Figure 2.6-37
BORING LOG—HOLE NO. B-5

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station		1 of 1	B-5

SITE	COORDINATES	ANGLE FROM HORIZ.	BEARING
Dry Cask ISFSI		90°	-

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE (INCHES)	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH (FT.)
5-6-82	5-6-82	Ayers & Ayers J. Ayers	CME 55	3½"	45.5'	-	45.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING (FT.)	GROUND EL. (FT.)	DEPTH/EL. GROUND WATER (FT.)	DEPTH/EL. TOP OF ROCK (FT.)
-	-	13	-	34.5'	23.3'/11.2'	-

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
140.25#/30"	None	K. R. Bell

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (in.)	LENGTH CORE RUN (Ft.)	SAMPLE RECOVERY (in.)	CORE RECOVERY (Ft.)	SAMPLE BLOWS "N"	PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"				7		4	3	4				ML	1	Brown, medium-stiff, clayey SILT	
SS	18"				10		3	4	6				CH	2	Brown, medium-stiff, silty CLAY	[L]
SS	18"				12		4	5	7		30	5		3	Gray, stiff, silty CLAY	
SS	18"				13		3	6	7					4		[L]
SS	18"				10		3	4	6			10		5	Gray, medium-stiff, silty CLAY, trace fine sand seams	
SS	18"				30		10	15	15				CL / SP	6	Top 6": Reddish-brown, stiff sandy CLAY / Bottom 12": Reddish-brown, medium-dense, fine to medium SAND	
SS	18"				17		6	8	9		20	15	SP	7	Tan, medium-dense, fine to medium SAND	[L]
SS	18"				19		7	8	11			20		8		
SS	18"				21		9	10	11		10	25	SP	9	Tan, medium-dense, medium to coarse SAND, some fine gravel	
SS	18"				14		5	7	7			30		10		[L]
SS	18"				7		2	2	5		0	35	SM / SM	11	Reddish-brown, loose, silty, fine SAND / Bottom 4": Greenish-gray, loose, silty fine SAND	
SS	18"				8		3	4	4			40		12		
SS	18"				7		3	3	4		-10	45		13		[L]
															Bottom of boring 45.5'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE Dry Cask ISFSI	HOLE NO. B-5

SI020640

GPD - 13234 Rev. 2/82 (Form 10070-1)

Figure 2.6-38
BORING LOG—HOLE NO. B-5U

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	1 OF 1	B-5U

SITE	COORDINATES	ANGLE FROM HORIZ.	BEARING
Dry Cask ISFSI		90°	-

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE (INCHES)	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH (FT.)
5-5-82	5-5-82	Ayers & Ayers J. Ayers	CME 55	3¼"	13.0'	-	13.0'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING (FT.)	GROUND EL. (FT.)	DEPTH/EL. GROUND WATER (FT.)	DEPTH/EL. TOP OF ROCK (FT.)
-	-	4	-	34.5'	-	-

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
-	None	K. R. Bell

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE LENGTH (In.) / CORE RUN (Ft.)	SAMPLE RECOVERY (In.) / CORE RECOVERY (Ft.)	SAMPLE BLOWS "N" / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6" / 4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
ST	24"	24"								1	Brown silty CLAY to clayey SILT	
ST	24"	22"					30	5		2	Gray silty CLAY	
ST	24"	24"								3	Gray silty CLAY	
								10				
ST	24"	22"								4	Top: Gray silty CLAY Bottom: Reddish-brown fine to medium SAND, trace iron nodules	
							20	15			Bottom of boring 13.0'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE	HOLE NO.
	Dry Cask ISFSI	B-5U

GPD - 13234 Rev. 2/82 (Form 10070-1)

SI020641

Figure 2.6-39 (SHEET 1 OF 2)
BORING LOG—HOLE NO. B-6

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	1 OF 2	B-6

SITE	COORDINATES	ANGLE FROM HORIZ.	BEARING
Dry Cask ISFSI		90°	

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE (INCHES)	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH (FT.)
4-29-82	4-29-82	Ayers & Ayers R. Ayers	CME 55	3¼"	100.5'	-	100.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING (FT.)	GROUND EL. (FT.)	DEPTH/EL. GROUND WATER (FT.)	DEPTH/EL. TOP OF ROCK (FT.)
-	-	24		34.8'	23.0'/11.8'	-

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
140.25#/30"	None	K. R. Bell

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (in.) / LENGTH CORE RUN (ft.)	SAMPLE RECOVERY (in.)	CORE RECOVERY (FT.)	SAMPLE BLOWS "N"	PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"			9		2	3	6				CL	1	Brown, medium-stiff, silty CLAY	
SS	18"			11		2	4	7				CH		Brown and gray, stiff, silty CLAY	[L]
SS	18"			12		3	4	8		30	5		2, 3		
SS	18"			10		3	4	6					4		
SS	18"			5		2	2	3			10		5	Brown, soft, silty CLAY, trace reddish-brown sand seams	
SS	18"			12		2	4	8				CL	6	Reddish-brown, stiff, silty CLAY	
SS	18"			32		13	13	19		20	15	SP	7	Reddish-brown, dense, medium to coarse SAND, trace fine gravel	
SS	18"			18		4	9	9			20		8	Top 9": Reddish-brown, medium-dense, medium to coarse SAND Bottom 9": Tan, medium-dense, fine to medium SAND	
SS	18"			10		4	4	6		10	25	SP	9	Tan, loose, medium to coarse SAND, some fine gravel	
SS	18"			17		6	8	9			30		10	Tan, medium-dense, medium to coarse SAND, some fine gravel	
SS	18"			40		8	32	8		0	35	SC	11	Reddish-brown, dense clayey SAND, trace fine gravel	
SS	18"	0"		14		5	8	6			40		12		
SS	18"			8		2	4	4				SM	12	Greenish-gray, loose, silty fine SAND, trace shells	
SS	18"			7		2	3	4		-10	45		13		[L]
SS	18"			8		2	4	4			50		14		
SS	18"			8		2	3	5		-20	55		15		
SS	18"			7		2	2	5			60		16		
SS	18"			10		2	4	6		-30	65		17		

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE	HOLE NO.
	Dry Cask ISFSI	B-6

SI020642

Figure 2.6-39 (SHEET 2 OF 2)
BORING LOG—HOLE NO. B-6

BECHTEL

BORING LOG

PROJECT: Vepco - Surry Power Station | JOB NO. 14569 | SHEET NO. 2 OF 2 | HOLE NO. B-6

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (In.)	LENGTH CORE RUN (Ft.)	SAMPLE RECOVERY (In.)	CORE RECOVERY (Ft.)	SAMPLE BLOWS "N"	PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"				11		4	5	6			70	SM	18	Greenish-gray, medium-dense, silty fine SAND, trace shells	
SS	18"				12		2	4	8		-40	75		19	Greenish-gray, medium-dense, silty fine SAND, some clay, trace shells	
SS	18"				13		2	4	9			80		20		
SS	18"				11		2	3	8		-50	85	CH	21	Greenish-gray, stiff, CLAY, trace silt, trace shells	
SS	18"				8		2	3	5			90		22	Greenish-gray, medium-stiff, CLAY trace silt, trace shells	
SS	18"				20		14	8	12		-60	95		23	Greenish-gray, stiff, CLAY, trace silt, some cemented shell zone	
SS	18"				12		3	4	8			100		24	Greenish-gray, stiff, CLAY, trace silt	
															Bottom of boring 100.5'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER

SITE: Dry Cask ISFSI | HOLE NO. B-6

GPD-13234-A Rev. 2/82 (Form 10070-2)

SI020643

Figure 2.6-40 (SHEET 1 OF 2)
BORING LOG—HOLE NO. B-7

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	1 of 2	B-7

SITE	COORDINATES	ANGLE FROM HORIZ.	BEARING
Dry Cask ISFSI		90°	-

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE (INCHES)	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH (FT.)
5-3-82	5-3-82	Ayers & Ayers J. Ayers	CME 55	3¼"	100.5'	-	100.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING (FT.)	GROUND EL. (FT.)	DEPTH/EL. GROUND WATER (FT.)	DEPTH/EL. TOP OF ROCK (FT.)
-	-	24	-	34.1'	23.3'/10.8'	-

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
140.25# /30"	None	K. R. Bell

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE LENGTH CORE RUN (FT.)	SAMPLE RECOVERY (in.)	CORE RECOVERY (FT.)	SAMPLE BLOWS "N"	PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6" / 4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"			6		2	4	4			CL	1	Brown and gray, medium-stiff, silty CLAY	
SS	18"			11		3	4	7			CL-CH	2	Brown and gray, stiff, silty CLAY, some reddish-brown sand seams	
SS	18"			9		3	4	5	30	5	CL-CH	3	Gray, medium stiff, silty CLAY, some reddish-brown sand seams	
SS	18"			6		2	3	3				4		[L]
SS	18"			6		2	3	3		10		5		
SS	18"			9		4	5	5			SC	6	Reddish brown, loose, clayey fine SAND	
SS	18"			17		5	8	9	20	15	SP	7	Tan, medium-dense, fine SAND	
SS	18"			31		9	14	17		20		8	Reddish-brown, dense, fine to medium SAND	
SS	18"			11		3	5	6	10	25	SP	9	Tan, medium-dense, medium to coarse SAND, some fine gravel	∇
SS	18"			11		3	4	7		30		10		[L]
SS	18"			4		1	2	2	0	35	SM / SM	11	Middle 6": Reddish-brown, loose, silty fine SAND Bottom 6":Greenish-gray, loose, silty fine SAND	
SS	18"			11		4	5	6		40	SM	12	Greenish-gray, medium-dense, silty fine SAND, trace shells	
SS	18"			6		3	3	3	-10	45	SM	13	Greenish-gray, loose, silty fine SAND, trace shells	[L]
SS	18"			10		3	5	5		50		14		
SS	18"			12		4	5	7	-20	55	SM	15	Greenish-gray, medium-dense, silty fine SAND, trace shells	
SS	18"			10		3	4	6		60	SM	16	Greenish-gray, loose, silty fine SAND, trace shells	
SS	18"			9		3	3	6	-30	65		17		

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE	HOLE NO.
	Dry Cask ISFSI	B-7

GPD - 13234 Rev. 2/82 (Form 10070-1)

SI020644

Figure 2.6-40 (SHEET 2 OF 2)
BORING LOG—HOLE NO. B-7

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	2 OF 2	B-7

SAMPLER TYPE AND DIAMETER	SAMPLE ADVANCE (In.) / LENGTH CORE RUN (Ft.)	SAMPLE RECOVERY (In.) / CORE RECOVERY (Ft.)	SAMPLE BLOWS "N" / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6" / 4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"		11	3	4	7		70	SM	18	Greenish-gray, medium-dense, silty fine SAND, trace shells	
SS	18"		9	3	3	6	- 40	75	SM	19	Greenish gray, loose, silty fine SAND, trace clay, trace shells	
SS	18"		11	3	4	7		80	CH	20	Greenish-gray, stiff, CLAY, some silt, trace shells	
SS	18"		10	4	4	6	- 50	85		21		[L]
SS	18"		7	3	3	4		90	CH	22	Greenish-gray, medium-stiff, CLAY, some silt, some shells	
SS	18"		15	25	7	8	- 60	95	CH	23	Top: Cemented shell zone Bottom: Greenish-gray, stiff, CLAY, some shells, some silt	
SS	18"		11	3	4	7		100		24		[L]
											Bottom of boring 100.5'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER

SITE	HOLE NO.
Dry Cask ISFSI	B-7

SI020645

GPD - 13234 - A Rev. 2/82 (Form 10070 - 2)

Figure 2.6-41 (SHEET 1 OF 2)
BORING LOG—HOLE NO. B-8

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	1 OF 2	B-8

SITE	COORDINATES	ANGLE FROM HORIZ.	BEARING
Dry Cask ISFSI		90°	-

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE (INCHES)	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH (FT.)
5-4-82	5-4-82	Ayers & Ayers J. Ayers	CME 55	3¼"	85.5'	-	85.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING (FT.)	GROUND EL. (FT.)	DEPTH/EL. GROUND WATER (FT.)	DEPTH/EL. TOP OF ROCK (FT.)
-	-	24	-	34.2	23.0'/11.2'	-

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
140.25# /30"	None	K. R. Bell

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (in.) / LENGTH CORE RUN (FT.)	SAMPLE RECOVERY (in.) / CORE RECOVERY (FT.)	SAMPLE BLOWS "N" / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"		6	2	2	4				CL	1	Brown, medium-stiff, silty CLAY	
ST	24"	16"								CH-CL	2	Gray, silty CLAY	[L]
SS	18"		13	3	6	7		30	5		3	Gray, stiff, silty CLAY	[L]
ST	24"	22"									4	Gray, silty CLAY	[L]
SS	18"		7	3	3	4			10		5	Gray, medium-stiff, silty CLAY, some reddish-brown fine sand seams	
ST	24"	24"								SC	6	Top: Gray, silty CLAY Bottom: Reddish-brown, clayey SAND	[L]
SS	18"		9	3	4	5		20	15		7	Reddish-brown, loose, clayey SAND	[L]
SS	18"		10	3	4	6			20	SP	8	Tan, loose, fine to medium SAND	▽
SS	18"		10	3	5	5		10	25	SP	9	Tan, loose, medium to coarse SAND, trace fine gravel	
SS	18"		11	3	4	7			30		10	Tan, medium-dense, medium to coarse SAND, some fine gravel	[L]
SS	18"		12	7	3	9		0	35	SM	11	Greenish-gray, medium-dense, silty fine SAND, trace shells	
SS	18"		8	3	3	5			40		12	Greenish-gray, loose, silty fine SAND, trace shells	
ST	24"	23"									13		[L]
SS	18"		8	3	4	4		-10	45		14		
SS	18"		8	3	3	5			50		15		
SS	18"		8	3	3	5		-20	55		16		
SS	18"		8	2	3	5			60		17		
ST	24"	24"									18		[L]
SS	18"		8	2	3	5		-30	65		19		

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE	HOLE NO
	Dry Cask ISFSI	B-8

SI020646

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Figure 2.6-41 (SHEET 2 OF 2)
BORING LOG—HOLE NO. B-8

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	2 of 2	B-8

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (In.) LENGTH CORE RUN (Ft.)	SAMPLE RECOVERY (In.) CORE RECOVERY (Ft.)	SAMPLE BLOWS "N" PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6" / 4TH 6"	ELEVATION (FT.)	DEPTH, FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"		10	3	4	6		70	SM	20	Greenish-gray, loose, silty fine SAND, trace shells	
SS	18"		10	3	4	6	-40	75	CH	21	Greenish-gray, medium-stiff CLAY, some silt, trace shells, trace fine sand	
SS	18"		10	3	4	6		80		22	Greenish-gray, medium stiff CLAY trace silt, trace shells	
ST	24"	24"								23		[L]
SS	18"		9	2	3	6	-50	85		24		[L]
											Bottom of boring 85.5'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER

SITE: Dry Cask ISFSI

HOLE NO.: B-8

SI020647

GPO - 13234 - A Rev. 2/82 (Form 10070 - 2)

Figure 2.6-42 (SHEET 1 OF 2)
BORING LOG—HOLE NO. B-9

BECHTEL

BORING LOG

Project	Job No.	Sheet No.	Hole No.
Vepco - Surry Power Station	14569	1 of 2	B-9

Site	Coordinates	Angle from Horiz.	Bearing
Dry Cask ISFSI		90°	-

Begun	Completed	Driller	Drill Make and Model	Hole Size (Inches)	Overburden (Ft)	Rock (Ft)	Total Depth (Ft)
4-29-82	4-29-82	Ayers & Ayers R. Ayers	CME 55	3¼"	101.5'	-	101.5'

Core Recovery (Ft./%)	Core Boxes	Samples	El. Top of Casing (Ft.)	Ground El. (Ft.)	Depth/El. Ground Water (Ft.)	Depth/El. Top of Rock (Ft.)
-	-	24	-	36.1	25.0'/11.1'	-

Sample Hammer Weight/Fall	Casing Left in Hole: Dia./Length	Logged By:
140.25# /30"	None	K. R. Bell

Sampler Type and Diameter	Sampler Advance Length Core Run (Ft.)	Sample Recovery (In.) Core Recovery (Ft.)	Sample Blows "N" Percent Core Recovery	Penetration Blows 1st 6"	2nd 6"	3rd 6"	4th 6"	Elevation (Ft.)	Depth-Ft	Unified Soil Classification	Sample	Description and Classification	Notes on: Water Levels, Water Return, Character of Drilling, Etc.
SS	18"		7	2	3	4				CH	1	Brown and gray, medium-stiff, silty CLAY	[L]
SS	18"		7	2	3	3					2		
SS	18"		11	2	5	6		30	5		3	Gray, stiff, silty CLAY	
SS	18"		9	2	3	6					4	Gray, medium-stiff, silty CLAY	
SS	18"		8	2	4	4			10		5		[L]
SS	18"		5	2	2	3					6	Gray, soft, silty CLAY	
SS	18"		8	1	2	6		20	15		7	Gray, medium-stiff, silty CLAY, some reddish-brown fine sand seams	
SS	18"		25	3	11	14			20	SC	8	Gray, medium-dense, clayey SAND	
SS	18"		24	4	6	18		10	25	SP	9	Reddish-brown, medium dense, medium SAND, some fine gravel	▽
SS	18"		24	7	12	12			30	SP	10	Tan, medium-dense, medium to coarse SAND, some fine gravel	
SS	18"		21	5	7	14		0	35		11		
SS	18"		7	2	4	3			40	SM	12	Gray, loose, silty fine SAND, some shells	
SS	18"		4	2	2	2		-10	45	SM	13	Greenish-gray, loose, silty fine SAND, trace shells	[L]
SS	18"		8	2	4	4			50		14		
SS	18"		11	3	5	6		-20	55		15		
SS	18"		10	3	5	5			60		16		
SS	18"		8	2	4	4		-30	65		17		

	Site	Hole No.
SS = Split Spoon; ST = Shelby Tube; D = Dennison; P = Pitcher; O = Other	Dry Cask ISFSI	B-9

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SI020648

Figure 2.6-42 (SHEET 2 OF 2)
BORING LOG—HOLE NO. B-9

BECHTEL

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Vepco - Surry Power Station	14569	2 OF 2	B-9

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE (In.) LENGTH CORE RUN (Ft.)	SAMPLE RECOVERY (In.) CORE RECOVERY (Ft.)	SAMPLE BLOWS "N" PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6" / 4TH 6"	ELEVATION (FT.)	DEPTH-FT	UNIFIED SOIL CLASSIFICATION	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
SS	18"		12	3	5	7		70	SM	18	Greenish-gray, medium dense, silty fine SAND, some clay, trace shells	
SS	18"		11	2	4	7	-40	75		19		
SS	18"		13	2	4	9		80	CH	20	Greenish-gray, stiff, CLAY, some silt, trace shells, trace fine sand	
SS	18"		16	3	6	10	-50	85		21	Greenish-gray, very stiff, CLAY, trace fine sand, trace shells	[1.]
SS	18"		7	1	3	4		90		22	Greenish-gray, medium stiff, CLAY, trace fine sand, trace shells	
SS	18"		11	2	4	7	-60	95		23		
SS	18"		19	8	8	11		100		24	Greenish-gray, stiff, CLAY, some shells Bottom of boring 100.5'	

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE	HOLE NO.
	Dry Cask ISFSI	B-9

SI020649

GPD - 13234 - A Rev. 2/82 (Form 10070 - 2)

Figure 2.6-43
BORING LOCATION PLAN

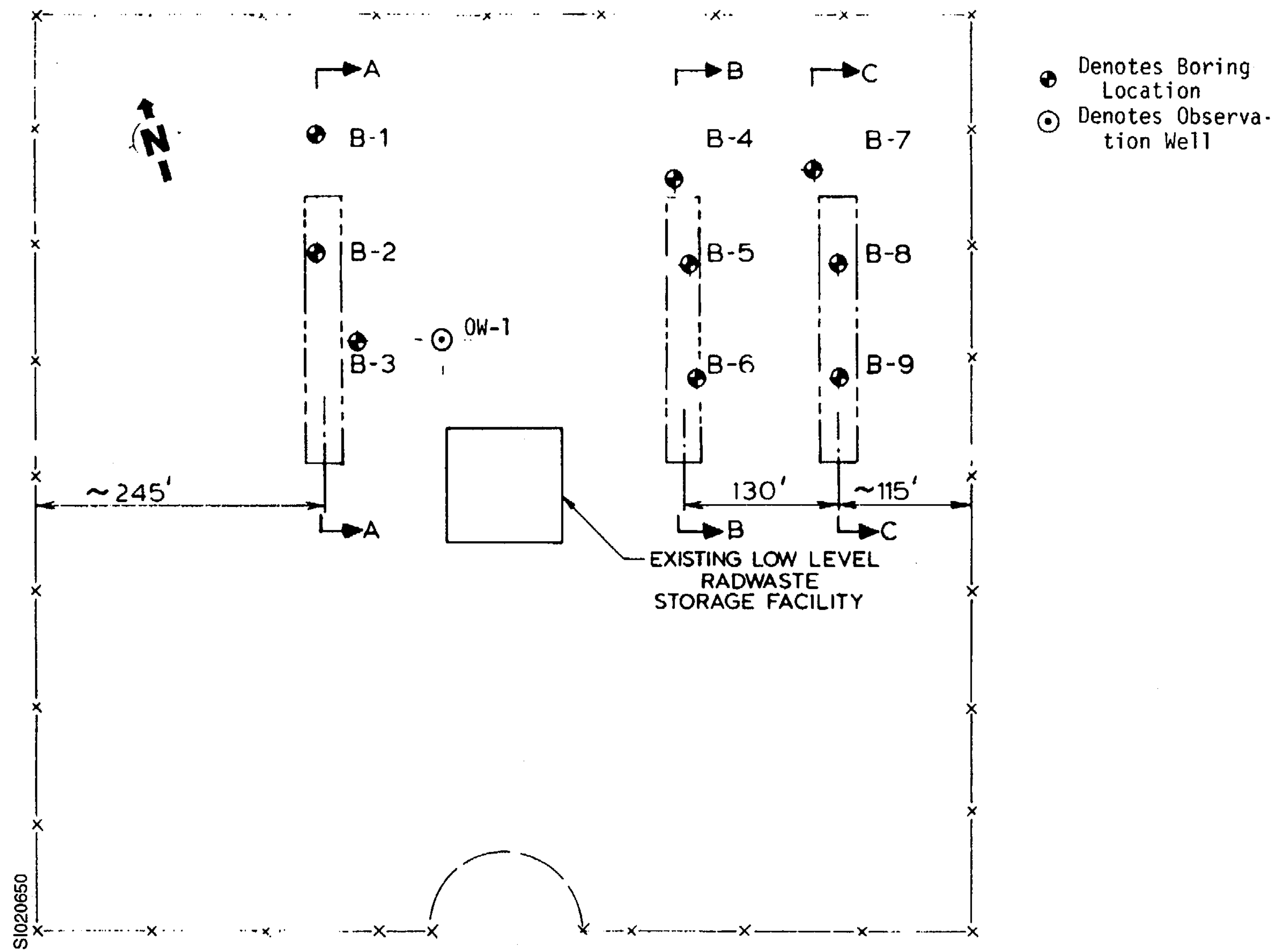
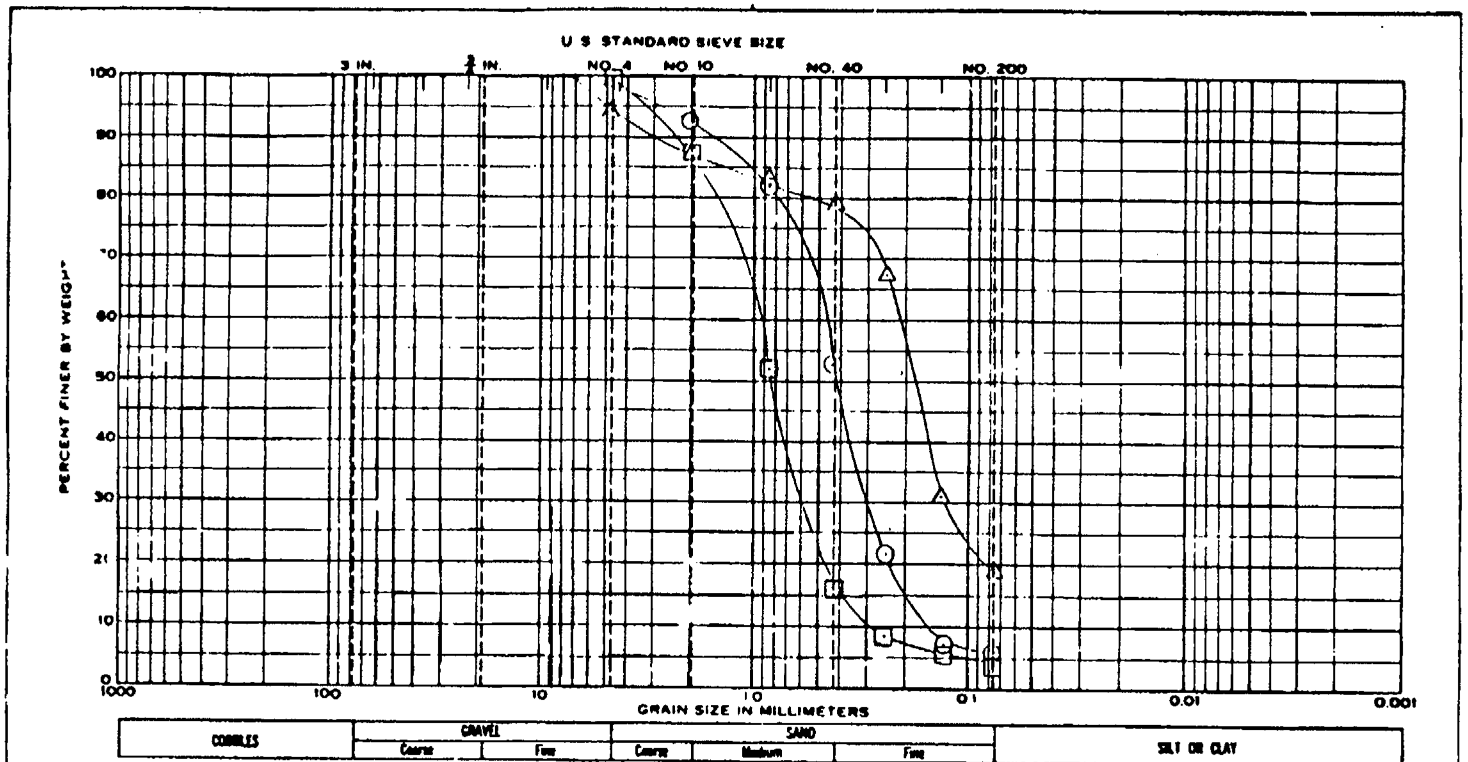


Figure 2.6-44 (SHEET 1 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASSIF.	M.C.	LL	PI
⊙	B-1	[illegible]	FINE TO COARSE SAND, TRACE SILT, WITH GRAVEL - TAN	SP-SM	-	-	-
⊡	B-1	[illegible]	FINE TO COARSE SAND, TRACE SILT AND FINE GRAVEL - TAN	SP-SM	-	-	-
△	B-1	[illegible]	FINE TO MEDIUM SAND, SOME SILT, WITH SHELL FRAGMENTS - BROWN	SM	24.4	NP	NP

SCHNABEL ENGINEERING ASSOCIATES

GRADATION CURVES

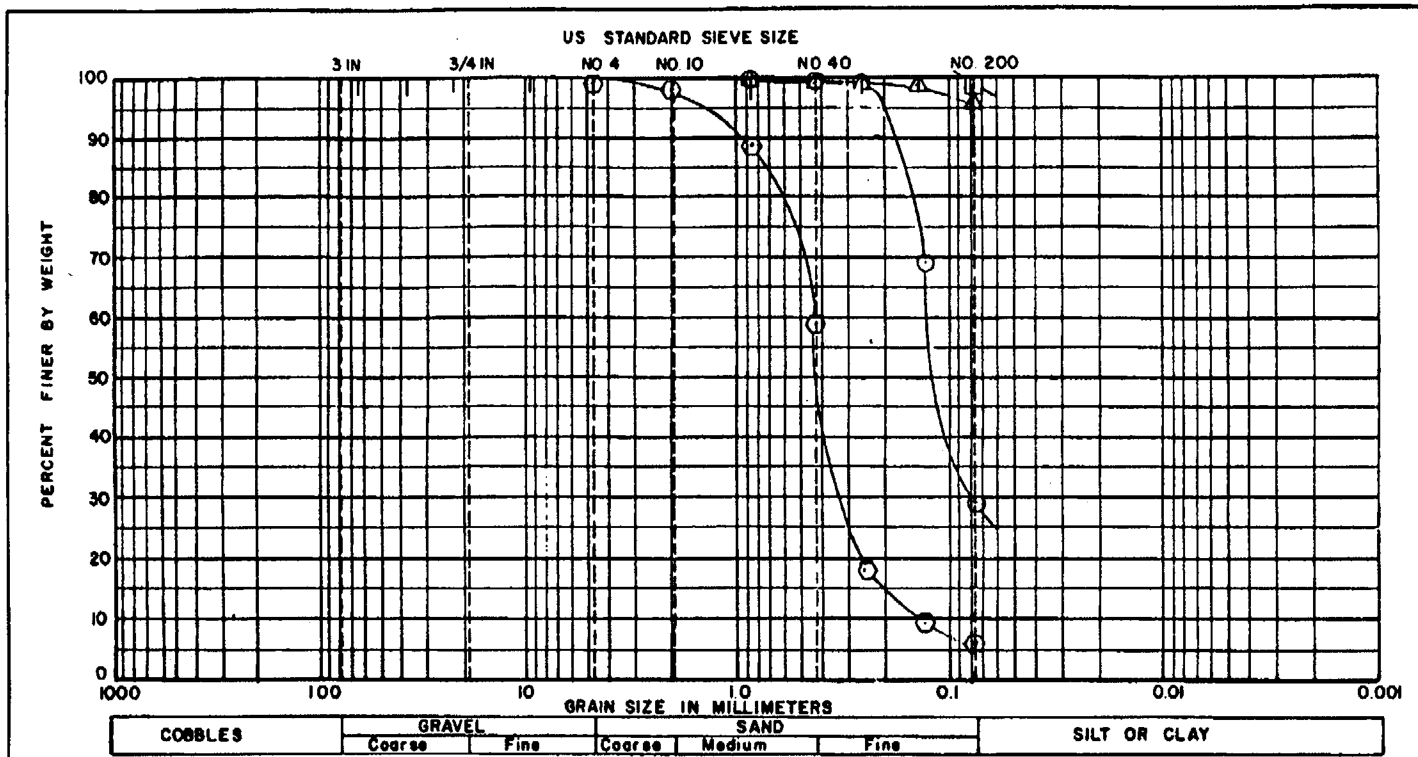
PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE: 6/11/82

CONTR. NO.: [illegible]

SI020651

Figure 2.6-44 (SHEET 2 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
⊙	B-1	49.5-50.5'	FINE SAND, SOME SILT, WITH SHELL	SM	30.5	NP	NP
			FRAGMENTS - GRAY				
⊡	B-3	0-1.5'	SILTY CLAY, TRACE FINE SAND - BROWN	CL	21.7	24	11
△	B-3	4'-5.5'	CLAY, TRACE FINE SAND - GRAY	CH	24.3	58	38
⊙	B-3	14-15.5'	FINE TO COARSE SAND, TRACE SILT -	SP-SM	-	-	—
			TAN AND BROWN				

SCHNABEL ENGINEERING ASSOCIATES

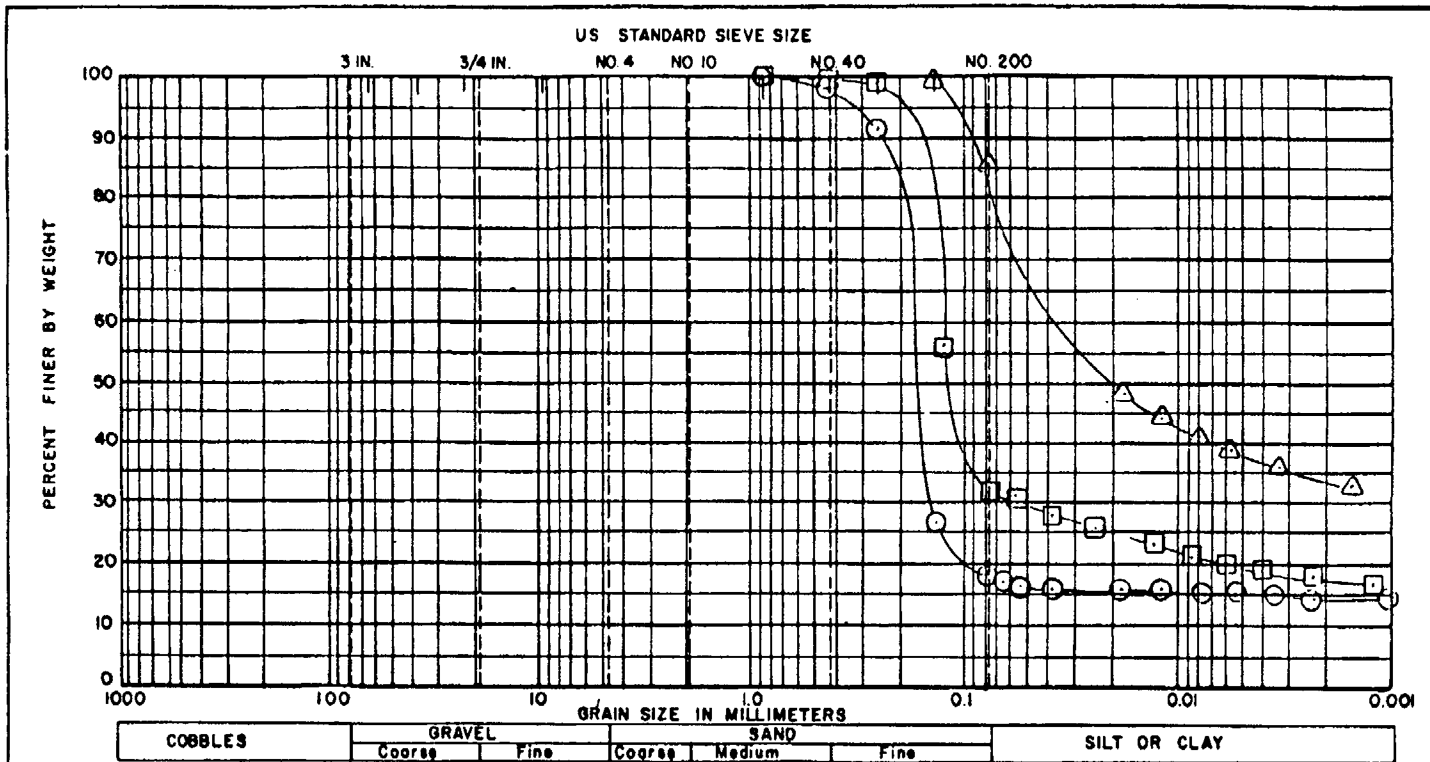
GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE: 6/11/82 CONTRACT NO. VA2263

SI020652

Figure 2.6-44 (SHEET 3 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
○	B-3	31'-35'	FINE SAND, SOME SILT - BROWN	SM	-	NP	NP
□	B-3	44'-45'	FINE SAND, SOME SILT, WITH SHELL FRAGMENTS - GREENISH GRAY	SM	31.6	NP	NP
△	B-3	14'-15'	CLAY, SOME FINE SAND, WITH SHELL FRAGMENTS - GREENISH GRAY	CH	44.9	57	30

SCHNABEL ENGINEERING ASSOCIATES

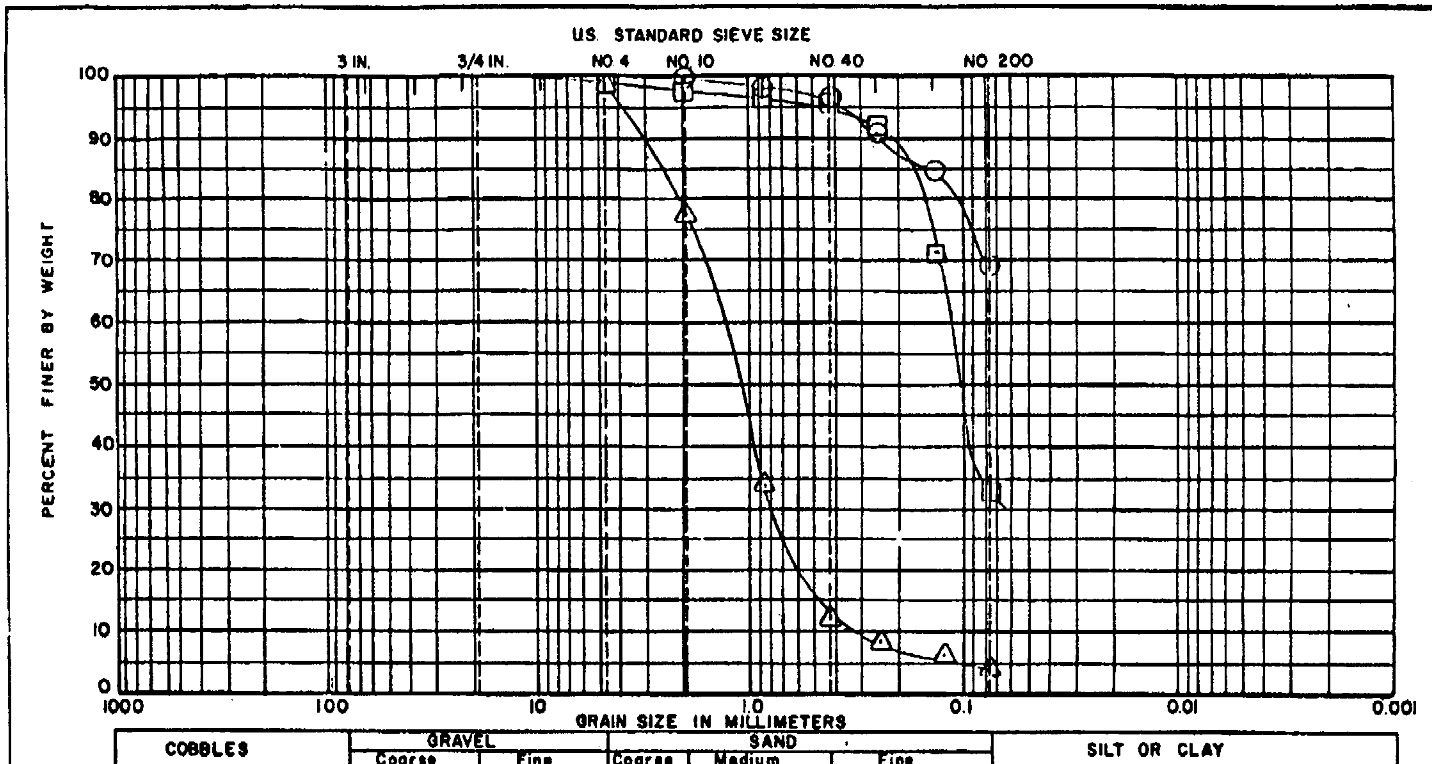
GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE: 6/11/82 CONTRACT NO. [illegible]

SI020653

Figure 2.6-44 (SHEET 4 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
○	B-4	[illegible]	FINE SILT, SOME SAND, WITH ORGANIC MATTER AND SHELL FRAGMENTS GRAY AND BROWN	ML	-	-	-
□	B-4	14'-15.5'	FINE SAND, SOME SILT - BROWN	SM	-	-	-
△	B-4	29'-30.5'	FINE TO COARSE SAND, TRACE SILT AND FINE GRAVEL - TAN	SP-SM	-	-	-

SCHNABEL ENGINEERING ASSOCIATES

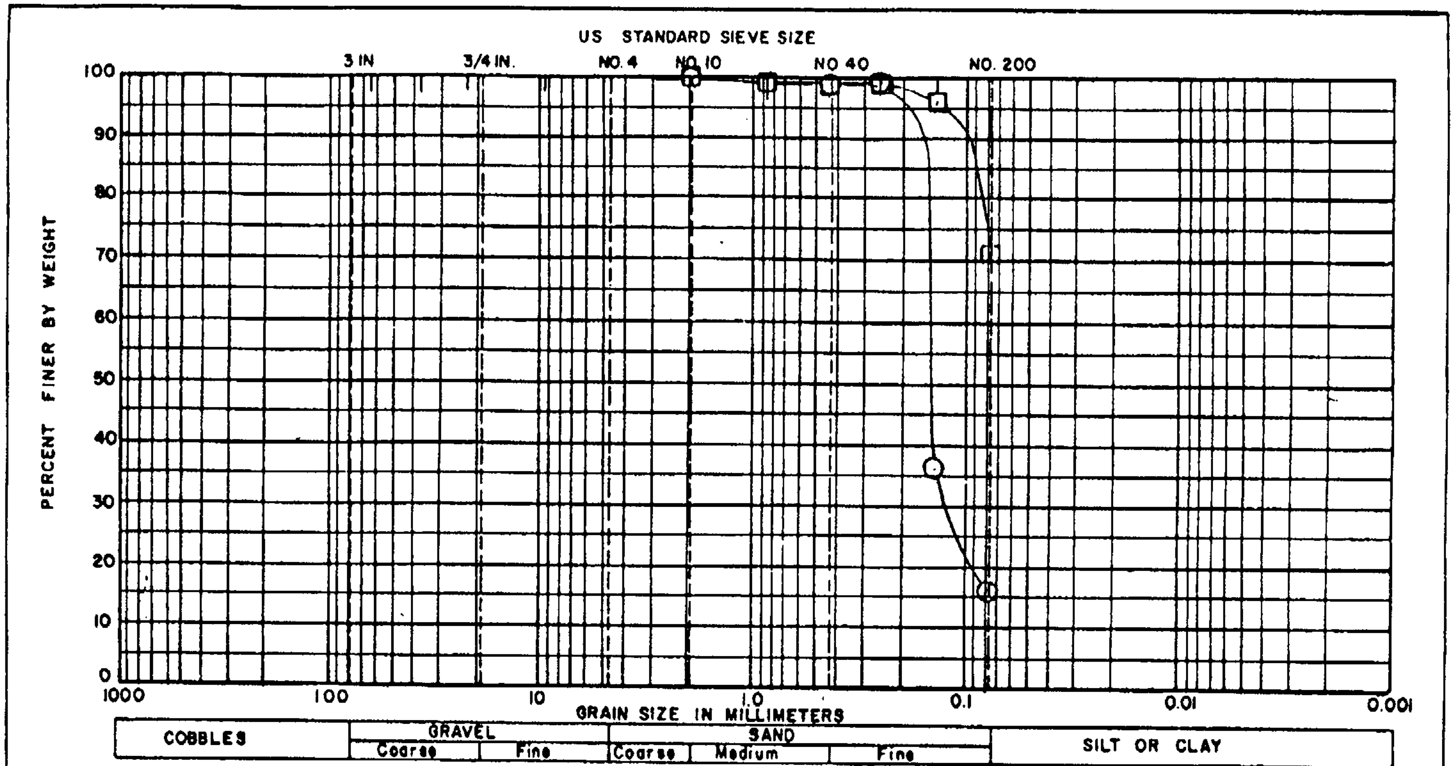
GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO. VA.

DATE: 6/11/82 CONTRACT NO. VB2182

SI020654

Figure 2.6-44 (SHEET 5 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
O	B-4	44-45.5	FINE SAND, SOME SILT, WITH SHELL FRAGMENTS - DARK GREENISH GRAY	SM	29.3	NP	NP
□	B-4	84-85.5	CLAYEY SILT, SOME FINE SAND, WITH SHELL FRAGMENTS - DARK GRAY	ML-MH	34.4	50	17

SCHNABEL ENGINEERING ASSOCIATES

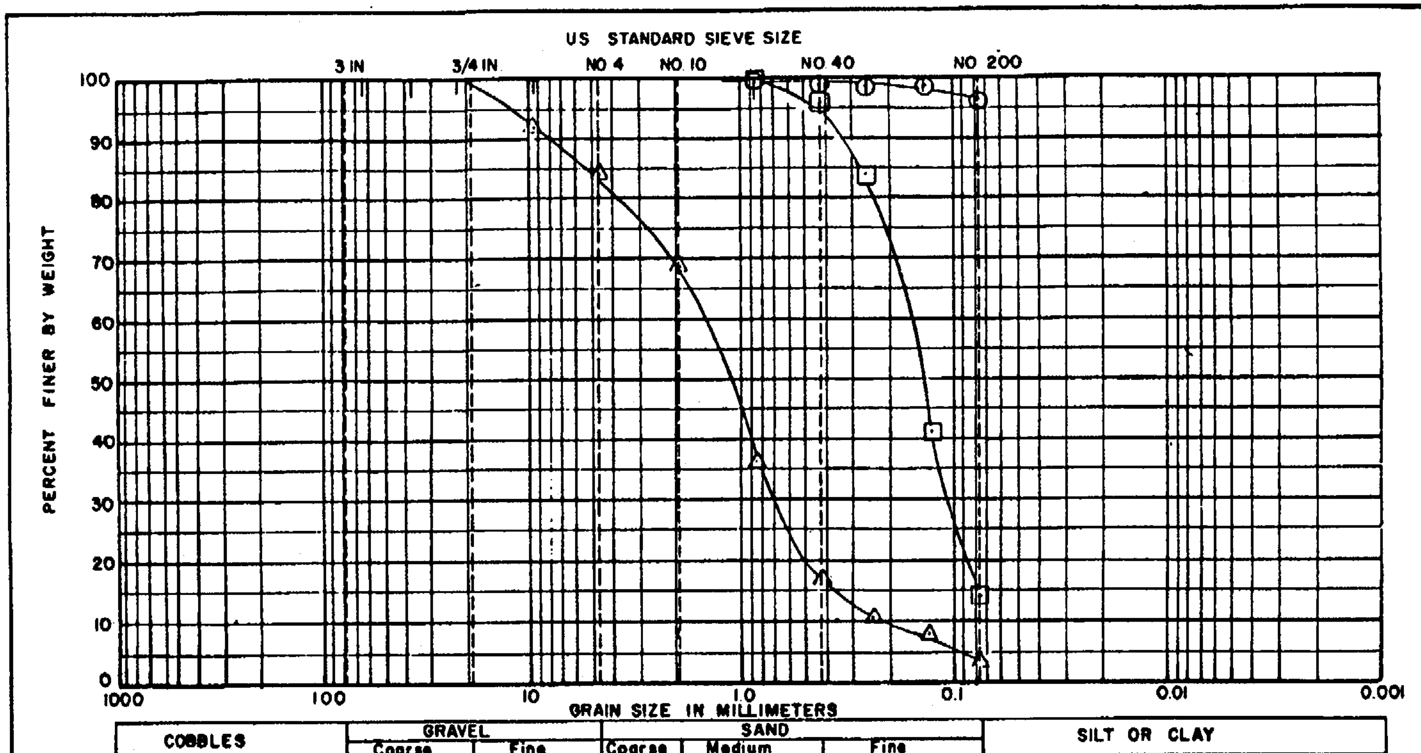
GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO, VA.

