20o-45°	
Very favorable	Favorable	Fair	Unfavorable	Very Unfavorable	Fair	Unfavorable

Note: (1) This classification is after Bieniawski, 1979.

TABLE 8. GEOMECHANICS CLASSIFICATION OF ROCK MASSES AT SSC SITE

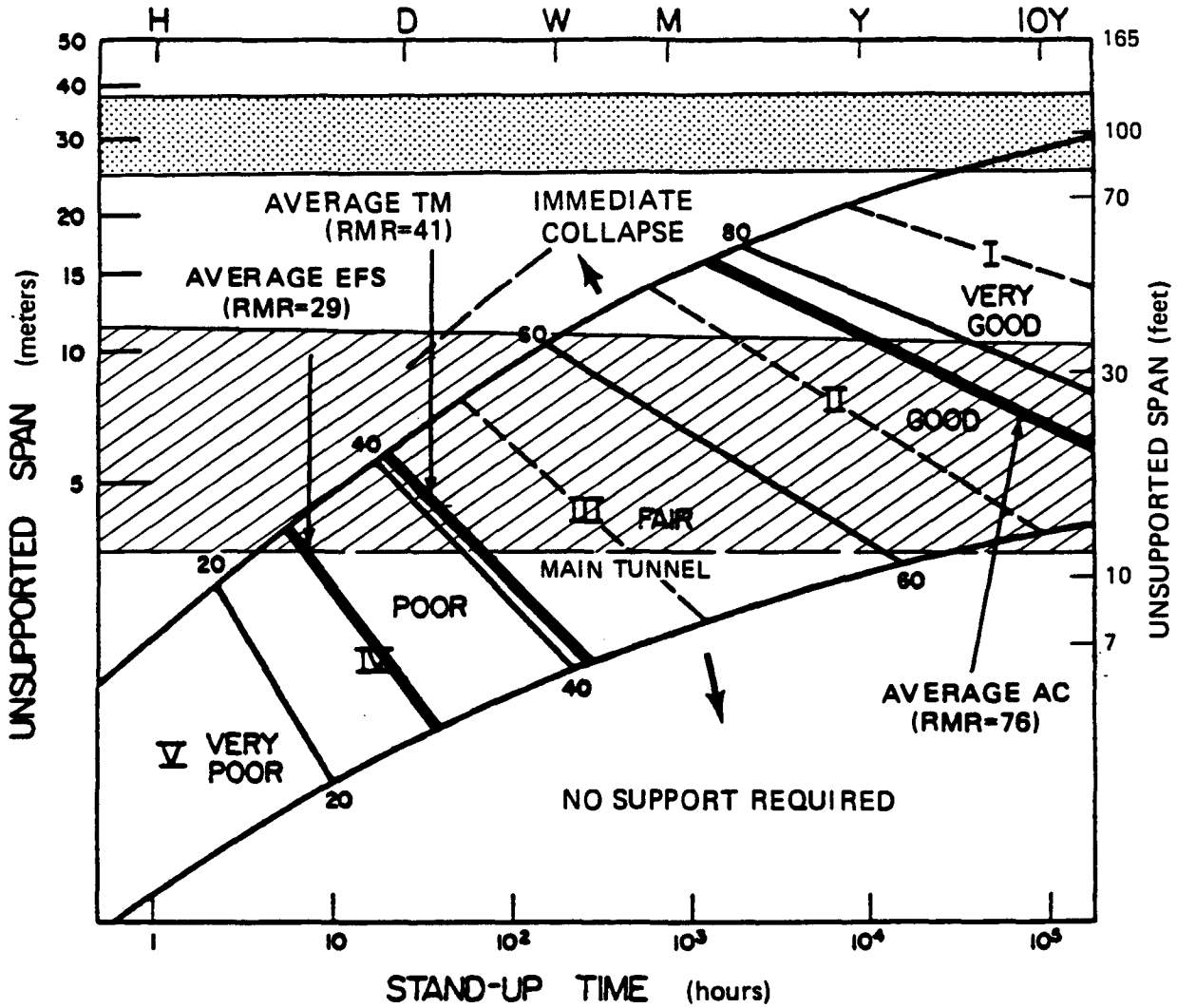
Rating Item	Taylor Marl		Austin Chalk		Eagle Ford Shale	
	Quantity	Rating	Quantity	Rating	Quantity	Rating
Uniaxial Compressive Strength	409 psi	1	2131 psi	2	297 psi	1
RQD	90 ~100%	20	90 ~100%	20	90 ~100%	20
Spacing of Joint(3)	<2 in	5	5.1 ft	15	<2 in	5
Conditions of Joint	Slightly Rough	20	Slightly Rough	25	Mostly Smooth	10
Groundwater	Moist Only	10	Moist Only	10	Moist Only	10
Joint Orientation with Respect to Tunnel Stability	Unfavorable	-10	Fair	-5	Unfavorable	-10
Basic Rating		46		67		36
Slake Adjustment Multiplier(1)	0.9		1.15		0.8	
Overall Rating		41		76.3		29
Class Number	III(4)		II(4)		IV(4)	
Rock Description(2)	Fair Rock(4)		Good Rock(4)		Poor Rock(4)	

Notes: (1) Slake adjustment multipliers (Newman and Bieniawski, 1985) are applied to strength, RQD, and joint parameters only. They are based on average second cycle slake durability index values of 52.8, 95.6 and 21.5% for TM, AC and EFS, respectively.

(2) This description is for tunnel excavation. For slope stability consideration and assuming favorable dip orientation, the overall ratings shown in the table will be 46 (Class III - fair rock), 67 (Class II - good rock), and 36 (Class IV - poor rock) for TM, AC and EFS, respectively.

(3) Because of potential separation of ubiquitous bedding planes, average spacing of bedding planes (Table 3) are conservatively used for TM and EFS. Average spacing for joint sets in AC is conservatively based on surficial mapping data (Table 3).

(4) The description is intended for about average in situ rock mass conditions other than the fault zones. As described in Table 1 and Section 3.1.2 the rock masses in the fault zone can be classified as "poor rock" (Class IV), "fair rock" (Class III), and "poor to very poor rock" (Class IV to Class V) for TM, AC and EFS, respectively.



EXPECTED SPAN RANGE FOR EXPERIMENTAL HALLS (75 TO 120 FEET)

EXPECTED SPAN RANGE FOR MAIN TUNNEL & WYES (12.5 TO 37 FEET)

NOTES: 1. THIS FIGURE IS AFTER BIENIAWSKI, 1979
2. NUMBERS I THROUGH V DENOTE GEOMECHANICS ROCK MASS RATING (RMR)

The Earth Technology Corporation	Project No.: 87-668-0017 RTK Joint Venture
Relationship Between the RMR Value and Stand-up Time of an Unsupported Underground Excavation Span	
12/90	Figure 10

Based on Figure 10, it can be shown that the average standup times are in terms of hours, days, and years for average conditions within EFS, TM, and AC, respectively.

Attempts were also made to classify the fault zones (Table 1) using the Bieniawski geomechanics classification system. The results indicate that the rock masses in the fault zone are generally one class lower than the in situ rock masses and can generally be classified as poor (Class IV) to very poor (Class V) in EFS, poor rock (Class IV) in TM and fair rock (Class III) in AC.

At the SSC site, the bedding planes in TM and EFS are mostly horizontal and the fractures are predominantly near vertical. Thus, it is anticipated that the strike/dip orientations at the site are favorable with respect to slope stability considerations. Based on Bieniawski's recommendations, the ratings for slope stability for TM, AC, and EFS are 46 (Class III - fair rock), 67 (Class II - good rock), and 36 (Class IV - poor rock), respectively.

It should be noted that these rock mass classification systems for underground application were developed based on case histories of tunnels and large underground facilities in rocks with little or no swell potential and low susceptibility to slake/alteration. Thus, the use of the geomechanics classification to classify rock masses for TM and EFS represents an initial qualitative engineering classification and will require further refinements as some site specific experience, such as the construction of a large-diameter drilled shaft or prototype installation facility (PIF) becomes available.

3.2 INDEX PROPERTIES

3.2.1 Density and Moisture Content

Density and moisture content tests yielded a broad range of results. As postulated previously, some of the scatter probably reflects the normal in situ variation and some may be caused by sample disturbance and sample size effect. Sample disturbance would tend to open cracks and microfissures within the rock cores and, thus, may result in decreasing the density. However, the expected extent of density and moisture content changes caused by disturbance is small. The recommended range and mean values of density and moisture content, and corresponding laboratory test results are tabulated below.

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Dry Density (pcf)			
		Available Data		Recommended Value for SSCD(2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	102.5 - 129.2	114.3	100 - 130	115
Overall AC	A-15 to A-27	96.8 - 143.5	124.1	95 - 145	124
Top AC	A-15 to A-18	100.0 - 140.5	129.8	100 - 145	130
Middle AC	A-19 to A-25	96.8 - 143.5	122.7	95 - 145	123
Bottom AC	A-26 to A-27	101.7 - 140.6	122.0	95 - 145	123
Overall EFS	E-20 to E-22	101.2 - 139.3	117.2	100 - 140	118
Top EFS	E-20	101.2 - 131.9	116.5	100 - 135	117
Bottom EFS	E-21 to E-22	110.4 - 139.3	118.8	110 - 140	119

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Total Density (pcf)			
		Available Data		Recommended Value for SSCD(2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	123.9 - 142.8	133.7	120 - 145	135
Overall AC	A-15 to A-27	113.6 - 159.9	139.6	110 - 160	140
Top AC	A-15 to A-18	124.1 - 155.8	143.7	120 - 160	145
Middle AC	A-19 to A-25	113.6 - 159.9	138.6	110 - 160	139
Bottom AC	A-26 to A-27	125.6 - 152.7	138.2	120 - 160	139
Overall EFS	E-20 to E-22	118.0 - 150.4	136.1	115 - 155	137
Top EFS	E-20	118.0 - 148.0	135.7	115 - 150	136
Bottom EFS	E-21 to E-22	129.1 - 150.4	137.1	125 - 155	138

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Moisture Content (percent)			
		Available Data		Recommended Value for SSCD(2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	10.5 - 23.1	16.9	10 - 24	17
Overall AC	A-15 to A-27	6.9 - 25.4	12.7	6 - 26	13
Top AC	A-15 to A-18	7.4 - 24.1	10.8	7 - 25	11
Middle AC	A-19 to A-25	6.9 - 22.9	13.2	6 - 24	13
Bottom AC	A-26 to A-27	8.5 - 25.4	13.3	8 - 26	13
Overall EFS	E-20 to E-22	6.2 - 23.0	16.1	6 - 24	17
Top EFS	E-20	10.4 - 23.0	16.5	6 - 24	17
Bottom EFS	E-21 to E-22	6.2 - 20.0	15.4	6 - 20	16

Notes: (1) Refer to Report No. SSC-GR-23
(2) Recommended for Site-Specific Conceptual Design

3.2.2 Specific Gravity and Degree of Saturation

Specific gravity for TM and EFS as determined by UT (ranging from 2.62 to 2.87) is higher than those determined by MJA/SWL (ranging from 2.57 to 2.79). The high specific gravity values from UT tests were attributed to higher pyrite content in the samples, which are not typically representative of the generally low pyrite content (test results shown an average of about 2 percent or less in TM and EFS). It is assumed that the following specific gravity values corresponding to the range and mean values as determined by MJA/SWL are appropriate:

Rock Type	Specific Gravity Values	
	Range	Best Estimate
TM	2.65 to 2.73	2.71
AC	2.62 to 2.71	2.67
EFS	2.57 to 2.79	2.72
MB	2.3 to 2.7	2.5

The above best estimates, available density/moisture content data (Report Nos. SSC-GR-66, GR-67, and GR-68), and field hydrogeologic data (Report No. SSC-GR-63) all lead to the following conclusions with respect to the degree of saturation:

- EFS and AC are mostly 100 percent saturated
- The portion of TM under the piezometric level is 100 percent saturated whereas the portion above the piezometric level is probably about 90 percent saturated.

3.2.3 Slake Alteration and Deterioration

Among the three bedrock units, EFS has the lowest slake durability. TM has low to moderate slake durability, and AC has the highest slake durability with little or no alteration when exposed to air or water. The ranges and mean values of second cycle slake durability summarized in the appendix and below are considered representative of the three rocks at the SSC site.

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: 2nd Cycle Slake Durability Index (percent)			
		Available Data		Recommended Value for SSCD(2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	14.0 - 86.6	52.8	14 - 90	52
Overall AC	A-15 to A-27	88.4 - 98.0	95.6	88 - 99	95
Top AC	A-15 to A-18	—	—	90 - 99	96
Middle AC	A-19 to A-25	88.4 - 98.0	95.9	88 - 99	95
Bottom AC	A-26 to A-27	93.4 - 97.4	95.8	90 - 98	95
Overall EFS	E-20 to E-22	0.4 - 51.9	21.5	0 - 55	21
Top EFS	E-20	0.40 - 51.9	21.9	0 - 52	21
Bottom EFS	E-21 to E-22	4.9 - 33.1	20.8	4 - 35	21

Notes: (1) Refer to Report No. SSC-GR-23
(2) Recommended for Site-Specific Conceptual Design

Humidity-controlled deterioration tests also reveal that both EFS and TM are susceptible to alteration and are expected to deteriorate when exposed to environments with relative humidity less than 95 percent. They are also likely to soften if exposed to environments with a relative humidity more than 95 percent. All these indicators dictate the need of minimizing exposure of TM or EFS to air or water by quickly applying protection and support during excavation.

Although AC has high slake durability, some rock spalling may occur if it is left exposed indefinitely. Additionally, AC contains weaker bentonite marker and bentonitic shale beds that have lower slake durability. Thus, protection of AC with a thin shotcrete layer may be advisable and local stabilization/strengthening of bentonite and shale beds may be necessary.

3.3 SWELLING PRESSURE

The swell pressures for TM and EFS, as measured and calculated by MJA/SWL, UT, and UI are presented in Table 6. As can be seen from this table, swell pressures measured and calculated by the

different laboratories vary significantly. The following factors are postulated to contribute to this large variation:

- Sample disturbance in the form of microfissures or cracks may result in underestimates of the true swell pressure and were not accounted for in MJA/SWL test results.
- UT may have over-adjusted for sample disturbance effects by assuming that all microfissures were caused by sample disturbance. This results in several calculated swell pressure values that are significantly higher than the corresponding effective overburden stress.
- Swell pressure data obtained by UI were not adjusted for sample disturbance. Most of UI data are of the same order of magnitude as those obtained by MJA/SWL, except for a few anomalous data points in which the measured swell pressures in EFS were significantly higher than the effective overburden stresses. The reason(s) for this anomaly is not clear.