DATE: 6/11/82 CONTRACT NO. VB.182

SI020655

Figure 2.6-44 (SHEET 6 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
⊙	B-5	7.5'-9'	CLAYEY SILT, TRACE FINE SAND - GRAY AND BROWN	MH	36.4	65	27
⊡	B-5	14'-15.5'	FINE SAND, SOME SILT - BROWN	SM	-	-	-
Δ	B-5	29'-30.5'	FINE TO COARSE SAND, SOME FINE GRAVEL, TRACE SILT - BROWN	SW-SM	-	-	-

SCHNABEL ENGINEERING ASSOCIATES

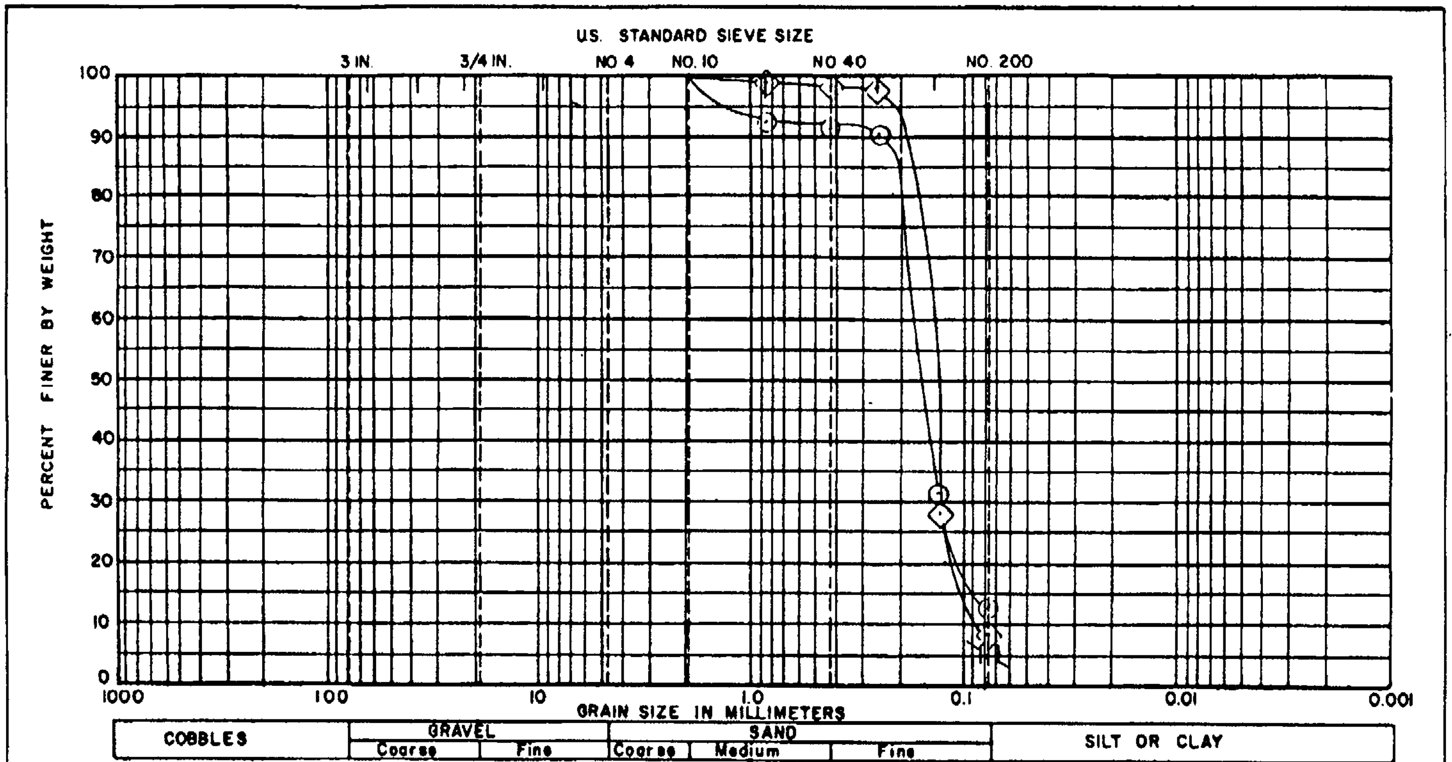
GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE 6/11/87 CONTRACT NO. [illegible]

SI020656

Figure 2.6-44 (SHEET 7 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
⊙	B-5	44-45	FINE SAND, SOME SILT, WITH SHELL FRAGMENTS - GRAY	SM	29.2	NP	NP
⊡	B-6	24-25	FINE TO COARSE SAND, TRACE SILT AND FINE GRAVEL - TAN	SP-SM	–	–	–
◇	B-6	44-45	FINE SAND, TRACE SILT, WITH SHELL FRAGMENTS	SP-SM	20.2	NP	NP

SCHNABEL ENGINEERING ASSOCIATES

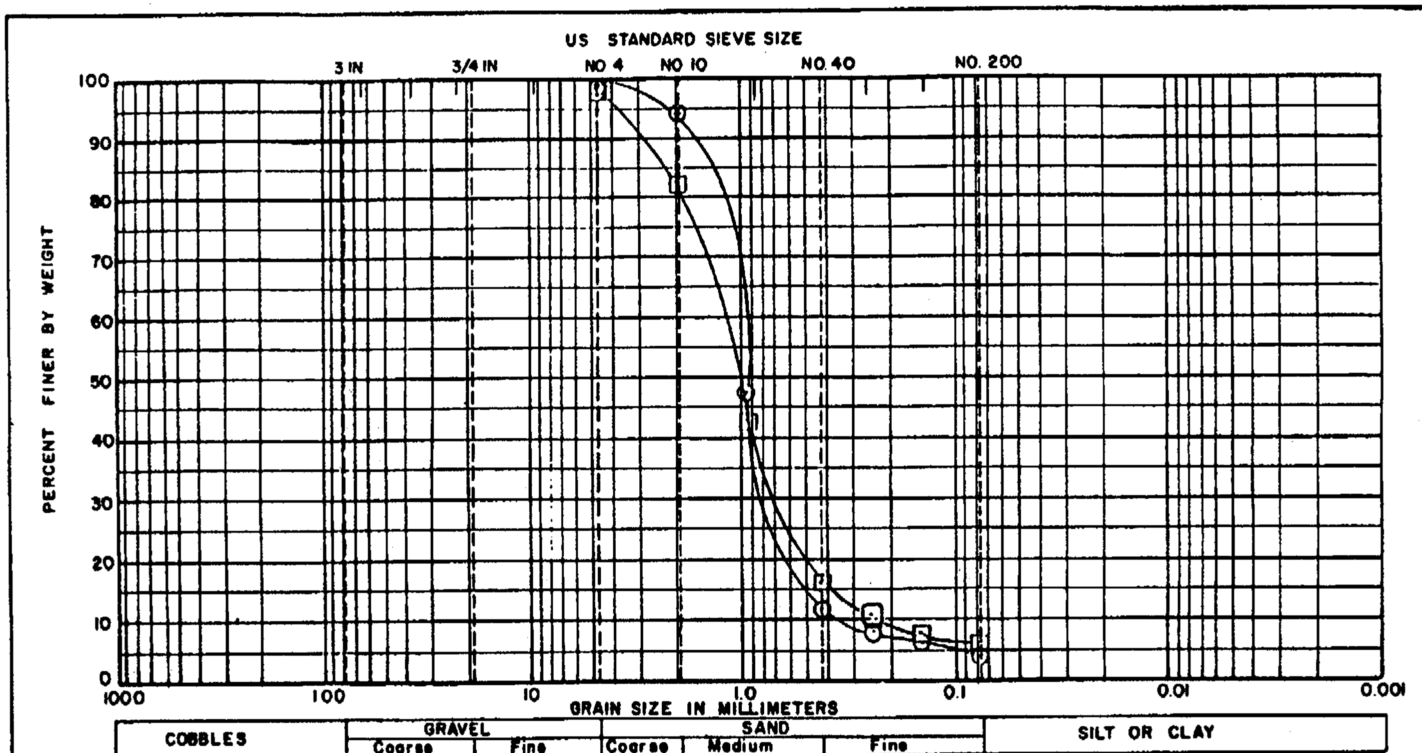
GRADATION CURVES

PROJECT: DRY CASK STORAGE FACILITY, SURRY [illegible]

DATE: 6/11/82 | CONTRACT NO. [illegible]

SI020657

Figure 2.6-44 (SHEET 8 OF 8)
GRADATION CURVES



KEY	BORING	DEPTH	DESCRIPTION OF SOIL SAMPLE TESTED	CLASS	MC	LL	PI
⊙	B-7	29'-30'	FINE TO COARSE SAND, TRACE SILT AND FINE GRAVEL - TAN	SP-SM	-	-	-
⊡	B-8	29'-30'	FINE TO COARSE SAND, TRACE SILT AND FINE GRAVEL - TAN	SN-SM	-	-	-

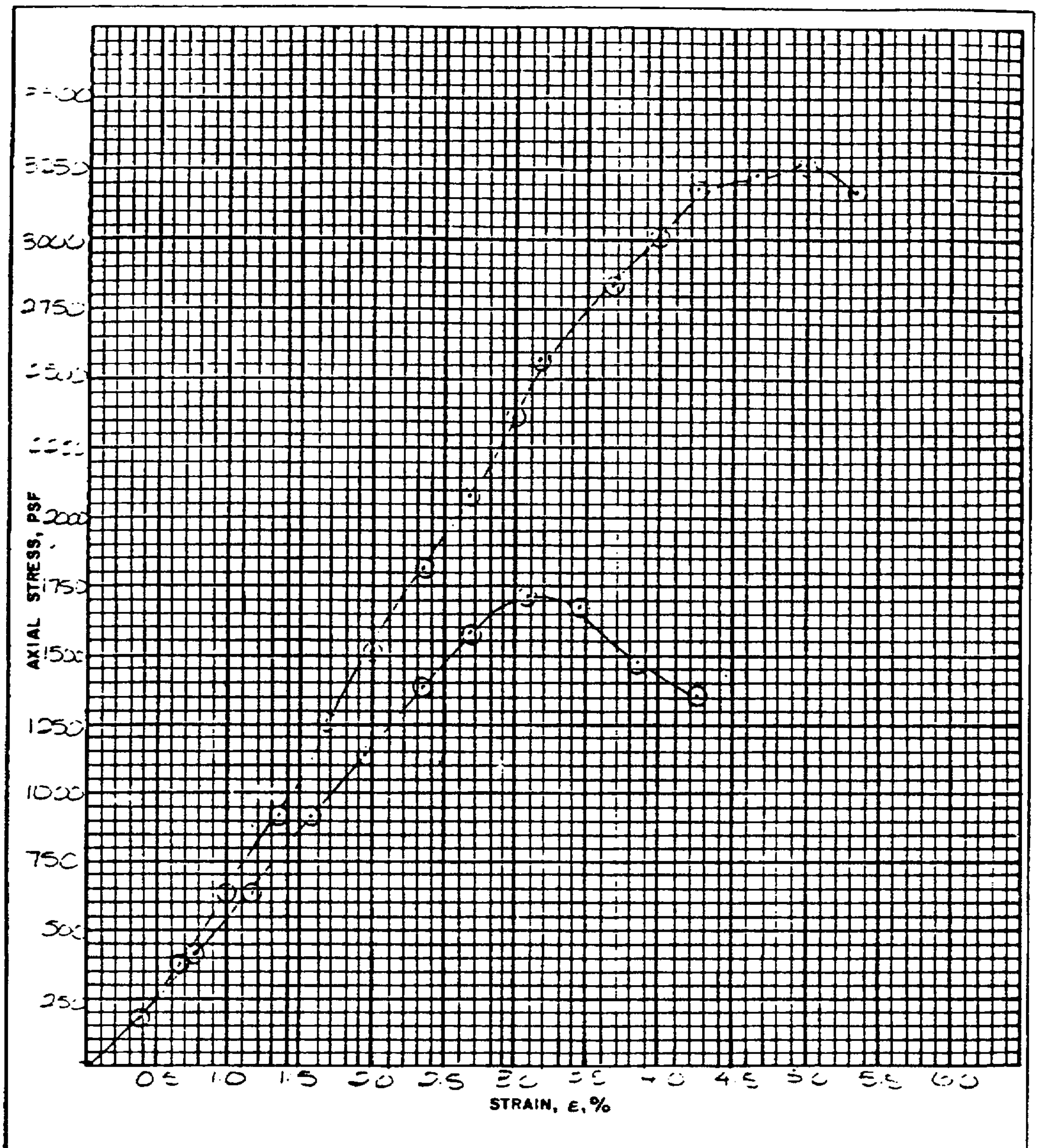
SCHNABEL ENGINEERING ASSOCIATES

GRADATION CURVES

PROJET: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

SI020658

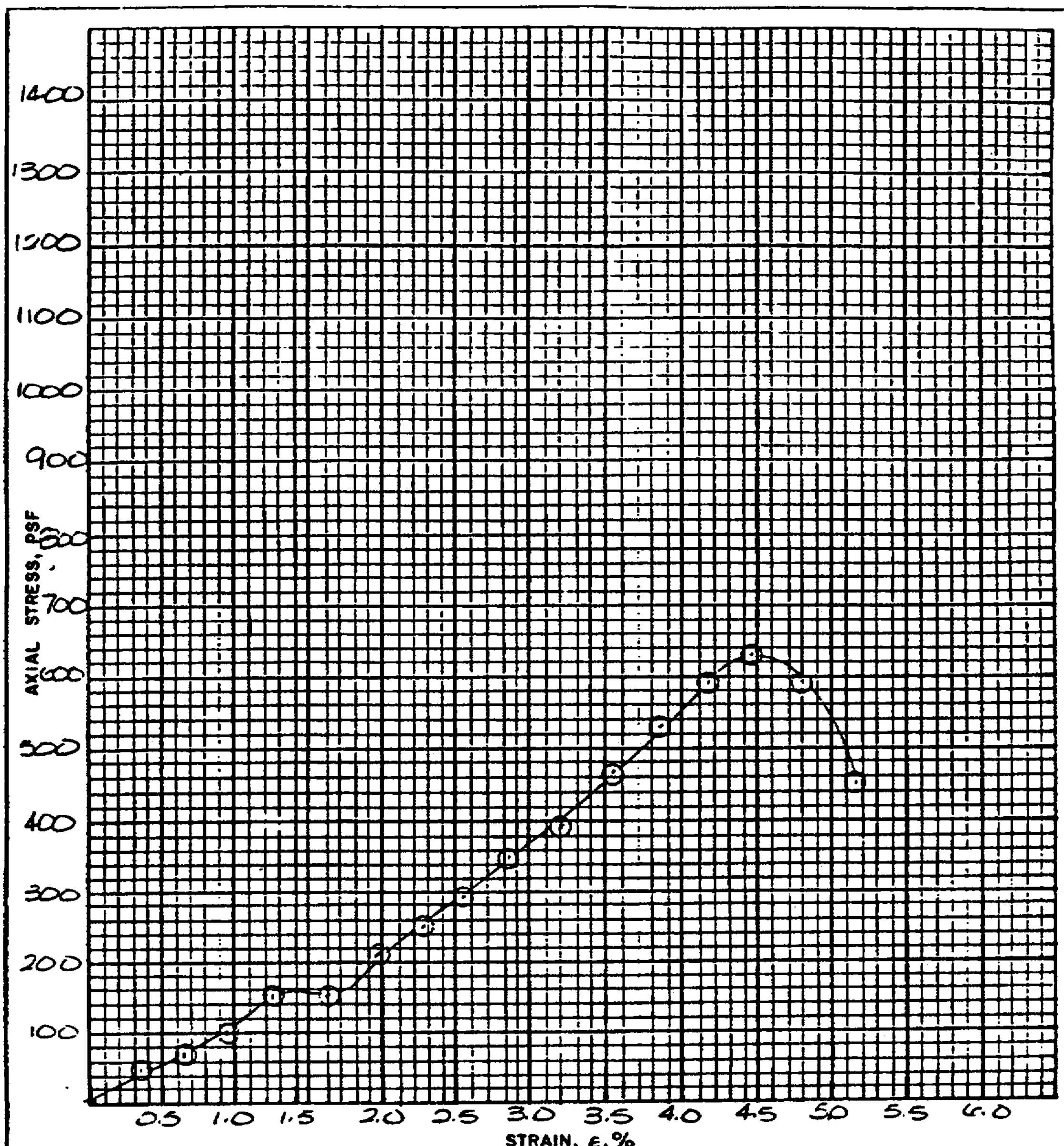
Figure 2.6-45 (SHEET 1 OF 4)
UNCONFINED COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED			SCHNABEL ENGINEERING ASSOCIATES
CLAYEY SILT, TRACE FINE SAND - [illegible] AND BROWN (CH)			UNCONFINED COMPRESSION TEST
BORING NO.: B-5U	DEPTH, FT.: 4'-6'		PROJECT: DRY CASK SPENT FUEL
MOIST CONTENT, %	WET DENSITY, PCF	DRY DENSITY, PCF	FACILITY, SURRY CO, VA.
[illegible]	120.4	98.0	DATE: 6/11/82 CONTRACT NO. [illegible]

SI020659

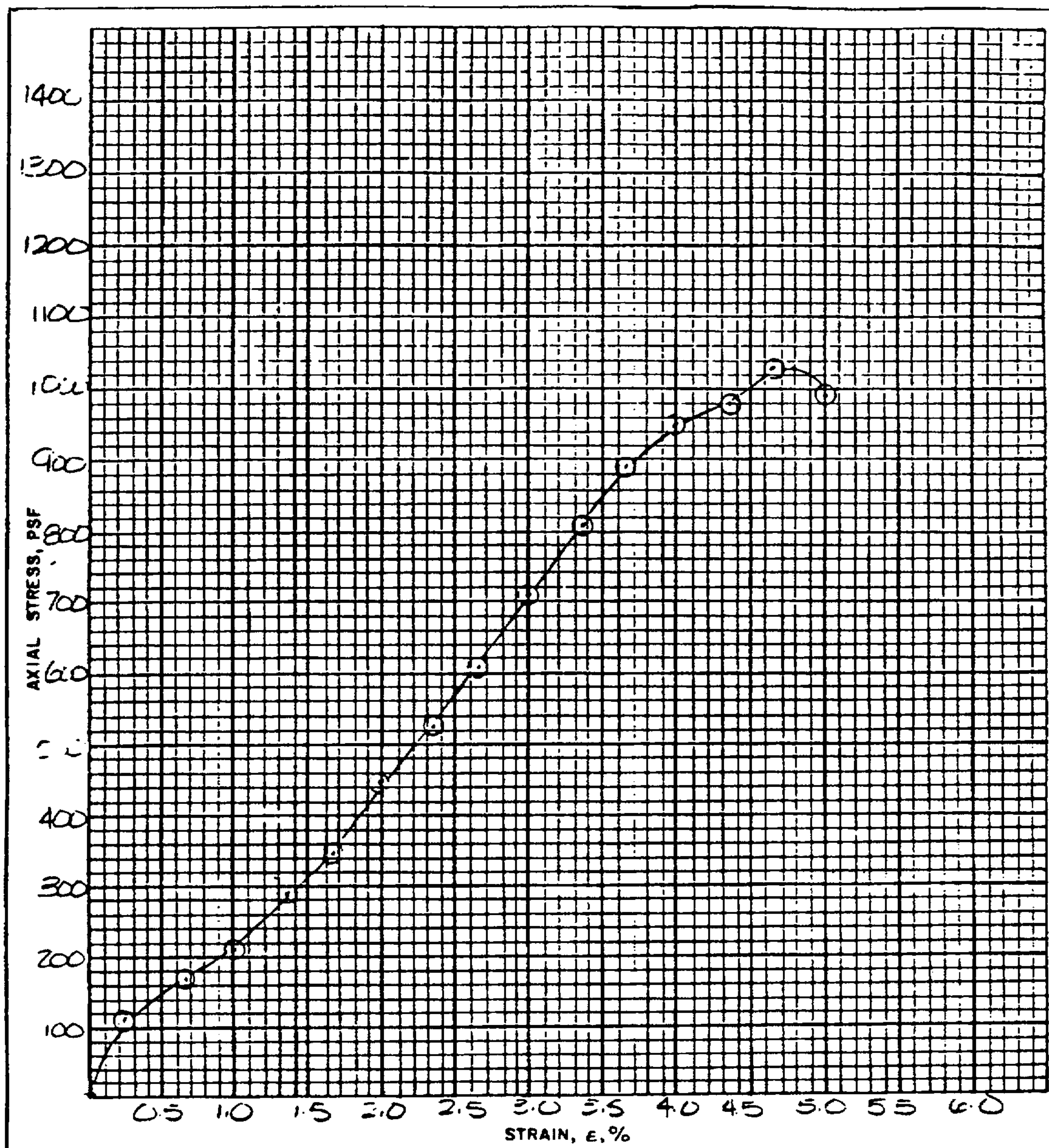
Figure 2.6-45 (SHEET 2 OF 4)
UNCONFINED COMPRESSION TEST



DESCRITION OF SOIL SAMPLE TESTED			SCHNABEL ENGINEERING ASSOCIATES	
CLAYEY SILT, SOME FINE TO COARSE SAND - BROWN & TAN (ML)			UNCONFINED COMPRESSION TEST	
BORING NO.: B-5U	DEPTH, FT.: 11' - 13'		PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO. VA.	
MOIST. CONTENT, %	WET DENSITY, PCF	DRY DENSITY, PCF		
31.4	116.5	88.7	DATE: 6/11/82	CONTRACT NO. VE-182

SI020660

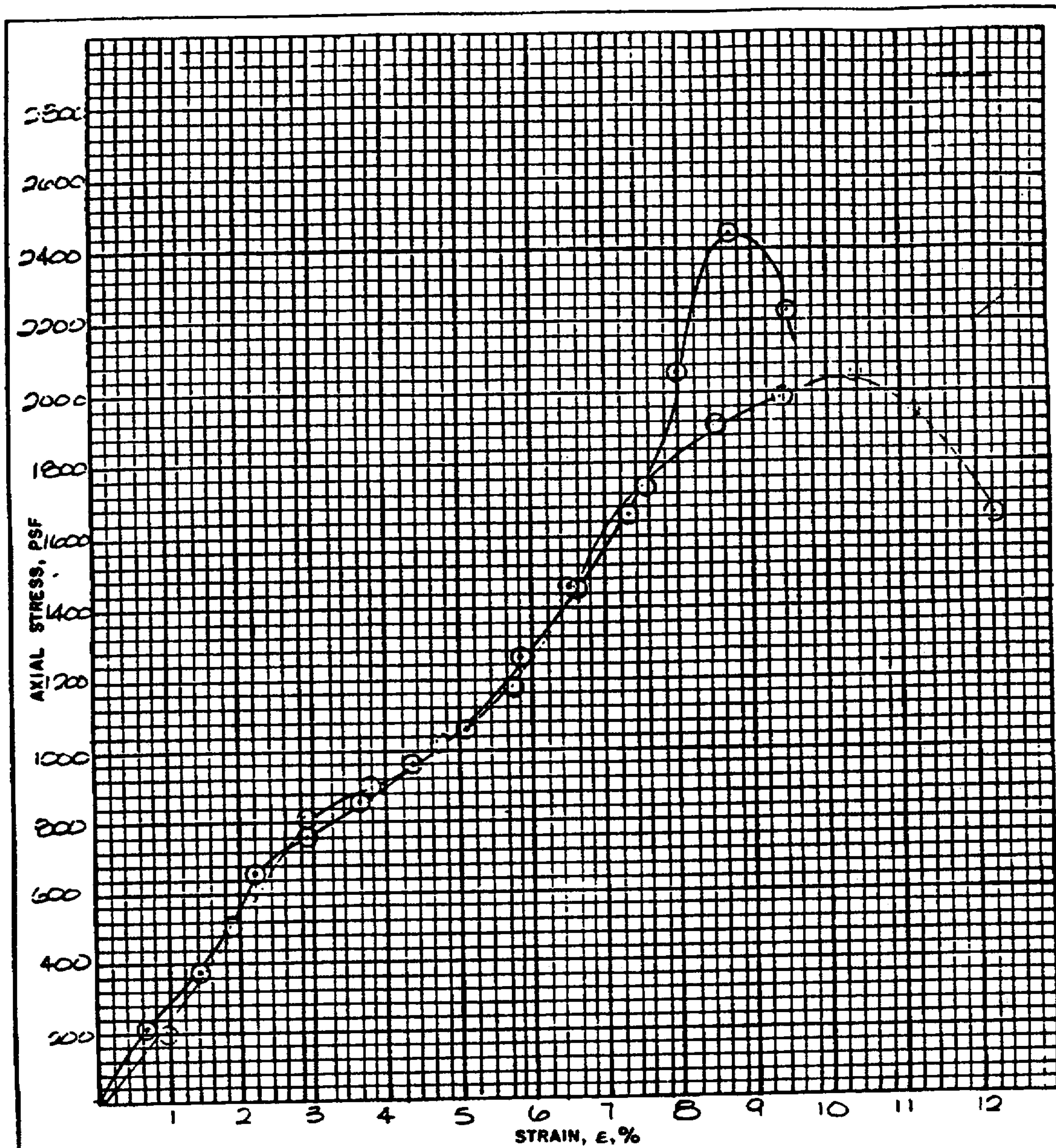
Figure 2.6-45 (SHEET 3 OF 4)
UNCONFINED COMPRESSION TEST



DESCRITION OF SOIL SAMPLE TESTED			SCHNABEL ENGINEERING ASSOCIATES
FINE TO MEDIUM SANDY SILTY CLAY - GRAY AND BROWN (CL)			UNCONFINED COMPRESSION TEST
BORING NO.: B-5U	DEPTH, FT.: 11'-13'		PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO, VA
MOIST. CONTENT, %	WET DENSITY, PCF	DRY DENSITY, PCF	
15.1	116.5	101.2	DATE: 6/11/82 CONTRACT NO. VE21E2

SI020661

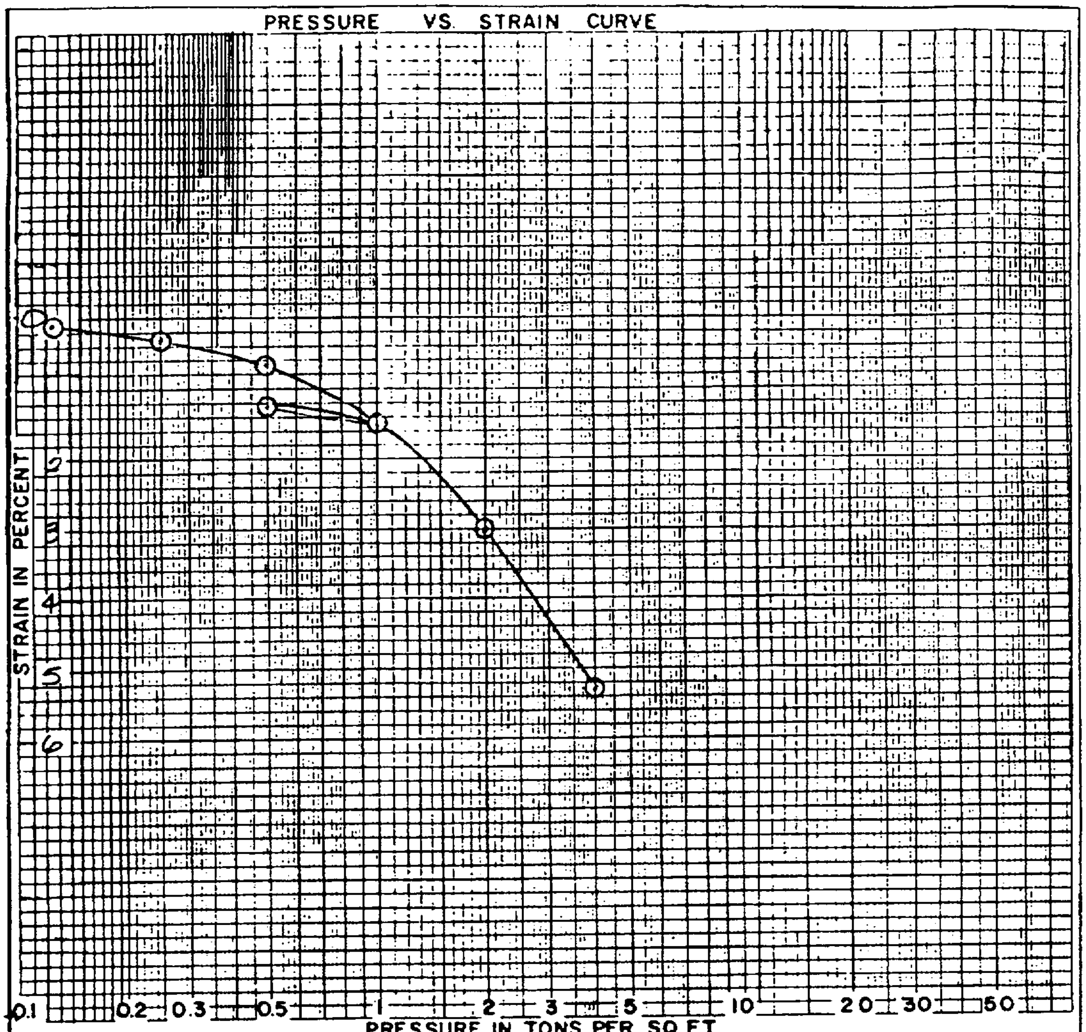
Figure 2.6-45 (SHEET 4 OF 4)
UNCONFINED COMPRESSION TEST



DESCRITION OF SOIL SAMPLE TESTED			SCHNABEL ENGINEERING ASSOCIATES	
SILTY CLAY, TRACE FINE SAND - LIGHT GRAY AND BROWN (CL)			UNCONFINED COMPRESSION TEST	
BORING NO.: B-8	DEPTH, FT.: 7' - 9'		PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA	
MOIST CONTENT, %	WET DENSITY, PCF	DRY DENSITY, PCF	DATE: 6/11/82	CONTRACT NO. V82182
36.5	120.9	88.2		

SI020662

Figure 2.6-46 (SHEET 1 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:
SILTY CLAY, TRACE FINE SAND - BROWN & TAN

CLASSIFICATION: CL STRATUM DESIGNATION: -

DIAMETER OF SPECIMEN (in.)	2.5
INITIAL THICKNESS OF SPECIMEN (in.)	1.0
INITIAL VOID RATIO, e_0	0.89
PROBABLE PRECONSOLIDATION STRESS(tsf)Pc	–
APPROXIMATE OVERBURDEN STRESS (tsf) Po	–
COMPRESSION RATIO; FROM 2 TO 4 (tsf)	0.076
COMPRESSION INDEX; FROM 2 TO 4 (tsf)	0.144
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.008
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.015

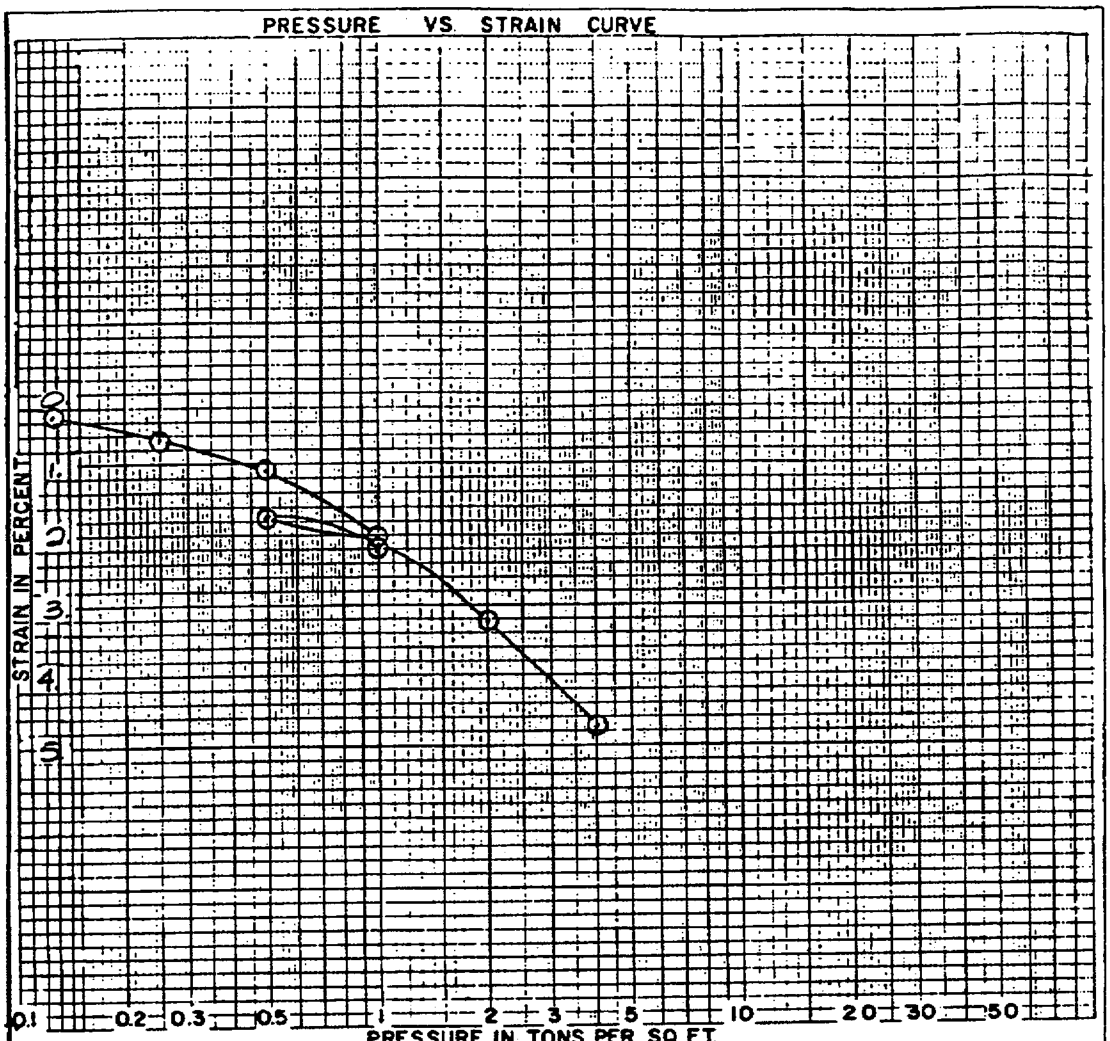
SCHNABEL ENGINEERING ASSOCIATE
CONSOLIDATION TEST

BORING NO.: B-2 DEPTH: 0-4'
PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., V

DATE	CONTRACT NO.
6/11/82	V82182

SI020663

Figure 2.6-46 (SHEET 2 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

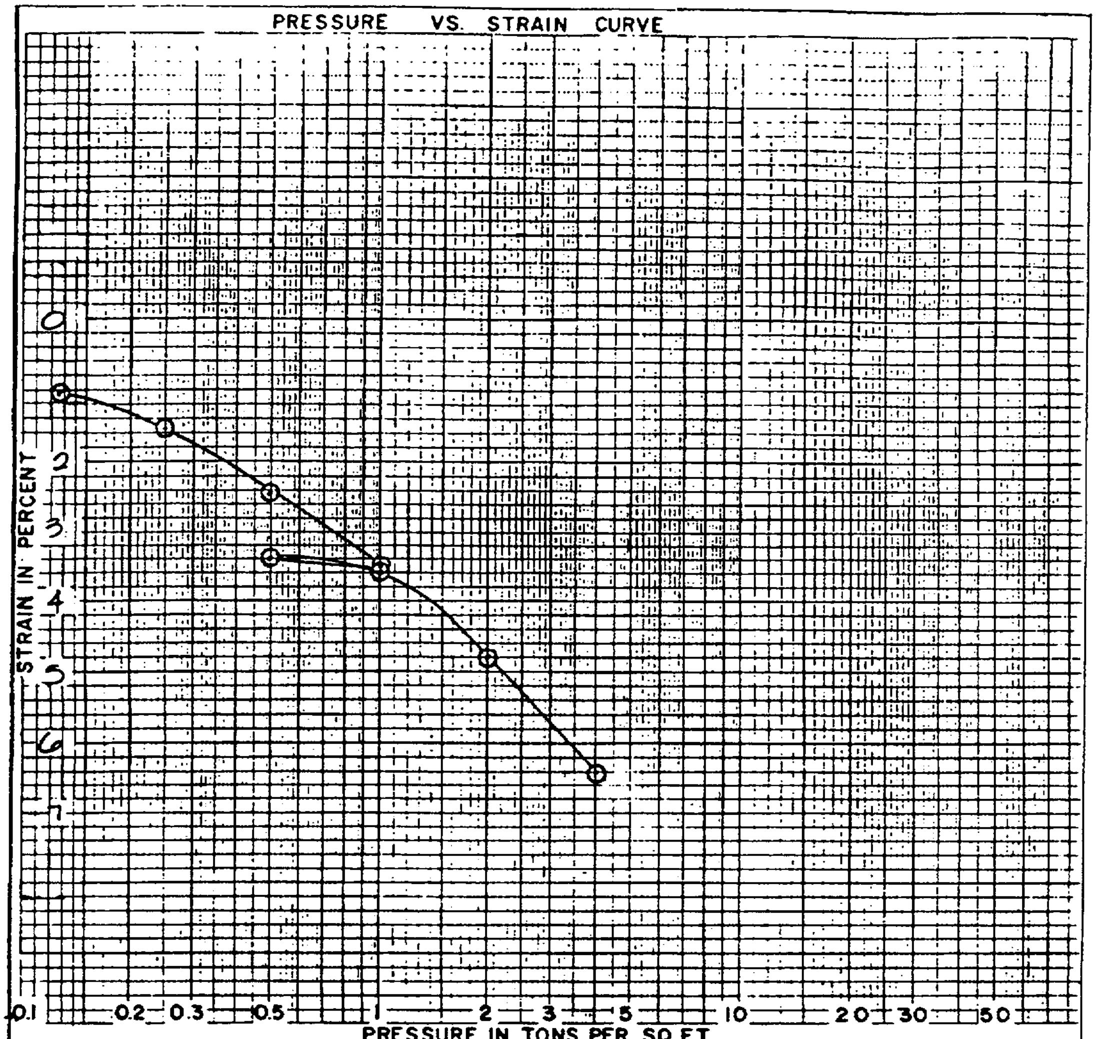
SILTY CLAY, TRACE FINE SAND - GRAY & BROWN

CLASSIFICATION: CL STRATUM DESIGNATION: –

DIAMETER OF SPECIMEN (in.)	2.5	SCHNABEL ENGINEERING ASSOCIATES
INITIAL THICKNESS OF SPECIMEN (in.)	1.0	CONSOLIDATION TEST
INITIAL VOID RATIO, e_o	0.75	
PROBABLE PRECONSOLIDATION STRESS (tsf) Pc	–	BORING NO.: B-2 DEPTH: 7-9'
APPROXIMATE OVERBURDEN STRESS (tsf) Po	–	PROJECT: DRY CASK SPENT
COMPRESSION RATIO; FROM 2 TO 4 (tsf)	0.050	
COMPRESSION INDEX; FROM 2 TO 4 (tsf)	0.090	FUEL FACILITY, SURRY CO, VA
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.037	DATE 6/11/82
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.065	CONTRACT NO. V82182

SI020664

Figure 2.6-46 (SHEET 3 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

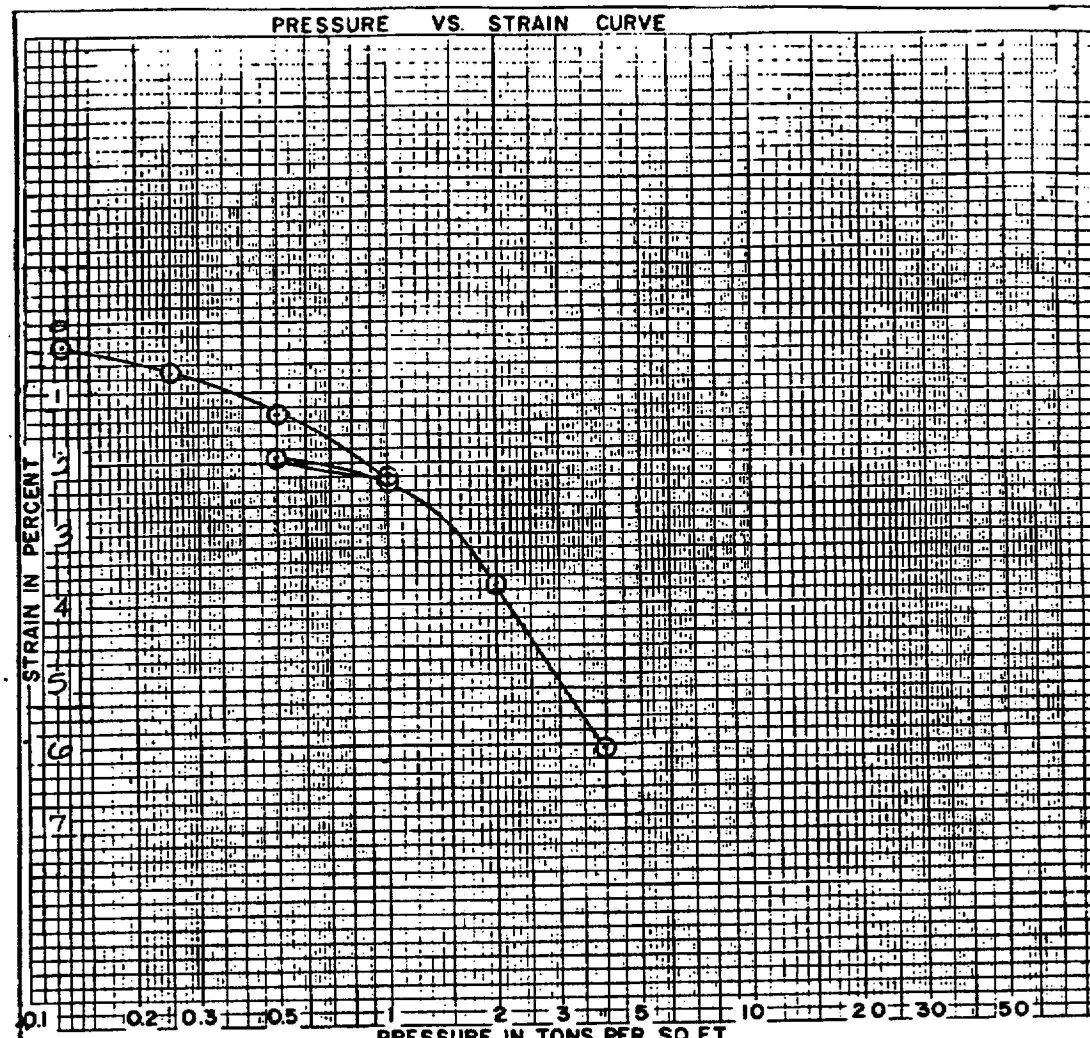
FINE SILTY SAND WITH SHELL FRAGMENTS - DARK GRAY

CLASSIFICATION: SM STRATUM DESIGNATION: -

DIAMETER OF SPECIMEN (in.)	2.5	SCHNABEL ENGINEERING ASSOCIATES
INITIAL THICKNESS OF SPECIMEN (in.)	1.0	CONSOLIDATION TEST
INITIAL VOID RATIO, e_o	0.79	
PROBABLE PRECONSOLIDATION STRESS (tsf) Pc	-	
APPROXIMATE OVERBURDEN STRESS (tsf) Po	-	BORING NO.: B-2 DEPTH: 52'-54'
COMPRESSION RATIO; FROM 2 TO 4 (tsf)	0.050	PROJECT: DRY CASK SPENT
COMPRESSION INDEX; FROM 2 TO 4 (tsf)	0.090	FUEL FACILITY, SURRY CO. V
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.007	DATE 6/11/82
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.012	CONTRACT NO. V82182

SI020665

Figure 2.6-46 (SHEET 4 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

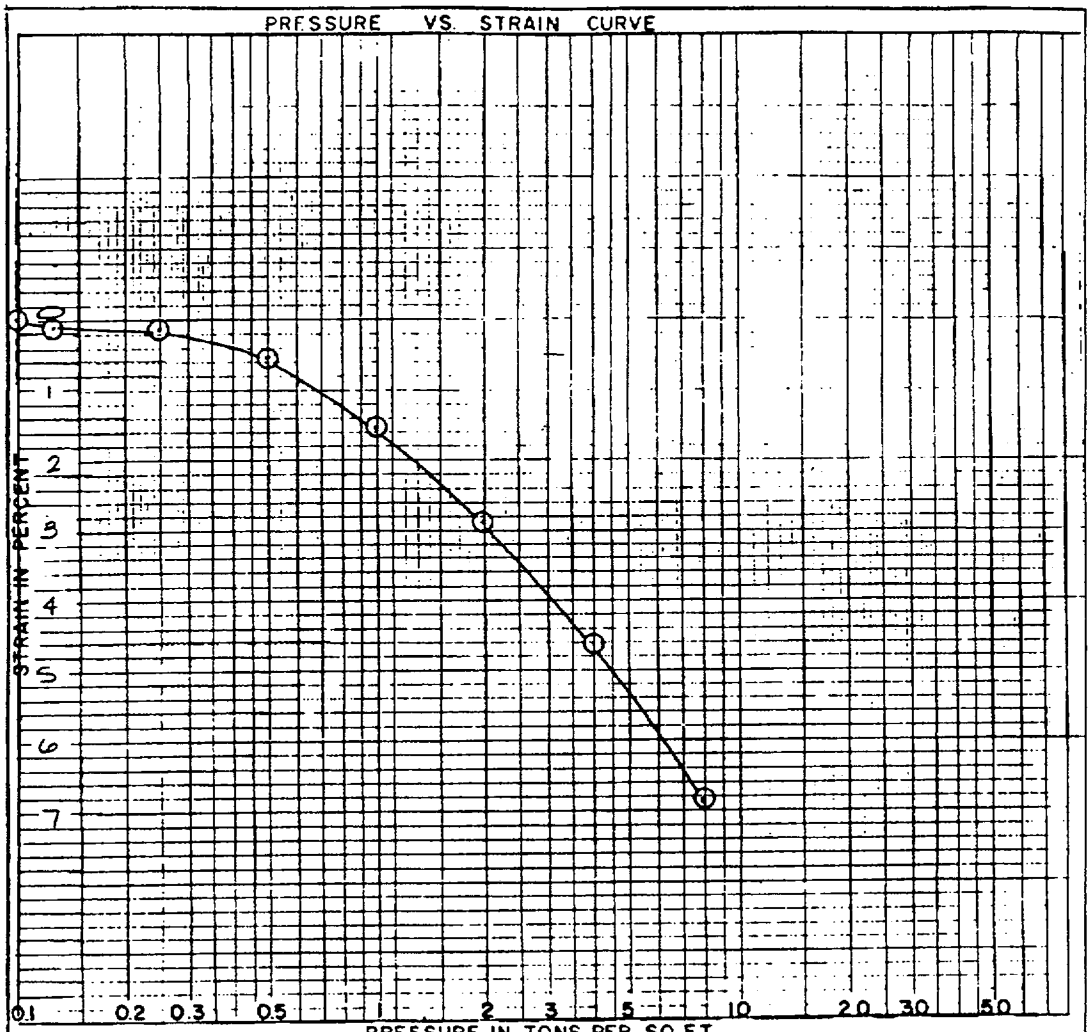
CLAY, TRACE FINE SAND - LIGHT BROWN AND TAN

CLASSIFICATION: CH STRATUM DESIGNATION:

DIAMETER OF SPECIMEN (in.)	2.5	SCHNABEL ENGINEERING ASSOCIATES
INITIAL THICKNESS OF SPECIMEN (in.)	1.0	CONSOLIDATION TEST
INITIAL VOID RATIO, e_0	0.85	
PROBABLE PRECONSOLIDATION STRESS(tsf)Pc	-	BORING NO.: B-5U DEPTH: 1' - 3'
APPROXIMATE OVERBURDEN STRESS (tsf) Po	-	PROJECT: DRY CASK SPENT
COMPRESSION RATIO; FROM 2 TO 4 (tsf)	0.078	FUEL FACILITY, SURRY CO.V
COMPRESSION INDEX; FROM 2 TO 4 (tsf)	0.144	
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.010	DATE 6/11/82
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.018	CONTRACT NO. V82182

SI020666

Figure 2.6-46 (SHEET 5 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

CLAYEY SILT, TRACE FINE SAND - GRAY AND BROWN

CLASSIFICATION: CH STRATUM DESIGNATION: -

DIAMETER OF SPECIMEN (in.)	2.50
INITIAL THICKNESS OF SPECIMEN (in.)	1.0
INITIAL VOID RATIO, e_o	0.76
PROBABLE PRECONSOLIDATION STRESS (tsf) Pc	–
APPROXIMATE OVERBURDEN STRESS (tsf) Po	–
COMPRESSION RATIO; FROM 4 TO 8 (tsf)	0.075
COMPRESSION INDEX; FROM 4 TO 8 (tsf)	0.133
RECOMPRESSION RATIO; FROM ___ TO ___ (tsf)	–
RECOMPRESSION INDEX; FROM ___ TO ___ (tsf)	–

SCHNABEL ENGINEERING ASSOCIATES

CONSOLIDATION TEST

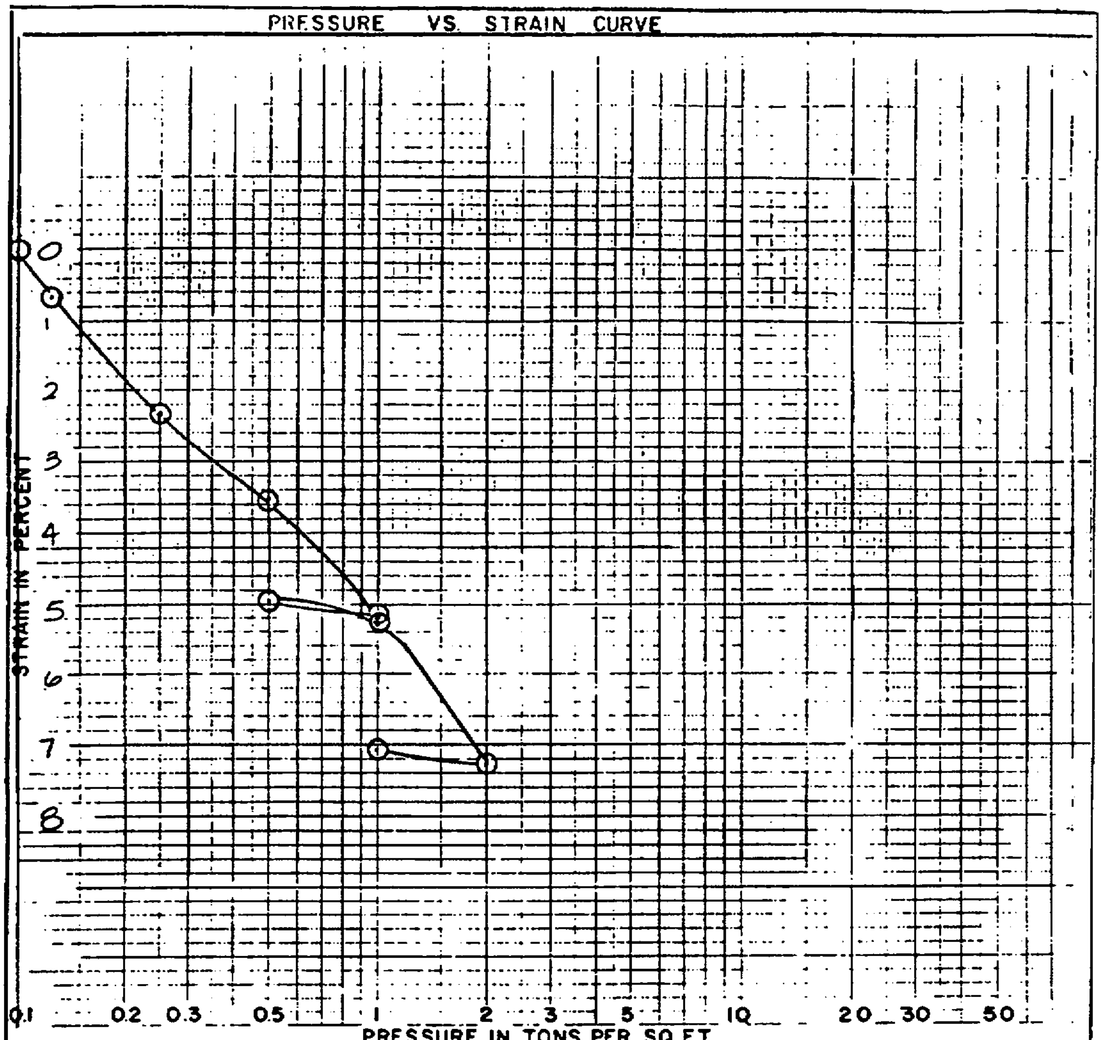
BORING NO.: B-5U DEPTH: 4'-6'

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE 6/11/82	CONTRACT NO. V82182

SI020667

Figure 2.6-46 (SHEET 6 OF 9)
CONSOLIDATION TEST



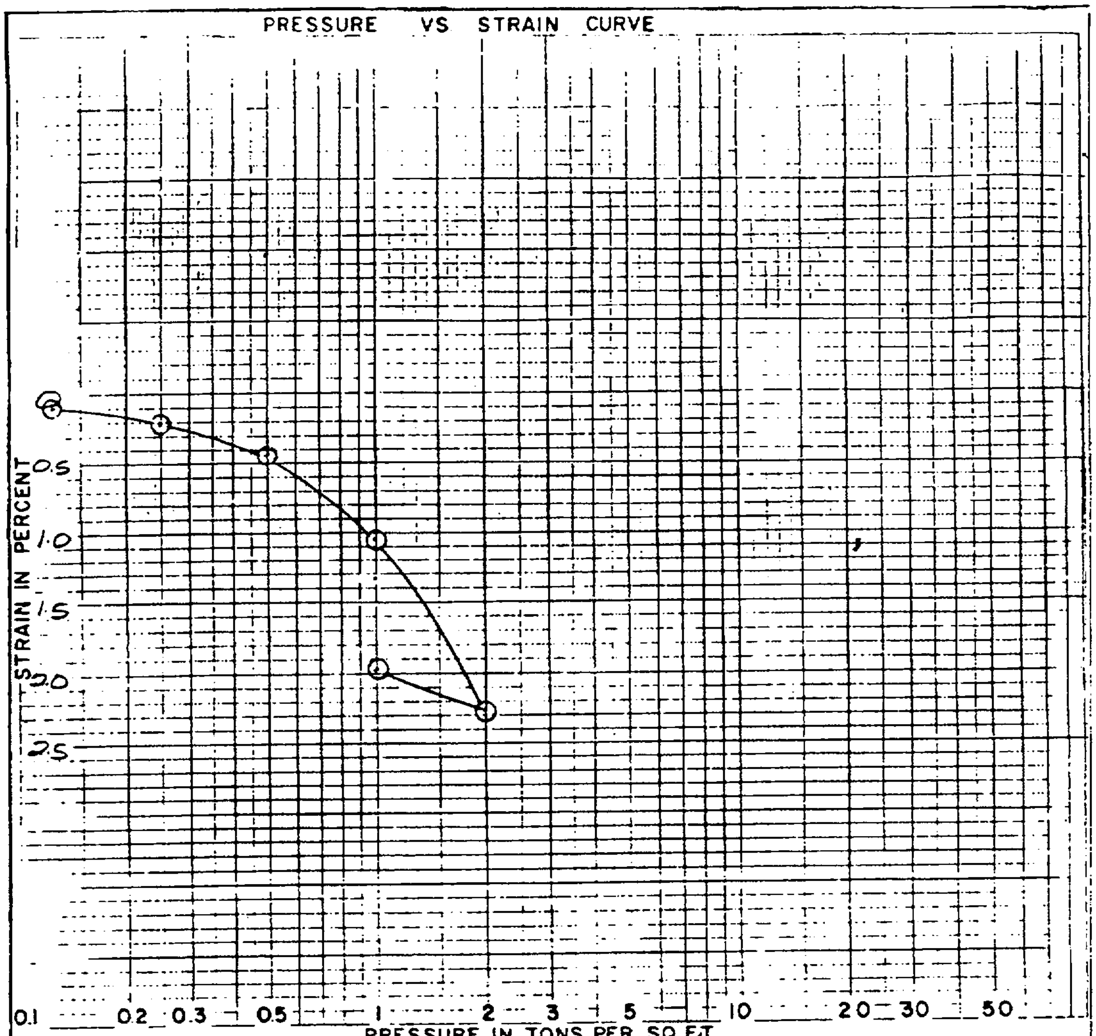
DESCRIPTION OF SPECIMEN:
SILTY CLAY TRACE FINE SAND - GRAY

CLASSIFICATION: CL STRATUM DESIGNATION: -

DIAMETER OF SPECIMEN (in.)	2.5	SCHNABEL ENGINEERING ASSOCIATES
INITIAL THICKNESS OF SPECIMEN (in.)	1.0	CONSOLIDATION TEST
INITIAL VOID RATIO, e_o	0.60	
PROBABLE PRECONSOLIDATION STRESS (tsf) Pc	-	BORING NO.: B-5U DEPTH: 7'-9'
APPROXIMATE OVERBURDEN STRESS (tsf) Po	-	PROJECT: DRY CASK SPENT
COMPRESSION RATIO; FROM 1 TO 2 (tsf)	0.066	FUEL FACILITY, SURRY CO. VA
COMPRESSION INDEX; FROM 1 TO 2 (tsf)	0.110	
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.010	DATE 6/11/82
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.018	CONTRACT NO. VE2162

SI020668

Figure 2.6-46 (SHEET 7 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

CLAY, TRACE FINE SAND - LIGHT BROWN

CLASSIFICATION: CH STRATUM DESIGNATION: –

DIAMETER OF SPECIMEN (in.)	2.50
INITIAL THICKNESS OF SPECIMEN (in.)	1.00
INITIAL VOID RATIO, e_o	–
PROBABLE PRECONSOLIDATION STRESS (tsf) Pc	–
APPROXIMATE OVERBURDEN STRESS (tsf) Po	–
COMPRESSION RATIO; FROM___TO___(tsf)	–
COMPRESSION INDEX; FROM___TO___(tsf)	–
RECOMPRESSION RATIO; FROM___TO___(tsf)	–
RECOMPRESSION INDEX; FROM___TO___(tsf)	–

SCHNABEL ENGINEERING ASSOCIATES

CONSOLIDATION TEST

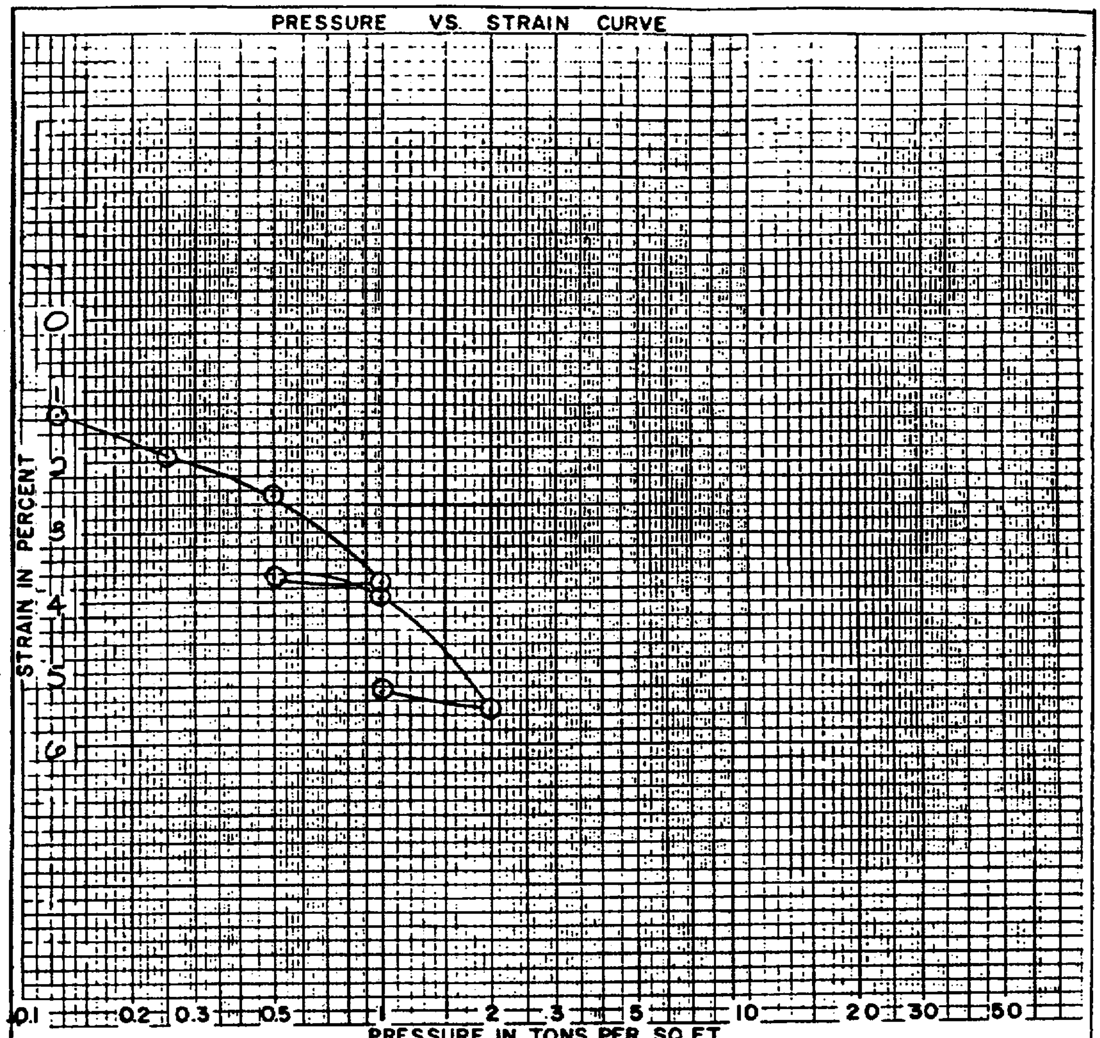
BORING NO.: B-8 DEPTH: 2'-4'

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE	CONTRACT NO.
6/11/82	VE21EZ

SI020669

Figure 2.6-46 (SHEET 8 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

SILTY CLAY, SOME FINE SAND - LIGHT GRAY AND BROWN

CLASSIFICATION: CL STRATUM DESIGNATION:

DIAMETER OF SPECIMEN (in.)	2.50
INITIAL THICKNESS OF SPECIMEN (in.)	1.00
INITIAL VOID RATIO, e_o	0.459
PROBABLE PRECONSOLIDATION STRESS(tsf)Pc	–
APPROXIMATE OVERBURDEN STRESS (tsf) Po	–
COMPRESSION RATIO; FROM 1 TO 2 (tsf)	0.056
COMPRESSION INDEX; FROM 1 TO 2 (tsf)	0.110
RECOMPRESSION RATIO; FROM 1/2 TO 1 (tsf)	0.010
RECOMPRESSION INDEX; FROM 1/2 TO 1 (tsf)	0.019

SCHNABEL ENGINEERING ASSOCIATE

CONSOLIDATION TEST

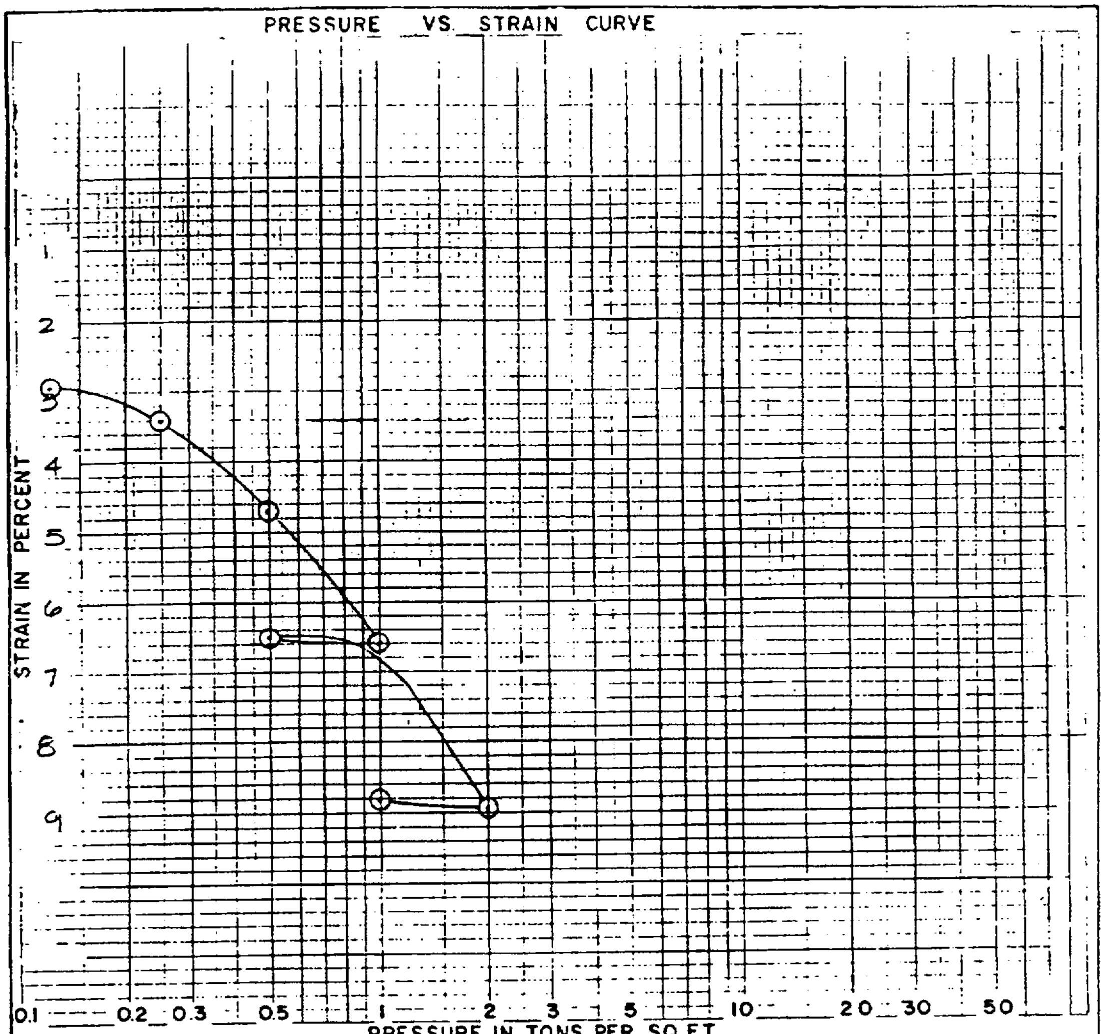
BORING NO.: B-8 DEPTH: 12'-14'

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO.V

DATE	CONTRACT NO.
6/11/82	V82182

SI020670

Figure 2.6-46 (SHEET 9 OF 9)
CONSOLIDATION TEST



DESCRIPTION OF SPECIMEN:

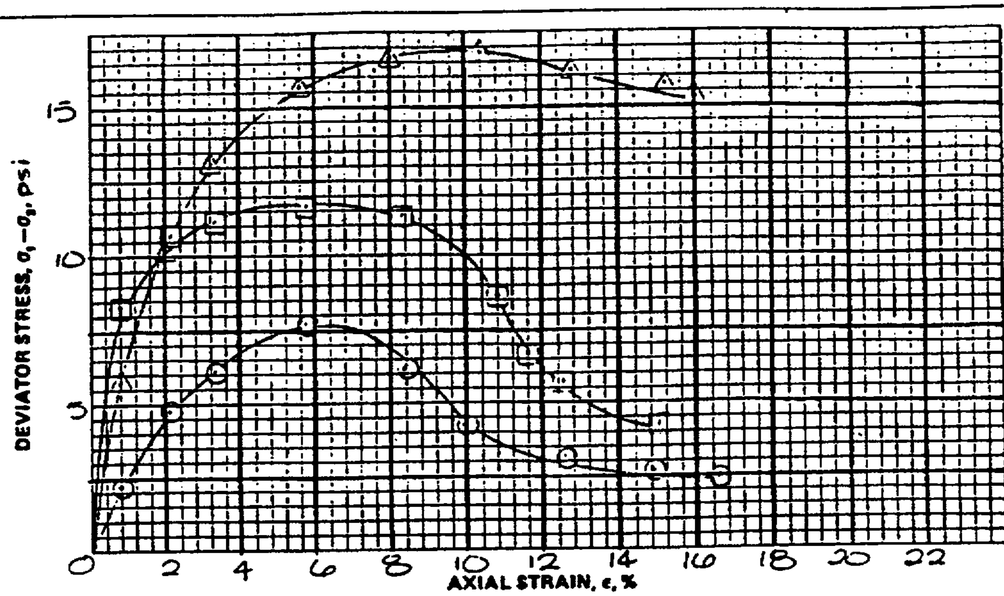
FINE SILTY SAND WITH SHELL FRAGMENTS - DARK GREENISH GRAY

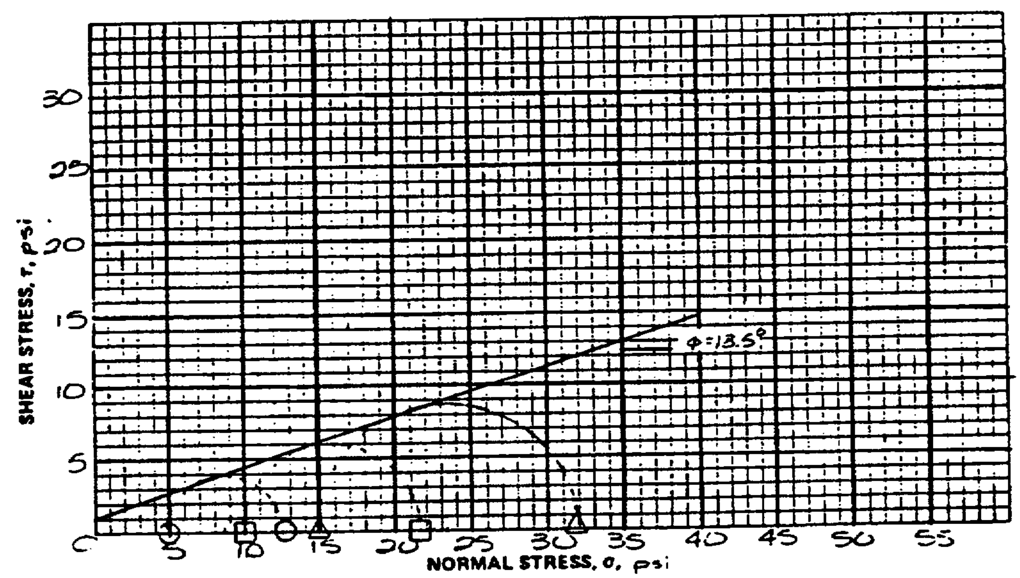
CLASSIFICATION: SM STRATUM DESIGNATION:

DIAMETER OF SPECIMEN (in.)	2.50	SCHNABEL ENGINEERING ASSOCIATES
INITIAL THICKNESS OF SPECIMEN (in.)	1.00	CONSOLIDATION TEST
INITIAL VOID RATIO, e_o	0.50	
PROBABLE PRECONSOLIDATION STRESS(tsf)Pc	-	BORING NO.: B-8 DEPTH: 42'-44'
APPROXIMATE OVERBURDEN STRESS (tsf) Po	-	PROJECT: D2V CASK SPENT
COMPRESSION RATIO; FROM 1 TO 2 (tsf)	0.041	
COMPRESSION INDEX; FROM 1 TO 2 (tsf)	0.061	FUEL FACILITY, SURRY CO, VA.
RECOMPRESSION RATIO; FROM __ TO __ (tsf)	-	DATE 6/11/82
RECOMPRESSION INDEX; FROM __ TO __ (tsf)	-	CONTRACT NO. VE2182

SI020671

Figure 2.6-47 (SHEET 1 OF 12)
TRIAXIAL COMPRESSION TEST





DESCRIPTION OF SOIL SAMPLE TESTED

SILTY CLAY, TRACE FINE SAND-
BROWN & TAN (CL)

KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
○	B-2	2'-4'	5 psi	23.1	24.9	—	—
□	B-2	2'-4'	10psi	23.1	25.9	9?.4	122.3
△	B-2	2-4'	15psi	23.1	26.2	95.7	117.6

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

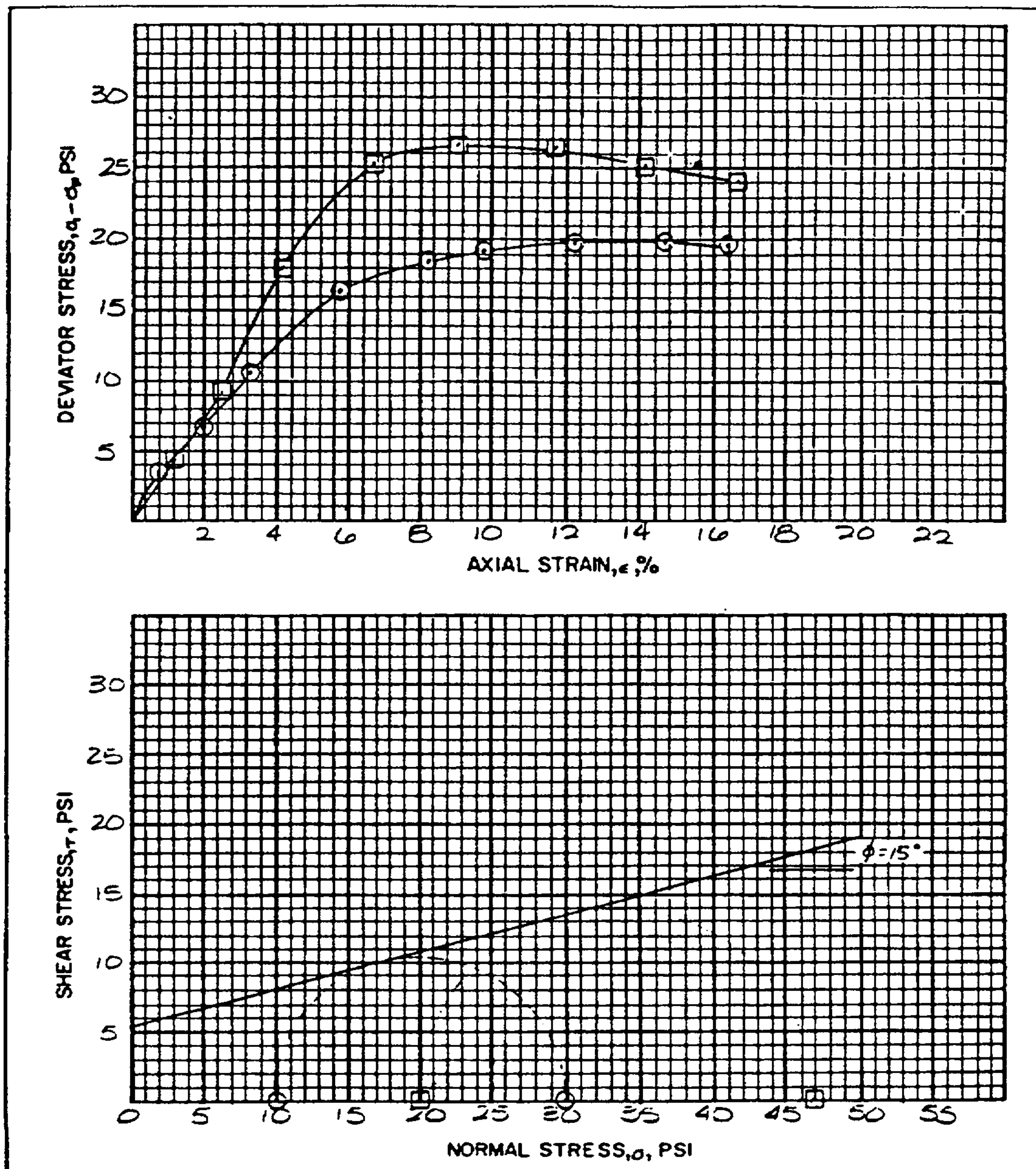
TYPE OF TEST: UU
RATE OF SHEAR: 1%/MIN

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO. VA.

DATE: 6/11/82 CONTR. NO.: [illegible]

SI020672

Figure 2.6-47 (SHEET 2 OF 12)
TRIAXIAL COMPRESSION TEST

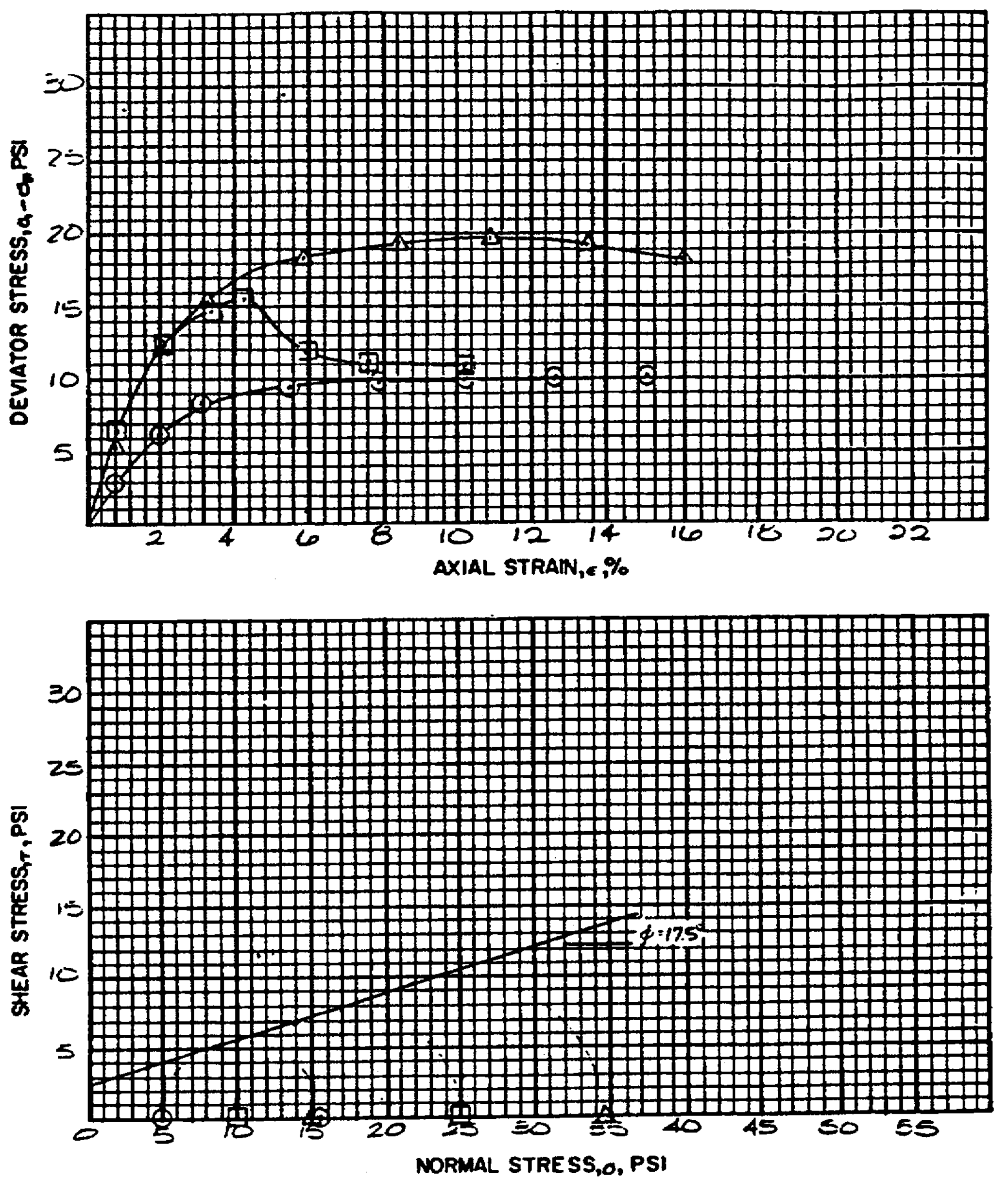


DESCRIPTION OF SOIL SAMPLE TESTED							
FINE SILTY SAND, WITH SHELL FRAGMENTS DARK GRAY (SM)							
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST CONT. % INITIAL	MOIST CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
○	B-2	52'-54'	10	31.3	27.0	92.6	119.0
□	B-2	52'-54'	20	31.3	30.0	90.7	119.1

SCHNABEL ENGINEERING ASSOCIATE	
TRIAXIAL COMPRESSION TEST	
TYPE OF TEST: UU RATE OF SHEAR: 1% /MIN	
PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.	
DATE: 6/11/82	CONTRACT NO.: V8218

SI020673

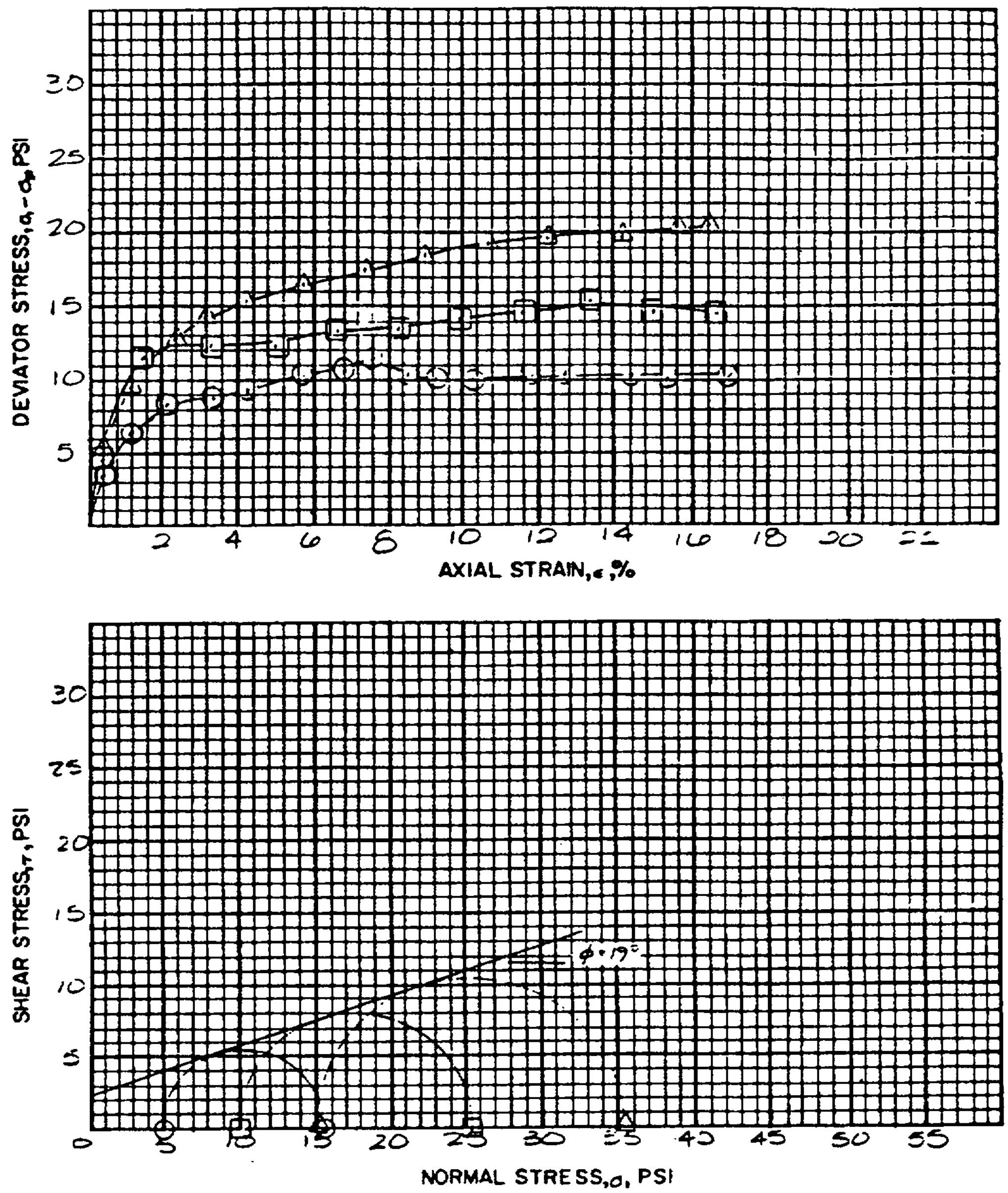
Figure 2.6-47 (SHEET 3 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED								SCHNABEL ENGINEERING ASSOCIATES
SILTY CLAY, SOME FINE SAND - GRAY & BROWN (CL)								TRIAXIAL COMPRESSION TEST
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET	TYPE OF TEST: UU. RATE OF SHEAR. 1%/MIN
⊙	B-2U	7'-9'	5	29.2	28.5	85.5	110.4	PROJECT: DRY CASK SPENT FUEL
⊡	B-2U	7'-9'	10	31.2	24.5	93.3	122.4	FACILITY, SURRY CO, VA.
△	B-2U	7'-9'	15	29.4	25.0	-	-	DATE: 6/11/82 CONTRACT NO.: V82182

SI020674

Figure 2.6-47 (SHEET 4 OF 12)
TRIAXIAL COMPRESSION TEST



KEY	BORING NO.	DEPTH FT	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
○	B-5U	1'-3'	5	29.0	35.8	90.5	116.7
□	B-5U	1'-3'	10	29.0	-	87.3	112.6
△	B-5U	1'-3'	15	29.0	33.4	87.9	113.4

DESCRIPTION OF SOIL SAMPLE TESTED: CLAY, TRACE FINE SAND - LIGHT BROWN AND TAN (CH)

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

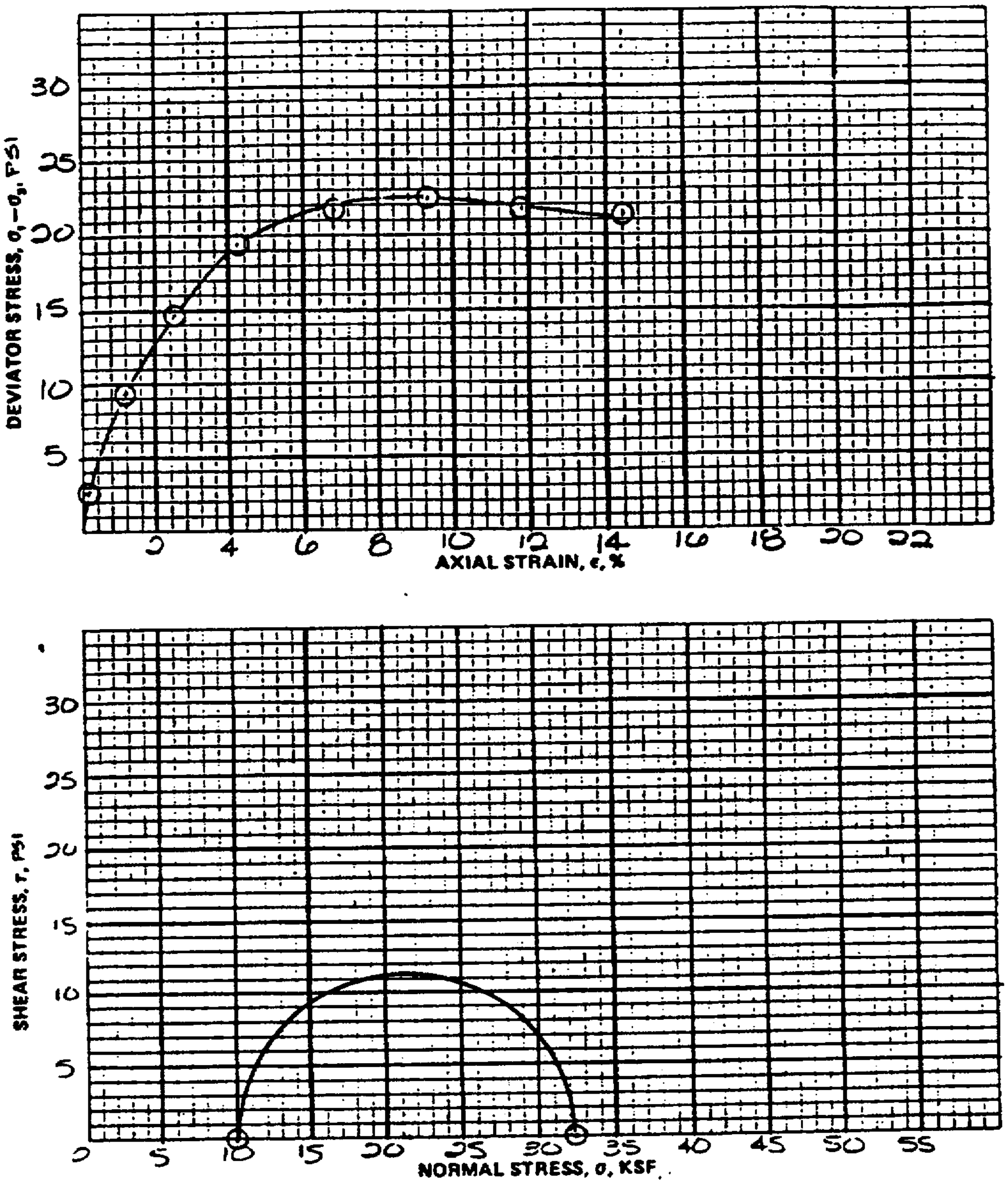
TYPE OF TEST: UU
RATE OF SHEAR: 1%/MIN

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO, VA.