Thus, the use of swell pressure data from MJA/SWL and UI (excluding some anomalous data points) will be unconservative while using data from UT may be over-conservative. Sample disturbance effects are difficult to quantity. For TM and EFS, the combined data from MJA/SWL were used and conservatively modified to account for sample disturbance effects as follows:

- The anticipated range is defined from minimal values from the test data to a maximum value equal to the estimated average effective overburden.
- The recommended best estimated value is taken to be equal to the maximum value from the test data (excluding anomalous data showing swell pressure greater then effective overburden stress).

Based on the above assumptions, the recommended swell pressures for various bedrock units are as follows:

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Swell Pressure (psi)			
		Available Data		Recommended Value for SSCD(2)	
		Range	Average	Range (3)	Best Estimate (4)
Overall TM	T-21 to T-23	6.5 - 57.9	19.9	20 to σ'_{max}	58
Overall EFS	E-20 to E-22	7.7 - 115.0	35.8	36 to σ'_{max}	115
Top EFS	E-20	7.7 - 115.0	37.2	37 to σ'_{max}	115
Bottom EFS	E-21 to E-22	9.0 - 86.8	32.6	33 to σ'_{max}	90

Notes:
(1) Refer to Report No. SSC-GR-23
(2) Recommended for Site-Specific Conceptual Design
(3) Recommended minimum value is set to be about the average value of available data to account for potential sample disturbance effect. σ'_{max} is equal to effective overburden or best estimate value, whichever is larger.
(4) Best estimate value is conservatively assumed to be about the maximum value from all available MJA/SWL test data.

It should be noted that the recommended maximum swell pressure values are also intended to account for the possibility that locally high swell pressure (as high as effective overburden) may exist in EFS and TM depending on their mineralogic composition, moisture content, grain size, etc. In situ monitoring and

verification, as well as provisions for remedial measures during construction, are prudent in design and construction in EFS and TM.

No test data (except for MB) are available for AC. However, AC (excluding MB) is not likely to be susceptible to swell.

3.4 CONSOLIDATION CHARACTERISTICS

The potential for settlement of facilities founded on TM and EFS as a result of consolidation requires consideration. Available laboratory-measured consolidation characteristics, based on one-dimensional consolidation tests in terms of recompression index and coefficient of consolidation, are summarized in the appendix. The laboratory-measured recompression index is expected to be higher than the in-situ value because of potential sample disturbance effects. Similarly, sample disturbance effects may indicate that the laboratory-measured coefficient of consolidation may be higher than the corresponding value in the field. However, it should also be noted that one-dimensional consolidation tests may also overestimate the consolidation time in situ, where consolidation is a three-dimensional phenomenon. Average and range of laboratory-measured consolidation characteristics are recommended for the following reasons:

- Use of laboratory-measured recompression index (without compensating for sample disturbance effects) is somewhat conservative and may provide compensation for some unexpected variation in the field
- Three-dimensional consolidation under in situ conditions will tend to compensate for the potential underestimate of consolidation rate (i.e., higher coefficient of consolidation) caused by sample disturbance in the laboratory under one-dimensional loading conditions.

Based on available test results, the following ranges and best estimate values for EFS are recommended:

Recompression Index		Coefficient of Consolidation (ft^2/day)	
Range	Best Estimate	Range	Best Estimate
0.001 to 0.123	0.05	0.001 to 0.014	0.005

No consolidation testing was performed for TM or AC. Judging from the similarity in index properties between EFS and TM, it is recommended that the above consolidation characteristics for EFS also be used for TM. Also, it is anticipated that AC will be less susceptible to consolidation (i.e., drained creep) deformation than either TM or EFS.

3.5 HYDRAULIC CONDUCTIVITY

Hydraulic conductivities of the in situ rock mass as determined by the field packer and slug tests are presented in Report No. SSC-GR-63 and summarized in Figure 4 in Section 2. These field-determined horizontal hydraulic conductivity values are considered reasonably representative of the field range variations. Laboratory tests (Report No. SSC-GR-81) on intact EFS indicate that the hydraulic conductivity in the vertical direction for the intact EFS is on the order of 10^{-10} to 10^{-9} cm/sec, which is one to two orders of magnitude less than the hydraulic conductivity of the in situ EFS rock mass as determined by the field packer and slug tests (Report No. SSC-GR-63). The recommended hydraulic

conductivities for in situ rock masses in areas other than the fault zones are similar to those determined by the field tests and are summarized as follows:

Hydraulic Conductivity (cm/sec)		
Rock Type	Range	Best Estimate
Overall TM	$2x10^{-8}$ to $9x10^{-6}$	$5x10^{-8}$
Overall AC	$3x10^{-8}$ to $3x10^{-5}$	$5x10^{-7}$
Overall EFS	$1.2x10^{-8}$ to $1.5x10^{-5}$	$2x10^{-7}$

As described in Section 2, the hydraulic conductivities of fault zones are expected to be about one order of magnitude higher than the above corresponding values.

3.6 STATIC STRENGTH

3.6.1 Intact Rock

Laboratory strength tests on cores from the three rock groups consisted mostly of uniaxial compression (UC) and unconsolidated, undrained triaxial compression (UU) tests. Although a limited number of consolidated "drained" direct shear (DS) and "drained" triaxial compression (CD) tests have been attempted by UT, the results cannot be used with confidence because time durations allowed for consolidation and rate of shearing may not have been sufficient to ensure complete consolidation or complete drainage during shear. Thus, the effective strength parameters determined by UT can be used as qualitative information only. Further work on the long-term effective strength parameters either by CD or consolidated undrained triaxial tests with pore pressure measurement will be needed.

Both UU and UC strength data on all three major rock units show significant scatter. The strength values, as determined by UU tests, are generally expected to be slightly higher than those determined by UC tests, because the applied confinement in UU tests may tend to partially close the fissures or cracks induced by sample disturbance. However, no definitive relationship was observed between strength and confinement. Other relevant observations are as follows:

- Because the carbonate content of TM is on the average about three times as much as that of EFS, TM is anticipated to be stronger than EFS. However, laboratory test results indicate that the UC or UU strengths of TM and EFS are about the same. It is postulated that intact core samples from TM may be more disturbed than those from EFS.
- The upper bound and mean UU and/or UC strengths from vertical borings are generally higher than corresponding strengths from inclined borings (about 45 to 55° from horizontal). Almost all test specimens from inclined borings failed along bedding or pre-existing fracture planes. Thus, bedding and pre-existing fracture planes appear to be the weakest direction in the rock mass. It should be noted that part of the apparent anisotropy may be due to the locations of the coreholes. In this project, the inclined borings were preferentially located in the fault zones where rock masses may have been previously weakened by past fault activities. However, data available are too limited to establish definitive conclusions. Further work will be necessary to resolve this issue.

Uniaxial compressive strength of intact cores is an important performance factor for rock mass characterization. Based on engineering judgement, laboratory test results were modified in an attempt to account for sample disturbance effects and are presented in Table 9. The rationale for recommended values and best estimates of uniaxial compressive strength are also presented in Table 9.

3.6.2 Discontinuities

Direct shear tests on samples were done under various conditions. The results in terms of peak strengths for TM, AC, and EFS are summarized in Figure 11a-c. The following observations relevant to strength assessment should be noted:

- A broad range in the test results is observed. This scatter may reflect the in situ variation.
- Conditions of discontinuities within each rock type vary, e.g., AC contains frequent argillaceous and shaly limestone beds. Specimens with argillaceous or shaly limestone discontinuities often exhibit lower shear strengths than discontinuities within clean chalk. It may be prudent to use lower bound peak and residual strengths in design to account for the presence of "weaker" discontinuities in individual rock units. A similar recommendation is applicable to EFS and TM as well.

The recommended range and best estimated values of peak shear strengths for discontinuities in the three bedrock units are also shown in Figures 11a-c. It should be noted that the recommended peak shear strength values are tentative and are subject to refinement after more laboratory and field data become available.

Available data (Nieto, 1990) on residual strength of discontinuities are limited and preliminary in nature. They are summarized as follows:

Rock Type	Residual Strength	
	Range	Best Estimate
TM and EFS	$c \sim 0$ $\phi = 5^\circ$ to 10°	$c \sim 0$ $\phi = 8^\circ$
AC	$c \sim 0$ $\phi = 22^\circ$ to 28° +	$c \sim 0$ $\phi = 25^\circ$

3.6.3 Rock Mass

The strength of the rock mass depends on rock type and characteristics of beddings, joints, and fractures. Attempts were made to evaluate peak strengths of rock masses at the SSC site using the empirical correlations developed by Hoek and Brown (1980) and Bieniawski (1979). The simplified strength correlation developed by Hoek and Brown (1980) correlating rock mass strength as a function of rock mass rating in terms of the Q system (Barton et al., 1974) or the Commonwealth Scientific and Industrial Research (CSIR) rating (the same as the 1974 rock mechanics rating version developed by Bieniawski, 1974) was used for this purpose. This correlation was developed based on model studies

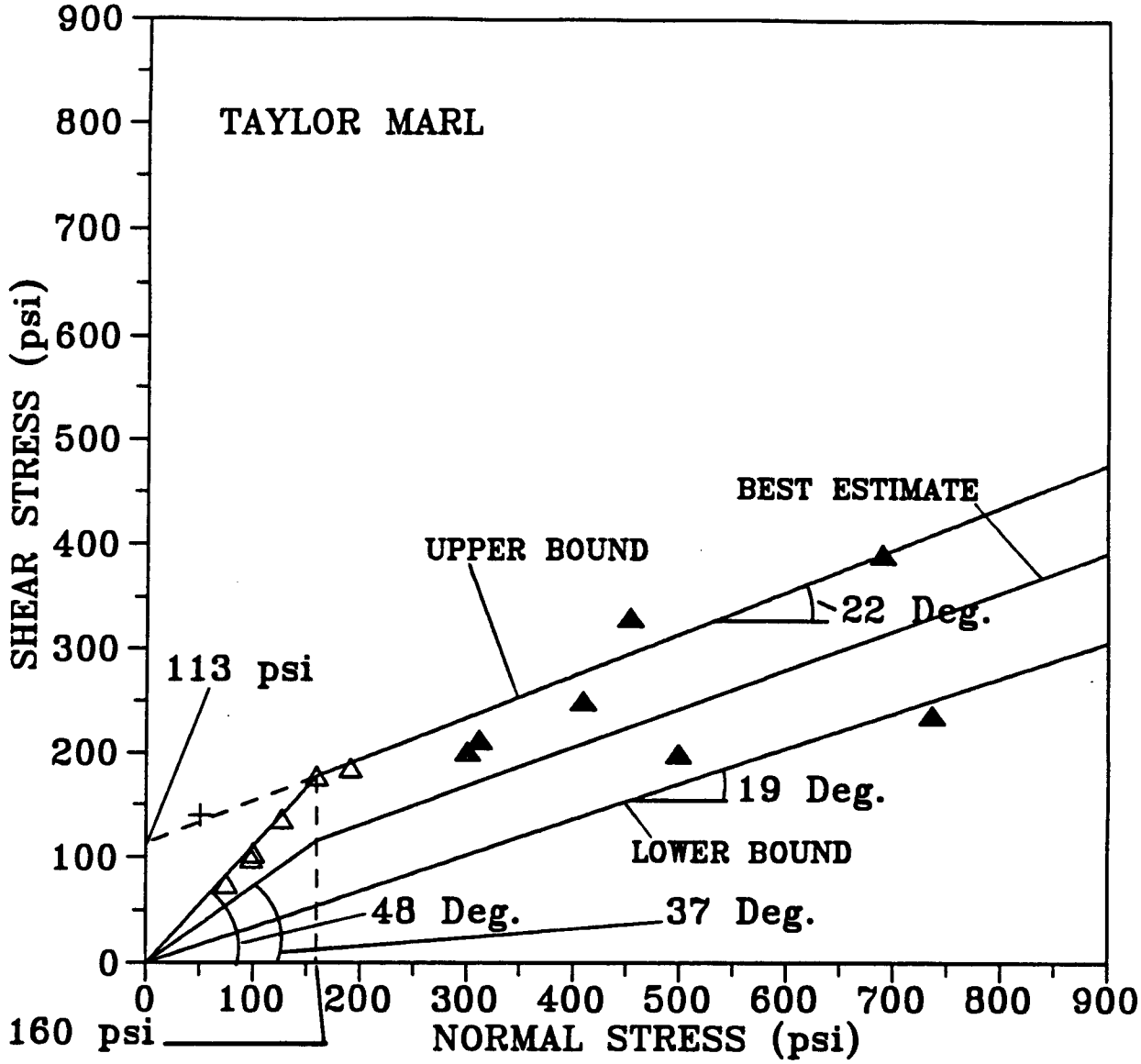
TABLE 9. RECOMMENDED VALUES OF UNIAXIAL COMPRESSIVE STRENGTH

Rock Type	Wireline Stratigraphic Units[1]	Geotechnical Parameter: Uniaxial Compressive Strength (psi)			
		Available Data[3]		Recommended Value for SSCD[2]	
		Range	Average	Range[4]	Best Estimate[4]
Overall TM	T-21 to T-23	77-746	409	250 to 1,100	550
Overall AC	A-15 to A-27	306-3,987	2,131	300 to 4,000	2,200
Top AC	A-15 to A-18	313-3,819	2,250	300 to 4,000	2,300
Middle AC	A-19 to A-25	306-3,987	--	300 to 4,000	2,000
Bottom AC	A-26 to A-27	426-3,790	2,117	400 to 4,000	2,100
Overall EFS	E-20 to E-22	9-844	297	100 to 1,200	400
Top EFS	E-20	9-522	257	100 to 700	350
Bottom EFS	E-21 to E-22	85-844	325	100 to 1,200	450

Notes: (1) Refer to Report No. SSC- GR-65
(2) Recommended for Site-Specific Conceptual Design
(3) Only laboratory data on samples from vertical coreholes are used
(4) A. To account for potential sample disturbance effects for TM and EFS the following approximate relations are used based on engineering judgment:
- Recommended minimum value ≅ a minimum value of available data
- Recommended best estimate ≅ average of available data $+\sigma$
- Recommended maximum value ≅ maximum value of available data $+2\sigma$

where σ = Standard deviation of available data

B. Recommended values are similar to these exhibited by the available data.



△△△△△ UC-INCLINED BEDDING PLANE SHEAR
▲▲▲▲▲ UU-INCLINED BEDDING PLANE SHEAR
+++++ UC-VERTICAL SHEAR ALONG PRE-EXISTING FRACTURE