DATE: 6/11/82
CONTRACT NO.: V82182

SI020675

Figure 2.6-47 (SHEET 5 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED

CLAY, TRACE FINE SAND - LIGHT BROWN (CH)

KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	B-8	2'-4'	10 PSI	34.7	-	66.3	114.9

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

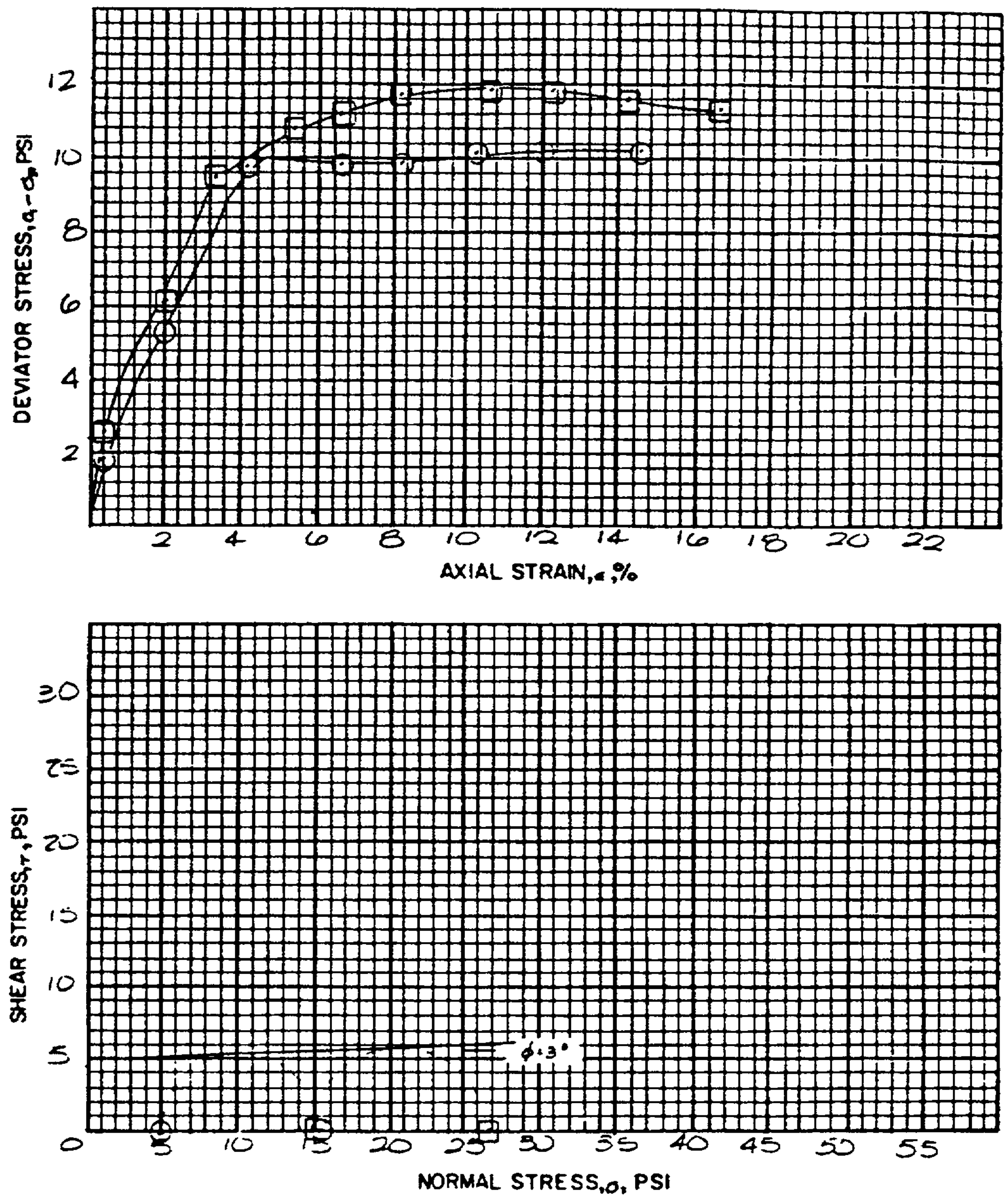
TYPE OF TEST: UU
RATE OF SHEAR: 1% /MIN

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY, VA.

DATE: 6/11/82 CONTR. NO. VB2182

SI020676

Figure 2.6-47 (SHEET 6 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED							
SILTY CLAY, SOME FINE SAND - LIGHT GRAY AND BROWN (CL)							
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST CONT. % INITIAL	MOIST CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	B-8	12'-14'	5	29.5	31.1	93.3	120.9
⊡	B-8	12'-14'	15	29.5	31.8	90.6	117.3

SCHNABEL ENGINEERING ASSOCIATE[S]

TRIAXIAL COMPRESSION TEST

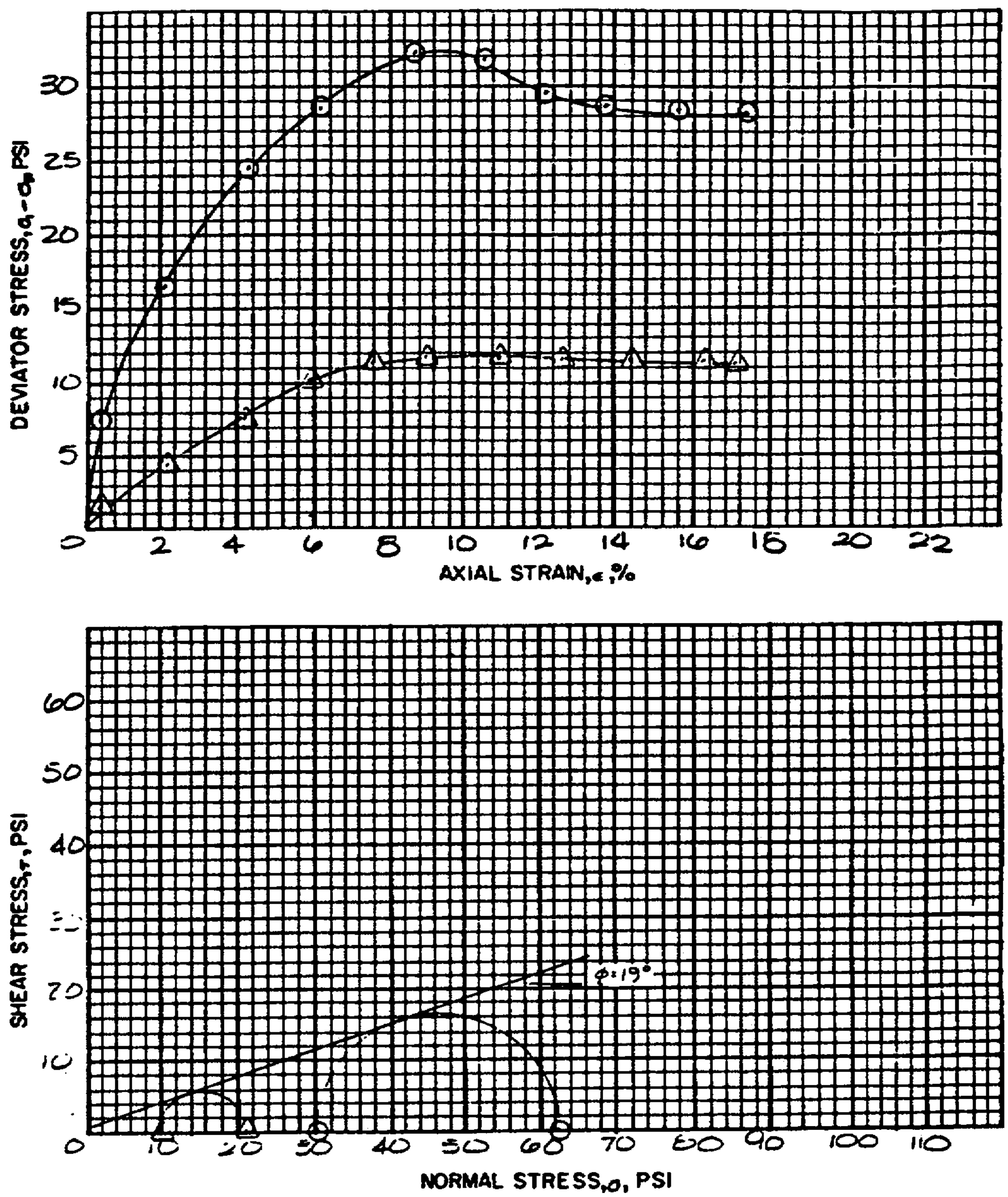
TYPE OF TEST: UU
RATE OF SHEAR: 1% /MIN

PROJECT: DRY CASK SPENT FUE[L]
FACILITY, SURRY CO, VA.

DATE: 6/11/82 CONTRACT NO.: VE21E

SI020677

Figure 2.6-47 (SHEET 7 OF 12)
TRIAXIAL COMPRESSION TEST

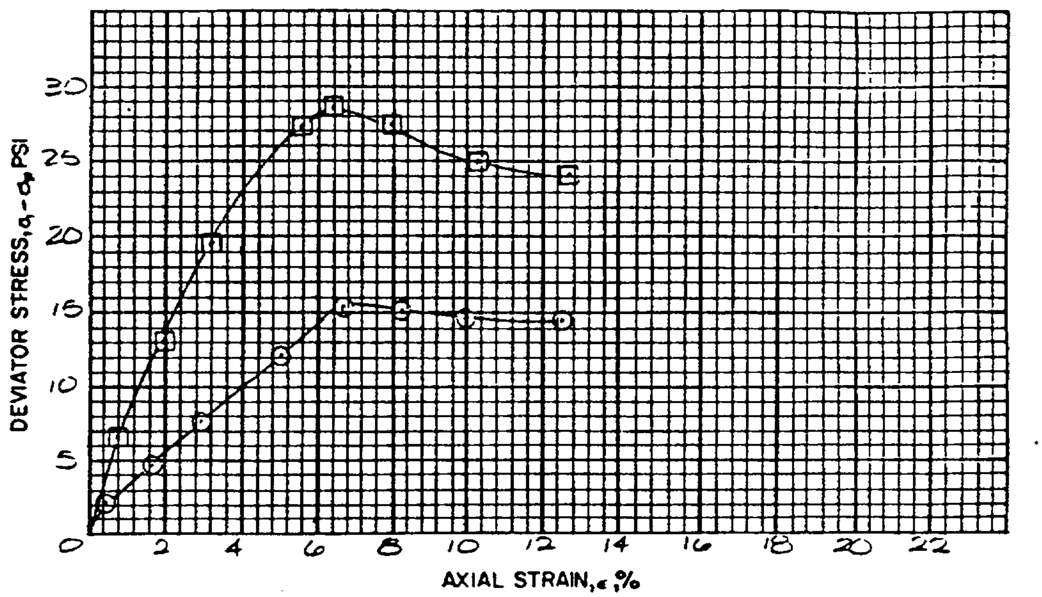


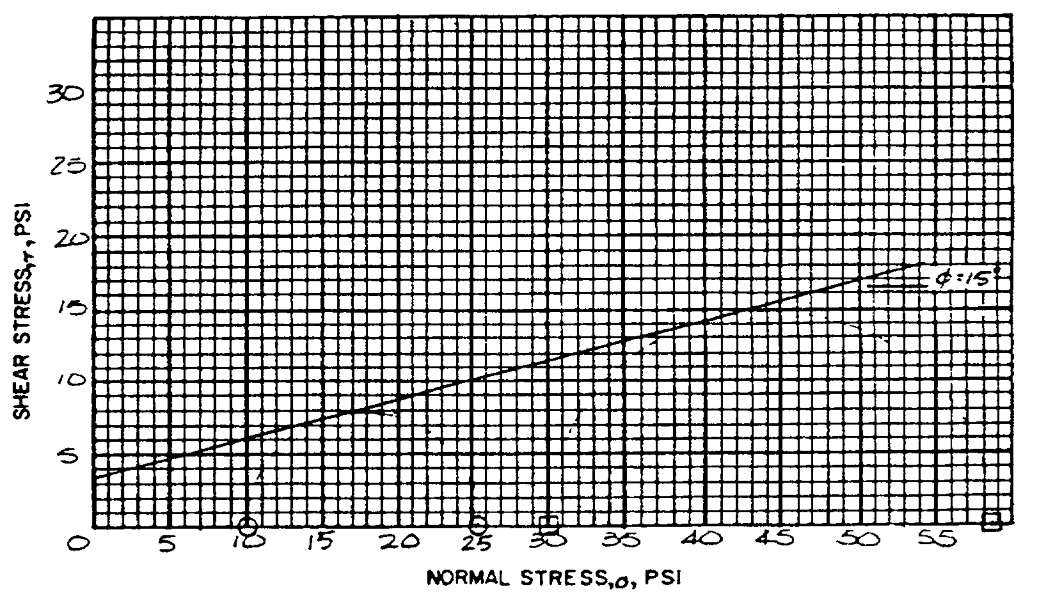
DESCRIPTION OF SOIL SAMPLE TESTED							
FINE SILTY SAND WITH SHELL FRAGMENTS - DARK GREENISH GRAY (SM)							
KEY	BORING NO.	DEPTH FT	LATERAL PRESS.	MOIST. CONT. %		DENSITY, PCF	
				INITIAL	FINAL	DRY	WET
⊙	B-8	42'-44'	20 psi	32.9	27.6	88.7	117.9
△	B-8	42'-44'	10 psi	32.9	25.2	90.8	120.7

SCHNABEL ENGINEERING ASSOCIATES	
TRIAXIAL COMPRESSION TEST	
TYPE OF TEST: UU RATE OF SHEAR: 1%/MIN	
PROJECT: DRY CASK SPENT	
FUEL FACILITY, SURRY CO. VA.	
DATE: 6/11/82	CONTRACT NO.: V821E2

SI020678

Figure 2.6-47 (SHEET 8 OF 12)
TRIAXIAL COMPRESSION TEST





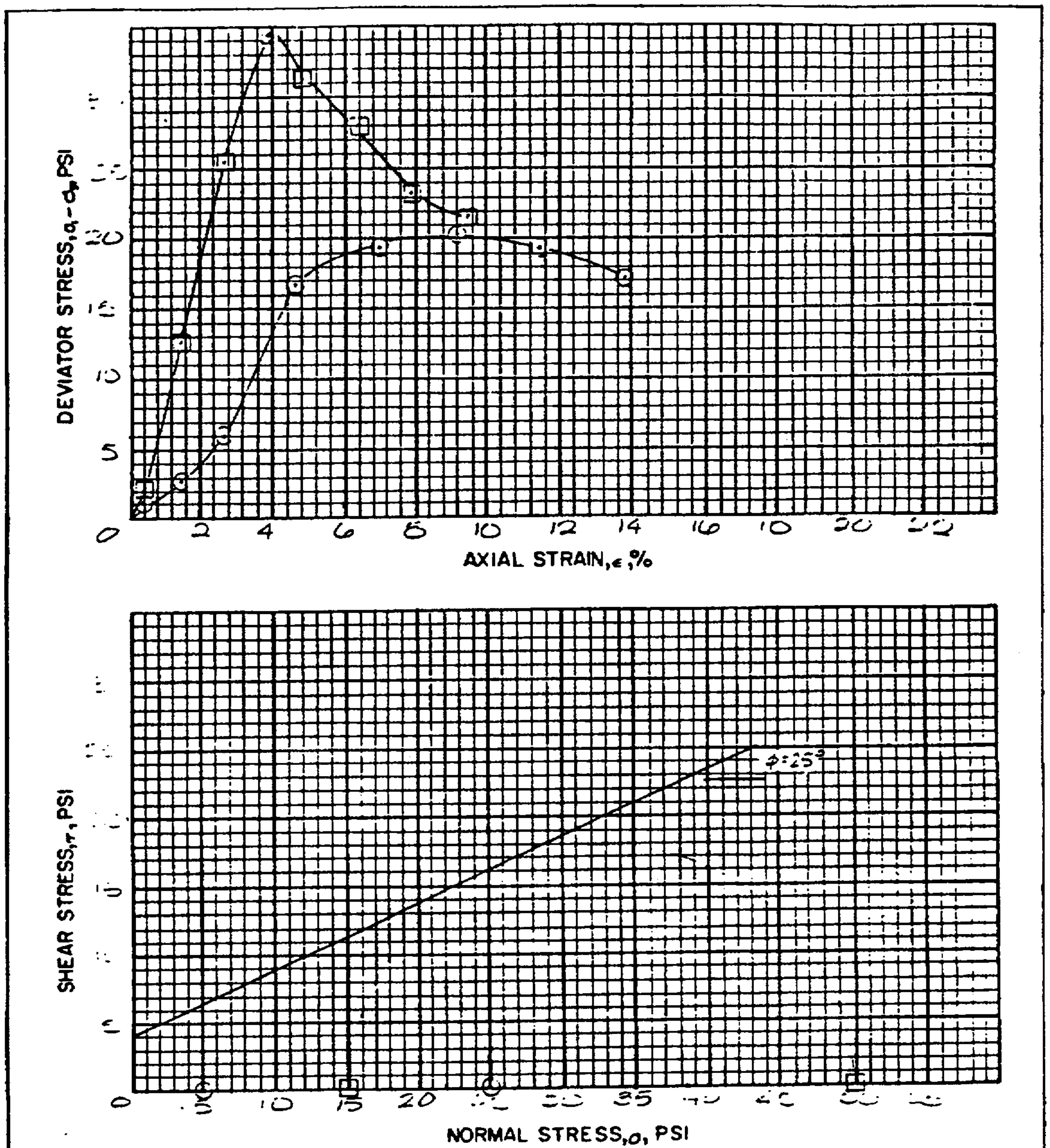
DESCRIPTION OF SOIL SAMPLE TESTED							
FINE SILTY SAND WITH SHELL FRAGMENTS - DARK GRAY (SM)							
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST CONT. % INITIAL	MOIST CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊡	B-8	62'-6"	30	33.8	32.7	86.0	117.7
⊙	B-6	62'-6"	10	30.8	28.9	90.6	118.0

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST
TYPE OF TEST: UU
RATE OF SHEAR: 1% /MIN
PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO, VA.
DATE: 6/11/82 CONTRACT NO: V82182

SI020679

Figure 2.6-47 (SHEET 9 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED
CLAY, TRACE FINE SAND, WITH SHELL FRAGMENTS - DARK GREENISH GRAY (CH)

KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST. CONT. % INITIAL	MOIST. CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
○	B-6	82-84	5	44.7	45.[illegible]	75.5	10.8
□	B-8	82-84	15	45.0	45.6	73.6	105.9

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

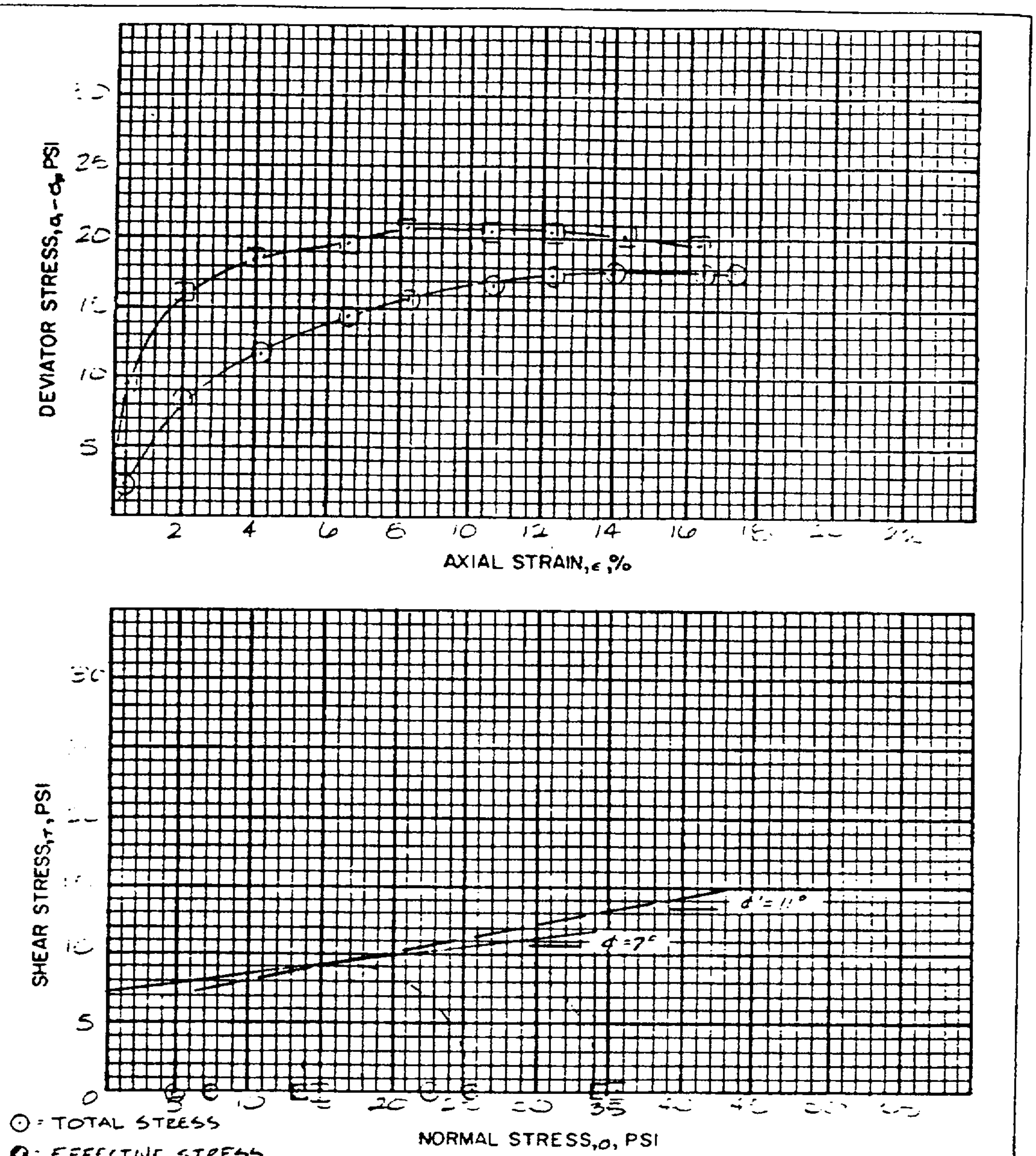
TYPE OF TEST: UU
RATE OF SHEAR: 1%/MIN

PROJECT: DRY CASK STORAGE [illegible]
FACILITY, SURRY CO, VA.

DATE: 6/11/82 CONTRACT NO: [illegible]

SI020680

Figure 2.6-47 (SHEET 10 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED							
SILTY CLAY, TRACE FINE SAND-BROWN AND TAN (CL)							
KEY	BORING NO.	DEPTH FT.	LATERAL PRESS.	MOIST CONT. % INITIAL	MOIST CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	B-2	2-4'	5	34.9	-	85.3	114.1
⊡	B-2	2-4'	15	[illegible]	28.7	81.5	[illegible]

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

TYPE OF TEST: CU

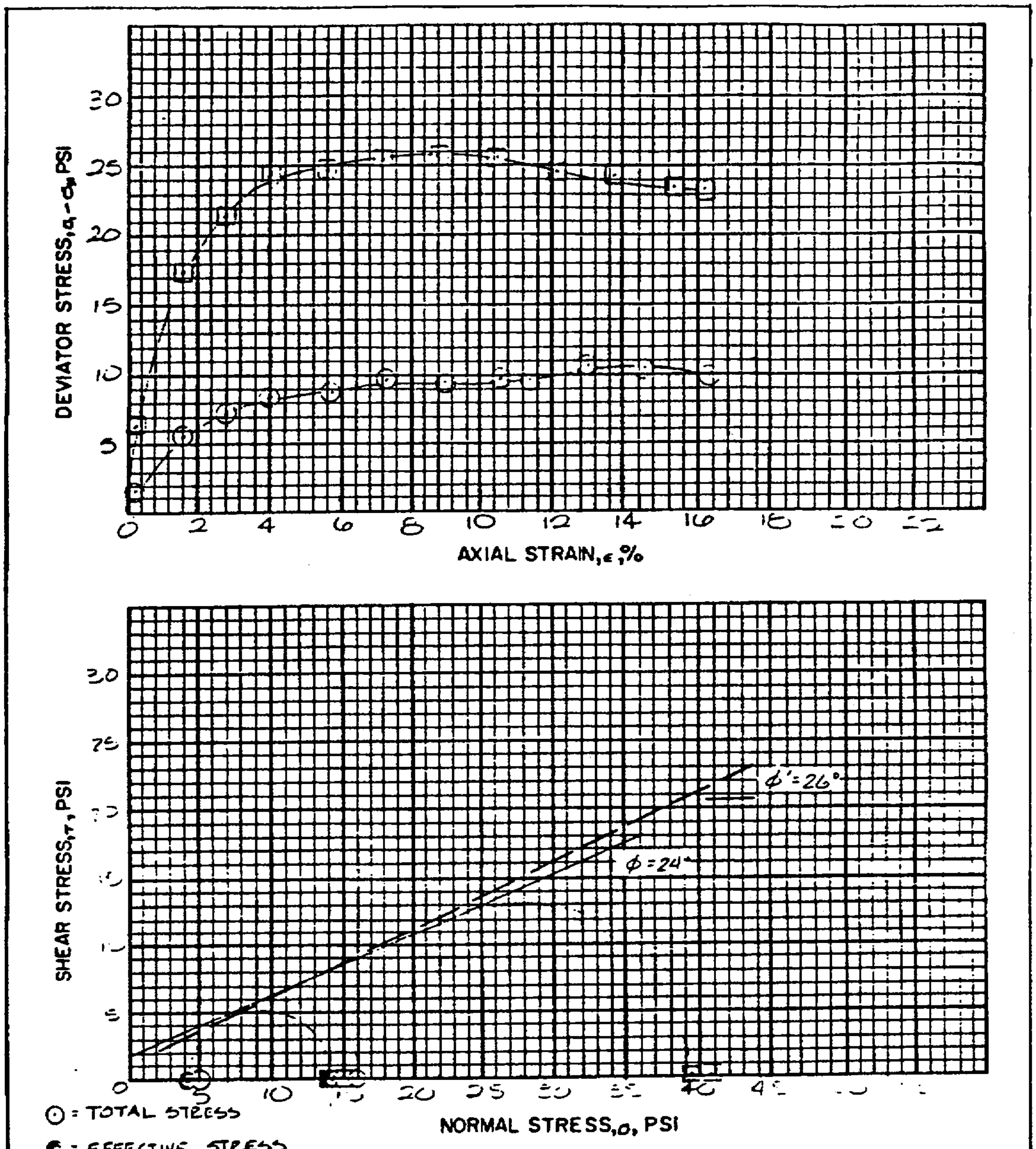
RATE OF SHEAR: 1% /MIN

PROJECT: DRY CASK SPENT FUEL FACILITY, SURRY CO., VA.

DATE: 6/11/82 CONTRACT NO.: [illegible]

SI020681

Figure 2.6-47 (SHEET 11 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED							
CLAY, TRACE FINE SAND - GRAY & BROWN (CH)							
KEY	BORING NO.	DEPTH FT	LATERAL PRESS.	MOIST CONT % INITIAL	MOIST CONT % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
⊙	[illegible] 1	7-[illegible]	[illegible]	[illegible]	[illegible]	92.4	[illegible]
☐	[illegible] 2	7-[illegible]	[illegible]	27.4	27.5	9[illegible]4	[illegible]

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

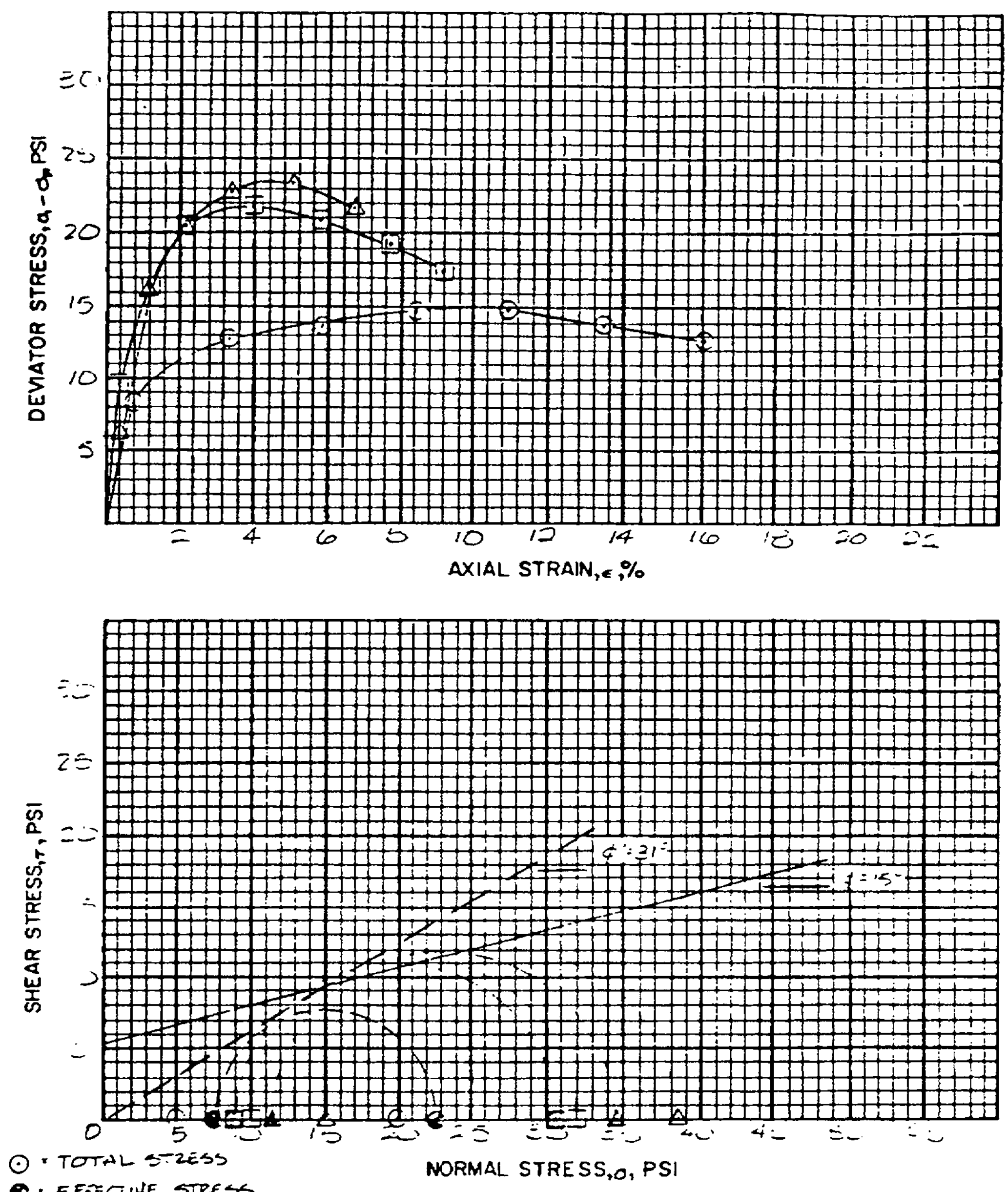
TYPE OF TEST: [illegible]
RATE OF SHEAR: [illegible]

PROJECT: [illegible]
FACILITY, SURRY CO. VA

DATE: 6/11/82
CONTRACT NO.: [illegible]

SI020682

Figure 2.6-47 (SHEET 12 OF 12)
TRIAXIAL COMPRESSION TEST



DESCRIPTION OF SOIL SAMPLE TESTED

SILTY CLAY, TRACE FINE SAND - GRAY (CL)

KEY	BORING NO	DEPTH FT	LATERAL PRESS	MOIST CONT. % INITIAL	MOIST CONT. % FINAL	DENSITY, PCF DRY	DENSITY, PCF WET
○	B-50	7-9'	5	24.1	33.6	91.7	113.8
□	B-50	7-9'	10	24.1	-	94.1	116.8
△	B-50	7-9'	15	24.1	-	93.9	116.5

SCHNABEL ENGINEERING ASSOCIATES

TRIAXIAL COMPRESSION TEST

TYPE OF TEST: CU
RATE OF SHEAR 1%/MIN

PROJECT. DRY CASK SPENT FUEL
FACILITY, SURRY CO. VA.

DATE 6/11/82 CONTRACT NO: VE[illegible]

SI020683

Figure 2.6-48
SOIL PROFILE A-A'

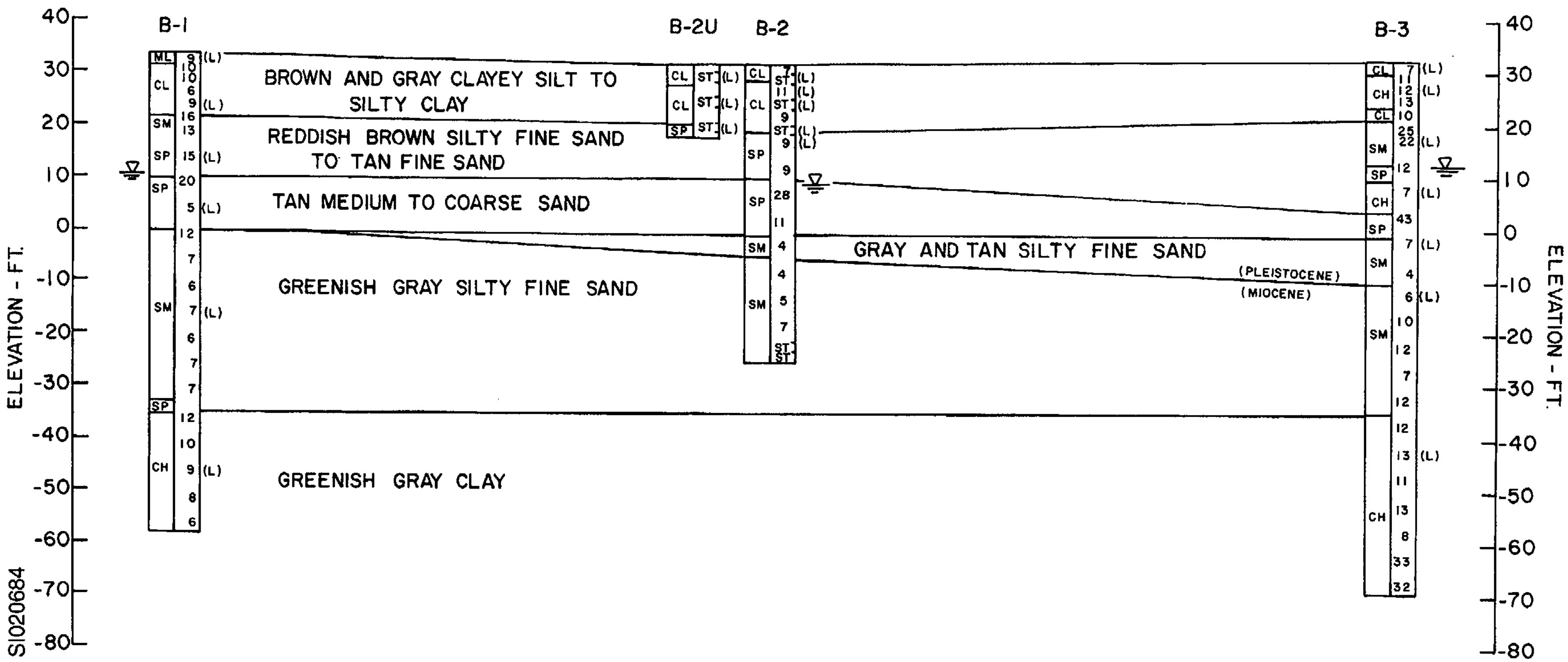


Figure 2.6-49
SOIL PROFILE B-B'

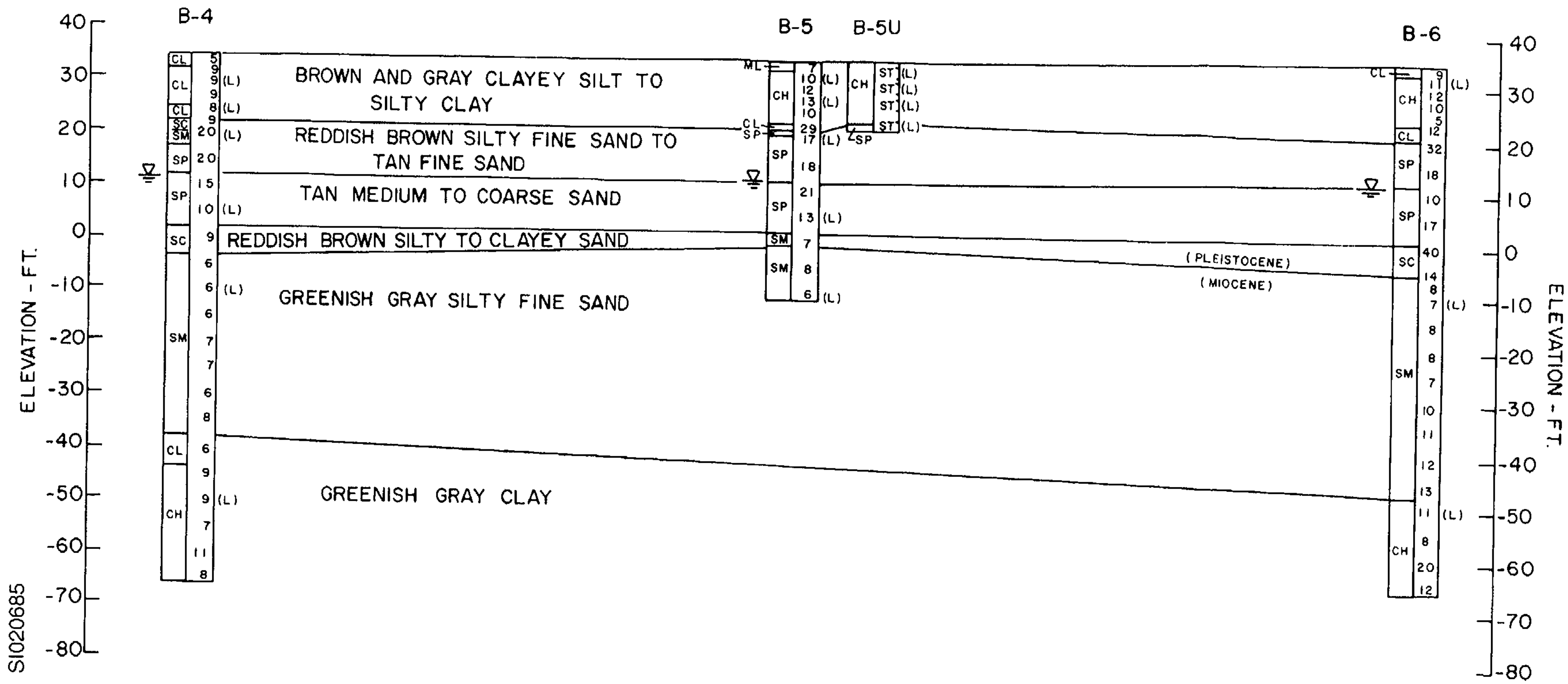


Figure 2.6-50
SOIL PROFILE C-C'

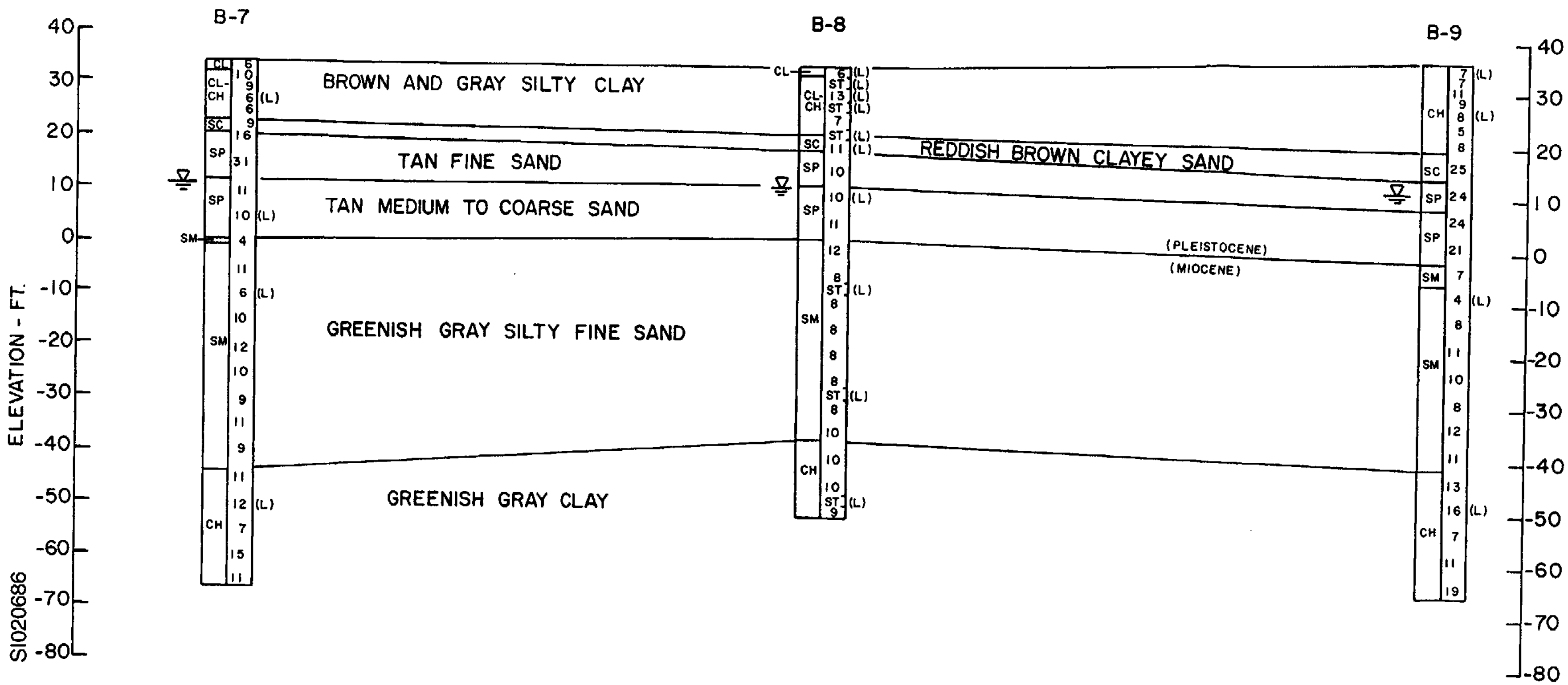


Figure 2.6-51
EXCAVATION PLAN AND PROFILE

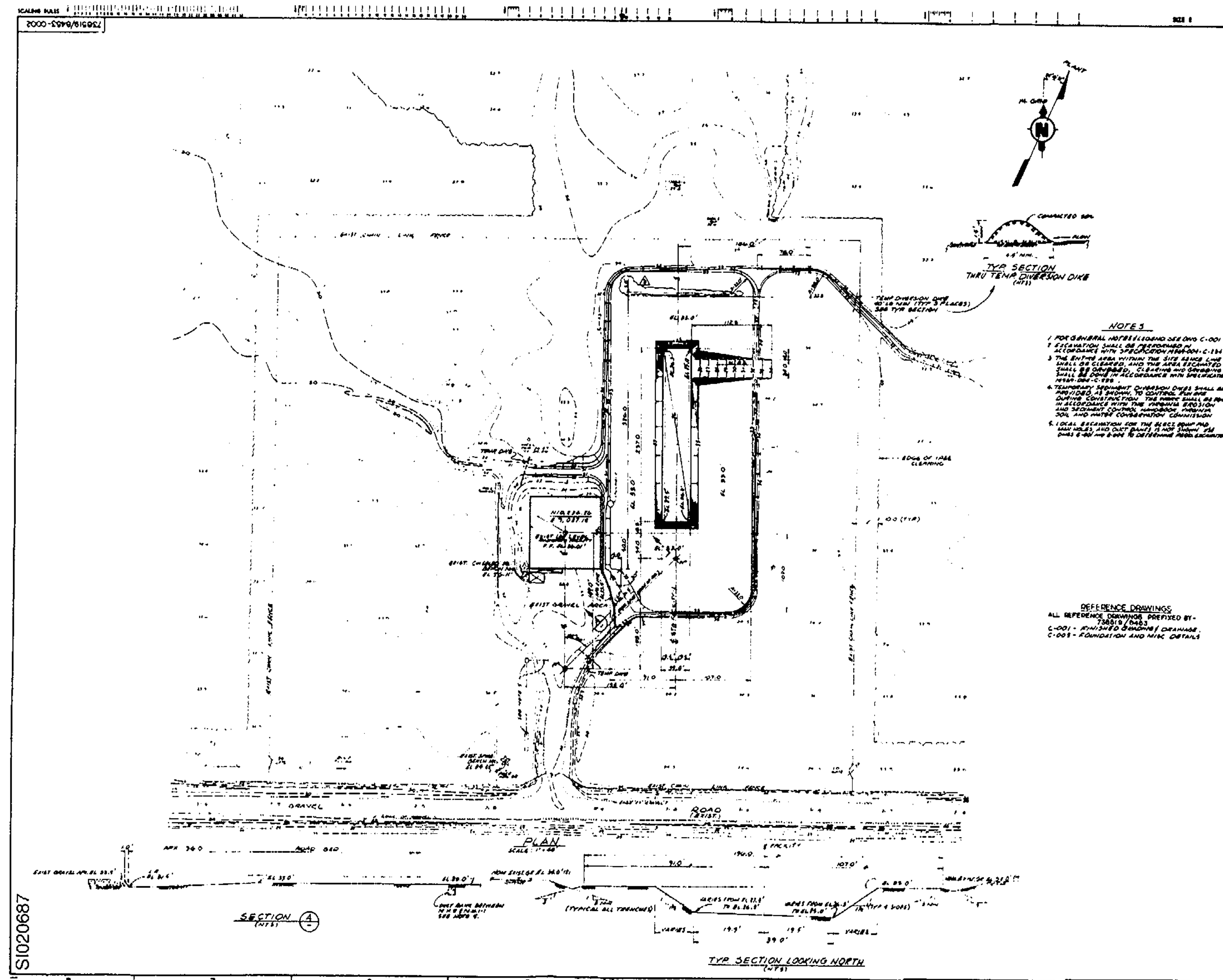
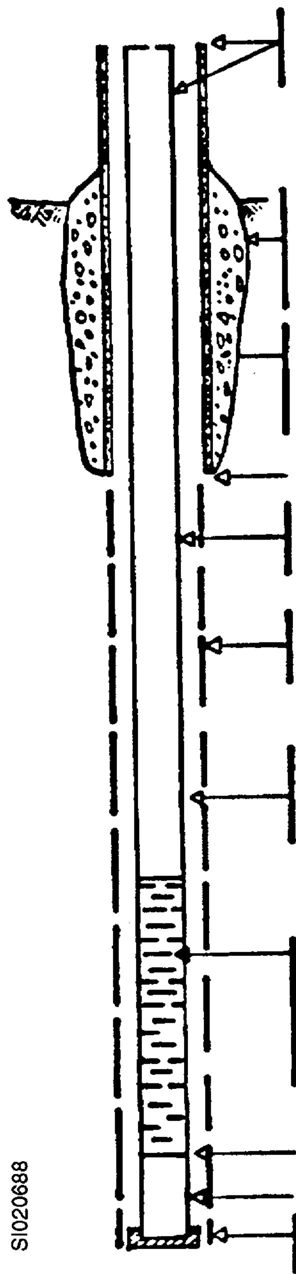


Figure 2.6-52
OBSERVATION WELL CONSTRUCTION DETAIL



Height of top of surface casing above ground surface	2.0 ft.
Height of top of well casing above ground surface	1.5 ft.
Depth of surface seal below ground surface	5.0 ft.
Type of surface seal: Concrete	
I.D. of surface casing	4.0 in.
Type of surface casing: Steel	
Bottom of surface casing below ground	3.0 ft.
I.D. of well casing	1.5 in.
Type of well casing: Schedule 80 PVC	
Depth of top of gravel pack below ground surface	5.0 ft.
Type of gravel pack: Sand	
Diameter of borehole	6 in.
Depth of top of screened section	22.0 ft.
Type of screened section: Schedule 80 PVC	
Type of perforations: 15 horizontal grooves, 1-inch long, 0.01-inch opening per inch.	
I.D. of screened section	1.5 in.
Depth of bottom of screened section	32.0 ft.
Length of blank section	0.5 ft.
Depth of bottom of plugged blank section	32.5 ft.
Total depth of hole	33 ft.