Note: Based on test data from report GR-68

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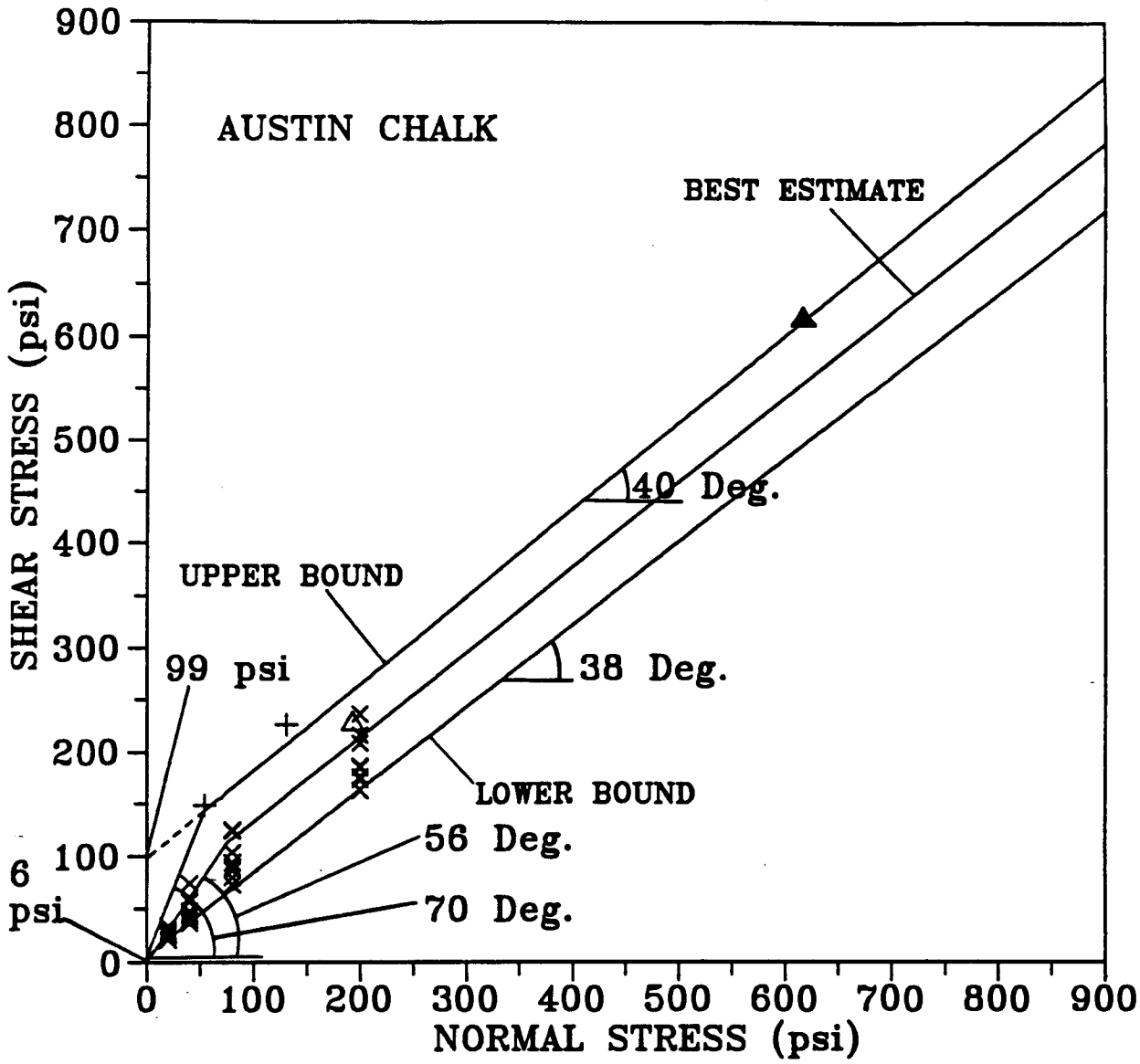
Peak Shear Strength of Discontinuities - Taylor Marl	
12/90	Figure 11a



△△△△△ UC–INCLINED BEDDING PLANE SHEAR
▲▲▲▲▲ UU–INCLINED BEDDING PLANE SHEAR
+++++ UC–VERTICAL SHEAR ALONG PRE EXISTING FRACTURE
XXXXX DISCONTINUITY SHEAR – OUTCROP SAMPLES

Note: Based on test data from report GR-67

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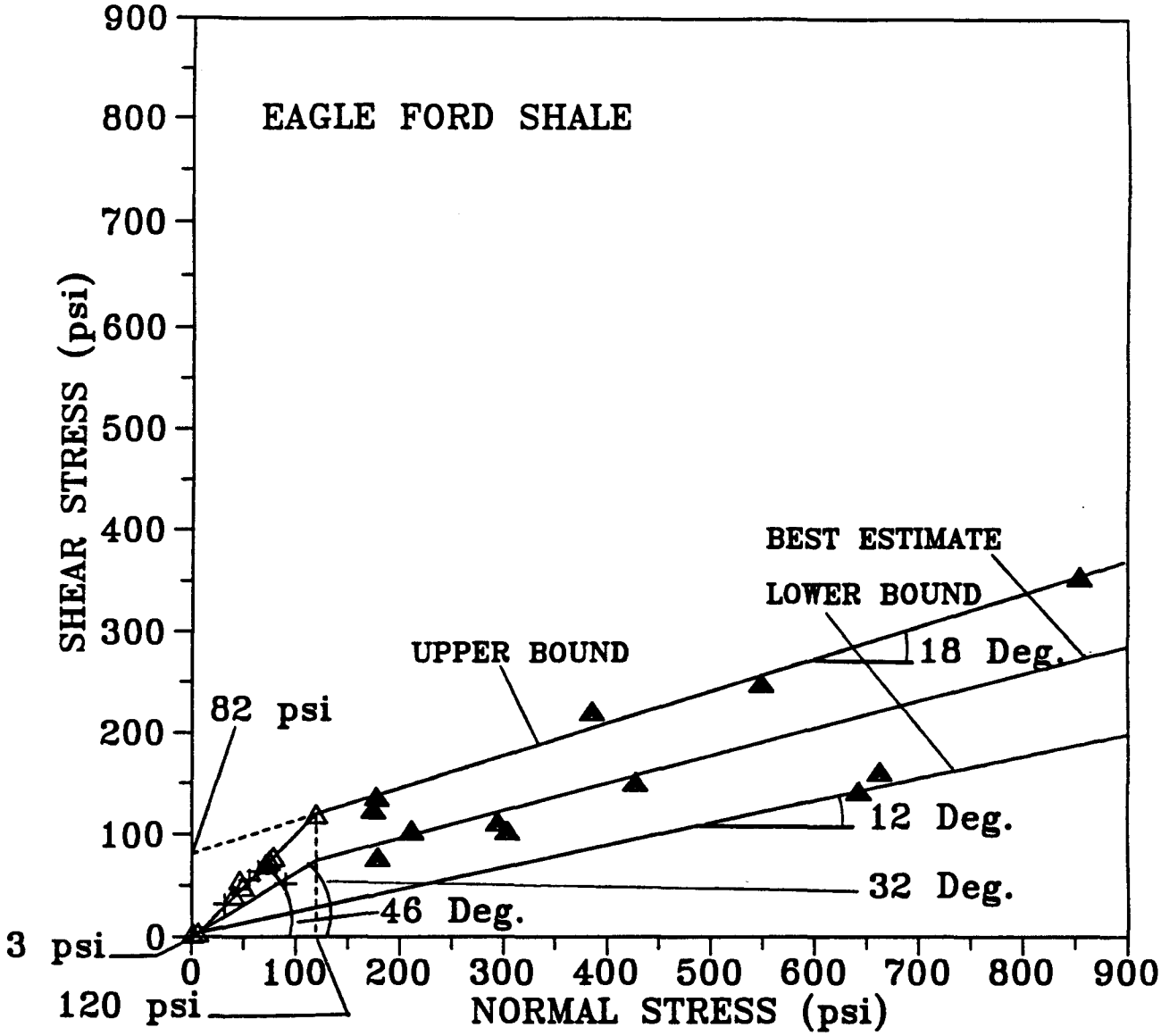
Peak Shear Strength of Discontinuities - Austin Chalk	
12/90	Figure 11b



△△△△△ UC-INCLINED BEDDING PLANE SHEAR
▲▲▲▲▲ UU-INCLINED BEDDING PLANE SHEAR
+++++ UC-VERTICAL SHEAR ALONG PRE-EXISTING FRACTURE

Note: Based on test data from report GR-66

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Peak Shear Strength of Discontinuities - Eagle Ford Shale	
12/90	Figure 11c

and analytical formulations by Ladanyi and Archambault (1970, 1972). The basic empirical equation for rock mass strength can be expressed as:

$$\tau = A\sigma_c (\sigma/\sigma_c - T)^B$$

where:

τ = shear strength of rock mass
σ_c = uniaxial compressive strength of intact rock
σ = normal stress
A,B,T = dimensionless constants depending on rock types and rock mass rating

Assuming favorable joint orientation for slope stability evaluation purpose, the following parameters were utilized:

Rock Type	CSIR Rating	Uniaxial Compressive Strength (psi)	A	B	T
TM	40	550	0.21	0.672	-0.00048
AC	70	2,200	0.45	0.674	-0.01
EFS	38	450	0.2	0.67	-0.00045

Calculated shear strengths of rock mass based on the above parameters are shown in Figures 12a-c. As can be seen from these figures, the shear strengths of rock masses are very close to the lower bound peak strength of discontinuities in the corresponding rock units.

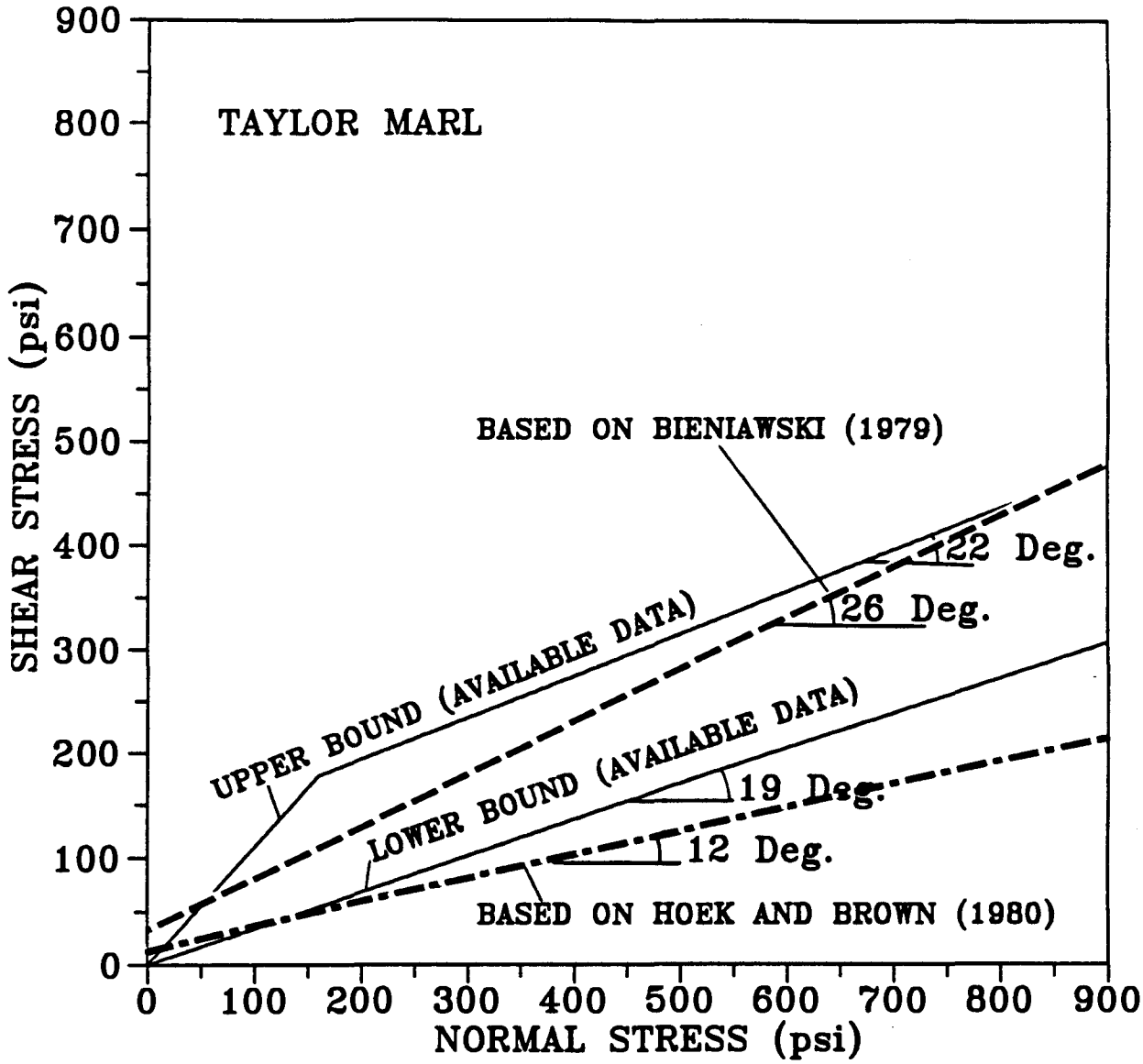
Similarly, the empirical correlations between shear strength and geomechanical classification rating as developed by Bieniawski (1979) and shown in Table 7 were also used to estimate the shear strength of rock masses. The results are also shown in Figures 12a-c. The results shown in Figures 12a-c appear to indicate that the ranges of shear strength of in situ rock mass are similar to the bounded range of peak shear strengths of discontinuities in corresponding rock units. Thus, in lieu of further field testing or performance monitoring data, the range and best estimated values of peak shear strength of discontinuities shown in Figures 11a-c are recommended for use as in-situ rock mass shear strength values.

It should be noted that the above shear strength recommendation is subject to refinement after more information from large-scale field testing, as well as from performance monitoring associated with engineering evaluation from the planned large-diameter drilled hole of PIF.

3.7 DEFORMATION/STIFFNESS CHARACTERISTICS

3.7.1 Static Deformation Modulus of Rock Mass

The rock type as well as, to a large extent, the frequency and nature of discontinuities determine its deformation characteristics. Reliable estimates of deformation characteristics can be obtained only from either large-scale field load tests and/or from back calculations based on performance monitoring. Only limited in-situ data are available from pressuremeter tests and geophysical investigations at the SSC site (Section 2). Thus, the geomechanical characterization presented herein is primarily based on laboratory test data.



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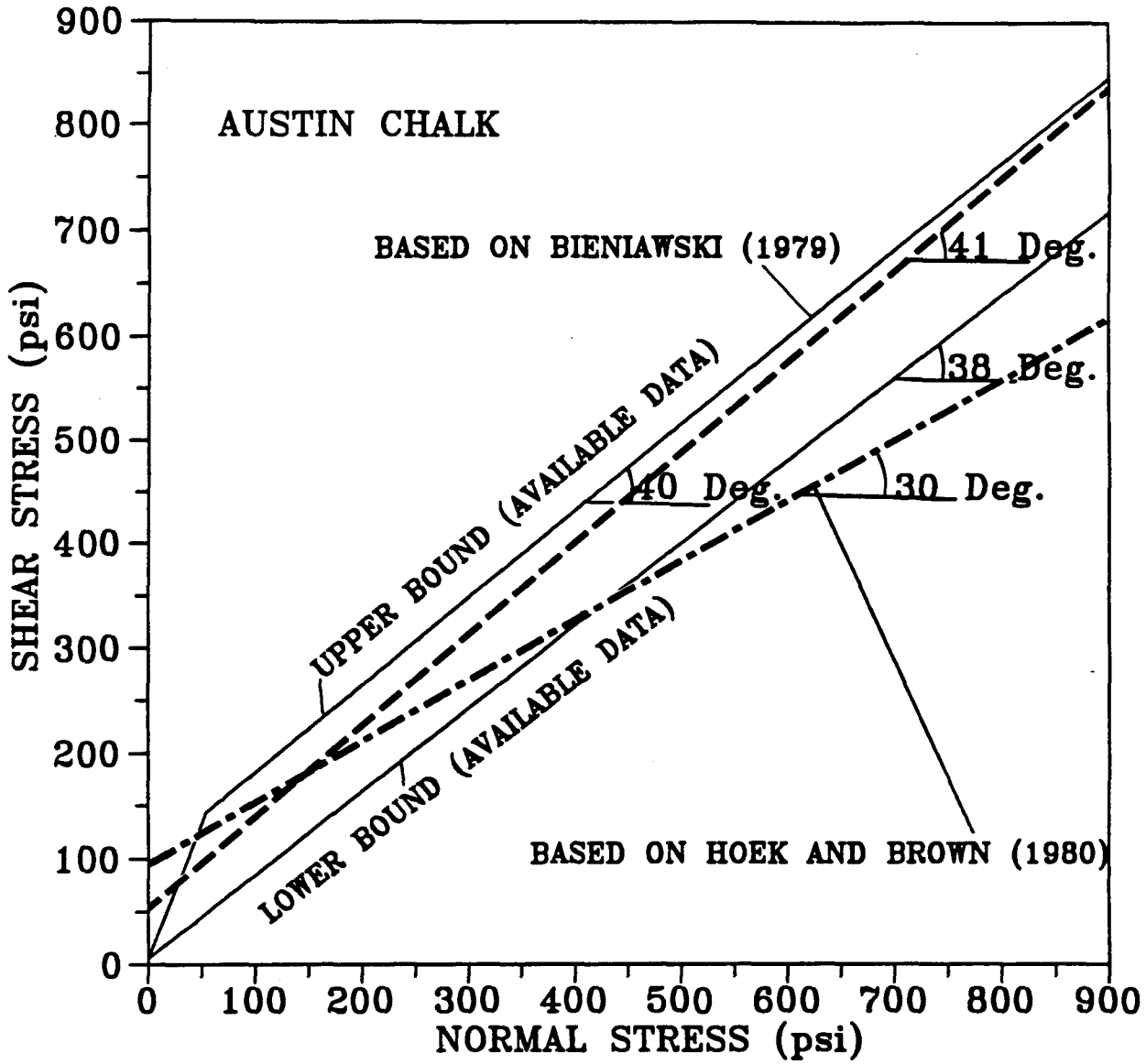
Shear Strength of Rock Mass Based on Empirical Correlations - Taylor Marl

12/90 Figure 12a



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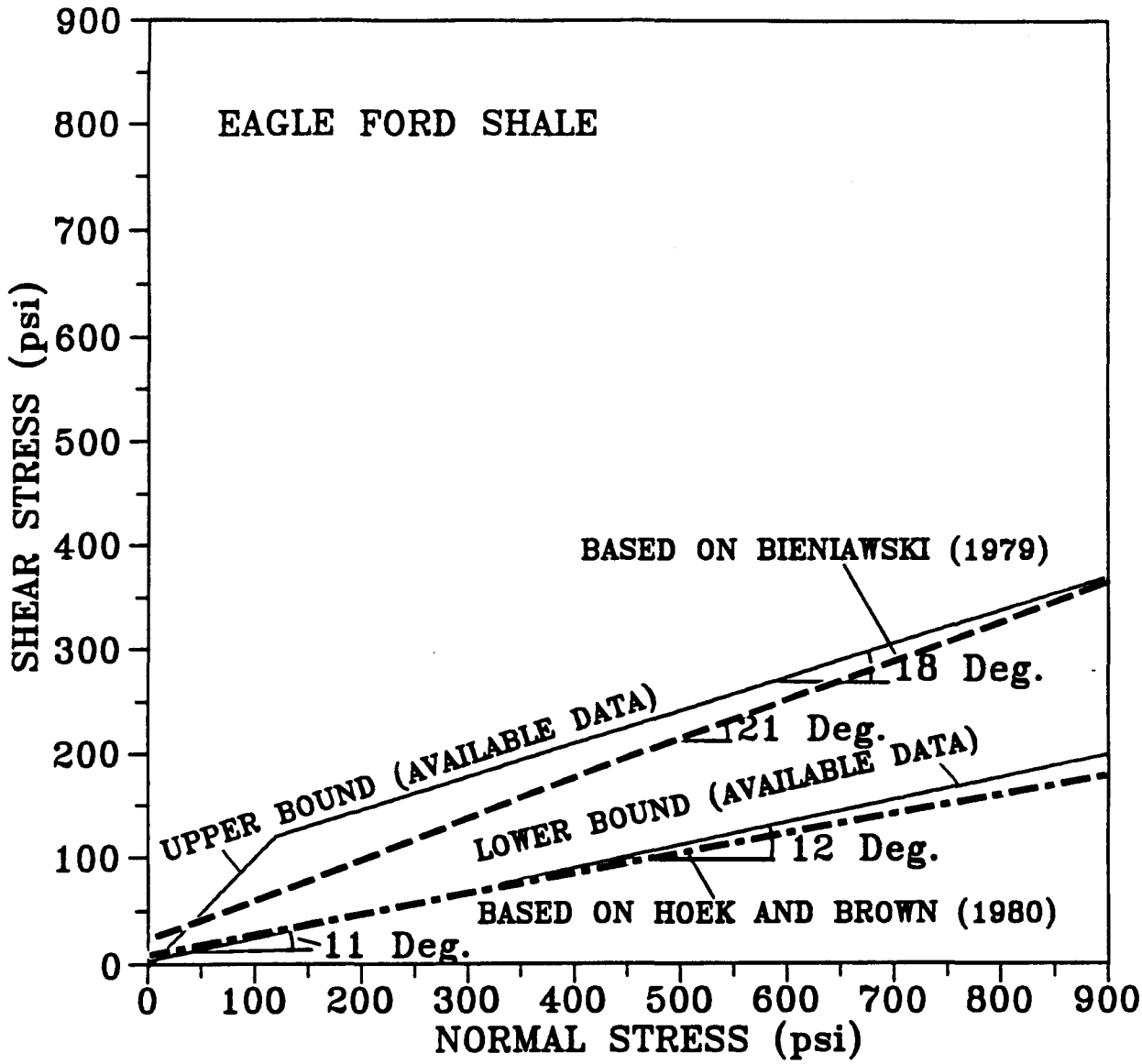
Shear Strength of Rock Mass Based on Empirical Correlations - Austin Chalk

12/90

Figure 12b



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Shear Strength of Rock Mass Based on Empirical Correlations - Eagle Ford Shale

12/90

Figure 12c

The modulus of a rock mass is always smaller than the static modulus determined from "true" intact rock cores. The literature contains various empirical relations as the ratio of deformation modulus of rock mass measured in the field to the laboratory modulus in terms of RQD, or seismic velocity ratio (Deere et al., 1966), or geomechanics classification rating (Bieniawski, 1979). These empirical correlations may not be applicable to the three rock units, especially TM and EFS at the SSC sites, for the following reasons:

- These correlations were developed mostly based on field and laboratory data on various rock types with strengths significantly higher than those at the SSC site.
- The intact cores of the three rock units at the SSC sites, particularly the TM and EFS, may be disturbed to some extent. Thus, the deformation characteristics determined on the "intact cores" may not be representative of the "true" intact modulus.

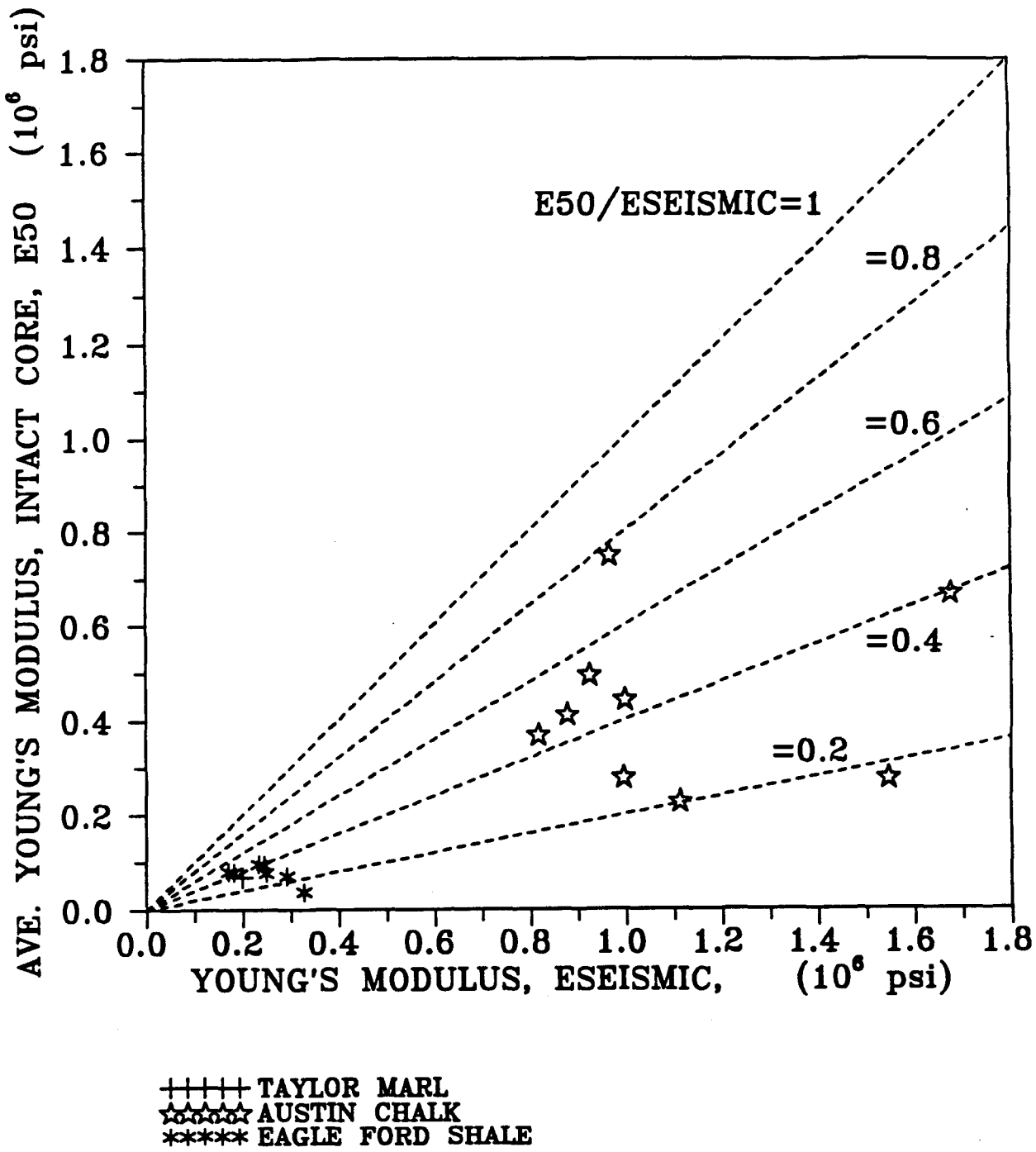
Figures 7a and 7b show the range of modulus (E_{50}) determined from laboratory tests vs. depth. Limited modulus data from pressuremeter tests (E_m) are also superimposed in these figures. Comparison of these two modulus profiles indicates that:

- The E_m profile of TM rock mass is approximately $2\frac{1}{4}$ to 4 times that of the laboratory-determined E_{50} values.
- The deformation modulus (E_m) of AC as determined from the pressuremeter tests is about 0.5 to 3 times that of E_{50} determined from the laboratory tests.
- The E_m profile of EFS rock mass is approximately 1.5 to 2 times that of the laboratory E_{50} profile.

The seismic Young's modulus (E_{seis}) as determined from geophysical field surveys is generally higher than the in-situ deformation modulus of rock mass, even though E_{seis} inherently reflects the intact rock and discontinuities within the rock mass. This is primarily a result of the fact that the deformation of rock mass during a seismic survey is so small that the deformation is entirely elastic. Comparison of E_{seis} with corresponding E_m and average E_{50} are shown in Figures 7 and 13, respectively. The following observations are made upon examination of these figures:

- The limited data shown in Figure 7 indicate that the E_{seis} of AC is about 0.8 to 2.2 times the corresponding E_m and the E_{seis} of EFS is about 1 to 2.2 times its corresponding E_m.
- The ratios of E_{seis} to E_{50} based on the results shown in Figure 13 are summarized in Table 10. An overall summary of relations among E_{50}, E_m, and E_{seis} is also presented in Table 10.

Although the data shown in Table 10 are relatively consistent with expected trends, this needs further validation based on additional in situ, large-scale tests. In particular, the few in situ data available are too limited and all data show significant scatter. The causes for large differences between E_{50} and E_m remain speculative and unresolved and require large-scale field tests for better resolution.



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Average Young's Modulus (E50) vs Seismic Young's Modulus

12/90

Figure 13

TABLE 10. ESTIMATED RELATIONSHIPS BETWEEN VARIOUS MODULUS VALUES

Rock Type	E_{50}/E_m		E_{seis}/E_m		E_{50}/E_{seis}	
	Range	Approximate Average	Range	Approximate Average	Range	Approximate Average
Taylor Marl	0.25 to 0.45	0.35	0.5 to 2	1	--	0.38
Austin Chalk	0.5 to 3	0.8	0.8 to 2.2	1.5	0.2 to 0.8	0.55
Eagle Ford Shale	0.5 to 0.7	0.6	1 to 2.2	1.7	0.2 to 0.6	0.35

Notes:

E_{50} = Young's Modulus at 50% of Ultimate Strength (load applied perpendicular to bedding)

E_m = Deformation modulus as determined from pressuremeter tests (load applied parallel to bedding)

E_{seis} = Seismic Young's Modulus as determined from geophysical investigation assuming isotropic behavior

Based on the aforementioned reasons and engineering judgement, the recommended values for deformation modulus for the three rock types TM, AC, and EFS are as follows:

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Young's Modulus of Rock Mass (10E5, psi)			
		Available Data (E_{50})		Recommended Value for SSCD E_m (2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	0.10 - 2.39	0.71	0.25 to 5.0	2
Overall AC	A-15 to A-27	0.13 - 22.73	4.22	0.25 to 25	5
Top AC	A-15 to A-18	0.13 - 11.63	3.81	0.1 to 15	5
Middle AC	A-19 to A-25	0.21 - 22.73	4.04	0.3 to 25	5
Bottom AC	A-26 to A-27	0.56 - 10.53	4.72	1 to 15	5
Overall EFS	E-20 to E-22	0.03 - 2.95	0.78	0.1 to 4.5	1.3
Top EFS	E-20	0.03 - 1.41	0.77	0.1 to 2.5	1.3
Bottom EFS	E-21 to E-22	0.03 - 2.95	0.79	0.1 to 4.5	1.3

Notes: (1) Refer to Report No. SSC-GR-23
(2) Recommended for Site-Specific Conceptual Design

3.7.2 Dynamic Modulus and Seismic Wave Velocity

The ranges of dynamic modulus and seismic wave velocities based on available geophysical data are provided in Table 5.