SI020688

Figure 2.6-53
STRESS REDUCTION FACTOR

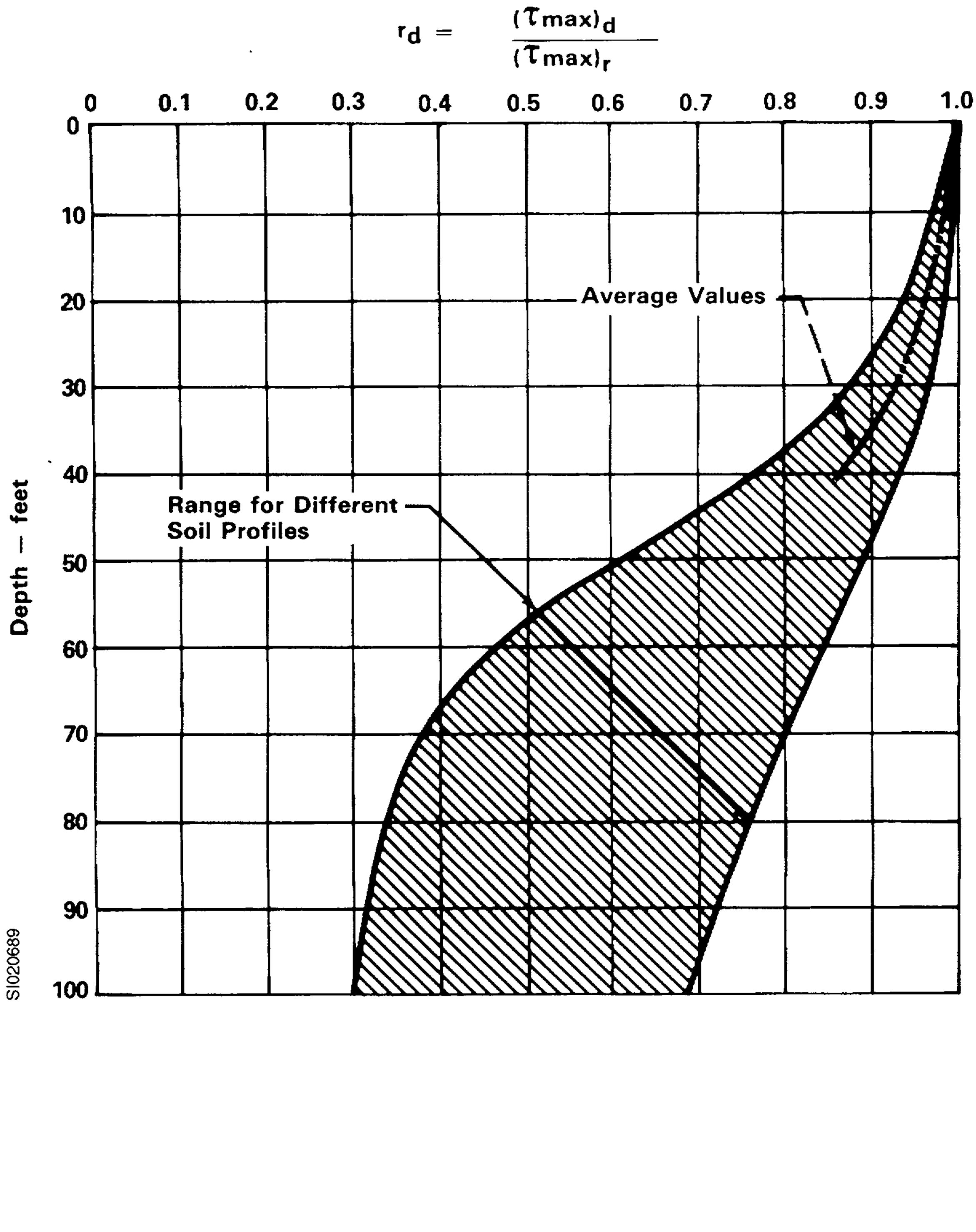


Figure 2.6-54
CHART FOR EVALUATION OF LIQUIFICATION POTENTIAL
FOR DIFFERENT MAGNITUDE EARTHQUAKES

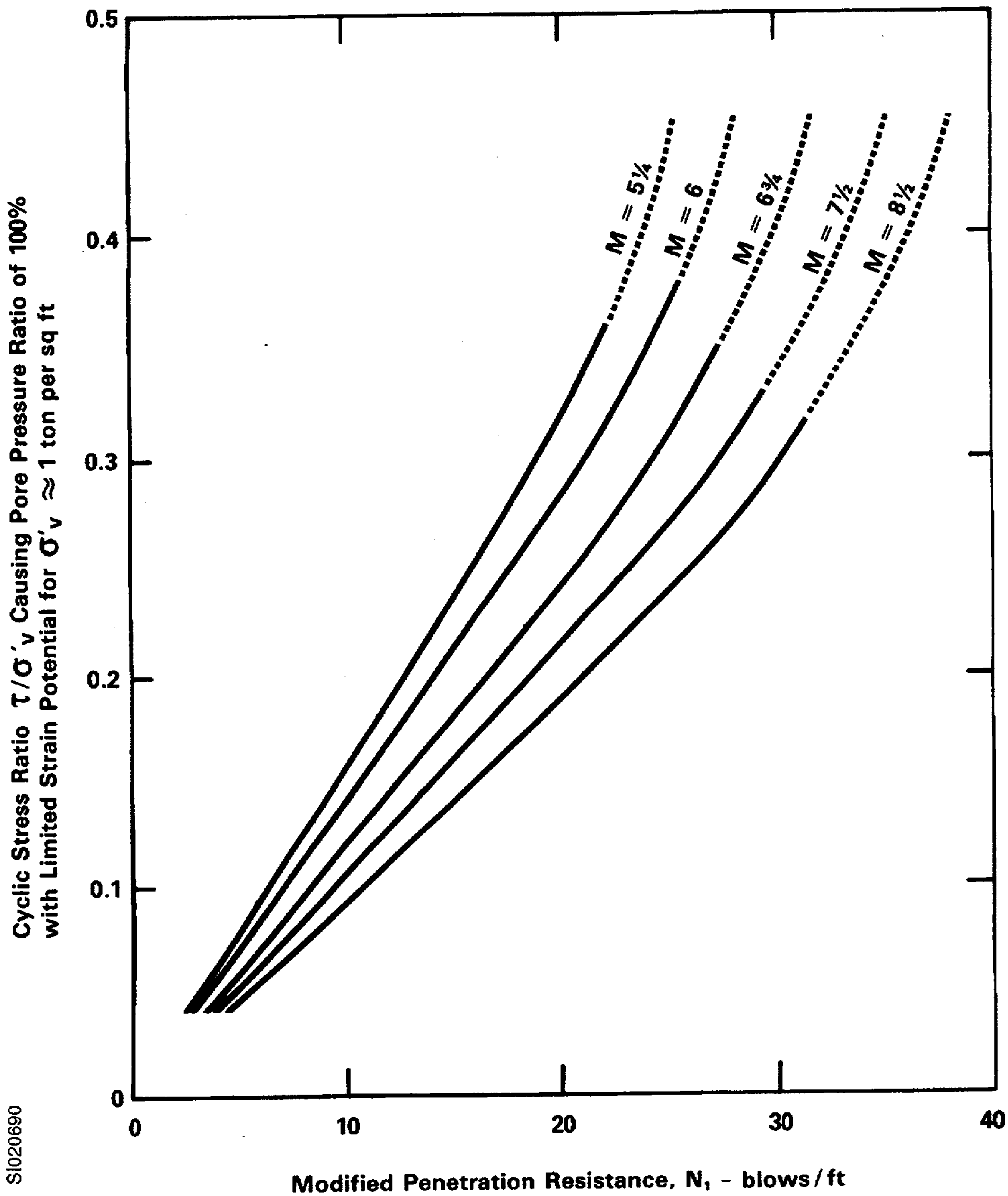
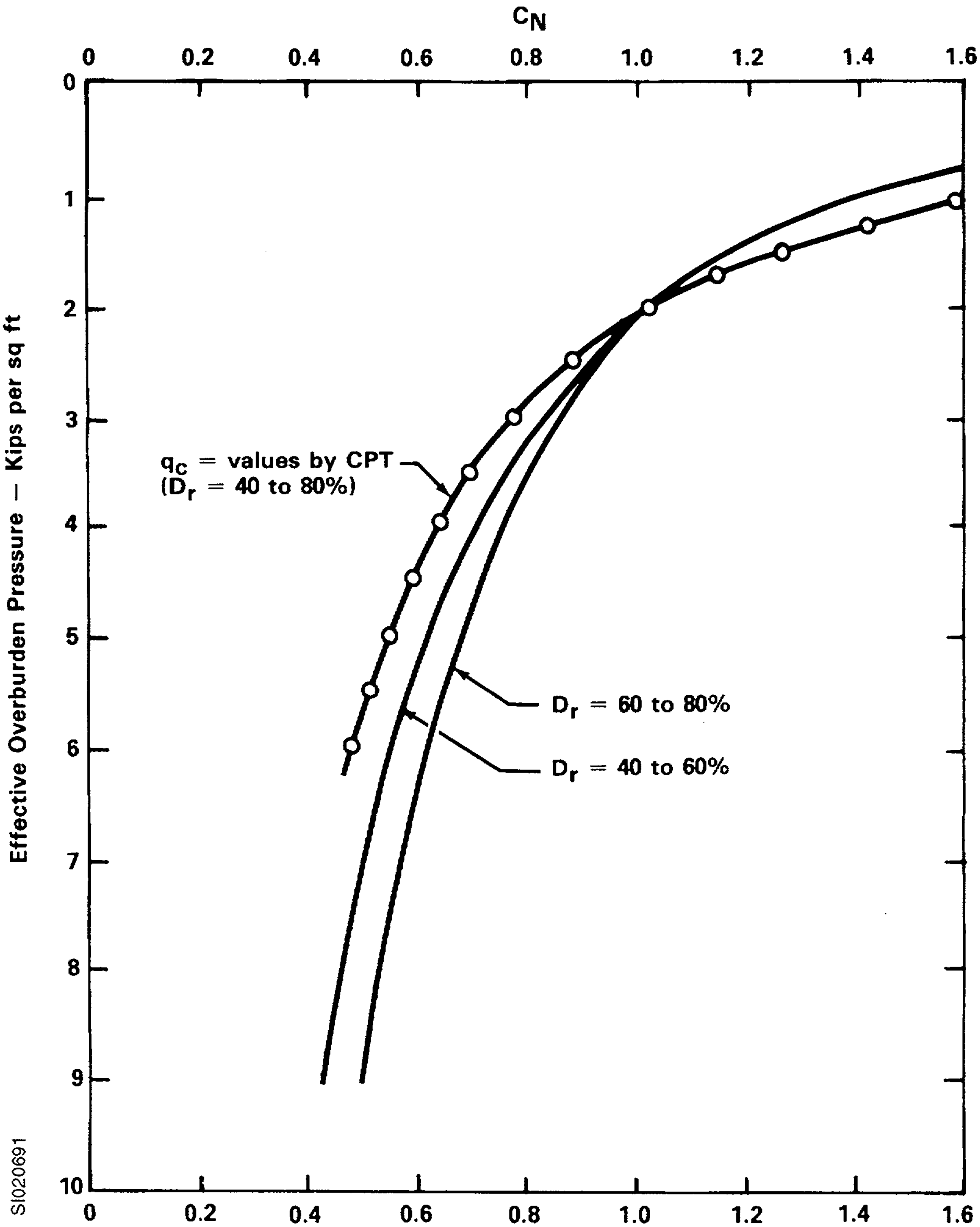


Figure 2.6-55
STANDARD PENETRATION ADJUSTMENT FACTORS



2.7 SUMMARY OF SITE CONDITIONS AFFECTING CONSTRUCTION AND OPERATING REQUIREMENTS

The site specific phenomena and characteristics described in this chapter have been used to define appropriate design criteria, as described in Chapter 3. See Table 2.7-1 for a summary of site specific information either newly established for the ISFSI or previously established for the Surry Power Station.

Table 2.7-1 (SHEET 1 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
1. Ambient temperature	2.3.1.1, 2.3.2.1, 3.2.1.1	-20°F to 115°F	Range bands extreme temperatures reported in ISFSI SAR Section 2.3.5, References 1-3 and 11-14 and actual site data. See also the response to NRC Question 1.3.1.	Newly developed for the ISFSI. References cited are identical to References 1-6 and 9 of SPS UFSAR Section 2.2. Site data are also discussed in SPS UFSAR Section 2.2.1.
2. Direct exposure to sunlight	3.2.1.1	0.800 cal/cm^2	Based on NRC Regulatory Guide 7.8; ISFSI SAR Section 3.2.6, Reference 1; and the response to NRC Question 1.3.2.	Newly developed for the ISFSI. Not applicable to SPS.
3. Ambient humidity	3.2.1.1	0 to 100%	Range encompasses all possible values.	Range encompasses all possible values.
4. Tornado pressure drop	2.3.1, 3.2.1.1	3 psi in 3 seconds	SPS UFSAR Section 2.2.1 and References 13 and 14 of SPS UFSAR Section 2.2.	Same value as established for Surry Power Station. See the discussion in SPS UFSAR Section 2.2.2.1.
5. Tornado winds	2.3.1, 3.2.1.1	Rot. vel. -300 mph Trans. vel. -60 mph or R.G. 1.76	SPS UFSAR Section 2.2.2.1 R.G. 1.76	R.G. 1.76 values may be used in lieu of those established for SPS in SPS UFSAR Section 2.2.2.1

Table 2.7-1 (SHEET 2 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
6. Wind direction	2.3.2.1.2	Predominantly from southwest and south southwest	Based on ISFSI SAR Section 2.3.5, Reference 1-3 and 11-14, and actual site data.	Predominant wind directions are the same as established for Surry Power Station in SPS UFSAR Section 2.2.1. References cited are identical to References 1-6 and 9 of SPS UFSAR Section 2.2.
7. Wind direction persistence	2.3.2.1.3	30 hrs (30.3 ft) 28 hrs (147.4 ft)	Based on ISFSI SAR Section 2.3.5, Reference 1-3 and 11-14, and actual site data.	Wind direction persistence is the same value as established for Surry Power Station in SPS UFSAR Section 2.2.1. References cited are identical to References 1-6 and 9 of SPS UFSAR Section 2.2.
8. Average wind speed	2.3.2.1.2	5.8 mph (30.3 ft) 9.8 mph (147.4 ft)	Based on ISFSI SAR Section 2.3.5, Reference 1-3 and 11-14, and actual site data.	Annual average wind speeds are the same as established for Surry Power Station in SPS UFSAR Table 2.2-5. References cited are identical to References 1-6 and 9 of SPS UFSAR Section 2.2.

Table 2.7-1 (SHEET 3 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
9. Maximum winds (V_{30})	2.3.1.3.1, 3.2.1.1	105 mph	ISFSI SAR Section 2.3.5, Reference 4	Same value as established for Surry Power Station in SPS UFSAR Section 2.2.2.2. Reference cited is the same as Reference 17 of SPS UFSAR Section 2.2.
10. Gustiness factor	2.3.1.3.1, 3.2.1.1	1.3	ISFSI SAR Section 2.3.5, Reference 5	Same value as established for Surry Power Station in SPS UFSAR Section 2.2.2.2. Reference cited is the same as Reference 18 of SPS UFSAR Section 2.2.
11. Maximum flood level	2.4, 3.2.2	28.2 ft msl.	Based on ISFSI SAR Section 2.4.10, References 1-6	Same value as established for Surry Power Station in SPS UFSAR Section 2.3.1.2. References cited are identical to References 3, 5, 7-9, and 11 of SPS UFSAR Section 2.3.

Table 2.7-1 (SHEET 4 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
12. Explosive peak over-pressure	2.2.3.1	1 psi	Established based on calculations and assumptions in ISFSI SAR Section 2.2.4, References 1 and 3-8.	Same value as established for Surry Power Station in SPS UFSAR Section 2.1.4.3. References cited are identical to References 9 and 11-15 of SPS UFSAR Section 2.1. Reference 5 of ISFSI SAR Section 2.2.4 was a personal communication that served to provide additional background information on the non-explosive behavior of unconfined gasoline vapor clouds.
13. Atmospheric dilution value (χ/Q)	2.3.4	1.56×10^{-3} sec/m^3	Calculations based on NRC Regulatory Guide 1.145. See also the responses to NRC Questions 1.3.5E and 1.3.6.	Same value as developed for the Low Level Waste Storage Facility.
14. Fires	2.2.3.2	Maximum increase of 8°F over ambient temperature	Calculations based on ISFSI SAR Sections 2.2.3.2 and 2.2.4, References 9-11.	Newly developed for the ISFSI.

Table 2.7-1 (SHEET 5 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
15. Population distributions	2.1.3	Updated population distributions are provided in the responses to NRC Questions 1.1.1 and 1.1.2E through 1.1.5E.	Population distributions were determined based on References 1 and 2 of the response to Question 1.1.1 and References l-5 of the response to Question 1.1.5E.	The population data contained in the responses to NRC Questions 1.1.1 and 1.1.2E through 1.1.5E update the information in SPS UFSAR Section 2.1. References cited were either previously submitted to NRC for SPS or are reports developed by state or federal agencies.
16. Lightning surge	2.3.1.3.6 (Future)	See the response to NRC Question 1.3.3.	See the response to NRC Question 1.3.3	Newly developed for the ISFSI.

Table 2.7-1 (SHEET 6 OF 6)
SITE CHARACTERISTICS SUMMARY

FACTOR	ISFSI SAR SECTION(S) REFERENCE	VALUE OR RANGE	SOURCE	COMPARISON TO SURRY POWER STATION UNITS 1 AND 2 (SPS)
17. Design earthquake peak acceleration	2.6, 3.2.3	0.07g	References contained in ISFSI SAR Section 2.6.6. See also the response to NRC Question 1.4.1.	Newly developed for the ISFSI. In ISFSI SAR Sections 2.6.1 and 2.6.2 the basic geology and tectonic information developed for the ISFSI is the same as established for the Surry Power Station. However, due to the passive safety function, the ISFSI design value is lower. ISFSI SAR Section 2.6.6 indicates references from the SPS PSAR for Units 3 and 4. Selected references utilized are taken from SPS UFSAR Sections 2.4 and 2.5.

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Appendix 2A

APPENDIX 2A
NRC COMMENT/RESPONSE 2.59 TO SURRY POWER STATION UNITS 3 & 4 PSAR

2A.1 Introduction

This Appendix contains the NRC Comment 2.59 to the PSAR for Surry Units 3 and 4 (1973) and the response generated for the Units 3 and 4 PSAR. It is presented in order to further explain the evidence or lack of evidence concerning the postulated Hampton Roads fault.

2A.2 General

COMMENT 2.59 (Section 2.5.1.1.(6), Tectonics) It is the staff's position that the applicant shall present evidence to demonstrate on sound geological and geophysical arguments whether the Hampton Roads fault postulated by Cederstrom, Bull, AAPG, Vol. 29, p 71, 1945, and supported by Rogers and Spencer, Bull GSA, Vol. 82, p 2314, 1971, is or is not a fault. If the feature proves to be a fault, the applicant is required to provide information to demonstrate the age of the most recent movement that it has experienced.

RESPONSE

The Hampton Roads fault was first proposed by Cederstrom in 1945 on the basis of well and geophysical data available at the time. The fault was proposed to explain apparent differences in thickness of Eocene sediments north and south of the James River. Primary in his hypothesis are three deep wells near Chesapeake Bay (Section F-F, Figure 2A-1). Figure 2A-2 shows the geologic cross section. The oil prospecting well at Mathews struck rock at El. -2297 and the well at Fort Monroe encountered rock at El. -2236. The well at Norfolk never reached bedrock before it was abandoned at El. -1750 ft.

Cederstrom (Reference 1) states on page 81:

> "Sets of samples from old deep wells at Fort Monroe were restudied in this laboratory and it was found that... Eocene foraminifers were present from 604 to 1440 feet; in addition, as already noted, Eocene macrofossils have been determined from material collected at 1440 feet; thus the lower boundary of the Eocene at Fort Monroe is about 725 feet lower than where the base of the Eocene was placed by early investigators." (Cederstrom (Reference 2) later changed the base of the Eocene to agree with that of the "early investigators.")

"The thickening of the Eocene deposits from Norfolk city waterworks to Fort Monroe is from 75 feet to more than 800 feet as shown in the cross sections EE' and FF'," Figures 2A-3 and 2A-2 respectively.

Using similar data and extrapolating known stratigraphic indexes westward, Cederstrom postulated a continuous trend of abrupt Eocene thickening along the James River and Hampton Roads area. Geologic cross sections developed by Cederstrom are located on Figure 2A-1 and presented on Figures 2A-2 to 2A-5.

Cederstrom summarizes his observations as follows:

Reference 1, page 85:

> "When the thicknesses of Eocene sediments on either side of James River and Hampton Roads are considered.... it is apparent that either subsidence occurred in the area north of the river in pre-Eocene time, allowing a much greater thickness of Eocene sediments to accumulate there than in the area on the south, or the pre-Eocene surface was deeply channeled with the same result.
>
> "The short distance in which thickening occurs, the apparent uniform thickness of the Eocene sediments in the whole Virginia Coastal Plain north of James River and Hampton Roads, and the progressive decrease in thickening upward seem to indicate that a basin formed in pre-Eocene time, probably by faulting action."

Reference 2, page 71:

> "The fault is thought to trend westward along the James River and approach the Fall Zone; the maximum displacement along the postulated fault, from 300 to 600 feet, occurs in the Hampton Roads area."

Reference 2, page 88:

> "In the Hampton Roads areas the Miocene boundaries, as shown in Section EE' and FF', are apparently unaffected, and it seems that movement along the fault ceased before Miocene time began."

Cederstrom postulated the fault occurred in the area of abrupt thickening, but refrained from showing it in his sections. He conceded that some of the northward thickening of the Eocene sediments might have resulted from deposition in a pre-Eocene channel. The topography of the Coastal Plan convinced Cederstrom that 700-foot erosion channels were improbable and therefore he postulated the Hampton Roads fault. Since the bottom of the Norfolk well in Figure 2A-2 was 486 feet higher than the rock encountered at Fort Monroe it was possible to postulate a fault with somewhat less than 486 feet of displacement. This reduced the required depth of pre-Eocene channeling to about 250 feet; something that Cederstrom considered "not too easily visualized." IL should be noted that rock was not encountered at Norfolk but that this line of reasoning amounts to assuming it was just below the bottom of the well.

Later in 1945, Cederstrom (Reference 3) expressed some concern about the classification of soils from wells south of the James River. He states:

"It may be recalled here that the Upper Cretaceous strata described by Darton are characterized by thin indurated layers but, on the other hand, recent studies show that indurated strata are by no means confined to Upper Cretaceous deposits and the possibility that these strata and overlying brightly colored beds may be of Eocene age must be borne in mind pending further information."

In Cederstrom's last study of the area (Reference 4), published in 1957, he concludes that his original (1945) classification and stratigraphic indexing was wrong. He explains (page 1) that "some previously held conceptions of Eocene and pre-Eocene stratigraphy have been greatly revised." He further states (page 25):

> "In previous publications (Cederstrom, 1945a, p. 36-37, pl. 1, and 1945c, p. 81-82, Fig. 6-7) the Eocene was said to be as much as 800 feet thick. This conclusion was based on the presence of Eocene foraminifera as reported by Cushman, on the presence of glauconitic sand in sediments thus designated, and by the report of Eocene macrofossils found at 1440 feet in the old U.S. Army well at Fort Monroe.
>
> "The pre-Eocene Mattaponi formation is characteristically glauconitic; the writer is satisfied that the Eocene foraminifera found at depth in the well cuttings from Fort Monroe are forms first appearing much higher and were washed down. The Eocene macrofossils found at 1440 feet at Fort Monroe are believed to have fallen from above or to have been improperly labeled when collected. It may be noted that no "rock" layer is reported in well 8c (Table 36) in which the fossils are said to have occurred but, on the other hand, a "calcareous rock crust and pebble conglomerate with some wood and shells" is logged between 840 and 850 feet in the Chamberlain Hotel well (9, Table 36). This log description is the only one in the two wells that fits the fossiliferous material shown to the writer by L. W. Stephenson.
>
> "The thickness of all the Eocene formations in Newport News may be as much as 240 feet, if the macrofossil was taken at that depth. The writer is inclined to believe it may not be much more than 125 feet thick. In any event, grating a thickness of 240 feet, the thickening of the Eocene section is hardly more than moderate."

Cederstrom's 1957 reclassification of Eocene and Cretaceous stratigraphy north and south of the James River shows only moderate Eocene thickening and no structural disturbance. The 1957 geologic cross sections are shown on Figures 2A-6 and 2A-7.

In effect, Cederstrom's interpretations of stratigraphy in 1957 were essentially the same as those of the earlier investigators referred to in his 1945 paper (Reference 1). There is no thickening in Eocene, no erosion channel and therefore no need for Cederstrom to postulate the Hampton Roads fault.

Brown's (Reference 5) work in 1972, based on closer well control and more reliable data than the limited regional data available to Cederstrom (Reference 1) in 1945, further substantiates

the lack of structural disturbance of the Eocene and other sedimentary units. Brown's structural contours shown on Figure 2.6-9 and Figures 2A-8 through 2A-18 show no structural disturbance in the James River area.

The bedrock structural contours on Figure 2.6-9 show no disturbance. The same applies for the isopach contours on Figures 2.6-10 through 2.6-20. The figures cover a range in time from Cretaceous through Pleistocene. No abrupt thickening nor asymmetric isopach contour patterns are present as would be expected for fault type subsidence. Rather, large gradually varying isopach patterns are evident. These may be formed by gradual regional downwarping, differential compaction, erosion, or as a function of distance from the sediment source (deposition). The isopach centers vary in location with geological time and are not correlative with any localized structural effect.

A geological cross section across the James River near the plant site is shown on Figure 2A-19. The location of the Hampton Roads fault as proposed by Rogers and Spencer (Reference 6) is shown on this section. No structural disturbance is evident.

Rogers and Spencer (Reference 6) list localized dip reversals observed by Cederstrom (Reference 1) in 1945 as a reason for Cederstrom postulating the Hampton Roads fault. Cederstrom cited the dip reversals as examples of anomolous deformations in the Coastal Plain. He never related them directly to the proposed fault. Cederstrom (Reference 1) cited examples of dip reversal from Washington, D.C. to North Carolina and related them to general regional deformation, lensing, or to localized differential compaction. The dip reversal near Yorktown, Virginia was formed by differential compaction of underlying sediments as discussed in response to Comment 2.16. At Waverly, Virginia, Cederstrom (Reference 3) described the following:

> "From Disputanta to Waverly (Section B-B') the base of the Miocene deposits descends a minimum of 93 feet 7-1/2 miles in a west-east direction, but at Waverly it rises 11 feet in less than 1 mile eastward. However, the base of the Eocene glauconite beds falls 24 feet in this distance and hence the structure may be due to lensing rather than to deformation."

The cited dip reversals are therefore probably controlled by general regional subsidence, lensing, or to localized differential compaction rather than to any faulting.

Differences in stratigraphic position (sequence) of sediments north and south of the river were first presented by Cederstrom (Reference 1) in 1945. South of the James River Eocene sediments overlie Upper Cretaceous sediments whereas north of the river they overlie thinned Lower Cretaceous sediments. This difference was postulated as due to erosion not faulting. Cederstrom in 1957 (Reference 4) presents new evidence which shows that the Upper Cretaceous is present on both sides of the James River.

Cederstrom (Reference 1) never reported different bedrock depths north and south of the James River. He postulated them to circumvent the need for a 700-foot erosion channel which he considered impossible. The erosion channel was necessary in 1945 to explain a 700-foot increase

in the thickness of Eocene sediments north of the river. As shown above, Cederstrom (Reference 4), in 1957, no longer shows an increased Eocene thickness north of the river and therefore his postulated bedrock depth is not necessary. In fact, Figure 2.6-9 from Brown et al. (Reference 5) based on recent data (1972) shows no structural bedrock details indicative of faulting in the Hampton Roads area.

The conclusion from the above is that the geologic data which led Cederstrom to postulate the Hampton Roads fault in 1945 were disproved by him in 1957.

Gravity and magnetic data show a generally featureless area near the site. Interpretations of these geophysical data presented in responses to Comments 2.13 and 2.17 also show no structure in the vicinity of the site.

Rogers and Spencer (Reference 6) in 1971 published a paper which claimed to support the existence of the Hampton Roads fault based on their interpretations of the following:

1. Differences in chloride content in ground water north and south of the James River.
2. Different piezometric surface north and south of the river drill
3. Reversal in dip of strata indicated on electric logs of wells.

These are considered in the following:

1. Rogers and Spencer (Reference 6) present contours of groundwater chloride content in the York-James Peninsula. In general, a wedge of high chloride concentrations was found north of the James River and low concentrations are found south of the river. This is in accordance with Cederstrom's data (Reference 2) published in 1943 and shown on Figure 2A-20. Rogers and Spencer note an abrupt change in chloride concentration and conclude this is a result of a fault. Figure 2A-21 shows that the log of chloride concentration varies smoothly with distance. This form of variation has been observed in coastal aquifers (Reference 7) and is not the result of structural control. It is the result of hydrodynamic dispersion occurring at the boundary between salt water and fresh water.

   The location of the chloride wedge was explained by Cederstrom in 1943 (Reference 2). He concluded that his zones of high chloride content were a depositional remnant that had not been flushed out by fresh ground water. The contours presented by Rogers and Spencer are not referenced to individual wells. Cederstrom's data is shown on Figure 2A-20. Well depths are shown along with the chloride concentration in the ground water. It may be seen that deeper wells generally have higher chloride concentrations.

   Cederstrom also reported that variations in chloride concentration result from differences in permeability. This is consistent with the flushing of saline water concept. Rogers and Spencer (Reference 6) state:

"The Cretaceous and Eocene water-bearing sands may be considered as a unit since fluid communication exists between them; the result is that there are no great differences in water quality in these sands (Cederstrom 1943, 1945a, 1957)."

Cederstrom (Reference 4) page 81 states the following for Newport News:

"A chloride concentration of 1080 ppm was found at 400 feet, 600 ppm at 813 feet, 690 ppm at 900 feet; and 1680 ppm of chloride was present in water from well 13 (Table 37) at a depth of 820 feet. The excessively high chloride water sample from 400 feet was from a poorly producing stratum. The two samples lowest in chloride are from wells that are rather good producers and are in constant use, and the sample second highest in chloride is from a poor producer."

On page 46:

"There was also the possibility that chloride content might increase with pumping."

Cederstrom therefore recognized that the effect of depth, pumping rate, and permeability of the strata as well as the location, controlled chloride concentration. Recent evidence (References 8 & 9) shows that the aquifers are separated by aquitards and therefore direct hydraulic and chloride communications does not exist between aquifers and their response will be very time dependent. Chloride concentrations have been observed as a function of time (References 10 & 11).

A further complicating factor in the analysis of chloride from wells is that many of the wells are screened in more than one aquifer and that increased ground water pumping is changing the hydrodynamic and dispersion behavior of the saline-fresh water zone.

In summary it appears from geological evidence (Reference 4) that the high chloride wedge is depositional in nature; that Rogers and Spencer's (Reference 6) "abrupt" change in chloride content is only the normal coastal contact between fresh and salt water; and that the assumption of hydraulic communication vertically is not true.

2. Rogers and Spencer (Reference 6) make frequent references to the structural interpretation proposed by Cederstrom in 1945. As shown earlier in this response Cederstrom in 1957 greatly revised his previously held conceptions of Eocene and pre-Eocene stratigraphy and the structural data supporting the proposed Hampton Roads fault was thereby destroyed.

   Rogers and Spencer contour piezometric data "based on Cretaceous and Eocene static levels because of their fluid communication." As discussed in part a above, aquifers in the York/James area can be separated by aquitards and therefore fluid communication is retarded. Static water levels can be influenced by adjacent pumping wells as shown in Reference 11. Recharge, which is considered to provide a significant percentage of water to the aquifers (Reference 9), is not considered steady state recharge to a peninsula between two saline rivers, and would show a potentiometric high between them similar to Figure 2 by Rogers and Spencer. Nonsteady conditions complicate the potentiometric surface by highs

and should probably still occur if fresh water recharge continues. Recent studies in the area (Reference 8) show that the anomolous lows and highs are influenced by pumping and aquifer (Reference 9) thickness and permeability. Figure 2A-22 shows the potentiometric surface in 1900, Figure 2A-23 from 1937 to 1939, Figure 2A-24 from 1945 to 1948, and Figure 2A-25 from 1966 to 1969. It may be seen that the potentiometric surface is dropping with largest drops in the areas of highest pumping. The pumping has been greatest on the south side of the James River as explained by Cederstrom (Reference 4) and the potentiometric level has therefore decreased most there. One area near Franklin has been pumped so heavily that the potentiometric surface has dropped as much as 180 feet (Reference 11).

It is therefore evident that the potentimetric surface will continue to change with time as a function of pumping rates, local stratigraphic conditions, the aquifer or aquifers from which the wells pump, the proximity to wells or well groups, and the recharge occurring to the aquifers and aquitards from the surface. To conclude that structural controls are present requires that the hydrodynamic effects be considered, corrected for and interpreted. Rogers and Spencer (Reference 6) have not considered these effects and it is therefore concluded that no indication of structural control is evident in the potentiometric data.

3. Rogers and Spencer (Reference 6) interpret electric logs to show a vertical offset at the James River. Rogers and Spencer's Figure 3 shows no wells closer than 8 miles to the proposed fault. In addition, they arbitrarily draw horizontal lines to represent the Eocene stratum. When these are projected 8 miles to the proposed fault there is a resulting offset of 60 feet. They appear to have correlated their electric logs by presupposing the existence of the Hampton Roads fault.

   It should first be pointed out that electric logs are no more than indirect geophysical methods and must therefore be considered interpretive not primary. In terms of clarity and uniqueness of interpretation, electric logs are no substitute for first-hand sampling of well materials. In this respect, Rogers and Spencer's section based on electric logs is subordinate to the stratigraphic sections by Cederstrom (Reference 24), Brown (Reference 9), and to interpretations of well data in the vicinity of the site as shown on Figure 2A-26. Since these stratigraphic sections show no fault, the electric logs cannot independently support a fault.

The following conclusions can be made from the above discussions:

1. The reversals in dip are not fault controlled.
2. There is no abrupt thickening of the Eocene sediments north of the James River as first proposed in 1945 and later refuted in 1957 by D. J. Cederstrom
3. No different stratigraphic positions in the Eocene north and south of the James River are evident. This was first proposed in 1945 and later refuted in 1957 by D. J. Cederstrom.
4. There is no evidence of different depths to basement north and south of the James River. In fact, recent evidence by Brown (Reference 6) shows that there is not a difference. The

original difference in depth was postulated to describe 1945 stratigraphic and coastal interpretations.

5. The high chloride wedge north of the James River is probably a result of incomplete flushing of sea water which once saturated the sediments. Chloride concentrations are a function of depth, permeability, flow or pumping rates, time and location, and are representative of coastal aquifer conditions.

6. The potentiometric surface is variable but does not indicate fault control. The potentiometric surface is variable depending on pumping rates, local stratigraphic conditions, the aquifer or aquifers from which the wells pump, the proximity to other wells or groups of wells and recharge from the surface to underlying aquifers and aquitards.

7. Electric log interpretation is an indirect method of developing geologic sections. Direct logging of wells does not show a fault. The data and geologic, geotechnical, and geohydrologic interpretations thereof show no evidence of fault control. The data and the anomolies have been reinterpreted and controls other than faulting are evidenced. The Hampton Roads fault therefore does not exist.

2A.3 References[1]

1. Cederstrom, D. J., *Structural Geology of Southeastern Virginia,* American Association of Petroleum Geologists, Bulletin, Vol. 29, No.1, 1945.(†)

2. Cederstrom, D. J., *Chloride in Ground Water in the Coastal Plain of Virginia,* Bulletin 58, Virginia Geological Survey, 1943.(†)

4. Cederstrom, D. J., *Geology and Ground-Water Resources of the York-James Peninsula, Virginia,* USGS Water Supply Paper 1361, 1957, pp. 237, 1957.(†)

5. Brown, P., Miller, and Swami, *Structural and Stratigraphic Framework, and Spatial Distribution of Permeability of the Atlantic Coastal Plain North Carolina to New York,* United States Geol. Survey Prof. Paper 796, 1972.(†)

6. Rogers, W. S., and Spencer, R. S., *Groundwater Quality and Structural Control in Southeastern Virginia,* GSA Bulletin 82, pp 2313-2318, 1971.(†)

7. Cooper, H. H., Jr., *A Hypothesis Concerning the Dynamic Balance of Fresh Water and Salt Water in a Coastal Aquifer,* Journal of Geophysical Research, Vol. 64, No. 4, April 1959.(†)

8. Virginia Division of Water Resources, *Ground Water of Southeastern Virginia,* Planning Bulletin 261, 1970.(†)

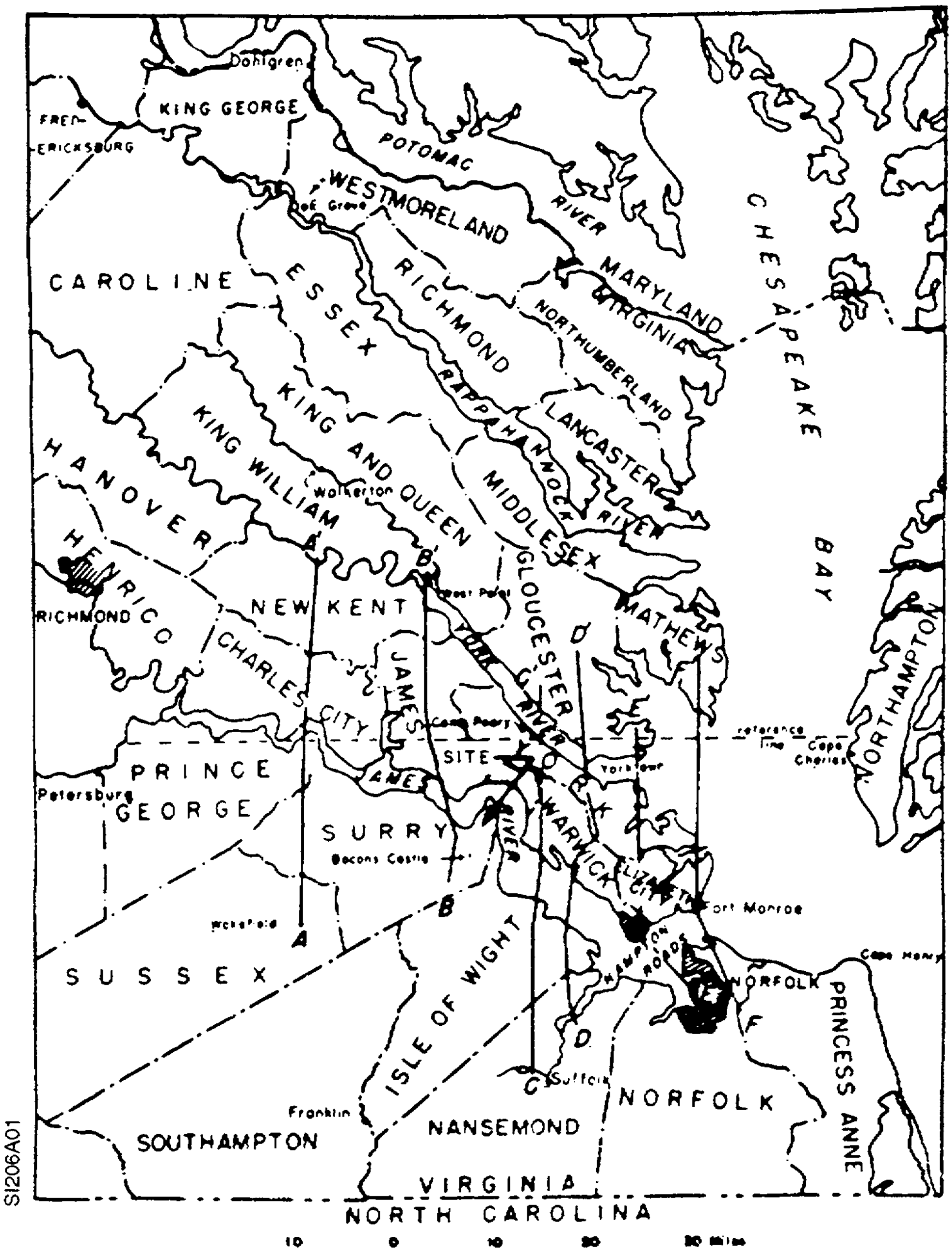
9. Virginia Bureau of Water Control Management, *Ground Water of the York-James Peninsula, Virginia,* Basic Data Bulletin 39, June 1973.(†)

1. References noted with an (†) indicate an original reference from Surry Power Station Units 3 and 4 PSAR.

10. Virginia Bureau of Water Control Management, *Ground Water in Virginia: Quality and Withdrawals,* Basic Data Bulletin 38, June 1973.(†)

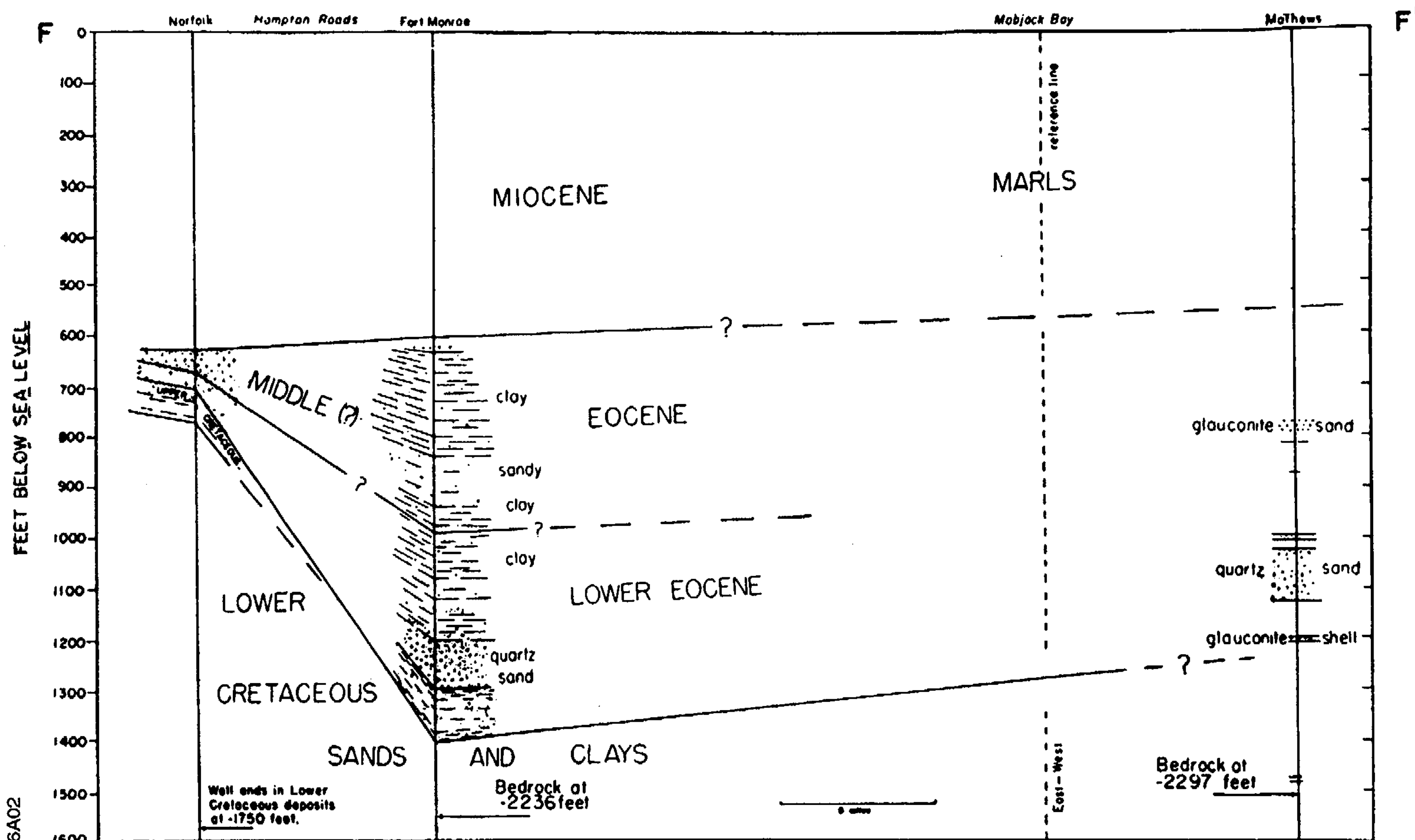
11. Brown, G. Allan, and Cosner, O. J., *Ground Water Conditions in the Franklin Area, Southeastern, Virginia,* Open File Report, USGS, Richmond Virginia, 1973.(†)