3.7.3 Poisson's Ratio

Statistics of Poisson's ratios based on tests on intact cores are presented in the appendix. The recommended range and mean values for rock units are summarized as follows:

Rock Type	Wireline Stratigraphic Units (1)	Geotechnical Parameter: Poisson's Ratio			
		Available Data		Recommended Value for SSCD (2)	
		Range	Average	Range	Best Estimate
Overall TM	T-21 to T-23	0.10 - 0.56	0.32	0.1 to 0.6	0.3
Overall AC	A-15 to A-27	0.06 - 0.48	0.19	0.05 to 0.5	0.2
Top AC	A-15 to A-18	0.07 - 0.48	0.20	0.05 to 0.5	0.2
Middle AC	A-19 to A-25	0.07 - 0.48	0.21	0.05 to 0.5	0.2
Bottom AC	A-26 to A-27	0.06 - 0.44	0.17	0.05 to 0.5	0.2
Overall EFS	E-20 to E-22	0.04 - 0.88	0.29	0.1 to 0.9	0.3
Top EFS	E-20	0.04 - 0.88	0.30	0.05 to 0.9	0.3
Bottom EFS	E-21 to E-22	0.09 - 0.42	0.26	0.05 to 0.5	0.3

Notes: (1) Refer to Report No. SSC-GR-65
(2) Recommended for Site-Specific Conceptual Design

These values are recommended for preliminary design purposes. They should be further refined as more data from laboratory tests, large-scale in-situ tests, and construction/ monitoring observations become available.

3.7.4 Anisotropy

The above discussion of the deformation modulus of the rock mass at the SSC site assumes linear and isotropic behavior. Available deformation and strength data on intact cores generally show that E_{50} and the strength of intact cores from vertical borings are generally slightly higher than the corresponding values from inclined borings, suggesting that the stiffest direction is perpendicular to bedding. However, available data are not sufficient to determine the extent of anisotropy. Further laboratory testing on large specimens cored at various orientations as well as large-scale, in situ tests will be necessary for this purpose.

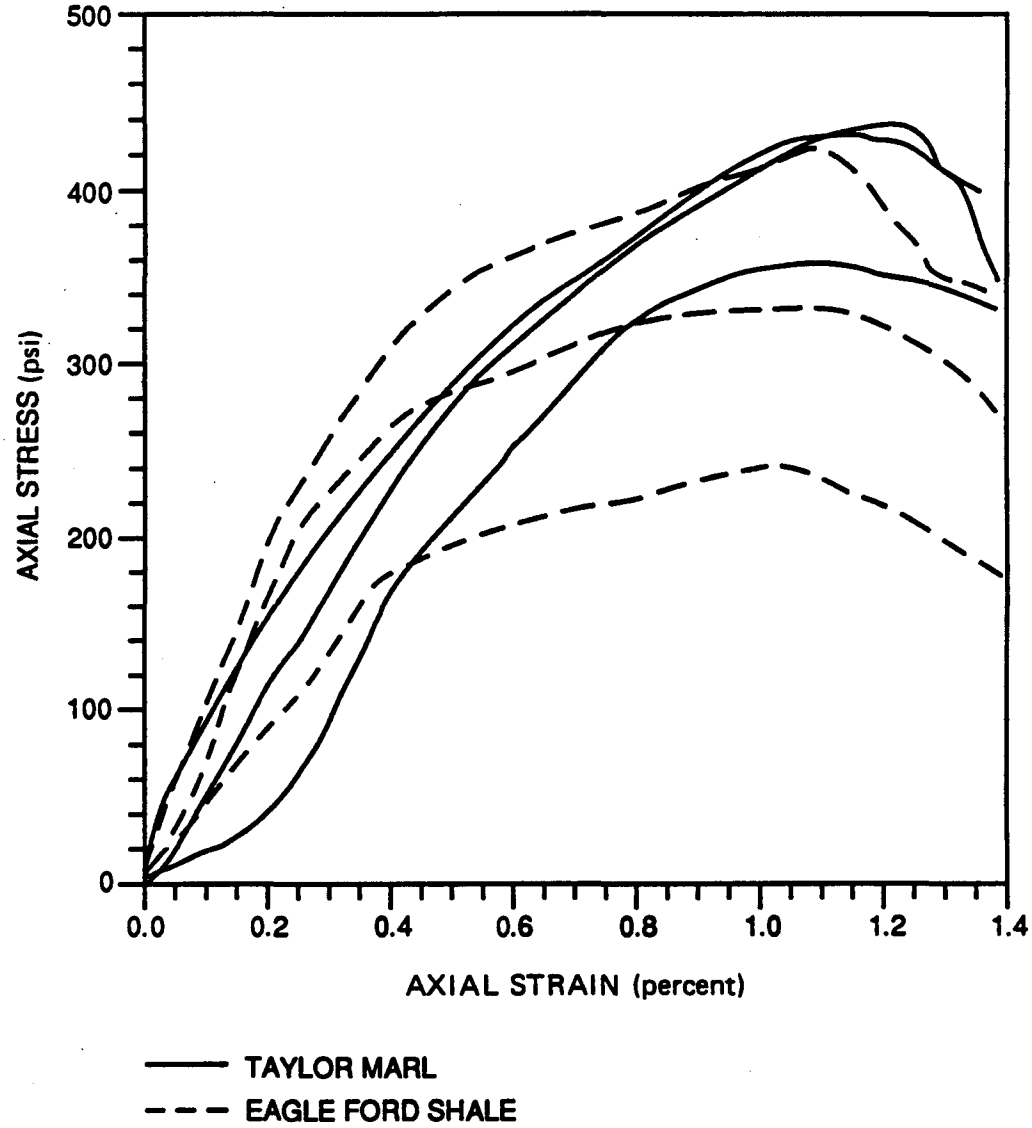
3.7.5 Nonlinearity

Typical stress-strain curves from UC and UU tests are presented in Figures 14 and 15. As can be noted from these figures, these curves are somewhat nonlinear. This nonlinearity is also evidenced by the fact that the reloading modulus values are generally about 1.5 to 2.5 times higher than E_{50}. The extent of nonlinearity cannot be estimated and in situ testing will be required so that a better estimate can be provided.

3.7.6 Time Dependency

The rock mass at the SSC site may undergo time-dependent creep deformation. This may arise from drained creep (secondary consolidation) or undrained creep deformations resulting from sustained shear stresses under undrained conditions.

Based on the results of index property, mineralogical evaluations, and known material property behavior, creep deformation concerns in AC should be negligibly small. However, time-dependent deformation in TM and EFS may be of concern. Undrained creep deformation associated with the construction and operation of structures founded on EFS and TM should be of particular concern. Because available consolidation data indicate small primary consolidation rebound, the potential for significant secondary consolidation (drained creep) may be even smaller in the absence of shear stresses. To increase the level of confidence in the design of facilities founded on TM or EFS, the extent of potential drained creep deformation may require further investigation in the presence of excavation and facility-loading induced stresses.



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Typical Stress - Strain Curves - UC Test

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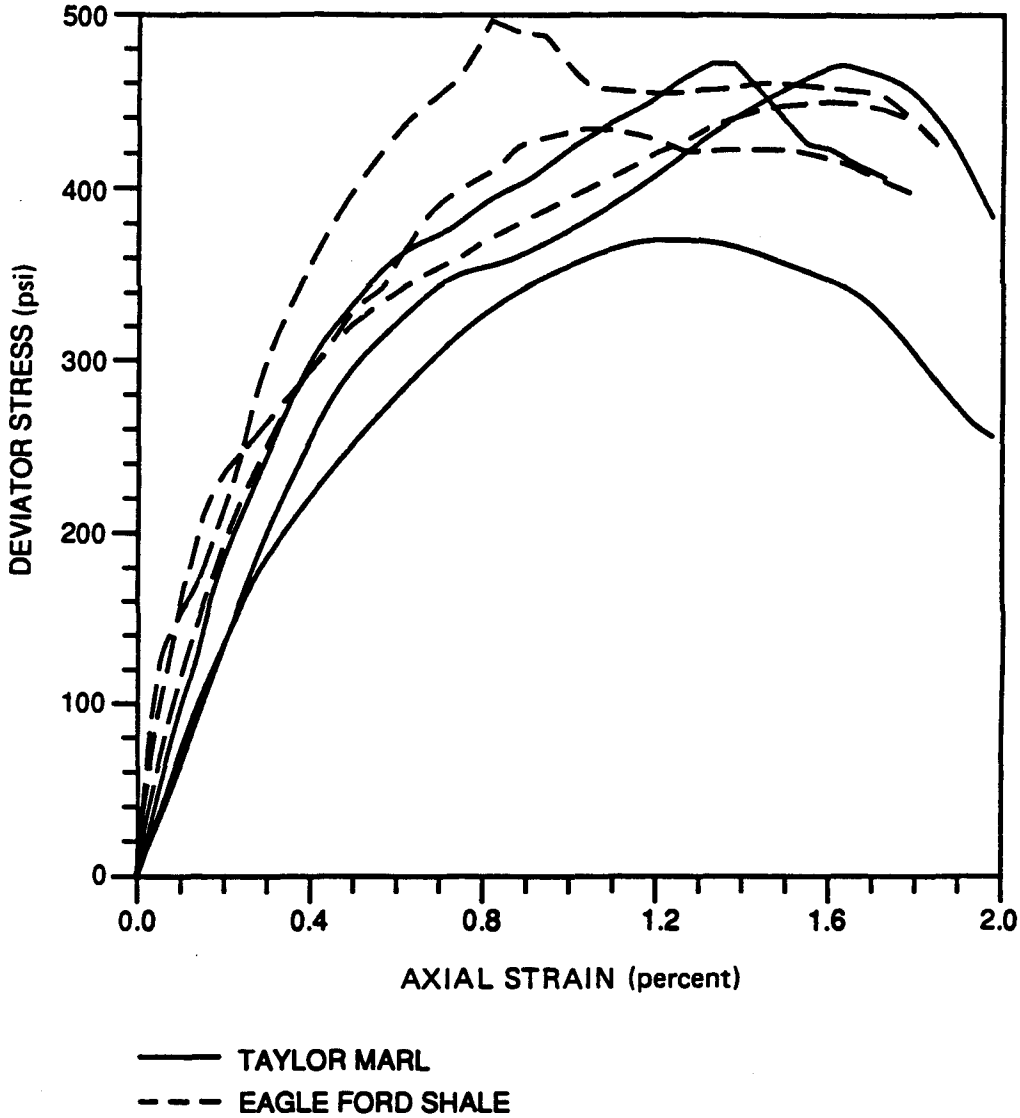
Figure 14



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Typical Stress - Strain Curves - UU Test

12/90

Figure 15

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

Geotechnical parameters for the three rock units (TM, AC, and EFS) in the SSC construction zone have been developed to support the site specific conceptual design of the SSC facilities. These geotechnical parameters were generated based on synthesis of available data, engineering interpretation, and engineering judgement and may be considered sufficient for site specific conceptual design at the SSC site. Based on the results of this evaluation, the following conclusions can also be made:

- The three rock units at the SSC site are classified as very low strength rocks.
- In terms of stability, standup time, or deformability characteristics for SSC underground facilities, AC can be classified as good quality rock and is significantly better than TM and EFS which are relatively poor quality rock because of their relatively high slaking and swelling potential and their comparatively low strength.
- The available data, especially laboratory test data, exhibit considerable scatter. Such scatter necessitates significant engineering judgement and postulation to reconcile laboratory data with field data, and to reconcile seeming contradictions between laboratory data and observed behavior.
- EFS and TM can be defined as soil-like rocks as defined by Goodman (1990) and behave neither completely like soils nor like rocks. Engineering data bases and experience with such subsurface materials are limited.

Various uncertainties include, but are not limited to, lack of site-specific data and design and construction experience, issues such as sample size effects and sample disturbance, lack of certain data such as in-situ stress conditions, the extent and potential effects of anisotropy, and nonlinearity and time-dependency (creep deformation) or long-term (drained) behavior of the in situ rock units. Accordingly, the geotechnical parameters developed in this report are intended for site-specific conceptual design purposes. They are not sufficient for final design and construction.

4.2 RECOMMENDATIONS

A number of uncertainties have been identified in the geotechnical parameters. Further investigation will be necessary to alleviate these uncertainties and to provide needed data and an appropriate level of confidence to support the final design and construction of the planned SSC facilities. The following additional studies are recommended:

- Available geotechnical parameters are based on widely-spaced borings and show a broad range of variations. There is need of a more detailed (closely spaced) local characterization for the design and construction of any specific facility component.

- Certain laboratory tests should be performed specifically to alleviate uncertainties in laboratory data, including long-term effective strength and deformation tests, time-dependent creep (drained and undrained conditions) characteristics, swell characterization and other index properties such as shrinkage limits, and activity index.

For strength and deformation properties, relevant issues that require implementation include:

- Obtain large-size (larger diameter) samples to minimize size and sample disturbance effects
- Develop appropriate laboratory test procedures considering the long term as well as material-specific nature of TM and EFS (i.e., high swell/slaking potential, low permeability, and soil-like rock nature).

- Appropriate ways and means of determining in situ stress conditions at the SSC site should be identified and field tests performed in the vicinity of planned structures; performance may be impacted by the presence of high horizontal in situ stresses.
- Site-specific field testing such as flat jack tests, plate load tests, and pressuremeter tests, and performance monitoring such as the work proposed for the large-diameter drill hole (Report No. SSC-GR-72) will be necessary to provide a data base for verification or modification of the geotechnical parameters. Use of performance monitoring data should also include backfitting as well as analyses to evaluate the behavior of in situ rock masses.
- Geotechnical parameter needs and facility design/construction are typically interactive. Additional geotechnical parameters that are not addressed herein may be identified and needed during the course of design. Provisions should be made to address such needs as they arise during the course of design and construction of the SSC facilities.