Figure 2A-1
MAP OF COASTAL PLAIN AREA IN VIRGINIA SOUTH OF POTOMAC RIVER
SHOWING LOCATIONS OF CROSS SECTIONS



SI206A01

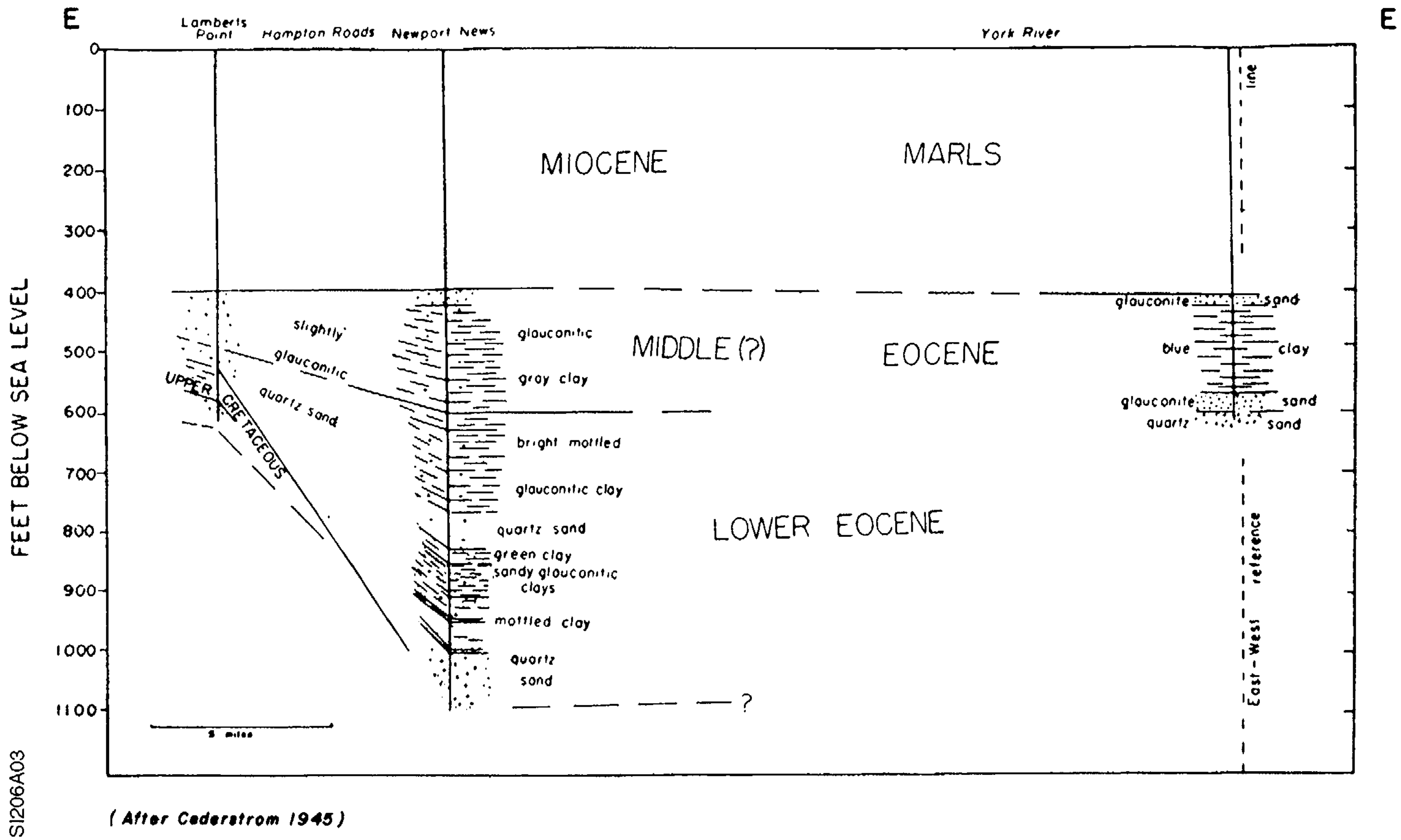
Figure 2A-2
GEOLOGICAL CROSS SECTION FF



SI206A02

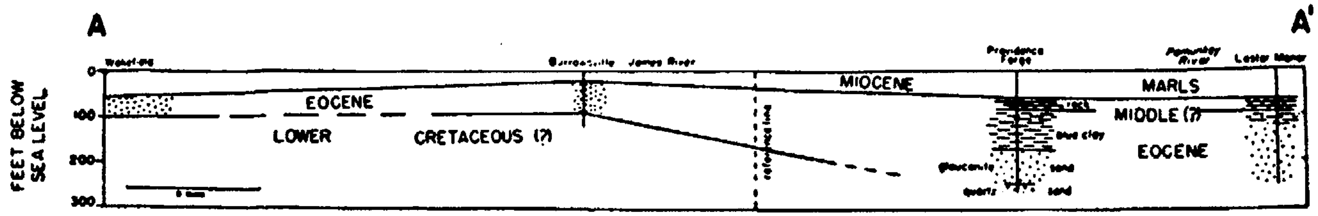
(After Cederstrom 1945)

Figure 2A-3
GEOLOGICAL CROSS SECTION EE



(After Cederstrom 1945)

Figure 2A-4
GEOLOGICAL CROSS SECTION BB



GEOLOGICAL CROSS SECTION AA

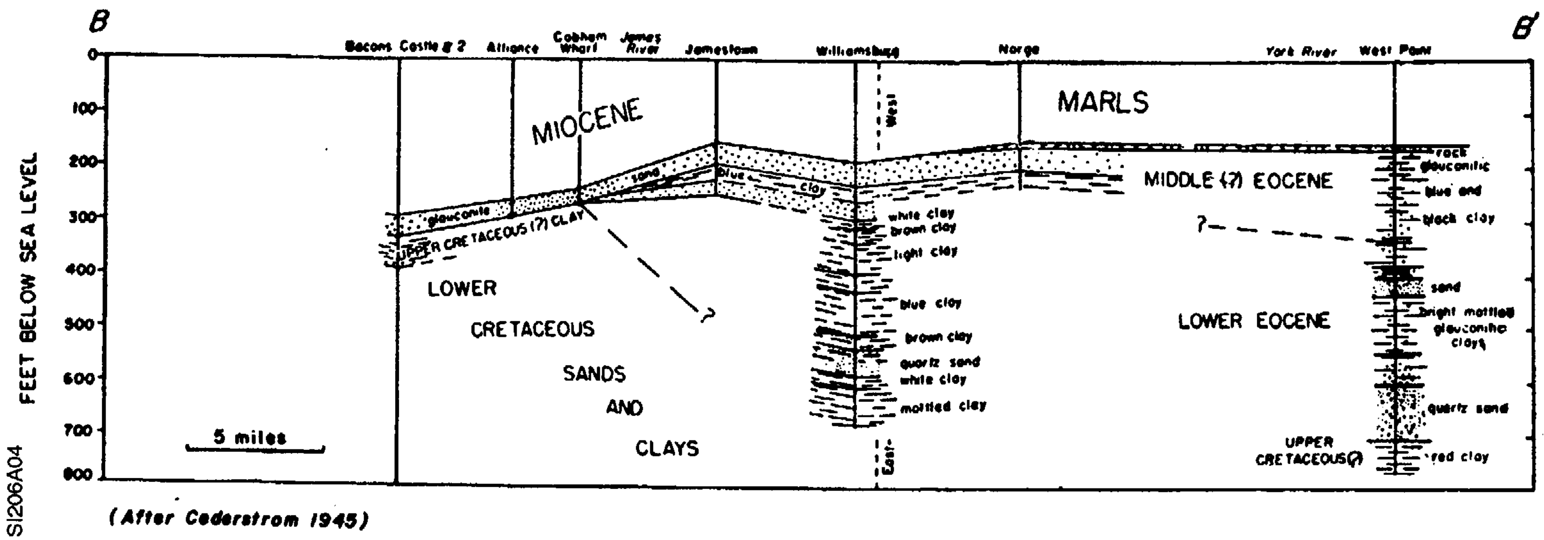
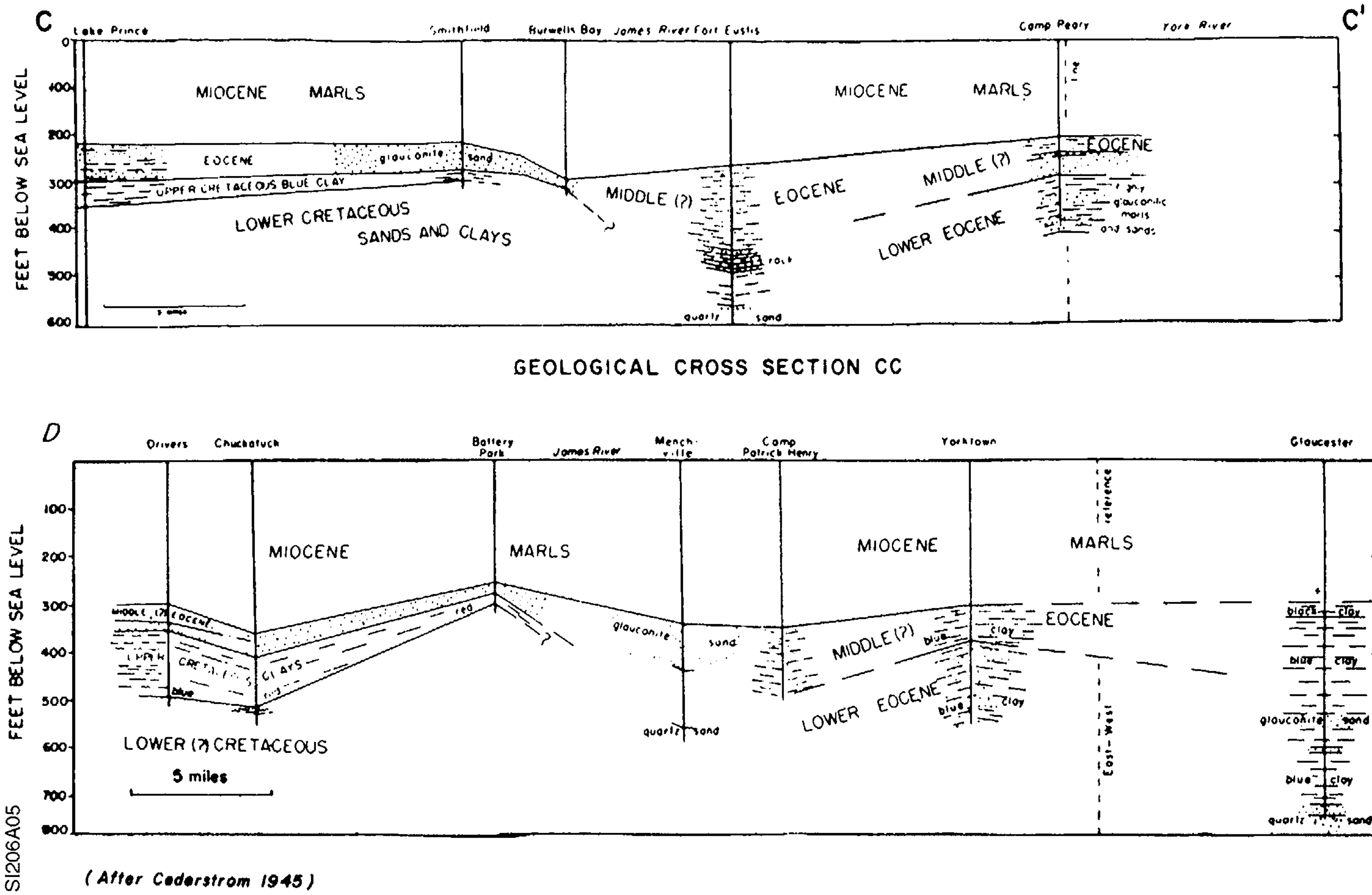


Figure 2A-5
GEOLOGICAL CROSS SECTION DD



(After Cederstrom 1945)

Figure 2A-6
INDEX SHOWING LOCATION
OF AREA AND OF CROSS SECTIONS

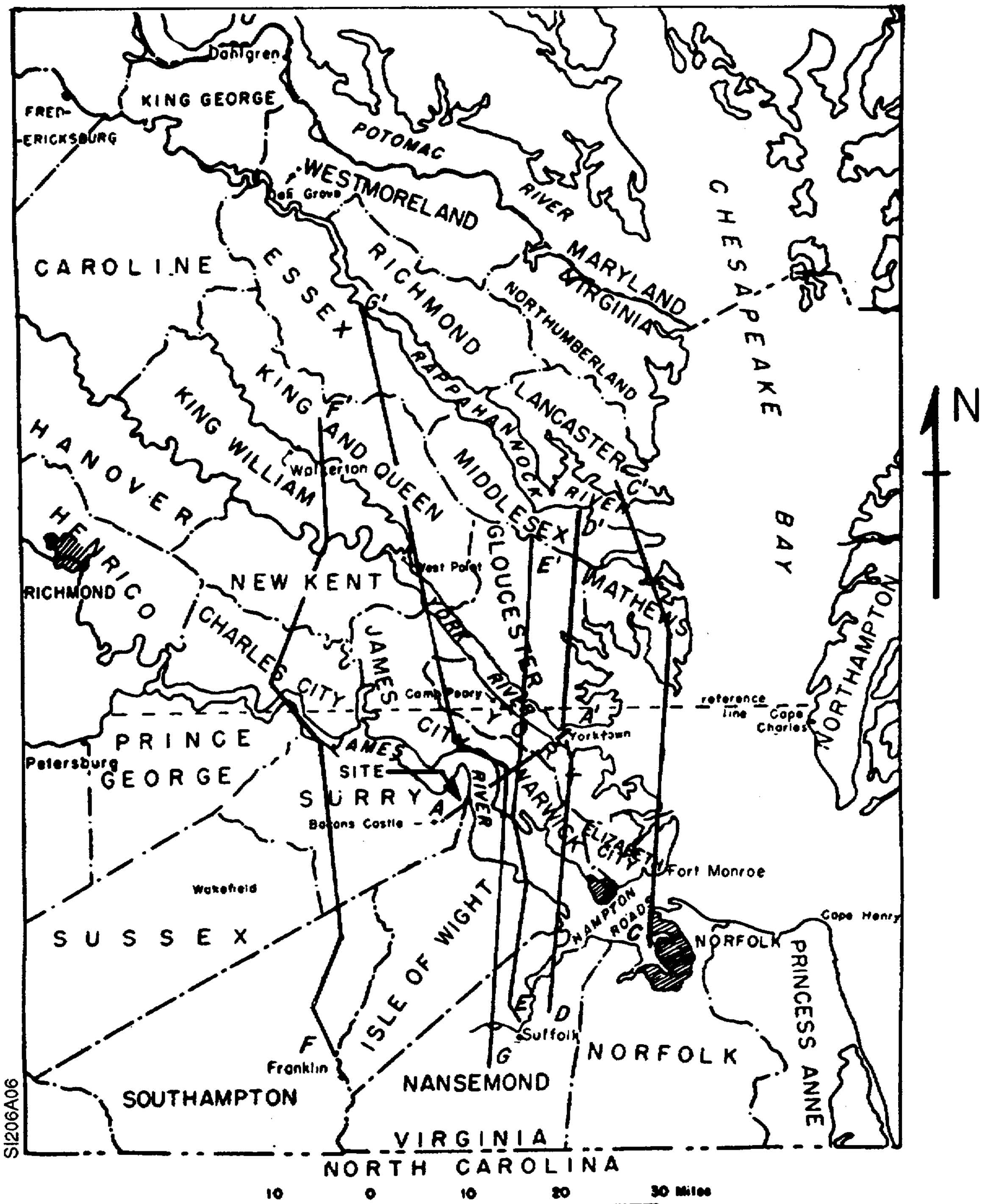
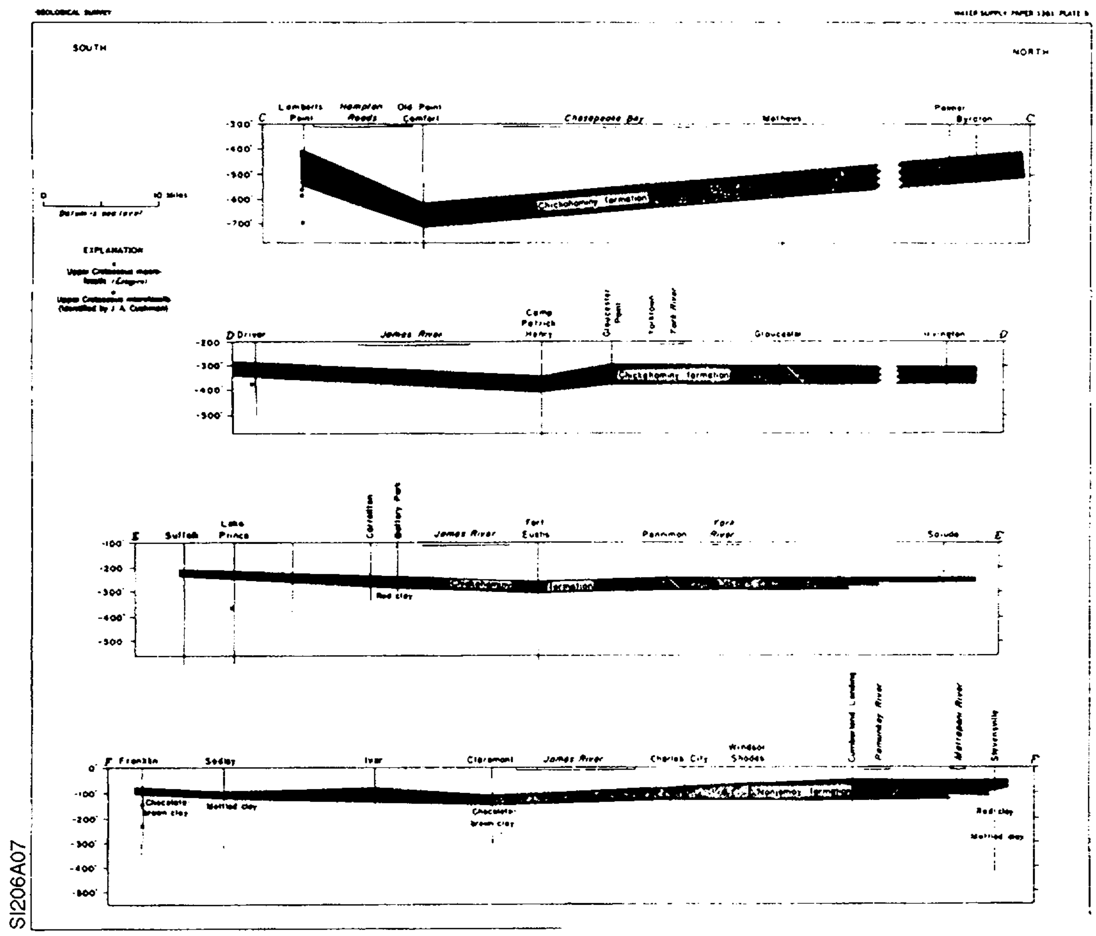
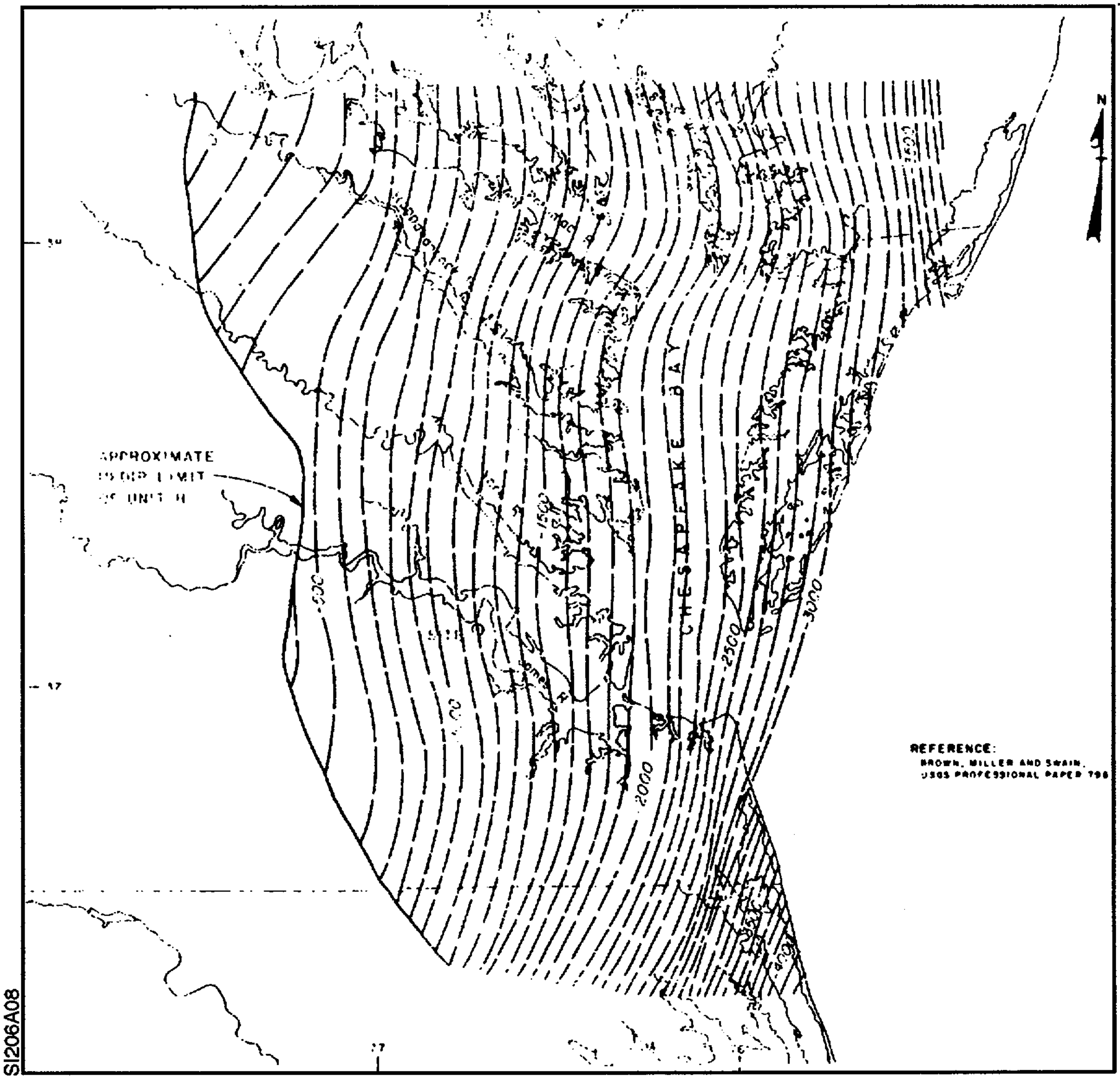


Figure 2A-7
CROSS SECTIONS SHOWING POSITION OF FORMATION IN THE YORK - JAMES PENINSULA, VIRGINIA RELATIVE TO AREAS NORTH AND SOUTH



(Cederstrom 1957)

Figure 2A-8
STRUCTURAL CONTOURS; CRETACEOUS AND LATE JURASSIC (UNIT H)



SI206A08

Figure 2A-9
STRUCTURAL CONTOURS; CRETACEOUS (UNIT G)

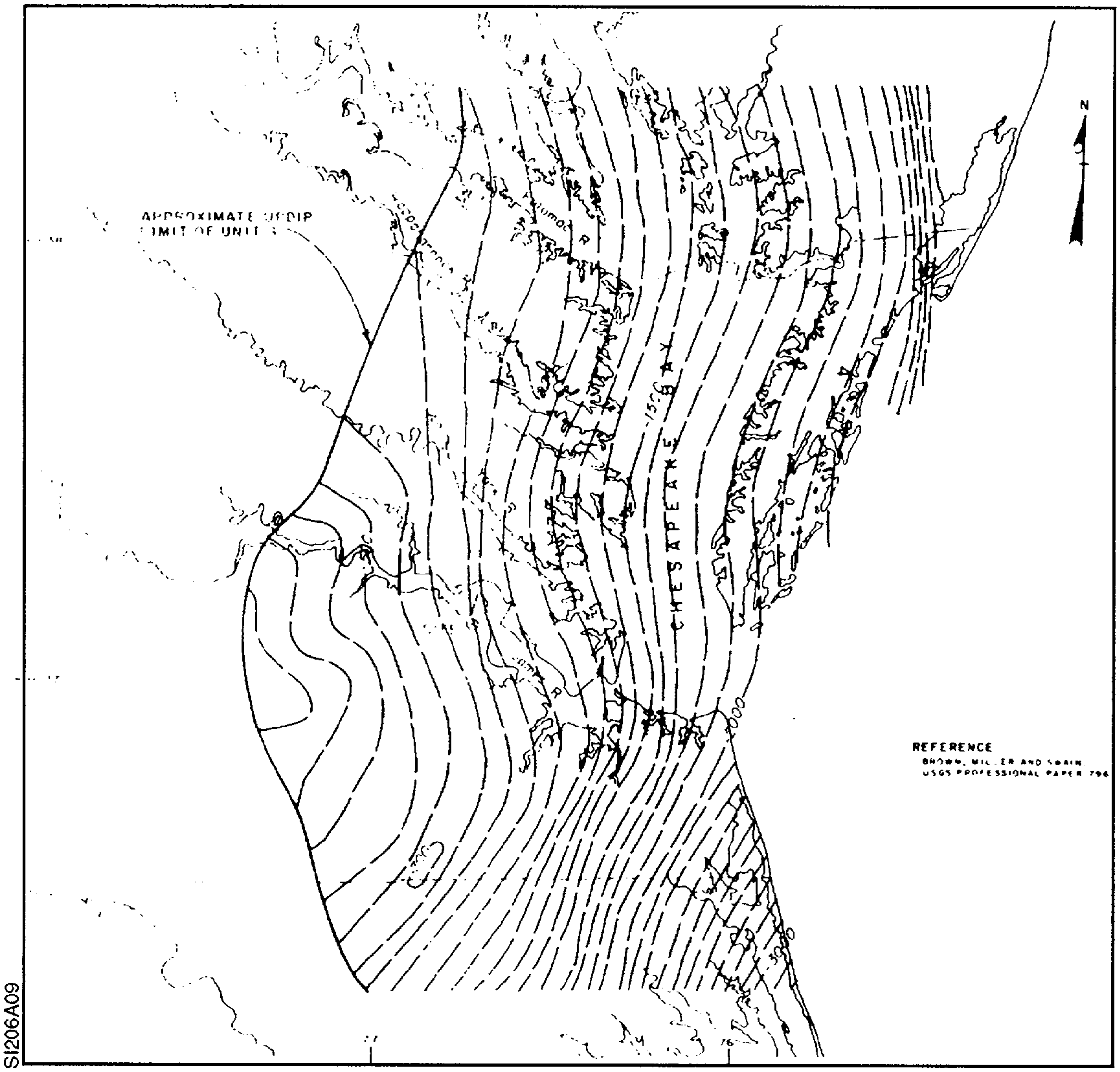


Figure 2A-10
STRUCTURAL CONTOURS; CRETACEOUS (UNIT F)

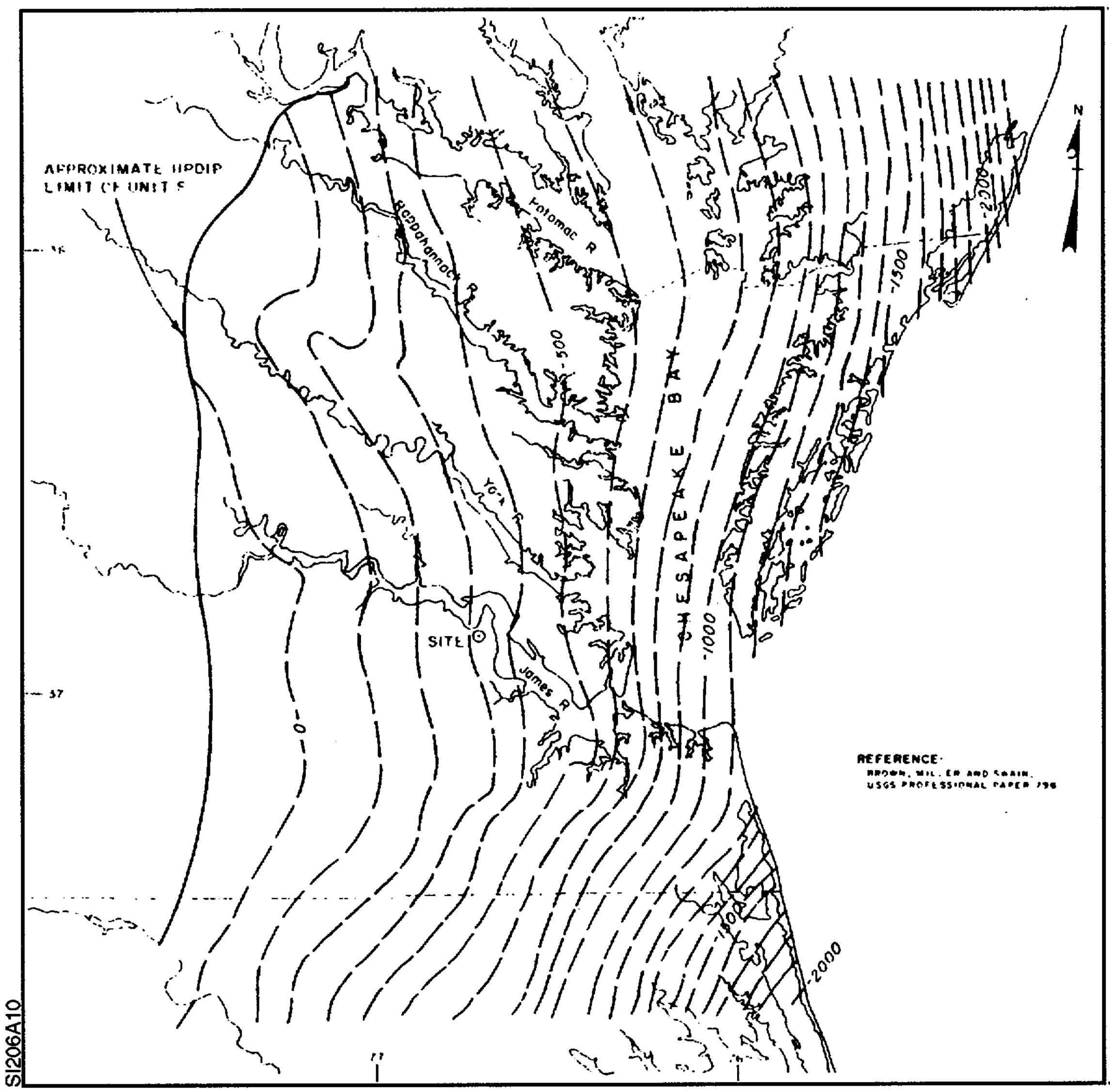
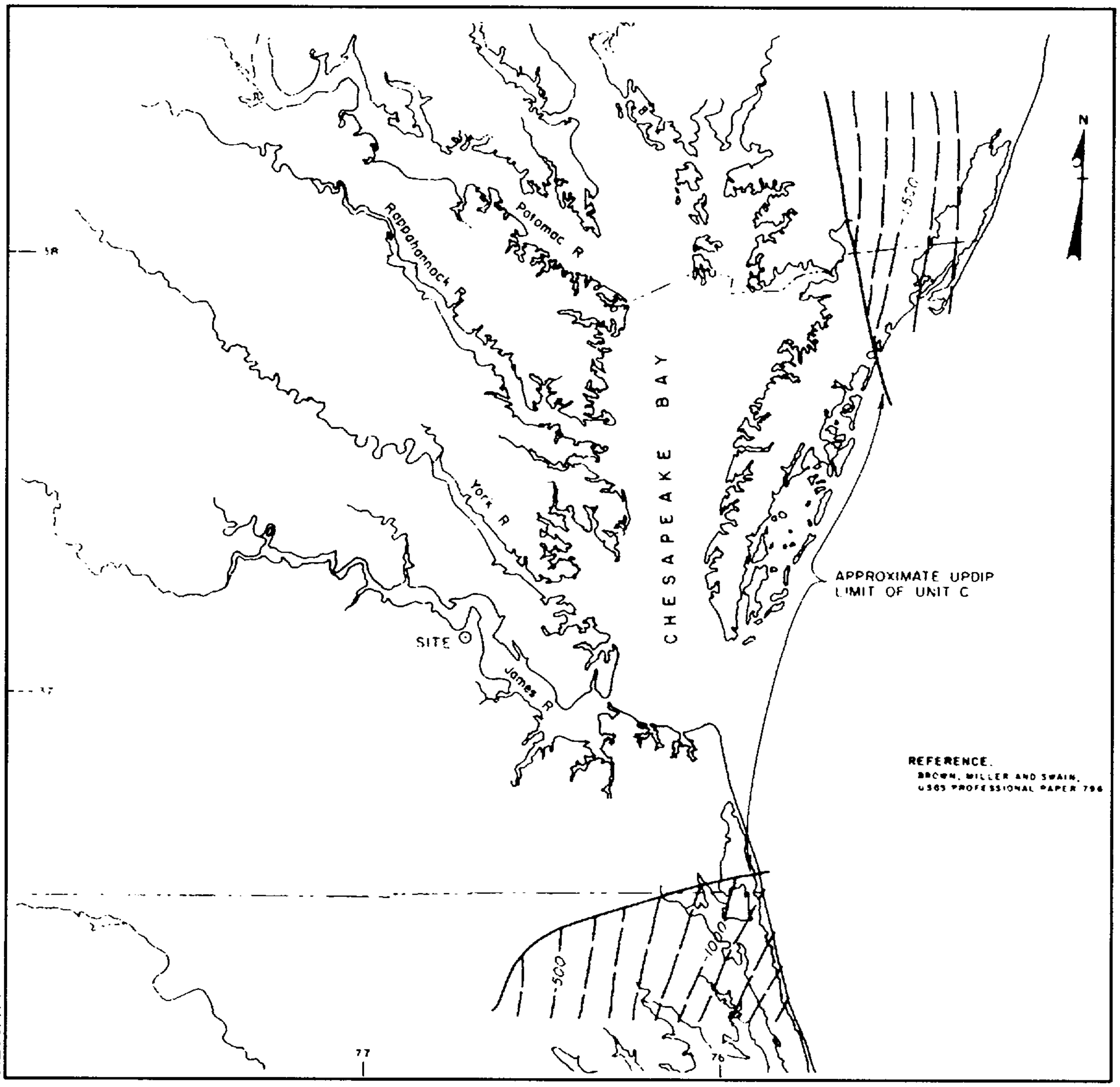
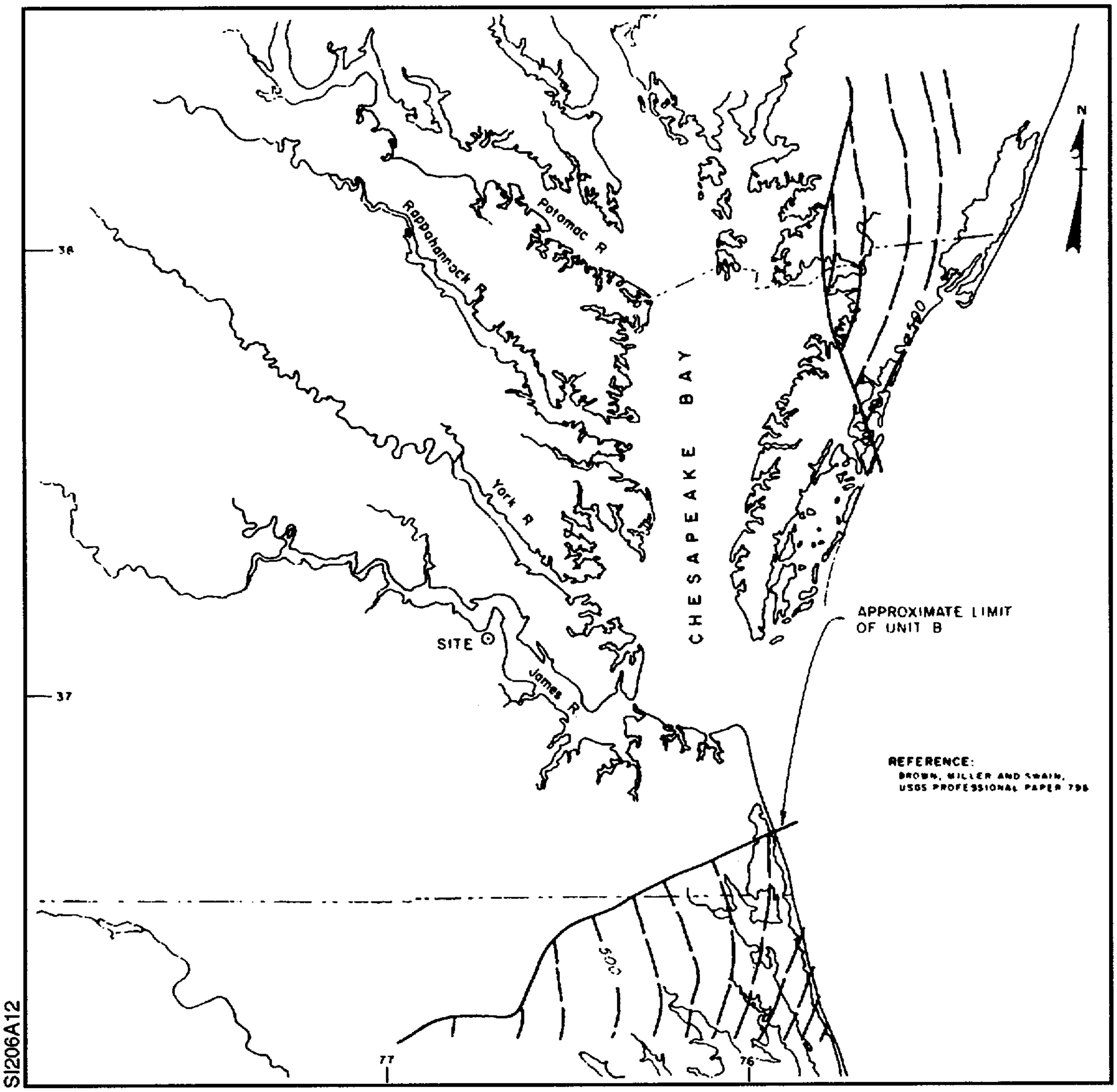


Figure 2A-11
STRUCTURAL CONTOURS; CRETACEOUS (UNIT C)



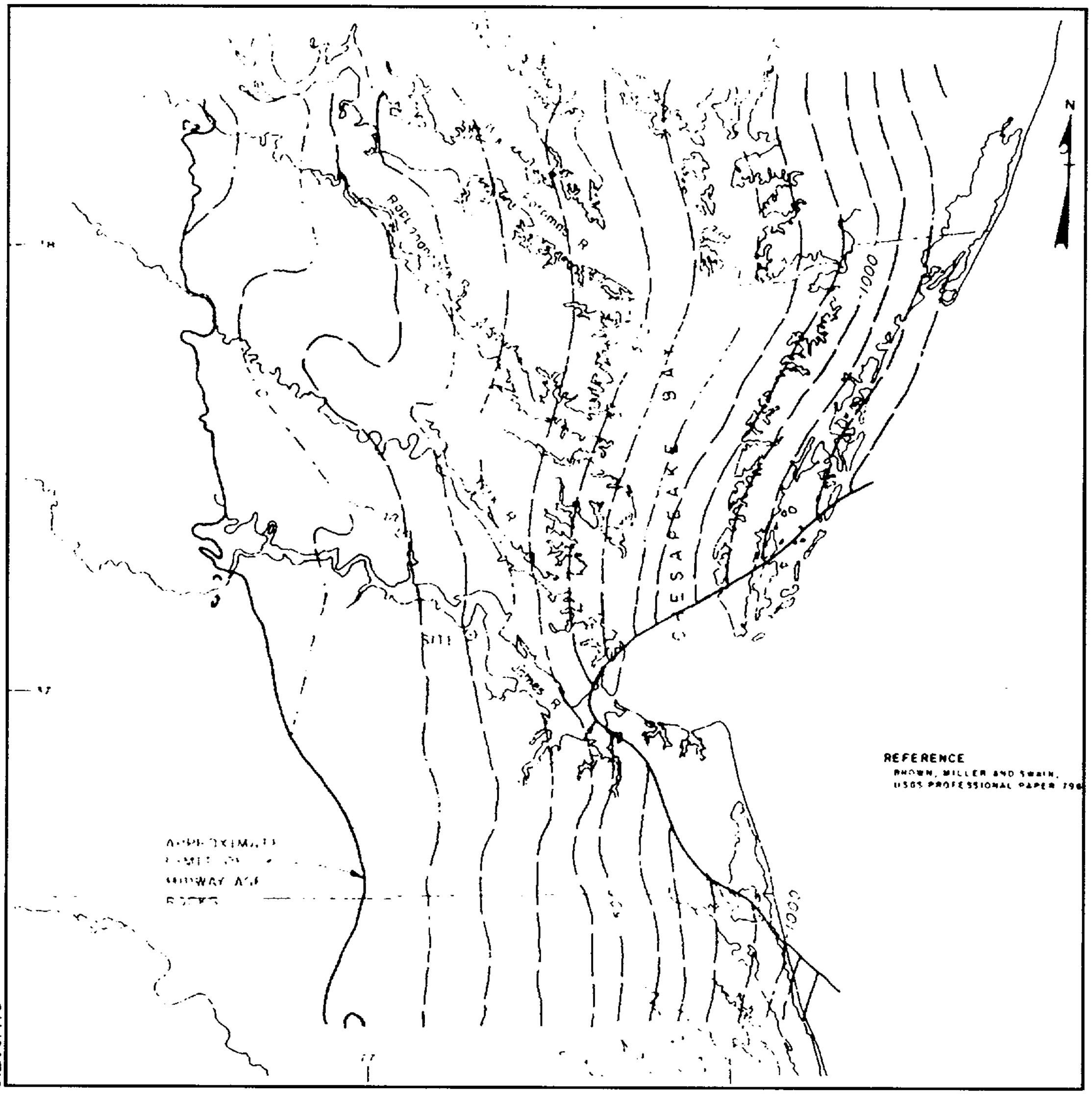
SI206A11

Figure 2A-12
STRUCTURAL CONTOURS; CRETACEOUS (UNIT B)