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GR-66	Geomechanical Characterization of the Eagle Ford Shale at the Superconducting Super Collider Site, 1990
GR-67	Geomechanical Characterization of the Austin Chalk at the Superconducting Super Collider Site, 1990
GR-68	Geomechanical Characterization of the Taylor Marl at the Superconducting Super Collider Site, 1990
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GR-72	Plan,, Schedule, and Cost Estimate for Large-Diameter Drilled Hole (RTK-produced)
GR-73	Seismic Reflection Investigation,, Superconducting Super Collider Ring near Waxahachie, TX (LCT-produced)
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APPENDIX

SUMMARIES OF GEOMECHANICAL TEST RESULTS

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GEOMECHANICAL TEST RESULTS

FOR TAYLOR MARL

TABLE A-1 SUMMARY OF GEOMECHANICAL TEST RESULTS FOR TAYLOR MARL BY COMMERCIAL LABORATORIES

	MOISTURE CONTENT	DRY DENSITY	TOTAL DENSITY	SPECIFIC GRAVITY	ATTERBERG LIMITS		FRACTION FINER THAN	CARBONATE CONTENT	UNIAXIAL COMPRESSIVE STRENGTH	TRIAXIAL COMPRESSION		FAILURE STRAIN	TANGENT YOUNG'S MODULUS		POISSON'S RATIO	BRAZIL TENSILE STRENGTH	2ND CYCLE SLAKE DURABILITY INDEX	SWELL PRESSURE INDEX
					LL	PI	# 200 SIEVE			CONFINING PRESSURE	DEVIATOR STRESS		E50	UNLOAD/ RELOAD				
	percent	pcf	pcf				percent	percent	psi	psi	psi	percent	(psi)x10E5	(psi)x10E5		psi	percent	psi

A. POST-FOOTPRINT PHASE-VERTICAL BORINGS

COUNT	300	217	217	48	111	111	90	104	56	50	50	106	63	11	37	11	16	43
MINIMUM	10.5	102.5	123.9	2.65	54	27	91.0	13.2	77	50	285	0.49	0.10	0.62	0.10	57	14.0	6.5
MAXIMUM	23.1	129.2	142.8	2.73	113	77	99.8	34.4	746	500	1147	2.48	1.96	3.72	0.56	176	86.6	57.9
MEAN	16.8	114.3	133.6	2.71	81	51	98.1	24.1	409		558	1.22	0.67	1.71	0.32	103	52.8	20.4
STD. DEV.	2.0	4.4	3.3	0.02	14	11	1.3	4.7	160		190	0.36	0.39	0.79	0.12	32	18.4	11.5

B. POST-FOOTPRINT PHASE-INCLINED BORINGS

COUNT	31	24	24	4	5	5	4	7	7	8	8	15	15	0	1	3	0	6
MINIMUM	14.1	107.5	127.9	2.69	52	28	98.2	17.1	150	50	398	0.33	0.31			41		10.6
MAXIMUM	21.1	119.4	137.1	2.73	98	64	99.6	31.1	370	500	760	0.80	2.39			67		22.6
MEAN	17.1	114.6	134.1	2.71	75	47	99.3	25.3	250		515	0.54	0.96		0.35	50.7		16.3
STD. DEV.	1.8	3.6	2.4	0.02	15	12	0.6	4.6	79		127	0.13	0.58					4.7

C. ALL BORINGS - TAYLOR MARL

COUNT	331	241	241	52	116	116	94	111	63	58	58	121	98	11	38	14	18	49
MINIMUM	10.5	102.5	123.9	2.65	52	27	91.0	13.2	77	50	285	0.33	0.10	0.62	0.10	41	14.0	6.5
MAXIMUM	23.1	129.2	142.8	2.73	113	77	99.8	34.4	746	500	1147	2.46	2.39	3.72	0.56	176	86.6	57.9
MEAN	16.9	114.3	133.7	2.71	81	51	98.1	24.2	391		552	1.14	0.71	1.71	0.32	92	52.6	19.9
STD. DEV.	1.9	4.3	3.2	0.02	14	11	1.3	4.7	161		183	0.41	0.44	0.79	0.12	36	18.4	10.9

D. SUMMARY OF SELECTED TESTS FOR STRENGTH DEFORMATION CHARACTERISTICS OF INTACT ROCK (1)

COUNT	24	24	24	2	9	9	8	9	7	17	17	24	18	6	13	0	0	0
MINIMUM	13.6	107.7	124.7	2.69	69	44	96.5	16.2	220	100	337	0.70	0.38	0.62	0.10			
MAXIMUM	19.8	122.4	139.0	2.70	104	77	98.6	28.0	648	500	647	2.46	1.67	3.72	0.49			
MEAN	17.0	114.1	133.5	2.70	64	56	97.9	22.3	419		520	1.29	0.76	1.67	0.31			
STD. DEV.	1.6	3.9	3.2		11	10	0.6	4.1	160		153	0.42	0.39	0.99	0.12			

E. SUMMARY OF SELECTED TESTS FOR STRENGTH DEFORMATION CHARACTERISTICS OF ROCK SAMPLES WITH DISCONTINUITIES (2)

COUNT	14	14	14	1	3	3	3	3	7	7	7	14	13	0	1	0	0	0
MINIMUM	14.6	110.3	131.3		75	47	98.2	21.0	150	100	398	0.33	0.31					
MAXIMUM	19.0	119.4	137.0		98	64	99.6	31.1	435	500	780	2.05	2.39					
MEAN	16.4	115.6	134.7	2.73	85	55	99.1	26.9	284		521	0.64	1.00		0.35			
STD. DEV.	1.5	2.6	1.5						98		135	0.41	0.61					

NOTES: (1)- These exclude test results of samples with failure along bedding planes or pre-existing fractures.
(2)- These include only the test results of samples with failure along bedding planes or pre-existing fractures.

TABLE A-2 MINERALOGICAL ANALYSES OF TAYLOR MARL BY MJA/SMU

TAYLOR MARL

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
BE 6	82.1	68			13				19	58	26			16
BE 6	82.3	0			2				98	7	16			77
BE 7A	81.9	33			13				54	46	21			33
BE 7A	82.1	0			3				97	9	19			72
BE 7A	280.6	0			2				98	5	16			79
BE 7A	280.8	76			9				15	33	33			34
BE 8	75.1	0			5				95	24	6			70
BE 8	75.6	19			24				57	55	12			33
BE 8	100.0	0			2				98	27	11			62
BIR 83	29.3	24			0				76	35	24			41
BIR 83	29.6	10			0				90	35	18			47
COUNT	11	11			11				11	11	11			11
MINIMUM		0			0				15	5	6			16
MAXIMUM		76			24				98	58	33			79
MEAN		20.9			6.6				72.5	30.4	18.4			51.3
STANDARD DEVIATION		26.5			7.1				30.3	17.5	7.2			20.6

TABLE A-3 MINERALOGICAL ANALYSES OF TAYLOR MARL BY SWL/CL

TAYLOR MARL

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
B 2758	152.5								39	13	10	4	73	
B 802	100.6								48	33	8	0	59	
BE 5	115.6	27	1	1	29	1	1	2	38	13	21	8	58	
BE 8	128.4	13	2	2	23	1	3	1	55	6	19	6	69	
BF 6	181.2	21	4	4	4	6	2	4	55	20	7	4	69	
BF 6	182.0	18	5	6	16	5	4	2	44	16	11	2	71	
BF 7	179.1	18	2	3	6	2	12	1	56	18	5	4	73	
BF 8	55.9	11	2	2	28	1	16	1	39	6	10	0	84	
BIR 51	88.1	16	2	2	26	1	3	3	47	13	15	8	64	
BIR 51	162.5	22	2	3	32	3	6	1	31	10	35	0	55	
BIR 52	131.9	21	5	3	36	3	8	2	22	14	50	0	36	
BIR 54	20.1	23	3	3	32	4	4	3	28	18	14	0	68	
BIR 54	117.1	24	1	5	30	3	0	2	35	11	17	0	72	
BIR 54	166.5								41	10	15	0	75	
BIR 84	63.0	25	1	0	36	2	6	1	29	10	21	0	69	
COUNT	15	12	12	12	12	12	12	12	15	15	15	15	15	
MINIMUM		11	1	0	4	1	0	1	22	6	5	0	36	
MAXIMUM		27	5	6	36	6	16	4	56	33	50	8	84	
MEAN		19.9	2.5	2.8	24.8	2.7	5.4	1.9	40.5	14.1	17.2	2.4	66.3	
STANDARD DEVIATION		4.6	1.4	1.6	10.3	1.6	4.5	1.0	10.1	6.4	11.3	2.9	10.7	

GEOMECHANICAL TEST RESULTS

FOR AUSTIN CHALK

TABLE A-4 SUMMARY OF GEOMECHANICAL TEST RESULTS FOR AUSTIN CHALK BY COMMERCIAL LABORATORIES

	MOISTURE CONTENT	DRY DENSITY	TOTAL DENSITY	SPECIFIC GRAVITY	UNIAXIAL COMPRESSIVE STRENGTH	TRIAXIAL COMPRESSION CONFINING PRESSURE	TRIAXIAL COMPRESSION DEVIATOR STRESS	FAILURE STRAIN	TANGENT YOUNG'S MODULUS E50	TANGENT YOUNG'S MODULUS UNLOAD/ RELOAD	POISSON'S RATIO	BRAZIL TENSILE STRENGTH	2ND CYCLE SLAKE DURABILITY INDEX	MODIFIED TABER ABRASION INDEX
	percent	pcf	pcf		psi	psi	psi	percent	(psi)x10E5	(psi)x10E5		psi	percent	
A. FOOTPRINT PHASE-VERTICAL BORINGS														
COUNT	29	27	27	0	16	10	10	8	7	0	3	14	2	2
MINIMUM	10	110.7	128.5		1224	11	169	0.37	0.89		0.06	135	94.1	0.263
MAXIMUM	19.3	131.9	148.1		3790	1000	3069	1.70	3.60		0.32	344	96.2	0.393
MEAN	14.2	121.0	137.9		1977	218	1491	0.90	2.46		0.18	244	95.2	0.328
STD. DEV.	2.2	5.3	4.7		655	278	857	0.48	0.80			60		
B. FOOTPRINT PHASE-INCLINED BORINGS														
COUNT	7	6	6	0	5	0	0	0	1	0	1	4	1	1
MINIMUM	11.9	113.3	132.2		1129							154		
MAXIMUM	16.7	128.8	144.1		1724							277		
MEAN	13.6	120.4	137.0		1451				4.10		0.12	218	93.4	0.425
STD. DEV.	1.5	4.9	4.1		195							44		
C. POST FOOTPRINT PHASE-VERTICAL BORINGS														
COUNT	911	608	608	64	269	142	142	408	380	22	90	122	11	43
MINIMUM	6.9	100	119.1	2.62	306	20	329	0.21	0.13	2.01	0.07	51	88.4	0.272
MAXIMUM	24.1	143.5	159.9	2.71	3987	500	4258	2.53	22.73	15.95	0.48	450	98.0	1.137
MEAN	12.7	124.4	139.8	2.67	2131		2492	0.81	4.25	7.27	0.20	247	96.3	0.542
STD. DEV.	2.5	6.8	4.9	0.02	752		742	0.33	2.55	3.95	0.09	65	2.6	0.198
D. POST FOOTPRINT PHASE-INCLINED BORINGS														
COUNT	88	72	72	5	26	14	14	40	41	0	4	11	0	1
MINIMUM	7.8	96.8	113.6	2.65	350	50	819	0.15	2.06		0.11	134		
MAXIMUM	25.4	137.7	148.4	2.68	3346	300	3632	1.02	9.52		0.21	397		
MEAN	13.0	122.9	138.6	2.67	1647		2196	0.56	4.28		0.16	216		0.588
STD. DEV.	2.9	8.4	6.1	0.01	792		763	0.22	1.84		0.04	85		
E. ALL BORINGS - AUSTIN CHALK														
COUNT	1035	713	713	69	316	166	166	456	429	22	98	151	14	47
MINIMUM	6.9	96.8	113.6	2.62	306	11	169	0.15	0.13	2.01	0.06	51	88.4	0.263
MAXIMUM	25.4	143.5	159.9	2.71	3987	1000	4258	2.53	22.73	15.95	0.48	450	98.0	1.137
MEAN	12.7	124.1	139.6	2.67	2073	183	2407	0.79	4.22	7.27	0.19	244	95.9	0.532
STD. DEV.	2.5	6.9	5.0	0.02	761	135	790	0.33	2.48	3.95	0.09	67	2.5	0.195

TABLE A-5 MINERALOGICAL ANALYSES OF AUSTIN CHALK BY MJA/SMU

AUSTIN CHALK

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
BE 9	153.5	2			98				0					0
BI 2A	65.9	0			95				5					0
BIR 44	3.0	0			100				0					0
SE 1.8	51.9	0			95				5					100
SI 2C	187.2	0			100				0					0
COUNT	5	5			5				5					5
MINIMUM		0			95				0					0
MAXIMUM		2			100				5					100
MEAN		0.4			97.6				2.0					20.0
STANDARD DEVIATION		0.8			2.2				2.4					40.0

BENTONITIC SHALE

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
BI 2A	66.2				5				95	27	5			68
BI 2A	97.4				10				90	28	13			59

TABLE A-6 MINERALOGICAL ANALYSES OF AUSTIN CHALK BY SWL/CL

AUSTIN CHALK

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
B 1533	6.6	2		0	97	0		0	1					
B 1540	8.3	1		0	99	0		0	0					
BF 1	128.6	3		1	95	0		1	0					
BI 2A	87.6	3		0	95	0		2	0					
BI 3	127.3	3		0	93	0		2	2					
BIR 11	52.4	2		0	97	0		1	0					
BIR 11	152.5	1		0	98	0		1	0					
BIR 21	42.2	4		0	93	1		2	0					
BIR 21	152.8	2		0	97	0		1	0					
BIR 23	103.4	4		1	93	0		2	0					
BIR 31	126.6	2		0	97	0		1	0					
BIR 33	37.4	1		0	98	0		1	0					
BIR 41	66.0	1		0	98	0		1	0					
BIR 44	3.3	3		0	96	0		0	1					
BIR 52	190.4	6		0	87	0		1	6					
COUNT	15	15		15	15	15		15	15					
MINIMUM		1		0	87	0		0	0					
MAXIMUM		6		1	99	1		2	6					
MEAN		2.5		0.1	95.5	0.1		1.1	0.7					
STANDARD DEVIATION		1.4		0.3	3.0	0.2		0.7	1.5					

SHALY LIMESTONE

B 1296	95.5								32				100	
B 1316	114.2								43	9			91	
BI 1	110.8								28	14			86	

ARGILLACEOUS LIMESTONE

B 1316	113.4								10	41			59	
BE 5	319	6			80			1	13	15			85	

CALCITIC SHALE

B 2052	190.4				47			3	50	14			86	

GEOMECHANICAL TEST RESULTS

FOR EAGLE FORD SHALE

TABLE A-7 SUMMARY OF GEOMECHANICAL TEST RESULTS FOR EAGLE FORD SHALE BY COMMERCIAL LABORATORIES

	MOISTURE CONTENT	DRY DENSITY	TOTAL DENSITY	SPECIFIC GRAVITY	ATTERBERG LIMITS		FRACTION FINER THAN	CARBONATE CONTENT	UNIAXIAL COMPRESSIVE STRENGTH	TRIAXIAL COMPRESSION		FAILURE STRAIN	TANGENT YOUNG'S MODULUS		POISSON'S RATIO	BRAZIL TENSILE STRENGTH	2ND CYCLE SLAKE DURABILITY INDEX	MODIFIED TABER ABRASION INDEX	SWELL PRESSURE INDEX
					LL	PI	# 200 SIEVE			CONFINING PRESSURE	DEVIATOR STRESS		E50	UNLOAD/ RELOAD					
	percent	pcf	pcf				percent	percent	psi	psi	psi	percent	(psi)x10E5	(psi)x10E5		psi	percent		psi

A. FOOTPRINT PHASE-VERTICAL BORINGS

COUNT	74	72	72	1	0	0	0	1	35	22	22	14	12	0	7	6	11	3	24
MINIMUM	11.2	108.9	128.7						127	50	114	0.43	0.13		0.04	31	0.4	0.209	22.0
MAXIMUM	19.8	133.5	150.4						792	1200	811	1.90	2.95		0.48	140	38.0	0.690	115.0
MEAN	16.5	117.1	136.4	2.71				7.4	326		405	1.07	0.80		0.24	80	12.6	0.441	41.5
STD. DEV.	1.3	3.9	3.6						136		199	0.47	0.70		0.18	38	10.2		21.3

B. FOOTPRINT PHASE-INCLINED BORINGS

COUNT	12	11	11	0	0	0	0	0	2	6	6	3	2	0	2	0	3	3	3
MINIMUM	15.8	111.5	130.1						85	80	94	1.70	0.60		0.09		9.0	0.353	34.0
MAXIMUM	18.8	115.9	134.8						105	1350	728	2.87	0.70		0.23		18.4	0.678	64.0
MEAN	16.9	113.3	132.5						95		333	2.18	0.65		0.16		13.0	0.516	48.7
STD. DEV.	1.2	1.3	1.7								225								

C. POST-FOOTPRINT PHASE-VERTICAL BORINGS

COUNT	460	332	332	50	123	123	100	112	72	75	75	147	134	19	39	1	39	4	101
MINIMUM	6.2	105.4	126.3	2.57	39	21	64.7	1.0	64	50	163	0.15	0.03	0.62	0.10		4.9	0.539	7.7
MAXIMUM	23.0	139.3	150.0	2.79	119	82	100.0	54.1	844	600	850	4.51	1.54	4.20	0.68		51.9	1.452	86.9
MEAN	16.0	117.5	136.3	2.72	88	56	97.0	8.7	297		396	0.86	0.79	1.60	0.31	336	24.7	0.816	35.6
STD. DEV.	1.9	4.3	3.2	0.03	17	13	5.4	8.7	137		126	0.51	0.33	0.85	0.15		10.5	0.372	15.7

D. POST-FOOTPRINT PHASE-INCLINED BORINGS

COUNT	88	69	69	7	26	26	19	25	14	12	12	26	26	0	3	0	6	0	21
MINIMUM	12.2	101.2	118.0	2.64	39	21	83.8	4.0	9	50	156	0.08	0.10		0.20		7.2		15.1
MAXIMUM	21.8	131.9	148.0	2.73	89	65	99.7	34.6	239	500	706	1.38	1.33		0.35		32.7		48.5
MEAN	18.2	116.7	135.5	2.70	68	42	98.0	8.2	125		324	0.41	0.71		0.26		21.3		28.5
STD. DEV.	1.2	5.0	4.9	0.03	13	11	3.4	6.3	61		149	0.25	0.31				8.1		7.6

TABLE A-7 SUMMARY OF GEOMECHANICAL TEST RESULTS FOR EAGLE FORD SHALE BY COMMERCIAL LABORATORIES (CONT.)

	MOISTURE CONTENT	DRY DENSITY	TOTAL DENSITY	SPECIFIC GRAVITY	ATTERBERG LIMITS		FRACTION FINER THAN	CARBONATE CONTENT	UNIAXIAL COMPRESSIVE STRENGTH	TRIAXIAL COMPRESSION		FAILURE STRAIN	TANGENT YOUNG'S MODULUS		POISSON'S RATIO	BRAZIL TENSILE STRENGTH	2ND CYCLE SLAKE DURABILITY INDEX	MODIFIED TABER ABRASION INDEX	SWELL PRESSURE INDEX
					LL	PI	# 200 SIEVE			CONFINING PRESSURE	DEVIATOR STRESS		E50	UNLOAD/ RELOAD					
	percent	pcf	pcf				percent	percent	psi	psi	psi	percent	(psi)x10E5	(psi)x10E5		psi	percent		psi

E. ALL BORINGS - EAGLE FORD SHALE

COUNT	634	484	484	58	149	149	119	138	123	115	115	190	174	19	51	7	59	10	149
MINIMUM	6.2	101.2	118.0	2.57	39	21	64.7	1.0	9	50	94	0.08	0.03	0.62	0.04	31	0.4	0.209	7.7
MAXIMUM	23.0	139.3	150.4	2.79	119	82	100.0	54.1	844	1350	850	4.51	2.95	4.20	0.88	336	51.9	1.452	115.0
MEAN	16.1	117.2	136.1	2.72	84	54	97.2	8.6	283		387	0.83	0.78	1.80	0.29	117	21.5	0.614	35.8
STD. DEV.	1.7	4.4	3.6	0.03	18	14	5.2	8.3	144		154	0.54	0.36	0.85	0.15	96	11.2	0.317	16.4

F. SUMMARY OF SELECTED TESTS FOR STRENGTH DEFORMATION CHARACTERISTICS OF INTACT ROCK (1)

COUNT	56	56	56	10	20	20	16	16	17	39	39	56	52	10	29	0	0	0	0
MINIMUM	7.7	108.9	128.7	2.57	54	30	92.4	3.8	114	50	114	0.18	0.11	0.90	0.10				
MAXIMUM	18.7	139.3	150.0	2.74	112	75	99.8	45.7	844	600	850	1.69	1.54	4.20	0.88				
MEAN	15.5	118.4	136.6	2.70	85	54	97.3	10.4	373		383	0.71	0.89	2.03	0.32				
STD. DEV.	2.3	5.7	3.8	0.05	17	13	2.6	11.1	201		153	0.36	0.32	0.95	0.16				

G. SUMMARY OF SELECTED TESTS FOR STRENGTH DEFORMATION CHARACTERISTICS OF ROCK SAMPLES WITH DISCONTINUITIES (2)

COUNT	28	28	28	3	5	5	5	6	16	12	12	28	27	0	2	0	0	0	0
MINIMUM	10.5	104.6	120.7	2.72	62	36	99.0	4.0	9	50	156	0.08	0.03		0.23				
MAXIMUM	19.4	131.3	145.1	2.74	116	77	99.9	12.1	240	500	706	1.61	1.33		0.35				
MEAN	15.7	117.6	136.0	2.73	85	53	99.5	7.9	119		324	0.51	0.65		0.29				
STD. DEV.	1.5	4.5	4.1		21	16	0.4	3.0	62		149	0.40	0.36						

NOTES: (1) - These exclude test results of samples with failure along bedding planes or pre-existing fractures.
(2) - These include only the test results of samples with failure along bedding planes or pre-existing fractures.

TABLE A-8 MINERALOGICAL ANALYSIS OF EAGLE FORD SHALE BY MJA/SMU

EAGLE FORD SHALE

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
B 1737	188.6									14	25			61
B 1807	163.2									14	17			69
B 2290	185.2									13	18			69
BE 1-90	225.5	31	0	0	0	0	0	0	69	43	25			32
BE 1-90	225.6	0	0	0	0	0	0	0	100	30	27			43
BE 2	184.8	26	0	0	0	0	0	0	74	17	29			54
BI 2A	236.9	0	0	0	0	0	0	0	100	54	16			30
BIR 13	259.8	18	0	0	0	0	0	0	82	33	35			32
BIR 13	260.3	0	0	0	0	0	0	0	100	26	25			49
BIR 14	282.9	0	0	0	0	0	0	0	100	34	48			18
BIR 32	232.9	22	0	0	8	0	0	3	67	39	27			34
BIR 32	233.1	16	0	0	10	0	0	2	72	43	31			26
BIR 32	259.4	10	0	0	0	0	0	0	90	38	20			42
BIR 32	259.7	27	0	0	0	0	0	2	71	44	18			38
BIR 42	249.8	9	0	0	0	0	0	2	89	49	15			36
BIR 42	250.1	0	0	0	0	0	0	0	100	40	20			40
BIR 42	271.5	5	0	0	0	0	0	0	95	33	27			40
BIR 42	272.1	18	0	0	0	0	0	0	82	45	22			33
SE 1.8	155.3	11	0	0	0	0	0	3	86	44	19			37
SE 1.8	195.9	13	0	0	0	0	0	2	85	45	22			33
SE 1.8	196.1	12	0	0	0	0	0	1	87	49	30			21
SF 10.1C	270.8									34	22			44
SIR 3B	302.0									25	48			27
COUNT	23	18	18	18	18	18	18	18	18	23	23	0	0	23
MINIMUM		0.0			0.0			0.0	67.0	13.0	15.0			18.0
MAXIMUM		31.0			10.0			3.0	100.0	54.0	48.0			69.0
MEAN		12.1			1.0			0.8	86.1	35.0	25.5			39.5
STAND. DEVIATION		9.9			2.8			1.1	11.4	11.8	8.6			13.2

TABLE A-9 MINERALOGICAL ANALYSIS OF EAGLE FORD SHALE BY SWL/CL

EAGLE FORD SHALE

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
B 1527	178.9								73	33	10	4	53	
B 1540	198.0								60	28	8	1	63	
BE 1-90	210.3	19	3	3	1	0	16	6	52					
BF 1	193.5	20	6	2	1	0	0	1	70	35	7	4	54	
BF 2	201.8	15	5	2	0	0	0	2	76	21	14	0	65	
BI 1	187.5	20	3	1	2	0	0	5	69	30	36	0	34	
BI 6	222.6	26	5	3	0	0	26	2	38	32	26	3	39	
BIR 11	225.0	27	3	1	2	0	0	2	65	6	18	2	74	
BIR 11	245.6	4	3	3	0	0	1	1	88	10	33	0	57	
BIR 11	261.1	19	3	2	1	0	0	4	71	21	23	0	56	
BIR 13	293.9								80	19	11	0	70	
BIR 14	253.1								70	54	16	3	27	
BIR 14	295.3								62	45	13	0	42	
BIR 23	216.1								88	12	26	0	62	
BIR 23	226.9	26	3	2	0	0	3	3	63	14	22	0	64	
BIR 31	171.6	14	11	0	7	0	4	4	60	22	10	1	67	
BIR 31	242.2								82	37	16	0	47	
BIR 33	176.5	20	6	1	2	0	0	6	65	23	35	0	42	
BIR 33	190.0								65	12	39	0	49	
BIR 41	205.0								56	30	14	1	55	
BIR 43	197.7								61	18	12	1	69	
BIR 43	218.6								74	35	20	0	45	
BIR 45	179.8								55	15	3	26	56	
SE 10.9C	296.6	29	0	1	1	0	5	2	62	26	26	0	48	
SE 10.9C	306.0	13	0	1	5	0	1	2	78	28	33	0	39	
SE 10.9C	316.5	18	1	4	0	0	2	5	70	26	27	0	47	
SE 10.9C	329.9	11	0	1	0	0	1	3	84	33	26	0	41	
SE 10.9C	335.4	33	0	1	3	0	2	4	57	42	44	0	14	
SE 10.9C	340.1	24	1	1	3	0	0	2	69	49	32	0	19	
SF 10.1C	277.3	13	2	1	1	0	1	1	81	26	26	0	48	
SF 10.1C	283.9	5	5	2	0	0	0	2	86	28	20	0	52	
SF 10.1C	296.5	24	5	5	2	0	4	3	57	21	44	0	35	
SF 10.1C	305.0	7	1	1	0	0	0	3	88	26	28	0	46	

TABLE A-9 MINERALOGICAL ANALYSIS OF EAGLE FORD SHALE BY SWL/CL (CONT.)