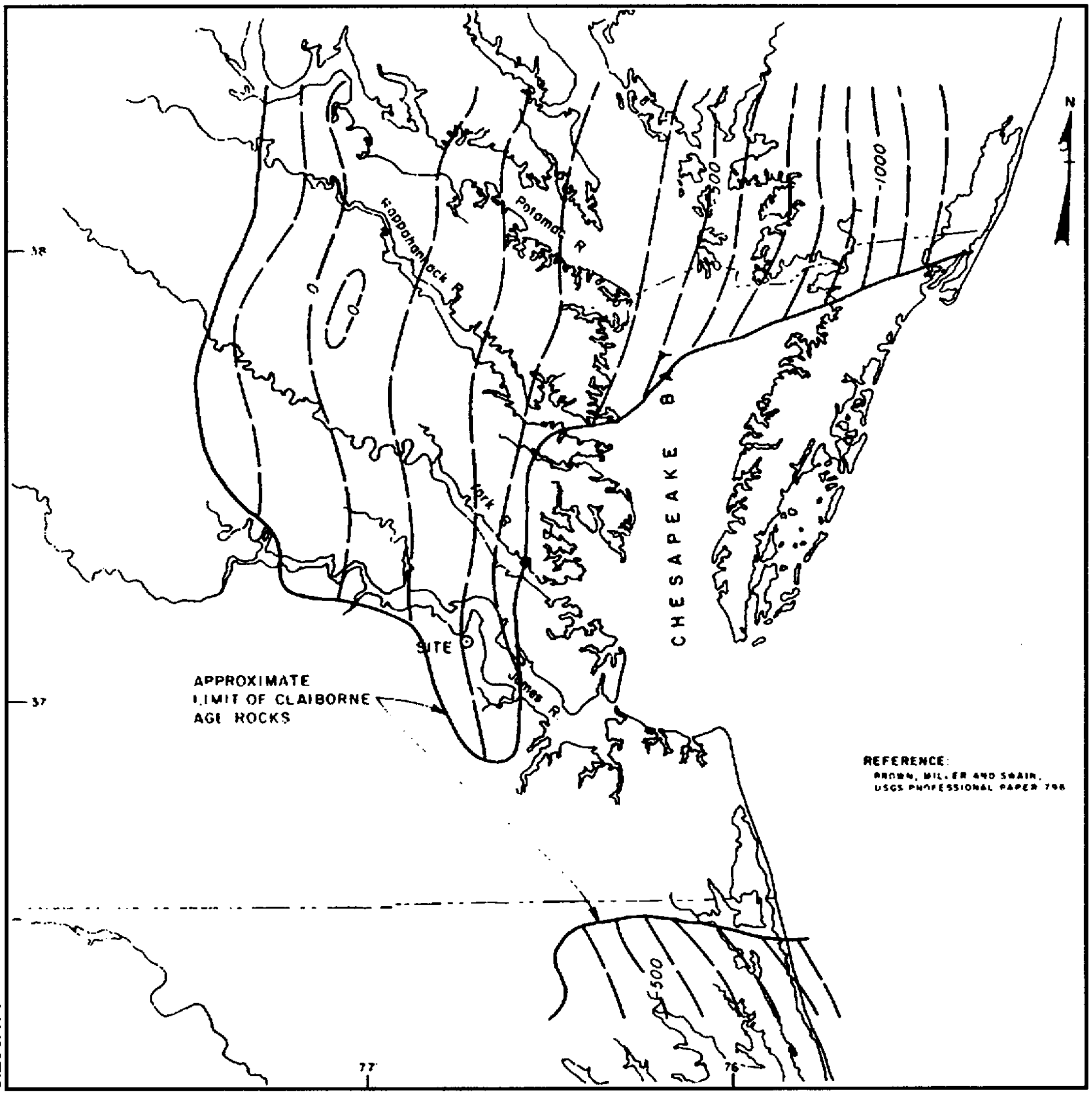
SI206A12

Figure 2A-13
STRUCTURAL CONTOURS; MIDWAY AGE ROCKS



SI206A13

Figure 2A-14
STRUCTURAL CONTOURS; CLAIBORNE AGE ROCKS



SI206A14

Figure 2A-15
STRUCTURAL CONTOURS; JACKSON AGE ROCKS

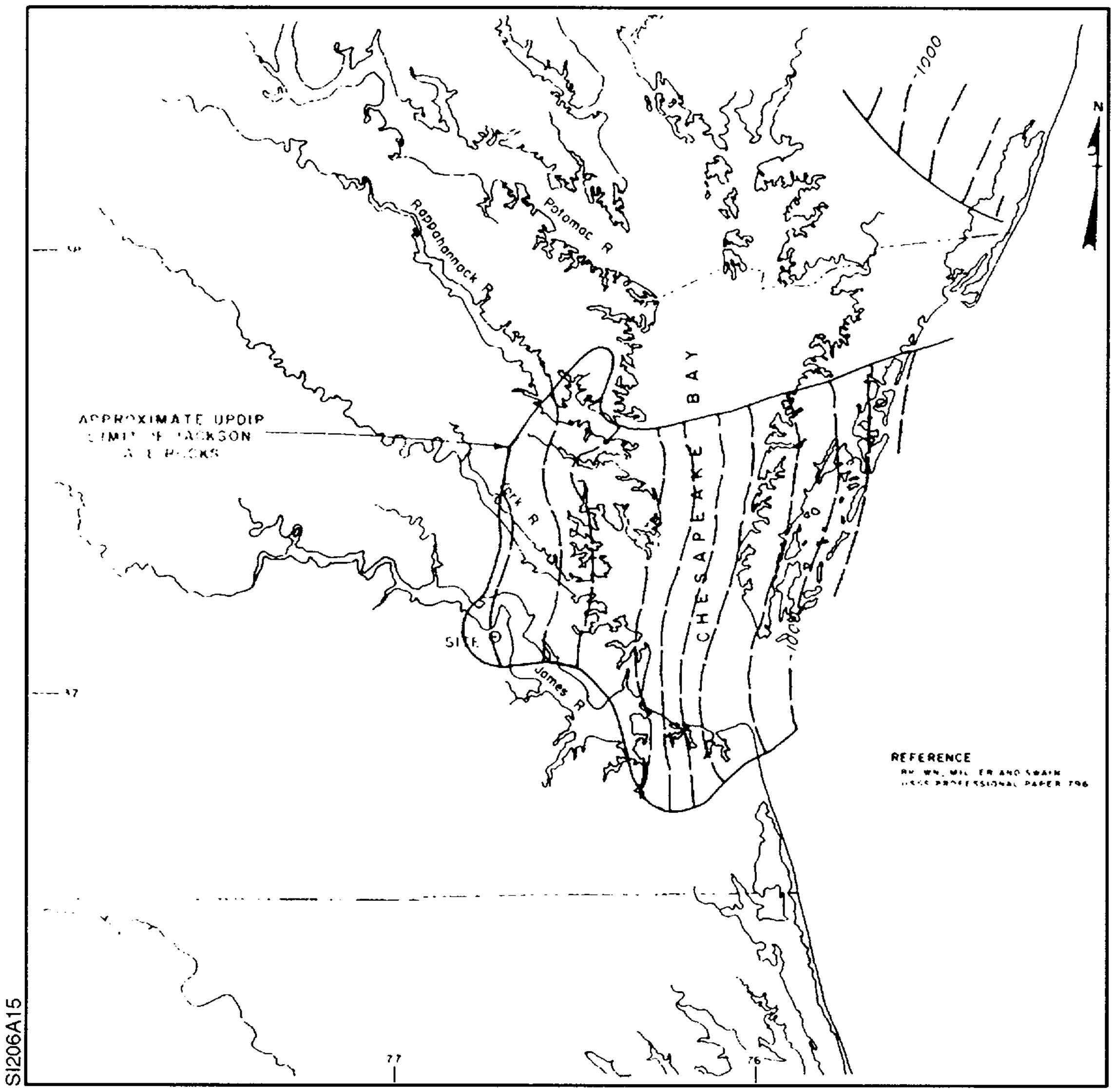


Figure 2A-16
STRUCTURAL CONTOURS; MIDDLE MIOCENE

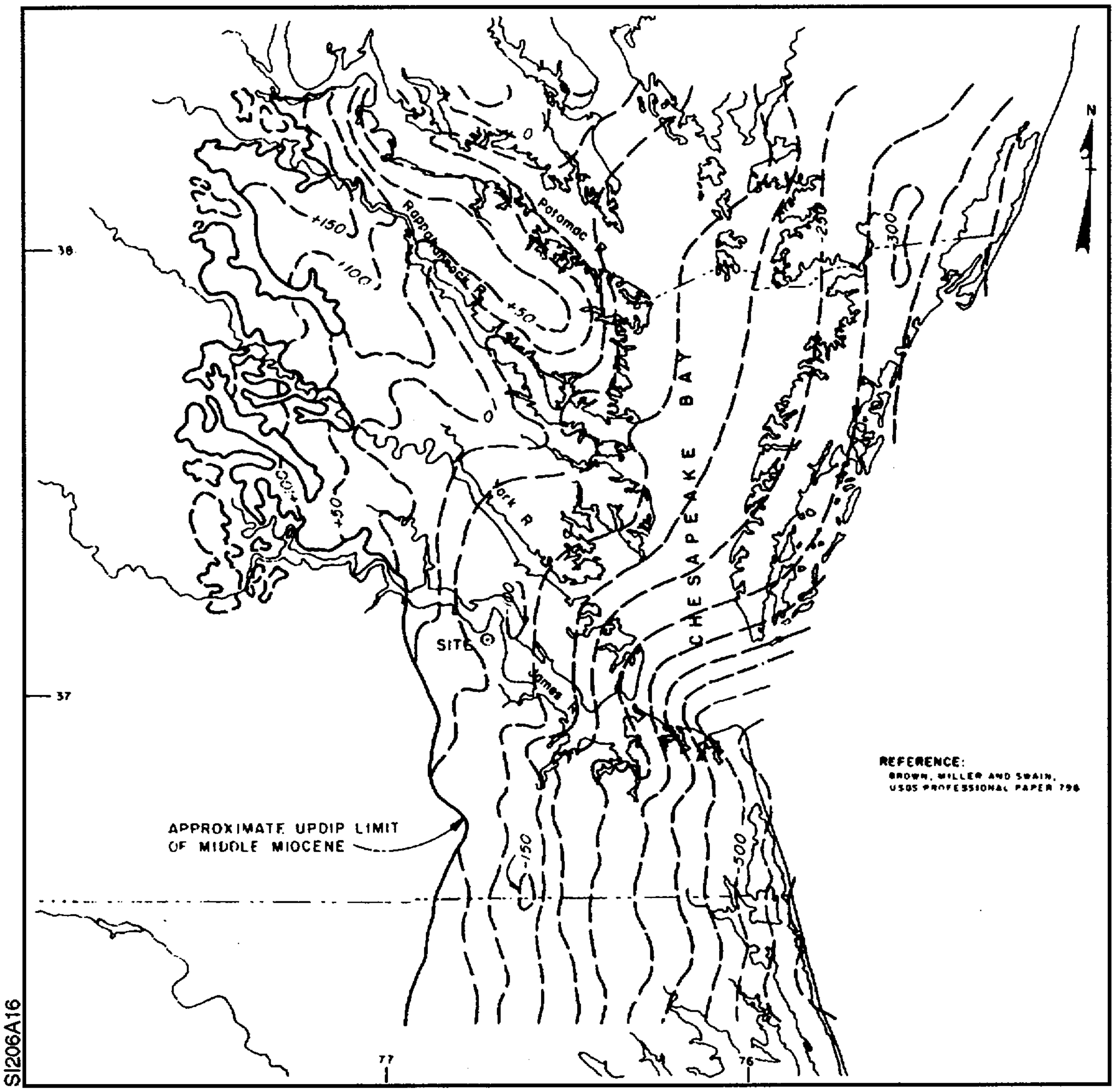
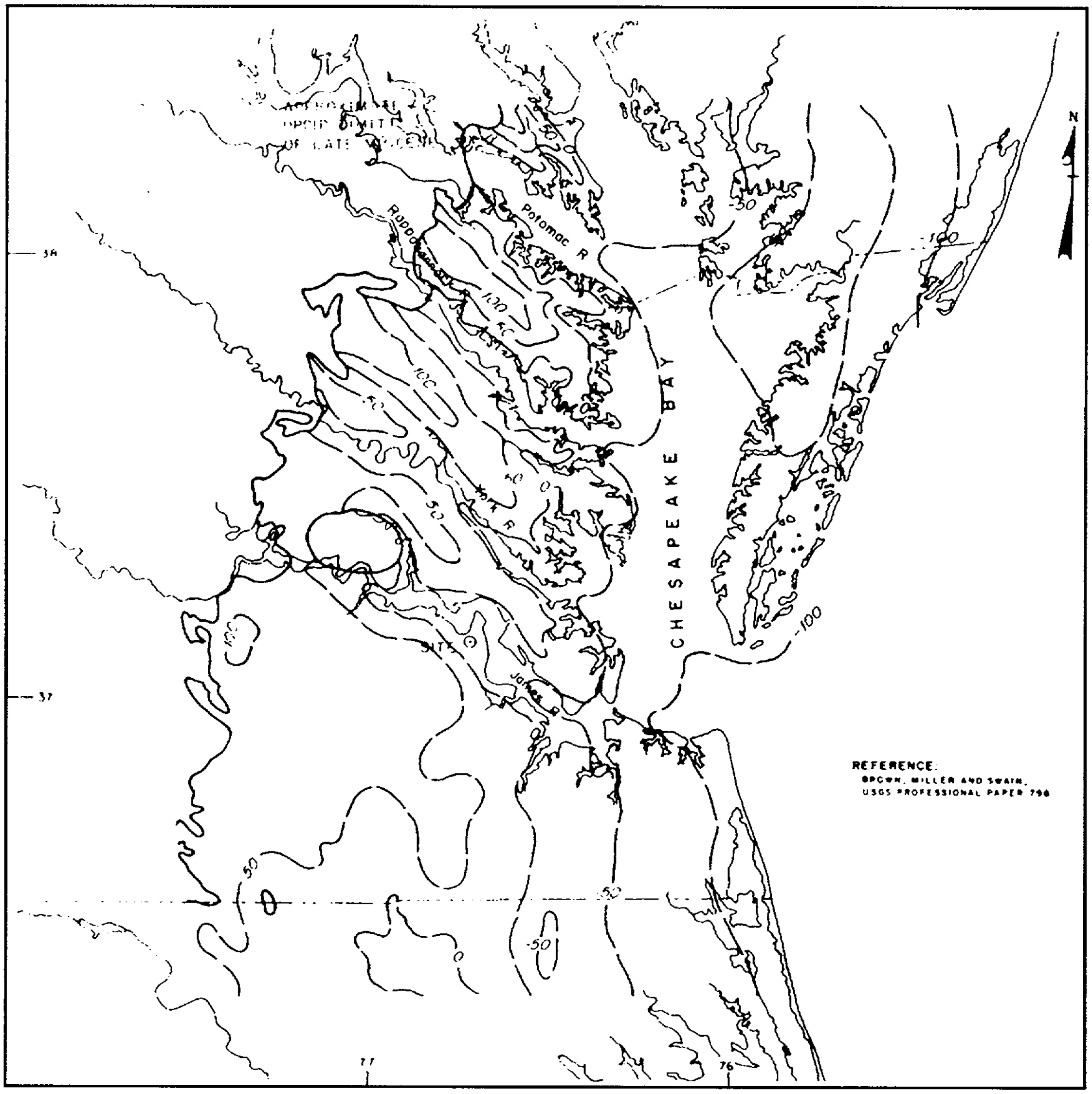


Figure 2A-17
STRUCTURAL CONTOURS; LATE MIOCENE



SI206A17

Figure 2A-18
STRUCTURAL CONTOURS; POST MIOCENE

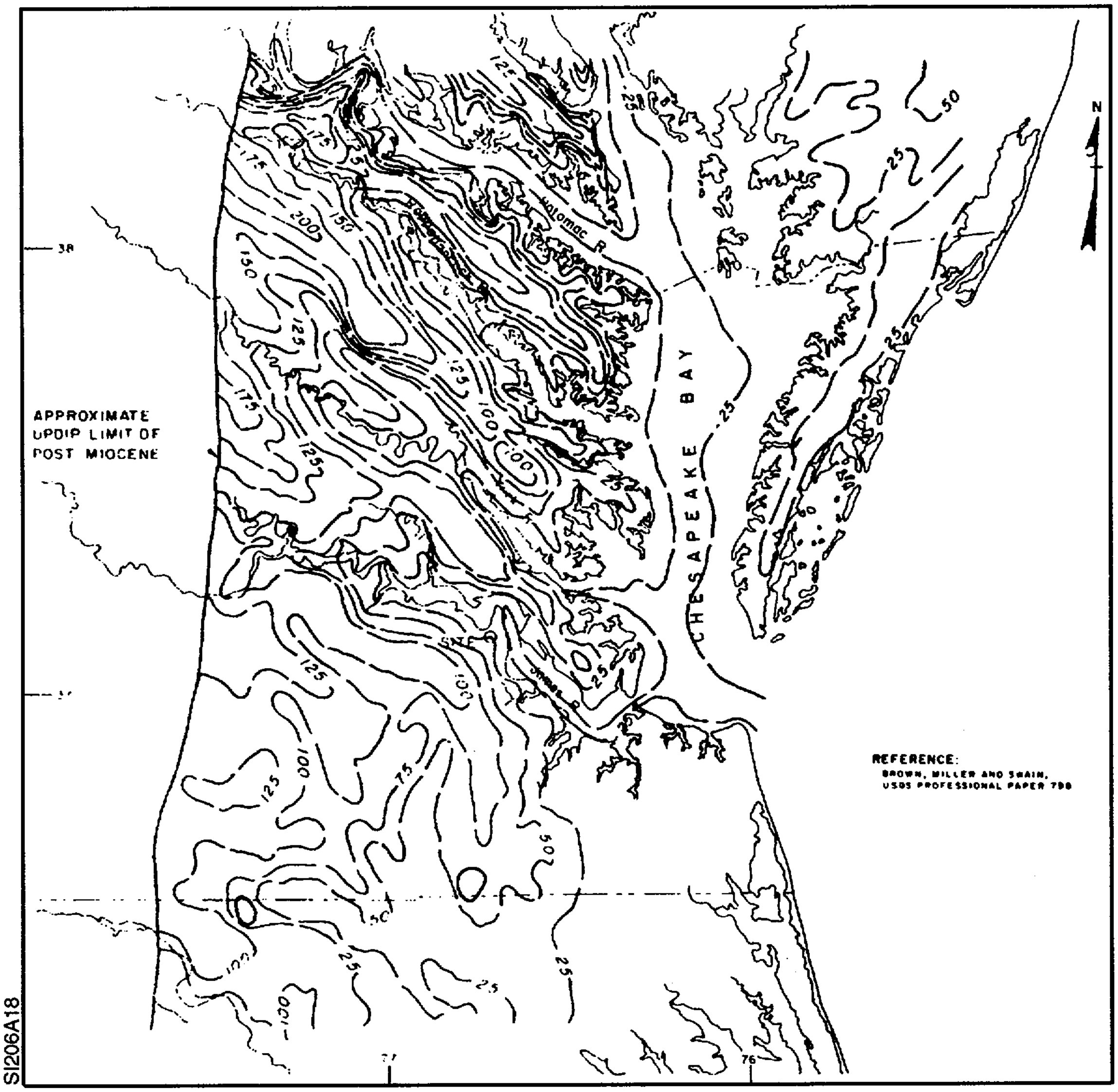
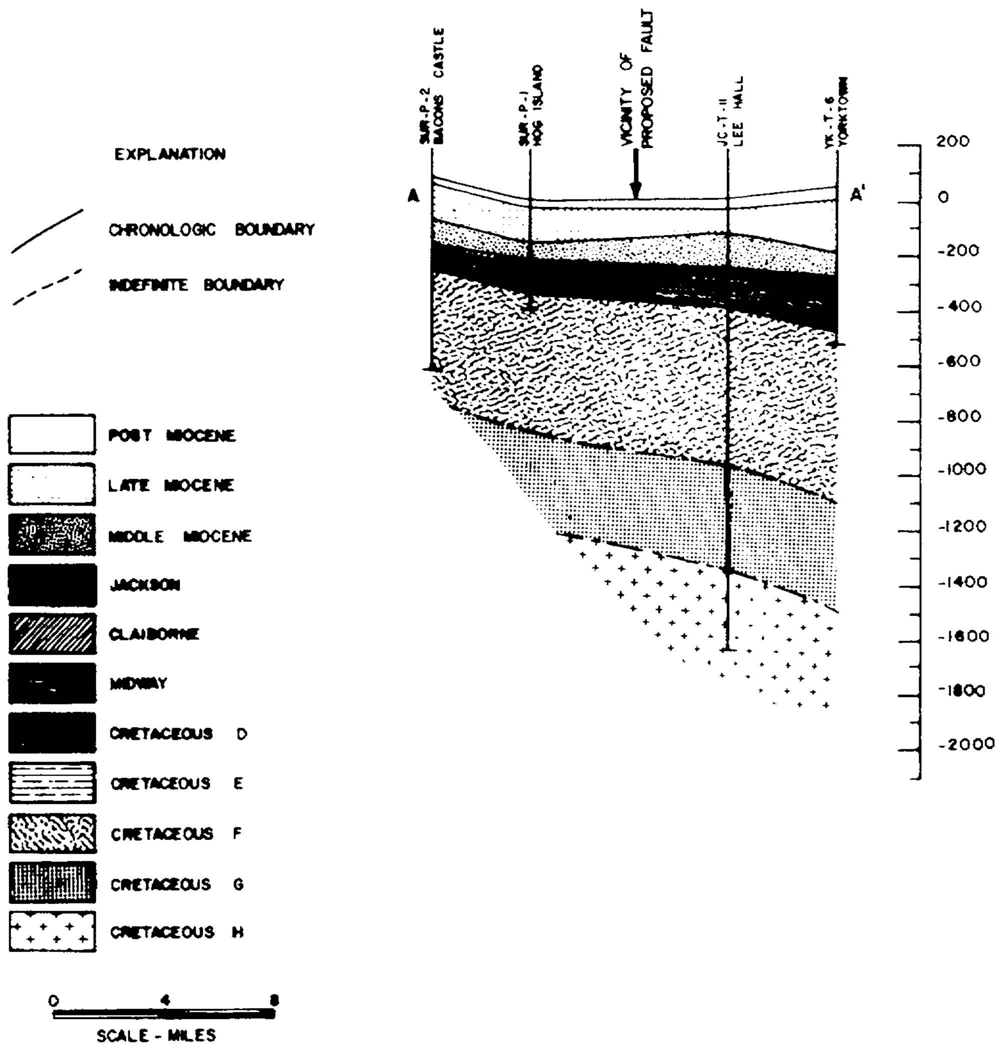


Figure 2A-19
GEOGRAPHICAL CROSS SECTION A-A' BACONS CASTLE TO YORKTOWN



SI206A19

Figure 2A-20
MAP SHOWING OCCURRENCE OF CHLORIDE IN ARTESIAN WATER
IN THE VIRGINIA COASTAL PLAIN SOUTH OF POTOMAC RIVER

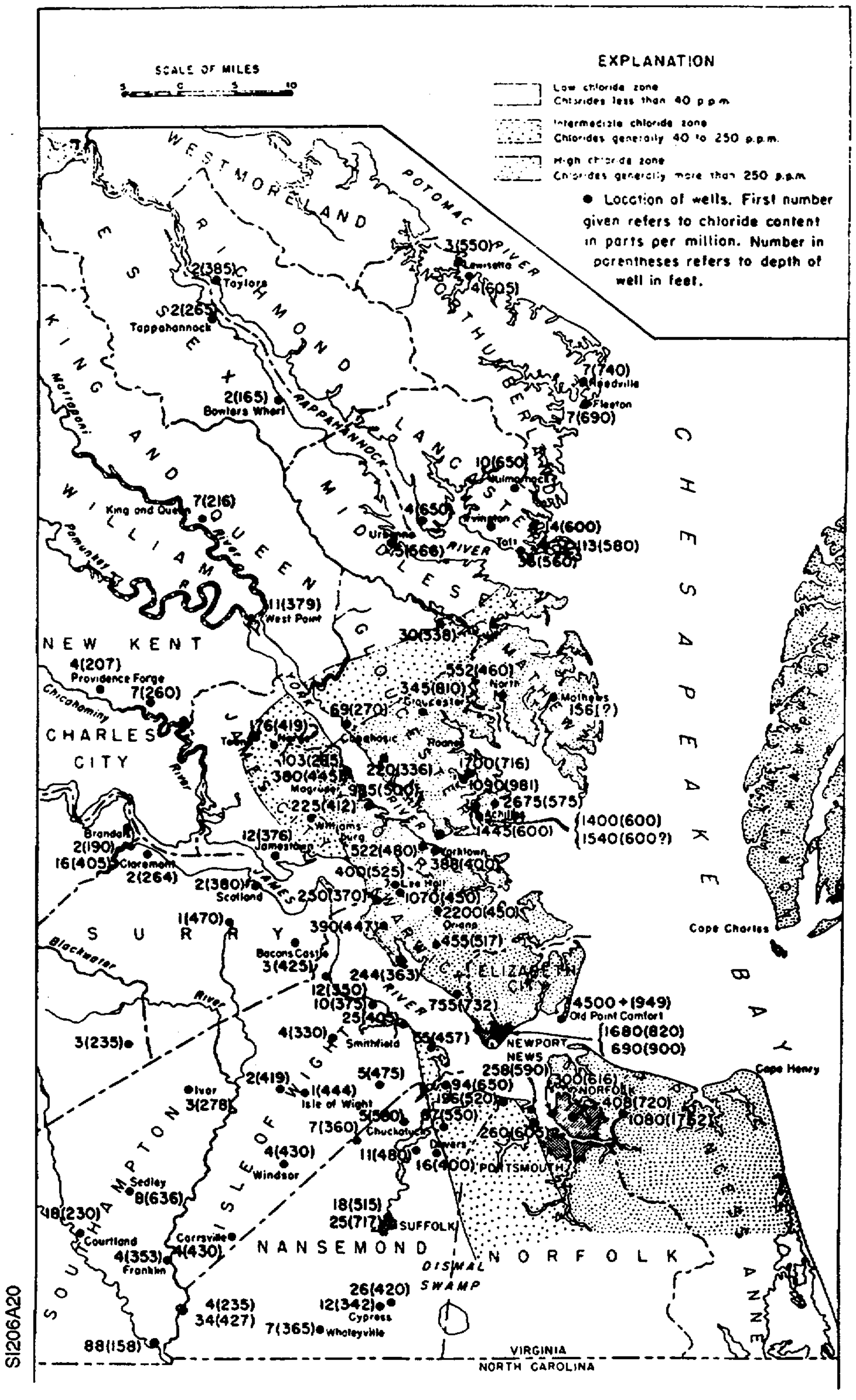
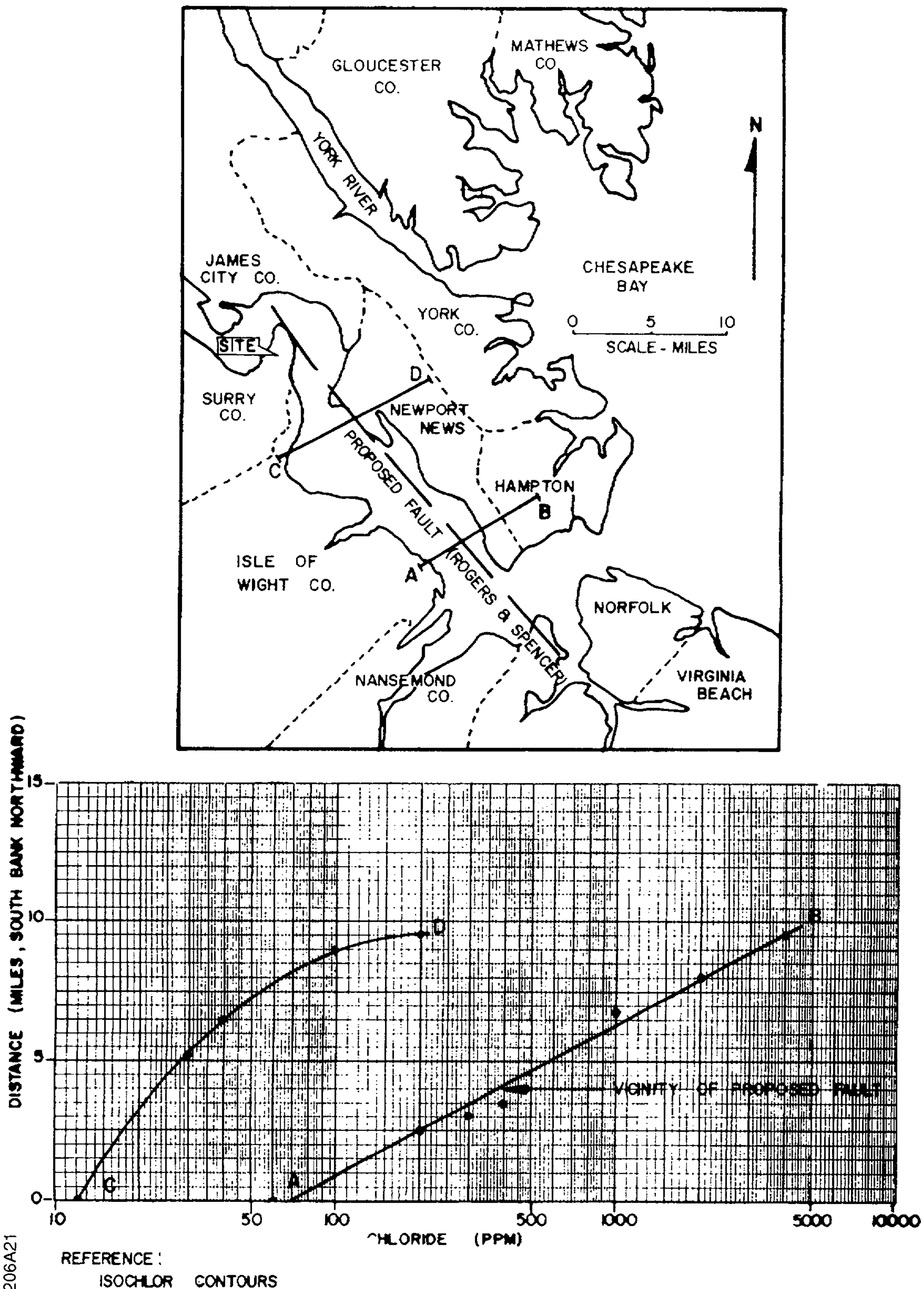


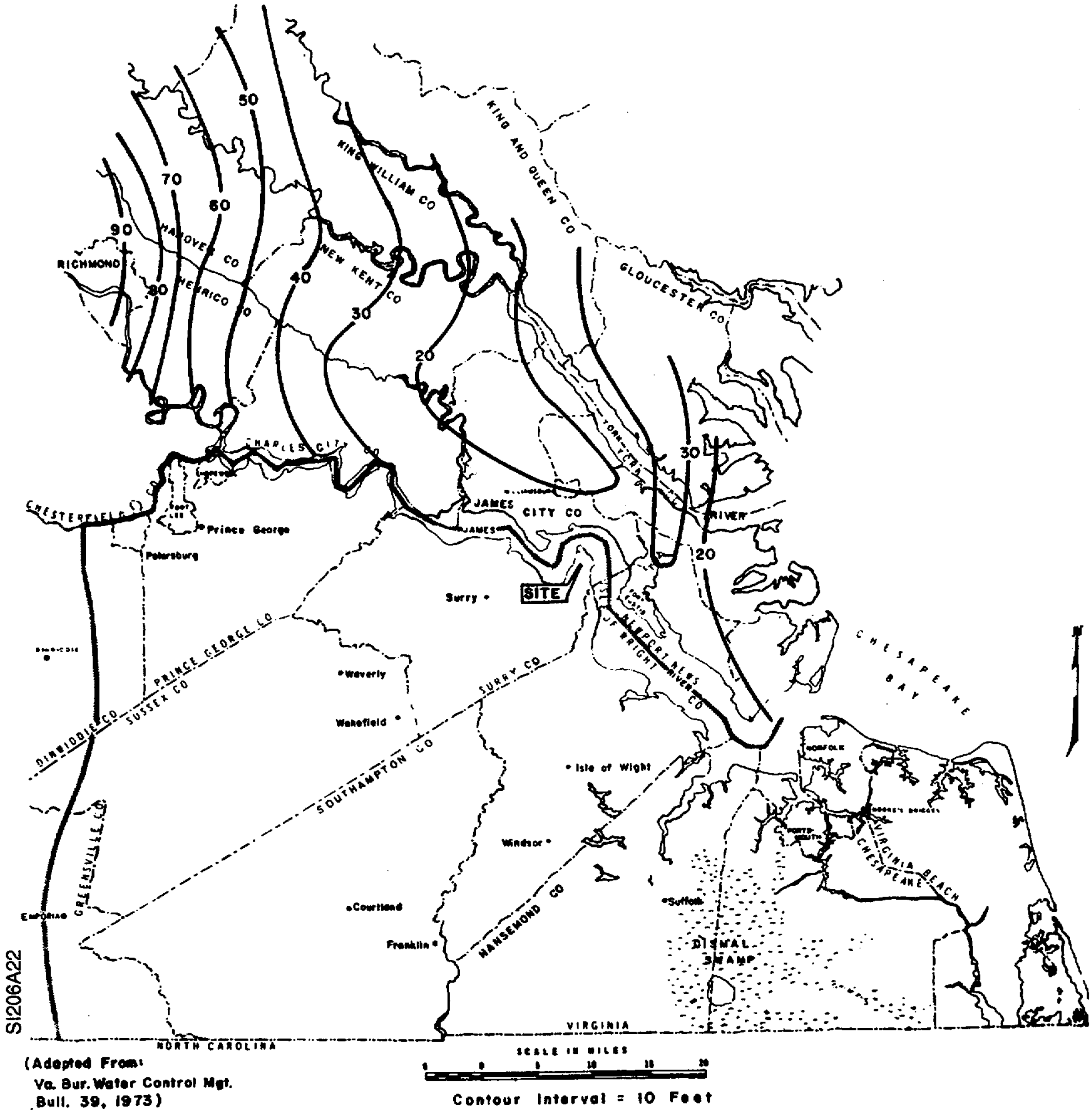
Figure 2A-21
CHLORIDE CONCENTRATION VS. DISTANCE SEMI-LOGARITHMIC PLOT



REFERENCE:
ISOCHLOR CONTOURS
(ROGERS AND SPENCER, 1971)

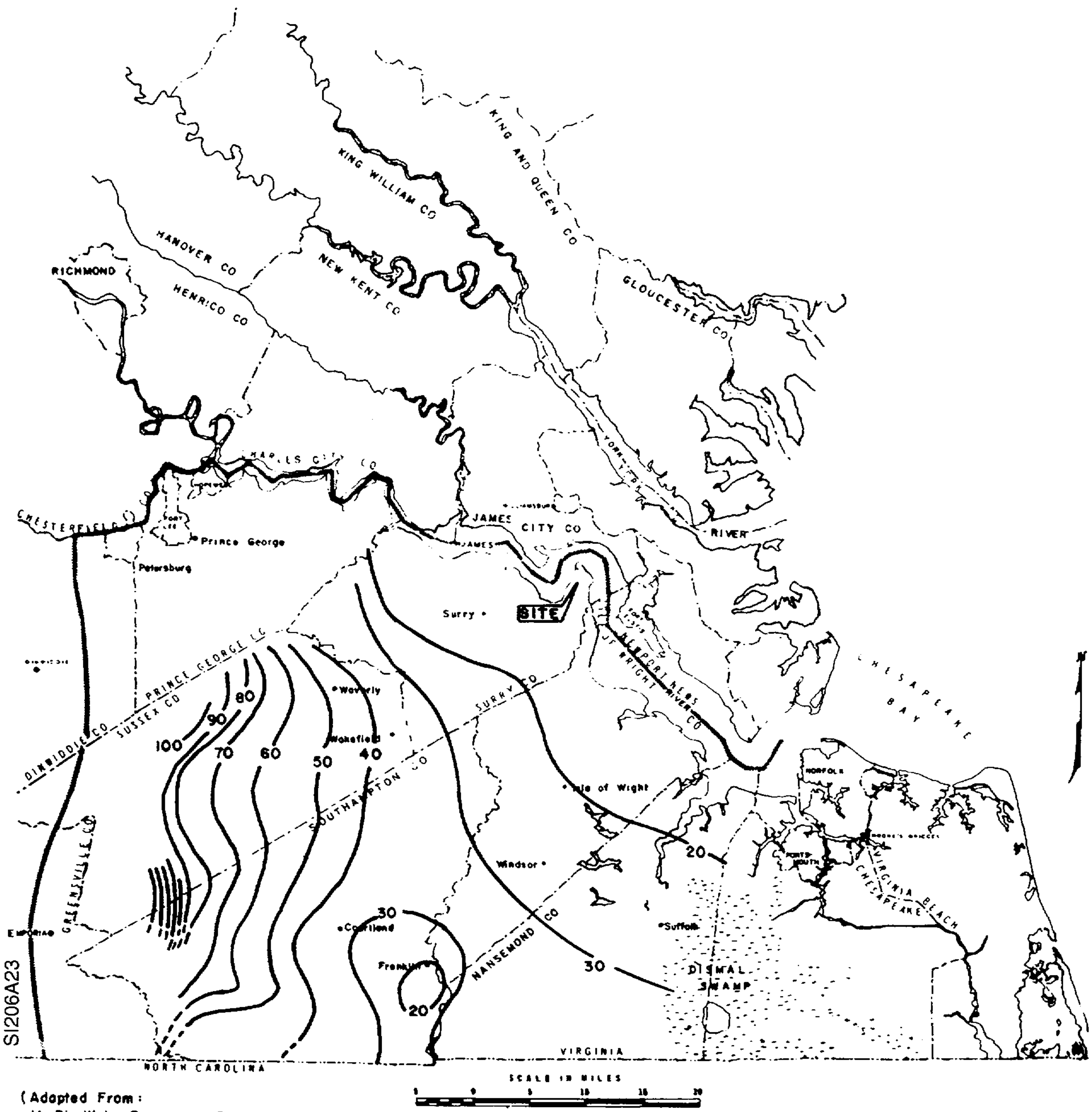
SI206A21

Figure 2A-22
POTENTIOMETRIC SURFACE PRINCIPAL AQUIFER SYSTEM CIRCA 1900



(Adapted From:
Va. Bur. Water Control Mgt.
Bull. 39, 1973)

Figure 2A-23
POTENTIOMETRIC SURFACE IN PRINCIPAL AQUIFER, 1937-1939



(Adapted From:
Va. Div. Water Resources, Plan Bull. 261, 1970)

Figure 2A-24
POTENTIOMETRIC SURFACE PRINCIPAL AQUIFER SYSTEM 1945 TO 1949

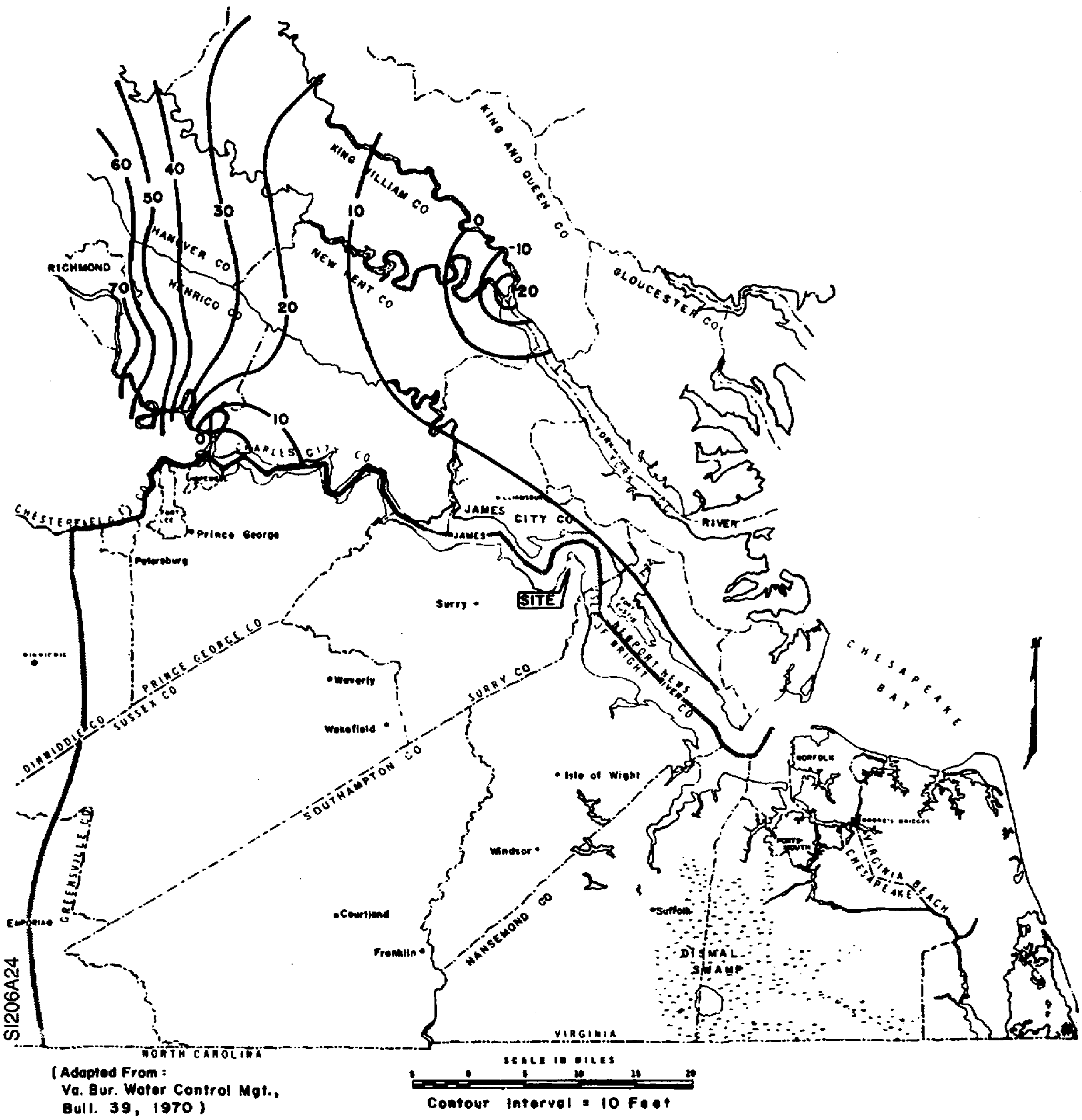


Figure 2A-25
POTENTIOMETRIC SURFACE IN PRINCIPAL AQUIFERS, 1966-1969

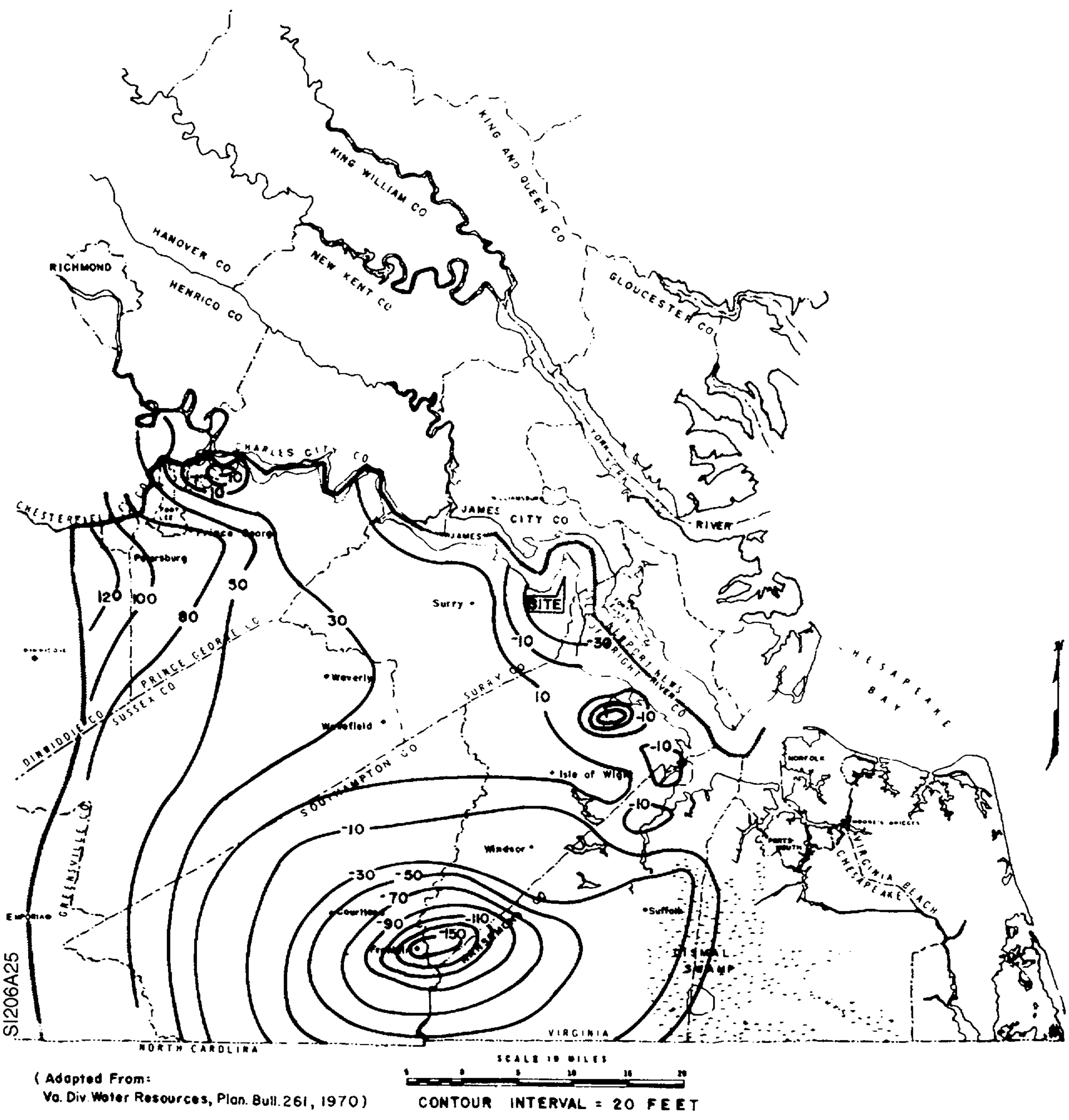
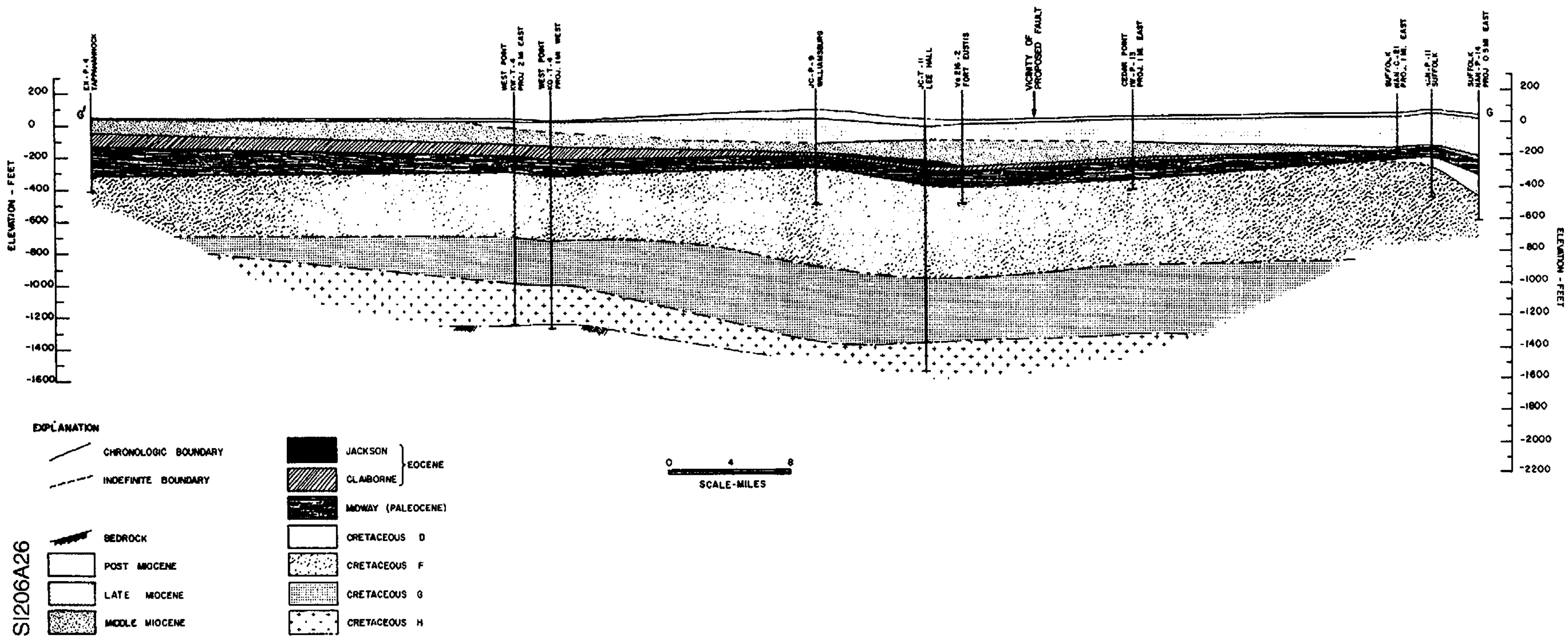


Figure 2A-26
GEOLOGICAL CROSS SECTION G-G' TAPPAHANNOCK TO SUFFOLK



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Appendix 2B

APPENDIX 2B
IN-SITU SEISMIC COMPRESSIONAL AND SHEAR WAVE VELOCITY MEASUREMENTS SURRY POWER STATION UNITS 3 AND 4

Presented herein is the excerpt from the Geotechnical Report for Surry Power Station Units 3 and 4 concerning the seismic velocity investigation and report from Weston Geophysical Engineers titled *In-Situ Seismic Compressional and Shear Wave Velocity Measurements.*

This data was obtained for Surry Power Station Units 3 and 4 located approximately 1/2 mile from the ISFSI site and is believed to be representative of the dynamic properties of the soil beneath the proposed installation.

Seismic Velocity Investigation

Ten borings were drilled and kept open for the detonating and monitoring devices of the seismic cross-hole investigation. The boreholes were cased to Elevation -150 with 3-1/2 in. o.d. flush joint casing. The borings were drilled within 1 inch of their planned location. Great care was taken to level and plumb the drill rigs, to ensure a vertical borehole. The appended report by Weston Geophysical Engineers, Inc. describes the seismic velocity investigation and presents the data.

IN-SITU SEISMIC COMPRESSIONAL AND SHEAR WAVE VELOCITY MEASUREMENTS SURRY POWER STATION UNITS 3 AND 4

Introduction

Seismic field measurements were performed at the location of the proposed Units 3 and 4, Surry Power Station of the Virginia Electric and Power Company, Surry, Virginia. Field work was conducted during the period of December 1972 through January 1973.