EAGLE FORD SHALE

BORING NUMBER	BORING DEPTH	BULK MINERALOGICAL COMPOSITION								CLAY MINERALOGICAL COMPOSITION				
		quartz	K-feldspar	plagioclase	calcite	dolomite	siderite	pyrite	clay minerals	illite	kaolinite	Fe-chlorite	illite/ smectite	montmori-llonite
	feet	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent	percent
SIR 3B	272.8	22	4	1	0	0	8	2	63	25	21	0	54	
SIR 3B	284.4	9	1	1	0	1	0	1	87	29	25	0	46	
COUNT	35	23	23	23	23	23	23	23	35	34	34	34	34	
MINIMUM		4.0	0.0	0.0	0.0	0.0	0.0	1.0	38.0	6.0	3.0	0.0	14.0	
MAXIMUM		33.0	11.0	5.0	7.0	1.0	26.0	6.0	88.0	54.0	44.0	26.0	74.0	
MEAN		18.2	3.1	1.7	1.3	0.0	3.2	2.9	69.3	26.8	22.5	1.4	49.4	
STAND. DEVIATION		7.7	2.6	1.2	1.8	0.2	6.0	1.5	11.8	10.7	10.4	4.4	13.7	

SUMMARY OF GEOMECHANICAL TEST RESULTS

BY UNIVERSITY OF TEXAS

TABLE A-10 SUMMARY OF GEOMECHANICAL TEST RESULTS BY UNIVERSITY OF TEXAS

BORING NO	GENERAL LITHOLOGY	BORING DEPTH	MOISTURE CONTENT	DRY DENSITY	SPECIFIC GRAVITY	ATTERBERG LIMITS		SWELL PRESSURE INDEX	SWELL INDEX		COMPRESSION INDEX		COEFFICIENT CONSOLIDATION		PERMEABILITY	
						LL	PI									
		feet	percent	pcf				psi					sft/day		(cm/sec)x10E-10	
									min	max	min	max	min	max	min	max
BE 6	TAYLOR MARL	34.3			2.81	66	37									
BE 8	TAYLOR MARL	34.5	20.7	111.1		63	36									
BE 8	TAYLOR MARL	34.7	24.3	105.9												
BE 8	TAYLOR MARL	34.9	20.2	109.5												
BE 8	TAYLOR MARL	35.0	19.8	110.2	2.87	66	37	118.1								
BE 6	TAYLOR MARL	35.2	20.8	109.4	2.87	69	40	20.8								
BE 6	TAYLOR MARL	35.4	19.7	108.9		67	40									
BE 6	TAYLOR MARL	173.0	17.6	117.3	2.87	77	52	76.4								
BIR 43	EAGLE FORD SHALE	182.7				88	58				0.001	0.123	0.012	0.019		
BIR 43	EAGLE FORD SHALE	182.7	17.1	114.6		90	61									
BIR 43	EAGLE FORD SHALE	182.8	17.5	112.6		82	49									
BIR 43	EAGLE FORD SHALE	182.9	17.5	113.7		88	51									
BIR 43	EAGLE FORD SHALE	183.0	19.0	112.6	2.78	82	52									
BIR 43	EAGLE FORD SHALE	183.3	17.6	112.8		82	53									
BIR 43	EAGLE FORD SHALE	183.4	17.7	114.1		87	56									
BIR 43	EAGLE FORD SHALE	183.5	18.8	113.1		86	54									
BIR 43	EAGLE FORD SHALE	183.8	18.9	112.0	2.78	88	58	69.4								
BIR 43	EAGLE FORD SHALE	189.0	16.1	120.1	2.78	75	46	62.5			0.000	0.098	0.001	0.005	1.0	4.5
BIR 43	EAGLE FORD SHALE	189.2	16.2	117.5		80	48									
BIR 43	EAGLE FORD SHALE	189.3				83	57									
BIR 43	EAGLE FORD SHALE	189.5	16.9	117.7	2.78	94	65									
BIR 43	EAGLE FORD SHALE	189.8	15.9	119.3		82	53									
BIR 43	EAGLE FORD SHALE	251.0				59	31				0.000	0.088	0.003	0.006		
BIR 43	EAGLE FORD SHALE	251.0				66	39				0.003	0.079	0.008	0.014	10.0	10.0
BIR 43	EAGLE FORD SHALE	251.0	18.7	110.8		64	40									
BIR 43	EAGLE FORD SHALE	251.4	18.3	115.0	2.78	59	31	118.1								
BIR 43	EAGLE FORD SHALE	251.5	16.0	116.1												
BIR 43	EAGLE FORD SHALE	251.6				63	34									
BIR 43	EAGLE FORD SHALE	251.7	15.7	118.3		64	36									
BIR 43	EAGLE FORD SHALE	252.0	15.8	120.0		67	37									
BIR 43	EAGLE FORD SHALE	252.2	14.0	112.0		63	39									
BIR 43	EAGLE FORD SHALE	252.4	16.1	118.8	2.78	66	39	118.1								
BIR 44	EAGLE FORD SHALE	227.5				64	39				0.001	0.085	0.001	0.005		
BIR 44	EAGLE FORD SHALE	227.5				73	45				0.000	0.088	0.001	0.003	1.5	2.5
BIR 44	EAGLE FORD SHALE	227.7	16.7	117.6		80	49									
BIR 44	EAGLE FORD SHALE	228.0	16.8	117.8	2.77	85	56									
BIR 44	EAGLE FORD SHALE	228.4	13.2	123.4	2.77	64	39	208.3								
BIR 44	EAGLE FORD SHALE	228.5				77	46									
BIR 44	EAGLE FORD SHALE	228.8	16.2	118.7		80	50									
BIR 44	EAGLE FORD SHALE	228.9	16.1	118.1												
BIR 44	EAGLE FORD SHALE	229.0	17.5	114.2	2.77	73	45	83.3								
SE 1-8	EAGLE FORD SHALE	197.1			2.62											
SE 1-8	EAGLE FORD SHALE	197.2	15.5	118.7	2.67	62	36	243.1								
SE 1-8	EAGLE FORD SHALE	197.4				62	36									
SE 1-8	EAGLE FORD SHALE	197.1				70	43				0.015	0.077				
SIR 3B	BENTONITE	191.0	36.8	83.3		120	70									
SIR 3B	BENTONITE	191.1	40.0	79.9		113	55									
SIR 3B	BENTONITE	191.3			2.87											
SIR 3B	BENTONITE	191.5	39.4	82.7		108	50									
SIR 3B	BENTONITE	191.7	38.0	85.1	2.87	110	57	76.4								

NOTES: 1. Coefficient of consolidation and recompression index values are over a stress range of 350 to 178000 psf.
See Table ---- for relevant parameters under insitu stress conditions.
2. Permeability test data were obtained over a void ratio range of 0.40-0.51.

SUMMARY OF GEOMECHANICAL TEST RESULTS

BY UNIVERSITY OF ILLINOIS

TABLE A-11 SUMMARY OF GEOMECHANICAL TEST RESULTS BY UNIVERSITY OF ILLINOIS

BORING NO	GENERAL LITHOLOGY	SAMPLE INTERVAL	MOISTURE CONTENT	DRY DENSITY	ATTERBERG LIMITS		TRIAXIAL COMPRESSION		TANGENT YOUNG'S MODULUS	SWELL PRESSURE INDEX	INITIAL VOID RATIO	SWELL INDEX		COMPRESSION INDEX		DIRECT SHEAR	
					LL	PI	CONFINING PRESSURE	DEVIATOR STRESS	E50			min	max	min	max	min	max
		feet	percent	pcf			psi	psi	(psi)x10E5	psi						degree	degree
BIR 43	BENTONITE	116.9 - 117.9														7	6
BIR 43	BENTONITE	116.9 - 117.9	39.6				200	155	0.12								
BIR 43	BENTONITE	116.9 - 117.9	39.6	81	200	152											
B 1533	EAGLE FORD SHALE	204.9 - 206.0	15.1	119.0			400	465	0.73								
B 1533	EAGLE FORD SHALE	204.9 - 206.0	14.9				200	448	0.64								
BF 1	EAGLE FORD SHALE	211.7 - 212.3	16.6							50.0	0.486						
BF 1	EAGLE FORD SHALE	211.7 - 212.3	16.3							27.8	0.526	0.03	0.03				
BF 1	EAGLE FORD SHALE	211.7 - 212.3	16.5		91	64											
BI 3	EAGLE FORD SHALE	264.4 - 265.6	16.8	113.0	121	92											
BI 3	EAGLE FORD SHALE	264.4 - 265.6	16.9							277.6	0.462						
BI 3	EAGLE FORD SHALE	264.4 - 265.6	16.6							111.1	0.522	0.04	0.04				
BI 4	EAGLE FORD SHALE	176.1 - 177.7	16.4	117.0	95	72	200	413	0.53								
BI 4	EAGLE FORD SHALE	176.1 - 177.7	16.6							72.2	0.462						
BI 4	EAGLE FORD SHALE	176.1 - 177.7	16.6							27.8	0.495	0.03	0.06	0.03	0.09		
BI 4	EAGLE FORD SHALE	176.1 - 177.7	16.4							58.3	0.463	0.03	0.06	0.02	0.07		
BI 6	EAGLE FORD SHALE	224.0 - 225.5														5	6
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.9	116.0	94	67											
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.6				200	425	0.56								
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.5				400	423	0.63								
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.9				100	424	0.63								
BI 6	EAGLE FORD SHALE	224.0 - 225.5														5	6
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.9				400	506	0.76								
BI 6	EAGLE FORD SHALE	224.0 - 225.5	16.5	116.0	93	68											
BIR 13	EAGLE FORD SHALE	216.8 - 217.6	18.4	106.0	126	87											
BIR 13	EAGLE FORD SHALE	216.8 - 217.6	18.4				200	196	0.41								
BIR 13	EAGLE FORD SHALE	240.9 - 242.4	16.6							16.7	0.522						
BIR 13	EAGLE FORD SHALE	240.9 - 242.4	17.8				200	237	0.36								
BIR 13	EAGLE FORD SHALE	240.9 - 242.4	17.1							12.5	0.532	0.04	0.04				
BIR 13	EAGLE FORD SHALE	240.9 - 242.4	17.8	115.0	102	75											
BIR 14	EAGLE FORD SHALE	217.0 217.8	17.8							27.8	0.526	0.03	0.03				
BIR 14	EAGLE FORD SHALE	217.0 217.8	17.1							11.1	0.625						
BIR 14	EAGLE FORD SHALE	292.0 - 293.0	15.1	119.0	86	60											

TABLE A-11 SUMMARY OF GEOMECHANICAL TEST RESULTS BY UNIVERSITY OF ILLINOIS (CONT.)

BORING NO	GENERAL LITHOLOGY	SAMPLE INTERVAL	MOISTURE CONTENT	DRY DENSITY	ATTERBERG LIMITS		TRIAXIAL COMPRESSION		TANGENT YOUNG'S MODULUS	SWELL PRESSURE INDEX	INITIAL VOID RATIO	SWELL INDEX		COMPRESSION INDEX		DIRECT SHEAR	
					LL	PI	CONFINING PRESSURE	DEVIATOR STRESS	E50			min	max	min	max	min	max
		feet	percent	pcf			psi	psi	(psi)x10E6	psi						degree	degree
BIR 14	EAGLE FORD SHALE	292.0 - 293.0	15.1				200	524	0.73								
BIR 41	EAGLE FORD SHALE	180.2 - 184.1	16.7	109.0	93	67											
BIR 41	EAGLE FORD SHALE	180.2 - 184.1	15.4							93.1	0.506						
BIR 41	EAGLE FORD SHALE	180.2 - 184.1	15.4	114.0	111	85											
BIR 41	EAGLE FORD SHALE	180.2 - 184.1	16.7							22.2	0.579	0.03	0.03				
BIR 43	EAGLE FORD SHALE	184.0 - 185.2	17.3							18.1	0.516	0.01	0.01				
BIR 43	EAGLE FORD SHALE	184.0 - 185.2	17.9							20.8	0.53						
BIR 43	EAGLE FORD SHALE	216.3 - 217.6	17.1							23.6	0.497	0.01	0.01				
BIR 43	EAGLE FORD SHALE	216.3 - 217.6	16.6							33.3	0.509						
BIR 43	EAGLE FORD SHALE	312.5 - 313.4	15.8				200	417	0.63								
BIR 43	EAGLE FORD SHALE	312.5 - 313.4	16.0				400	350	0.68								
BIR 43	EAGLE FORD SHALE	312.5 - 313.4	16.0	117.0	84	57											
BIR 44	EAGLE FORD SHALE	183.0 - 184.1	16.5	118.0	123	96											
BIR 44	EAGLE FORD SHALE	183.0 - 184.1	16.5				400	428	0.78								
BIR 44	EAGLE FORD SHALE	209.1 - 210.1	15.2							97.2	0.422	0.01	0.01				
BIR 44	EAGLE FORD SHALE	209.1 - 210.1	15.3							155.6	0.439						
BIR 44	EAGLE FORD SHALE	210.1 - 211.1	15.6	118.0	104	77											
BIR 44	EAGLE FORD SHALE	210.1 - 211.1	15.6				400	525	0.83								
BIR 44	EAGLE FORD SHALE	260.5 - 261.9	15.4							20.8	0.513	0.04	0.06	0.05	0.08		
BIR 45	EAGLE FORD SHALE	233.9 - 234.8	14.3	122.0	111	85											
BIR 45	EAGLE FORD SHALE	233.9 - 234.8	14.4				200	436	0.64								
BE 6	TAYLOR MARL	33.7 - 34.3	19.2				200	258	0.34								
BE 6	TAYLOR MARL	33.7 - 34.3	19.2	111	105	82											
BE 6	TAYLOR MARL	33.7 - 34.3	18.9							12.5	0.558						
BE 6	TAYLOR MARL	53.8 - 54.3	18.5				200	347	0.39								
BE 6	TAYLOR MARL	53.6 - 54.3	18.5	111	124	101											
BE 6	TAYLOR MARL	112.8 - 113.6	15.5							20.8	0.536						
BE 6	TAYLOR MARL	112.8 - 113.6	15.4							48.6	0.519	0.01	0.01				
BE 6	TAYLOR MARL	112.8 - 113.8	15.5							40.3	0.558	0.02	0.04	0.03	0.06		
BE 7A	TAYLOR MARL	82.3 - 83.0														4	4
BE 7A	TAYLOR MARL	82.3 - 83.0	17.7		112	90											
BE 7A	TAYLOR MARL	82.3 - 83.0														5	6
BE 7A	TAYLOR MARL	82.3 - 83.0	17.9				200	433	0.46								
BE 7A	TAYLOR MARL	82.3 - 83.0	17.7				200	419	0.51								
BE 7A	TAYLOR MARL	167.4 - 168.2	14.6		105	84											

TABLE A-11 SUMMARY OF GEOMECHANICAL TEST RESULTS BY UNIVERSITY OF ILLINOIS (CONT.)

BORING NO	GENERAL LITHOLOGY	SAMPLE INTERVAL	MOISTURE CONTENT	DRY DENSITY	ATTERBERG LIMITS		TRIAXIAL COMPRESSION		TANGENT YOUNG'S MODULUS E50	SWELL PRESSURE INDEX	INITIAL VOID RATIO	SWELL INDEX		COMPRESSION INDEX		DIRECT SHEAR	
					LL	PI	CONFINING PRESSURE	DEVIATOR STRESS				min	max	min	max	min	max
		feet	percent	pcf			psi	psi	(psi)x10E5	psi						degree	degree
BE 7A	TAYLOR MARL	206.6 - 207.8	14.2							54.2	0.509						
BF 7	TAYLOR MARL	169.4 - 170.9	16.8							27.8	0.5	0.01	0.01				
BF 7	TAYLOR MARL	169.4 - 170.9	16.3							22.2	0.6						
BF 8	TAYLOR MARL	38.0 - 39.2	23.3				100	275	0.34								
BF 8	TAYLOR MARL	38.0 - 39.2	23.3	101	102	78											
BF 8	TAYLOR MARL	38.0 - 39.2	22.8				400	257	0.34								
BF 8	TAYLOR MARL	78.3 - 79.1	19.9	109	115	93											
BF 8	TAYLOR MARL	78.3 - 79.1	19.9				100	445	0.56								
BF 8	TAYLOR MARL	78.3 - 79.1	19.4				400	477	0.73								
BF 8	TAYLOR MARL	107.0 - 107.5	17.8	113.0	116	93											
BF 8	TAYLOR MARL	107.0 - 107.5	17.8				400	484	0.73								
BF 8	TAYLOR MARL	107.0 - 107.5	18.5							41.7	0.565						
BF 8	TAYLOR MARL	107.0 - 107.5	18.2							55.6	0.53	0.02	0.04	0.02	0.07		
BI 4	TAYLOR MARL	171.6 - 174.1	17.2							20.8	0.55	0.02	0.02				
BI 4	TAYLOR MARL	171.6 - 174.1	17.5							54.2	0.546						