The purpose of this investigation was to measure both the in-situ "P" (compressional) wave and the "S" (shear) wave velocities of the geologic materials at the site. These velocities are used to compute values of Poisson's Ratio, Young's Modulus, Shear Modulus, and Bulk Modulus of these materials.

Field Procedures

Cross-hole velocity measurements were made using three orthogonal elements, containing one vertical and two horizontal geophones. Seismic energy was generated in one hole and detected by the geophones in four other holes with the seismic source and geophones at the same elevation level. This procedure was repeated using three combinations of shothole and detector hole as follows:

1. Shothole B-201
   Recording holes B-202, B-203, B-204, B-205
2. Shothole B-206
   Recording holes B-205, B-204, B-203, B-204
3. Shothole B-203
   Recording holes B-202, B-133S, B-137S, B-1357

Results

Figure 2B-1 shows the locations of the boreholes used for these measurements. The primary borehole array, Borings B-201 to B-206, is located along a line between the centers of the proposed Units 3 and 4. Shothole B-201 is at the center of the proposed Unit 3.

Table 2B-1 presents the results of this study from Elevation +5 to -140 feet. This table consists of the measured velocity values by elevation. Since there is some scatter on the travel-time curves plotted from the field data, these values are followed by a ± sign; this symbol indicates a range of ±50 ft/sec Also included are the elastic moduli values computed for the various velocity levels. Density values for these computations were provided by Stone & Webster Engineering Corporation.

Velocity values obtained from the three shothole-recording hole combinations were in excellent agreement with each other.

A limited amount of surface refraction data were obtained along the alignment between Units 3 and 4. The refraction data confirmed the "P" wave results of the cross-hole data above Elevation -50. It also indicated a near-surface material with a "P" wave velocity of 1500 ft/sec underlain by a thin layer of 2400 ft/sec "P" wave material.

Additional measurements using both cross-hole and uphole techniques were made at Surry Units 3 and 4. Two additional boreholes designated B-339 and B-340 were drilled as shot holes for the uphole and cross-hole surveys as shown on Figure 2B-2. Borehole B-340 is located at the eastern edge of Unit 4, as shown on the plan map of boreholes.

Cross-hole measurements were made using the following additional cross-hole patterns to supplement the original survey:

Shot Bole B-339 - Recording holes 201, 202, 204, and 205;

Shot Hole B-340 - Recording holes 202, 204, 205, and 206.

The cross-hole measurements using Shot Hole B-340, have been superimposed upon the travel-time plots of the original survey of January 1973 for comparison and show confirmation of the previous data as shown on Figure 2B-3.

An uphole survey was conducted in Boreholes B-339 and B-340. The location of surface detection arrays of vertical and horizontal geophones are shown on Figure 2B-2. Shots consisting of multiple cap arrays at 10-foot intervals were made using holes B-339 and B-340; these holes were uncased and drilling mud was used to keep them open. The travel-time plots for the uphole survey are shown on Figure 2B-3. Based on previous experience, an uphole survey rather than a down hole survey was conducted because of certain advantages in the control of energy generation, shot hole conditions and recording locations, including orientation of geophones. Seismic velocities measured in the uphole survey (Figure 2B-3) are the same as measured in the cross hole survey (Figure 2B-3).

Table 2B-1
SEISMIC VELOCITY AND DYNAMIC MODULE DATA

Elevation (feet)	"P" Wave Velocity (ft/sec)	"S" Wave Velocity (ft/sec)	Poisson's Ratio	Shear Modulus (psi)[a]	Young's Modulus (psi)[a]	Bulk Modulus (psi)[a]
+ 5 to 0	5200±	650±[b]	.492	1.09×10^4	3.26×10^4	68.57×10^4
0 to - 50	5600±	950±	.485	2.33×10^4	6.94×10^4	78.11×10^4
-50 to -90	5300±	950±	.483	2.33×10^4	6.93×10^4	69.64×10^4
-90 to -140	5500±	970±	.484	2.43×10^4	7.23×10^4	75.10×10^4

NOTES: ± Indicates range of ±50 ft/sec

a. Moduli calculation - based on a unit weight of 120 lb/ft^3.
b. Based on limited data.

Figure 2B-1
BORING LOCATION MAP
IN-SITU COMPRESSIONAL AND SHEAR VELOCITY MEASUREMENT

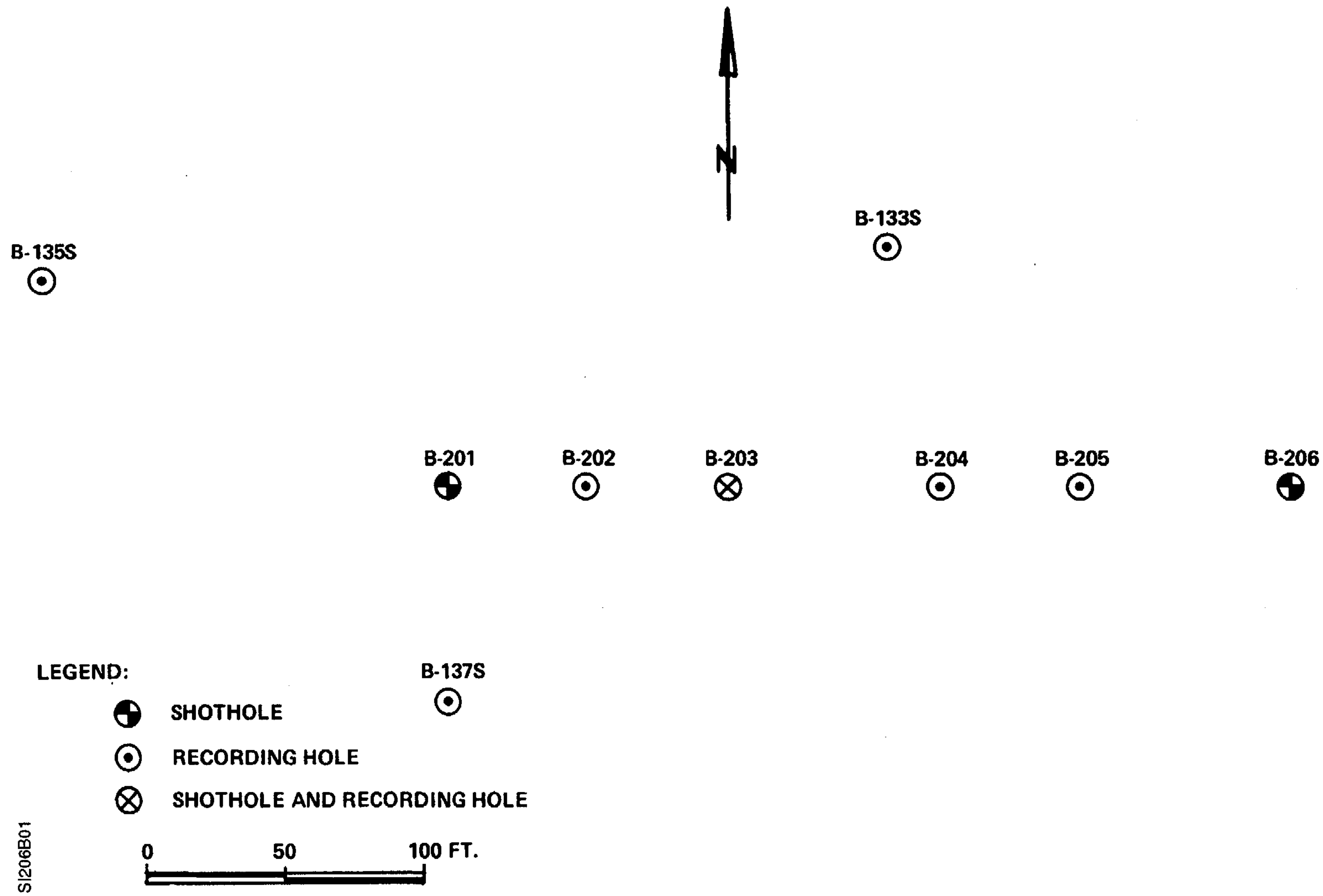

Figure 2B-2
SEISMIC UPHOLE LOCATIONS

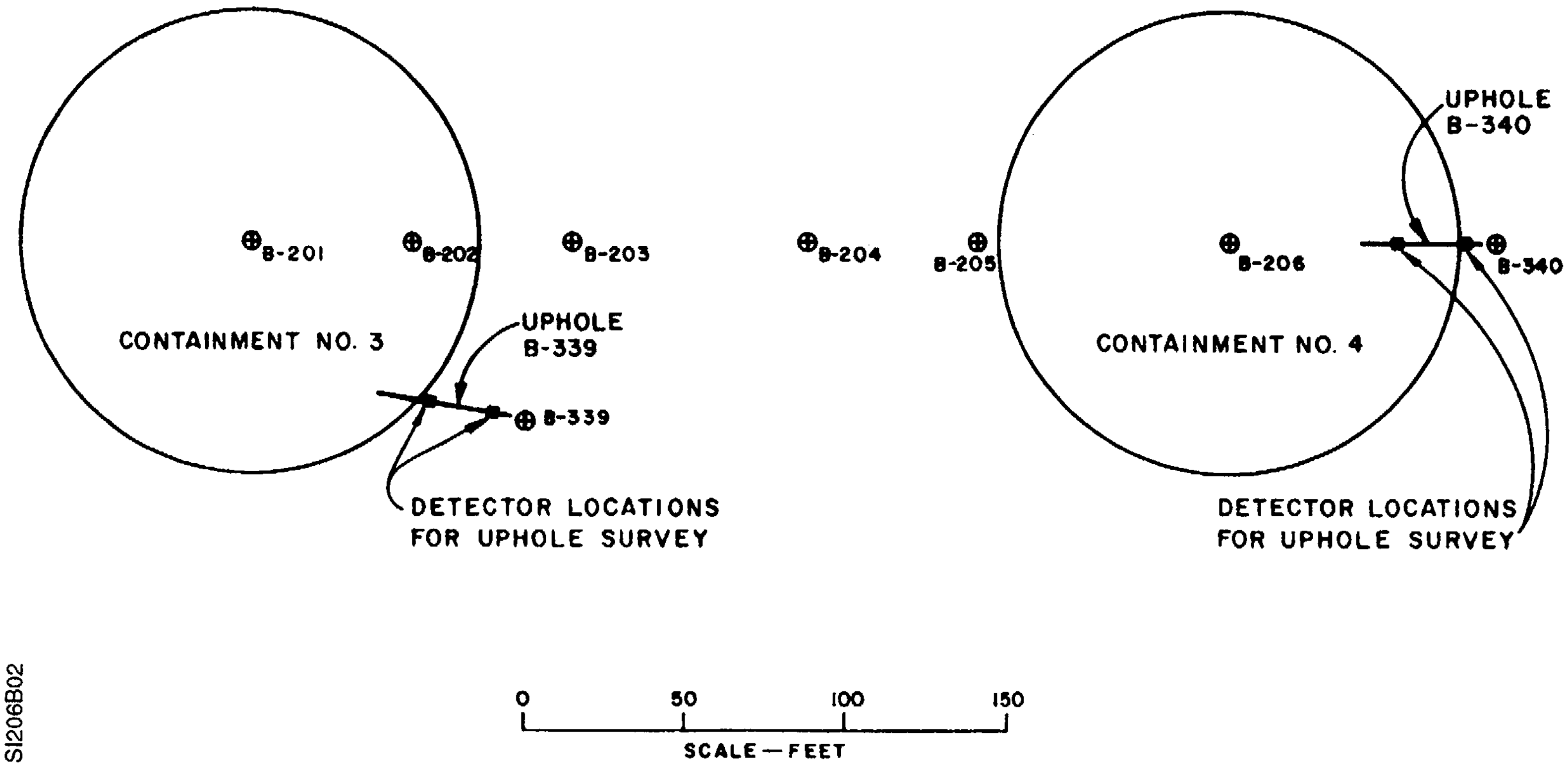

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Figure 2B-3 (SHEET 1 OF 5)
SEISMIC CROSSHOLE TIME DISTANCE PLOTS

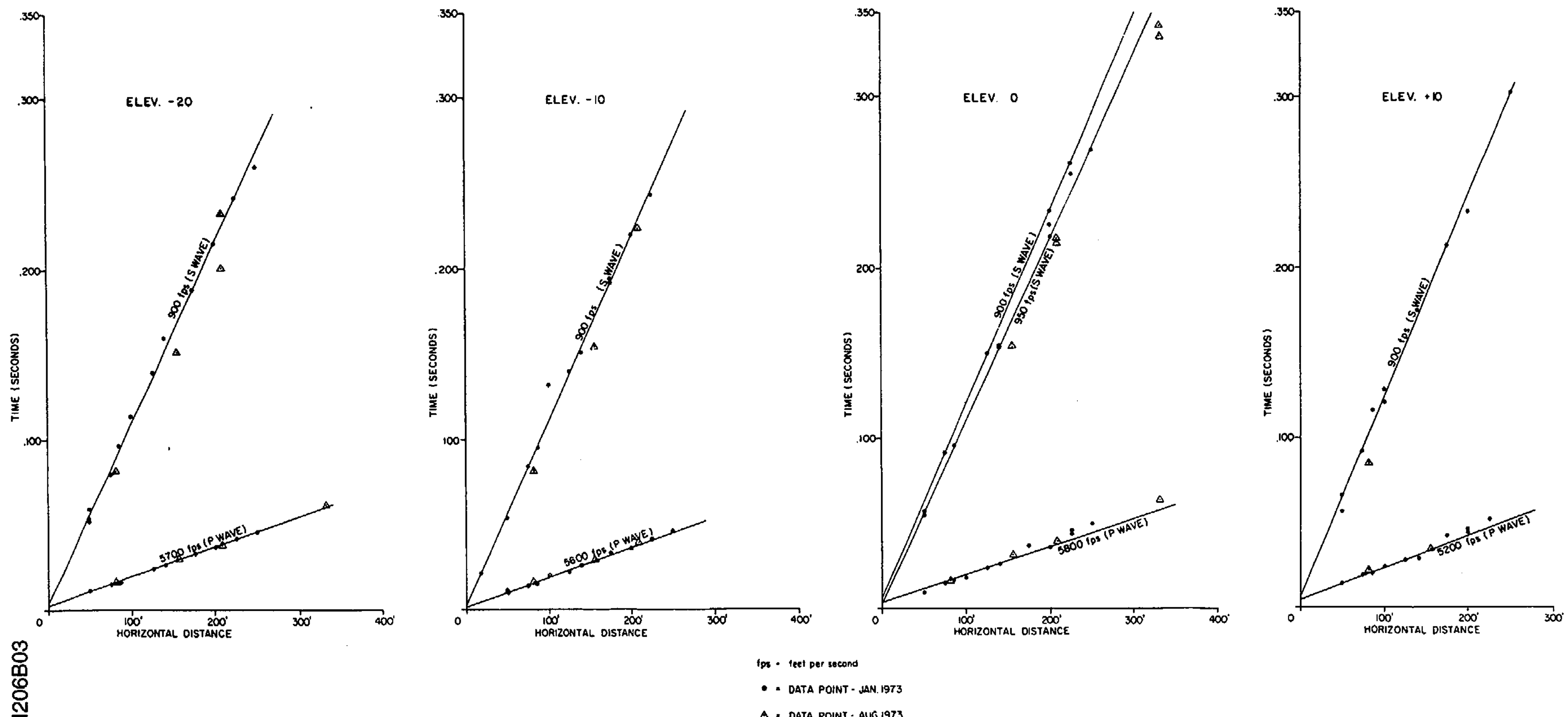

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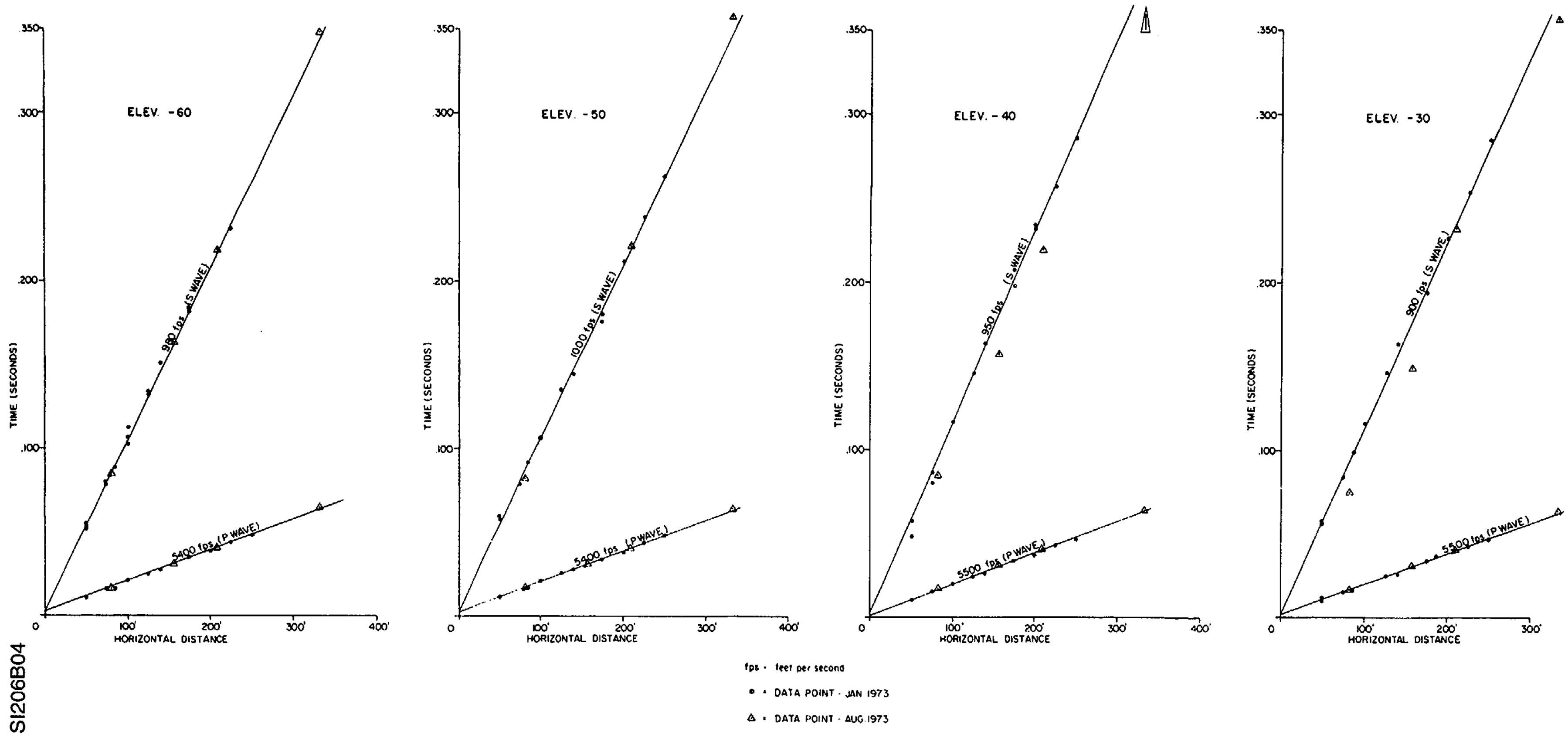
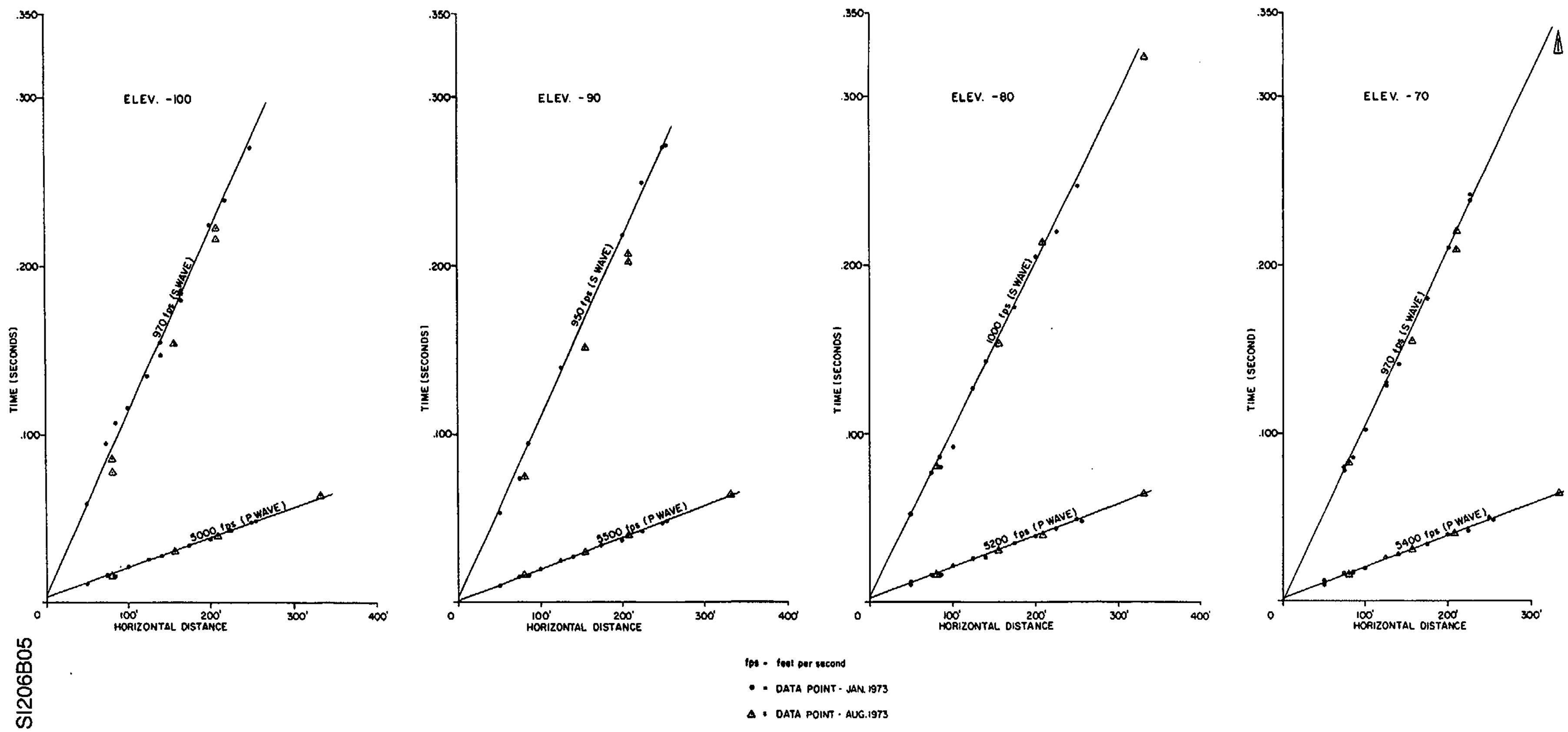
Figure 2B-3 (SHEET 2 OF 5)
SEISMIC CROSSHOLE TIME DISTANCE PLOTS



SI206B04

Figure 2B-3 (SHEET 3 OF 5)
SEISMIC CROSSHOLE TIME DISTANCE PLOTS



SI206B05

Figure 2B-3 (SHEET 4 OF 5)
SEISMIC CROSSHOLE TIME DISTANCE PLOTS

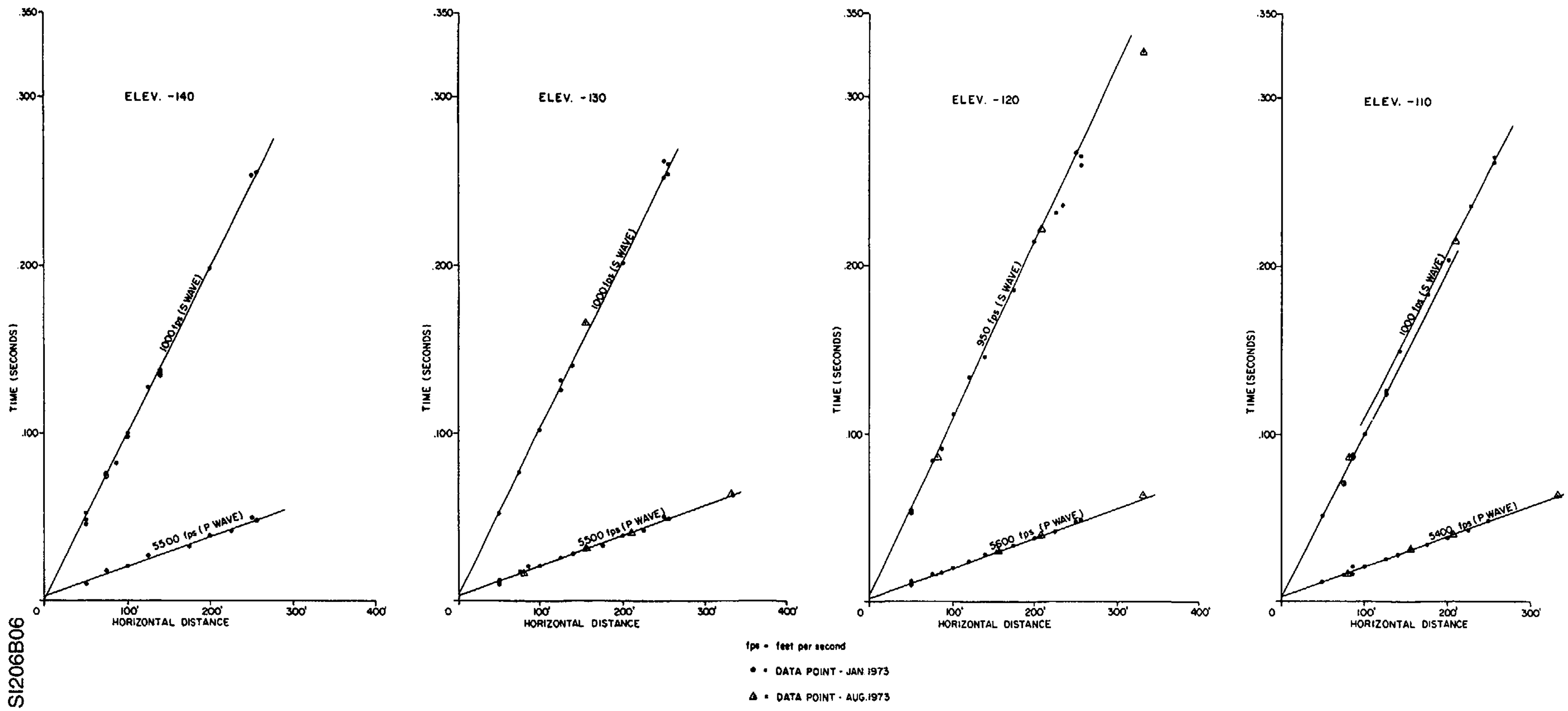
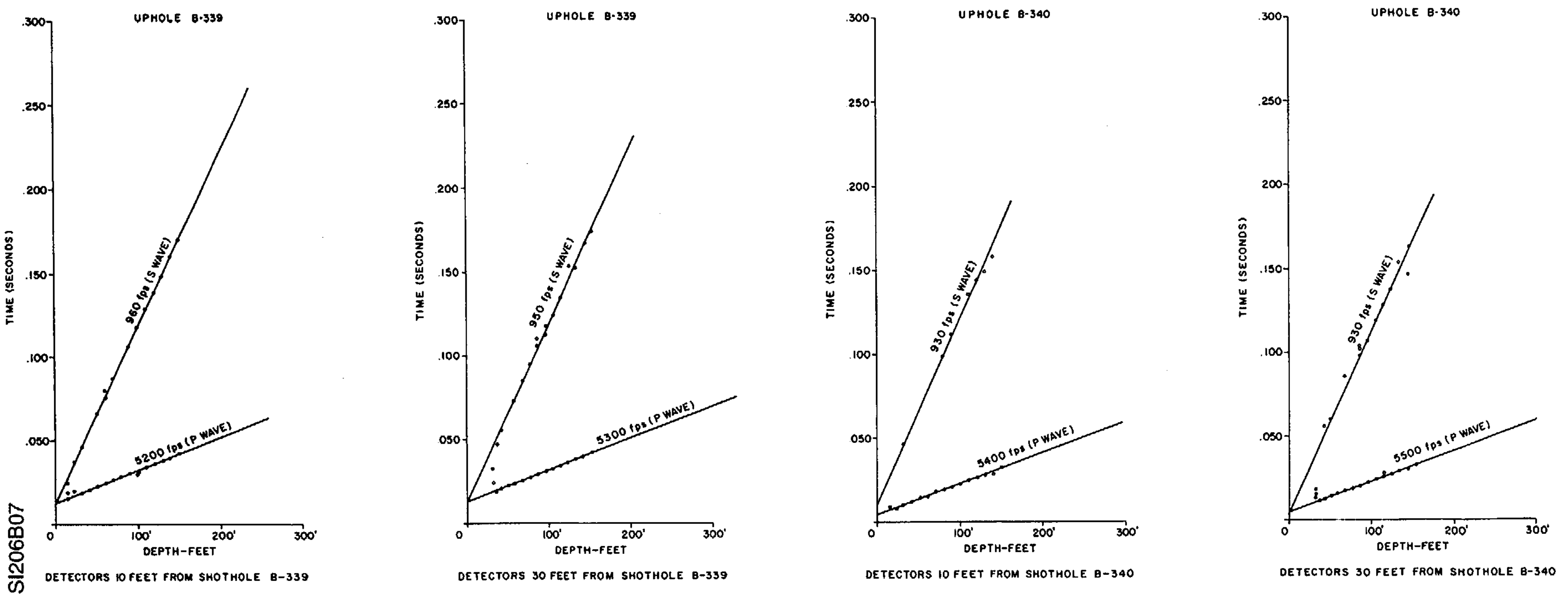


Figure 2B-3 (SHEET 5 OF 5)
SEISMIC CROSSHOLE TIME DISTANCE PLOTS



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3. Design Criteria

Chapter 3
DESIGN CRITERIA

This chapter describes the design criteria to be met by the SSSCs to be used in the Surry ISFSI. Compliance with these criteria ensures that the Surry ISFSI complies with the requirements of 10 CFR[1] Part 72.

3.1 PURPOSE OF INSTALLATION

The purpose of the Surry ISFSI is to provide additional interim storage capacity for the spent fuel resulting from the operation of the two pressurized water reactors at the Surry Power Station.

3.1.1 Materials to Be Stored

The ISFSI is designed to accommodate a total of 84 SSSCs. The ISFSI is capable of accommodating 1764 fuel assemblies. Each fuel assembly has 0.46 MTU. The total spent fuel storage design capacity of the facility is 811.44 MTU.

The physical characteristics of the fuel and fuel insert components to be stored at the ISFSI are described in detail in Chapter 3 of the Surry Power Station Units 1 and 2 FSAR and are summarized in Table 3.1-1. An evaluation of the storage of insert components with the fuel placed in SSSCs is provided in Appendix A for each SSSC design.

Fuel used during the first years of Surry Power Station Units 1 and 2 operation had initial enrichments not exceeding 3.5 weight percent U-235 and discharge burnup not exceeding 35,000 MWD/MTU. The Surry Power Station has been authorized to operate with fuel with higher initial enrichment and higher burnup. This SAR and the referenced SSSC topical reports, however, address only the fuel enrichments up to the maximum analyzed for the SSSCs as referenced in Appendix A and the SSSC topical reports.

The average heat generation rate for each cask at the time of storage will be as specified in the SSSC topical reports or Appendix A and the ISFSI Technical Specifications. |

3.1.1.1 Material Characteristics

The following fuel assembly characteristics constitute limiting parameters for storage of specific assemblies at the ISFSI:

a. Initial Fuel Enrichment

b. Fuel Burnup

c. Heat Generation

d. Spent Fuel Physical Configuration/Condition

1. Code of Federal Regulations, Title 10, *Energy*, January 1, 1982.

3.1.1.1.1 Allowable Limits

The allowable limits for each of these characteristics are discussed below.

3.1.1.1.1.1 *Initial Fuel Enrichment.* The initial fuel enrichment of any fuel that is stored in the ISFSI will be limited to the maximum enrichment specified in the SSSC topical reports or Appendix A and the ISFSI Technical Specifications.

3.1.1.1.1.2 *Fuel Burnup.* The fuel that is stored in the ISFSI will be limited to that specified in the SSSC topical reports or Appendix A and the ISFSI Technical Specifications.

3.1.1.1.1.3 *Heat Generation.* The heat generation rate by an individual fuel assembly is dependent on three factors: the initial fuel enrichment, the fuel burnup, and the amount of decay time after discharge. The maximum allowable heat generation rate and fuel temperature for a particular SSSC are specified in the SSSC topical reports or Appendix A and the Surry ISFSI Technical Specifications.

3.1.1.1.1.4 *Spent Fuel Physical Configuration/Condition.* Only spent fuel irradiated at Surry Power Station Units 1 and 2 with the physical configuration as listed in items 1, 2, and 3 of SAR Table 3.1-1 will be stored in the ISFSI. The fuel stored shall be intact (unconsolidated), shall not have gross cladding defects, and shall not have visible physical damage which would inhibit insertion or removal from the cask fuel basket.

3.1.1.1.2 Verification

The method of verification for each of these characteristics is discussed below.

3.1.1.1.2.1 *Initial Fuel Enrichment and Fuel Burnup.* Fuel management records shall be utilized to verify that the initial fuel enrichment and fuel burnup are within the above limits. Each fuel assembly is engraved with a unique identification number (based on ANSI/ANS 57.8) and a vendor identification, which is unique to the site for which the fuel assemblies were fabricated. This will allow visual confirmation of the identity of the fuel assemblies placed in the cask.

3.1.1.1.2.2 *Heat Generation.* The heat generation rate of a fuel assembly is based on three factors: initial fuel enrichment, burnup, and cooling time after discharge. Fuel management records will be used to obtain these three factors and an NRC approved code such as ORIGEN will be utilized to ensure that the heat generation is less than that specified in the SSSC topical reports and the Surry ISFSI Technical Specifications.

3.1.1.1.2.3 *Spent Fuel Physical Configuration/Condition.* Fuel management records will be reviewed to ensure that the assemblies to be put in the cask have not been previously identified as having gross cladding defects. The fuel assemblies shall also be visually inspected (e.g., using TV cameras) for physical damage which could potentially cause problems during insertion and/or removal from the storage cask.

3.1.2 General Operating Functions

The fuel assemblies will be stored unconsolidated and dry in sealed surface storage casks. The casks will rest on a reinforced concrete slab, and provide safe storage by ensuring a reliable decay heat path from the spent fuel to the environment and by providing appropriate shielding and containment of the fission product inventory.

Storage of spent fuel in SSSCs is a totally passive function, with no active systems required to function. Decay heat is removed via the cask surface to the environment by convective and radiant cooling.

The casks are to be handled with a lifting yoke, the fuel building cask handling crane, a transporter, or other appropriate equipment. The fuel building crane places the cask on the concrete pad in the crane enclosure. The cask is then picked up by the transporter which is pulled to the ISFSI by a haul vehicle. After the transporter has been maneuvered to locate the cask in its storage position, the cask is set down by the transporter.

The equipment in the fuel building is capable of handling casks and associated lifting equipment up to 125 tons fully loaded with the casks measuring no more than 16 feet in length with the top cover removed.

All the handling equipment to be used outside the fuel building will be sized to handle casks measuring up to the above specifications, as needed. This equipment will be designed according to appropriate commercial codes and standards, and will be operated, maintained, and inspected in accordance with the supplier's recommendations. Documentation shall be maintained to substantiate conformance with all applicable standards.

Table 3.1-1
CHARACTERISTICS OF FUEL USED AT SURRY POWER STATION [a]

1. Fuel Assemblies	
a. Rod array	15 x 15
b. Rods per assembly	204 (21 fuel rods are omitted to provide passage for control rods, insert components, and in-core instrumentation)
c. Length, including insert component	162.2 in.
d. Rod pitch	0.563 in.
e. Overall dimensions	8.426 in. x 8.426 in.
f. Total weight, including insert component	1525 lb
g. Active fuel length	144 in.
2. Fuel Rods	
a. Outside diameter	0.422 in.
b. Clad thickness	0.0243 in.
c. Clad material	Zircaloy-4
3. Fuel Pellets	
a. Material	UO_2 Sintered
b. Length	0.6 in.
4. Fuel Condition for Storage in SSSCs	
a. Maximum initial enrichment	[b]
b. Maximum burnup of storage	[b]
c. Average heat generation for one cask at time of storage	[b]

a. From Surry Power Station Units 1 and 2 FSAR. All dimensions are for cold conditions.
b. Specified in the SSSC topical reports or Appendix A and the Surry ISFSI Technical Specifications.

3.2 STRUCTURAL AND MECHANICAL SAFETY CRITERIA

The safe storage of the spent fuel assemblies depends only on the capability of the SSSCs to fulfill their design functions. The SSSCs are self-contained, independent, passive systems, which do not rely on any other systems or components for their operation. Therefore, the SSSCs are the only components at the Surry ISFSI which are important to safety. The criteria used in the design of the SSSCs ensure that exposure of the SSSCs to credible site hazards will not impair their safety functions.

3.2.1 Tornado and Wind Loadings

3.2.1.1 Applicable Design Parameters

The SSSC manufacturers will be required to meet either the design basis tornado and extreme wind used for the Class 1 (safe shutdown) systems and structures of the Surry Power Station, as described in Section *2.2.2* of the Surry Power Station FSAR and Section 2.3.1.3.2 of this SAR or alternately, those prescribed by Regulatory Guide 1.76, *Design Basis Tornado for Nuclear Power Plants*, April 1976. The design basis tornado for the Surry Power Station has a rotational wind velocity of 300 mph, a translational velocity of 60 mph, and a pressure drop of 3 psi in 3 seconds.

The design basis extreme wind is 137 mph at 30 feet above ground and with a gustiness factor of 1.3, as described in Section 2.3.1.3.1 of this SAR.

The design basis tornado and wind loadings for the casks are provided in the SSSC topical reports.

Design basis extreme ambient temperatures for the SSSCs have been selected to be -20°F and 115°F. These temperatures exceed the extreme temperatures experienced at the Surry site (Section 2.3.2.1.1), thus providing an additional level of conservatism. Other design criteria for the Surry ISFSI include 0- to 100-percent humidity and direct exposure to sunlight.

The daily solar radiation at the Surry site is estimated to be less than 800 cal/cm^2 (50 kW hours). This is a conservative estimate based on 90 percent transmissivity at the summer solstice (Reference 1). On this basis, a very conservative design criterion of an added heat load of 5 kW over 10-hour periods is imposed on the SSSCs.

3.2.1.2 Determination of Forces on Structures

The description of the methods used to convert the tornado and wind loading into forces on the casks is addressed in the SSSC topical reports.

3.2.1.3 Ability of Structures to Perform Despite Failure of Structures Not Designed for Tornado Loads

The safety function of the SSSCs is not dependent on any other structures or systems. In addition, there are no structures in the vicinity of the ISFSI, which, if failed under tornado loads, could damage the SSSCs.

3.2.2 Water Level (Flood) Design

The design basis flood used for the ISFSI is the same as that used for Class 1 (safe shutdown) structures of the Surry Power Station, and is described in Section 2.4.2 of this SAR. The maximum flood level calculated to occur at the ISFSI is 28.2 feet above msl. This is postulated to occur during the probable maximum hurricane, and includes wave runup.

The design finished grade elevation of the ISFSI is approximately 35.0 feet above msl, leaving a margin of more than 6 feet above the maximum flood. Therefore, the ISFSI site is flood dry.

3.2.3 Seismic Design

Section 2.6.2 describes the vibratory ground motions experienced in the region of the Surry site and defines a design earthquake peak acceleration value of 0.07 g for the ISFSI. As indicated in Section 2.6.2.3, an earthquake in excess of 0.05 g may be expected to have a recurrence interval of about 500 years. In view of the totally passive function of the SSSCs, and their inherent strength, a ground earthquake of 0.07 g is considered a conservative design criterion. See Appendix 3A. The SSSC topical reports describe the ability of the casks to withstand the design earthquake.

3.2.4 Snow and Ice Loadings

The rain and snow falls experienced at the Surry site are described in Section 2.3.1.2 of this SAR.

Snow and ice would melt soon after contacting the surface of the cask due to the decay heat generated by the stored fuel. These phenomena are not considered credible challenges to the SSSCs. Therefore, snow and ice loadings are not identified among the design criteria for the SSSCs.

3.2.5 Combined Load Criteria

The loads postulated as design criteria for the SSSCs have been described in this chapter.

Methods and assumptions made in analyzing the mechanical and structural behavior of the casks are described in the SSSC topical reports.

3.2.6 References

1. List, Robert J., *Smithsonian Meteorological Tables*, Sixth Revised Edition, 1951.
2. *Topical Safety Analysis Report for the CASTOR V/21 Cask Independent Spent Fuel Storage Installation (Dry Storage)*, GNSI, January 1985.

3.3 SAFETY PROTECTION SYSTEMS

3.3.1 General

The handling of the casks while they are being placed in the ISFSI requires that they be lifted by a transporter. Technical Specifications for the Surry ISFSI limit the height the SSSCs may be lifted while being transported to, and emplaced at, the ISFSI. The SSSCs are able to withstand a drop from these heights onto the ISFSI concrete slab without compromising their integrity and without resulting in physical damage to the fuel.

Because of the passive nature of the Surry ISFSI and the absence of support systems, no other items requiring special design consideration have been identified.

3.3.2 Protection by Multiple Confinement Barriers and Systems

3.3.2.1 **Confinement Barriers and Systems**

Confinement of radioactivity during the storage of spent fuel is achieved by (1) the uranium dioxide fuel pellet matrix, (2) the metallic tubes (cladding) in which the pellets are contained, and (3) the sealed cask in which the assemblies are stored.

The confinement function of the SSSCs is achieved by totally enclosing the spent fuel assemblies within a double-seal rigid metal vessel. The SSSCs are fabricated, delivered to the Surry site, loaded, sealed, and emplaced at the ISFSI in a manner that ensures their integrity, the capability to perform their safety functions, and compliance with all applicable rules and regulations.

The specific codes and standards to which the casks are fabricated, delivered to the site, and sealed are addressed in the SSSC topical reports. Compliance with applicable current nationally recognized codes and standards is expected. Codes and standards representing an acceptable level of design are:

a. American Welding Society (AWS) *The Structural Welding Code* (AWS Dl.1-1980)

b. American Iron and Steel Institute (AISI) *Steel Products Manual*

c. American Society of Mechanical Engineers (ASME) *Boiler and Pressure Vessel Code,* Section II

d. American Society of Testing and Materials (ASTM) Standards

As described in Chapter 11, the SSSC manufacturers will be required to maintain the necessary documentation to substantiate conformance with the specified codes and standards.

Construction materials are compatible with each other and with the expected radiation levels. In addition, the baskets or racks holding the fuel assemblies within the SSSCs are typical of those currently used in spent fuel pools throughout the industry, and are designed to protect the spent fuel assemblies from mechanical damage during insertion and removal operations and as a result of all credible events. Damage resulting from postulated accidents is limited to the extent that normal removal of the fuel assemblies is not precluded.

Once the casks are sealed, there are no credible events which could result in an unacceptable release of radioactive products to the environment. Similarly, there are no credible scenarios which could result in contamination of the outside surface of the SSSCs or in the generation of radioactive waste products.

3.3.2.2 **Ventilation—Offgas**

Natural air flow around the casks provides sufficient cooling. No forced ventilation is required. No radioactive releases during normal operation or accidents resulting in radioactive releases are considered credible. In addition, the gaseous releases postulated as the result of the hypothetical accidents described in Chapter 8 are of a very small magnitude. Therefore, no offgas system is required.

3.3.3 Protection by Equipment and Instrumentation Selection

3.3.3.1 **Equipment**

As discussed in Section 3.2, the SSSCs represent the only components of the ISFSI which are important to safety. Design criteria for the SSSCs are described in this section and summarized in Table 3.3-1.

3.3.3.2 **Instrumentation**

Due to the totally passive and inherently safe nature of the SSSCs, safety-related instrumentation is not necessary.

However, high quality commercial grade instrumentation will be provided to monitor the SSSCs functional performance. Instrumentation to survey and monitor cask parameters such as temperature and pressure will be furnished as recommended by the specific cask designs. Appropriate capabilities to check and recalibrate these monitors will also be provided. The casks are provided with temperature or pressure measuring systems as described in the SSSC topical reports.

3.3.4 Nuclear Criticality Safety

The criterion for ensuring that the fuel remains subcritical at all times is that the effective neutron multiplication factor (k_{eff}) be less than 0.95 (including any calculational uncertainties) for all normal and postulated accident conditions.

3.3.4.1 Control Methods for Prevention of Criticality

Methods to be used to ensure that subcriticality is maintained at all times in the casks are addressed in the SSSC topical reports or Appendix A.

3.3.4.2 Error Contingency Criteria

Error contingency criteria for the casks are presented in the SSSC topical reports or Appendix A.

3.3.4.3 Verification Analyses

The criteria for establishing verification of the models and programs used in the criticality calculations for the casks are presented in the SSSC topical reports or Appendix A.

3.3.5 Radiological Protection

Provisions for radiological protection by confinement barriers and systems are described in Section 3.3.2.1. No additional radiological protection design criteria are considered to be necessary.

3.3.5.1 Access Control

The Surry ISFSI does not require the continuous presence of operators or maintenance personnel. In addition, it is located within a fenced-in area shared only with a low level waste (LLW) storage facility and concrete pad for storage of contaminated material, which are not continuously manned. Access to the fenced-in area is limited to personnel needed during operations at the ISFSI or the LLW storage facility, e.g., periodic inspections of these facilities, emplacement of SSSCs, and security checks. These activities are controlled by station Health Physics and Security procedures.

3.3.5.2 Shielding

The SSSCs provide sufficient shielding to allow handling of the loaded casks with as low as reasonably achievable (ALARA) doses to the operators and to comply with the radiation limits in 10 CFR Part 72. For a description of the specific shielding provided by the casks, see the SSSC topical reports or Appendix A. For specific dose estimates, see Chapter 7 of this SAR.

3.3.5.3 Radiological Alarm Systems

There are no credible events which could result in unacceptable releases of radioactive products or unacceptable increases in direct radiation. In addition, the releases postulated as the result of the hypothetical accidents described in Chapter 8 are of a very small magnitude.

Therefore, radiological alarm systems are not necessary. However, as described in Sections 3.3.3.2, 4.3.7, and 5.4.1, other type nonsafety-grade monitors are provided with suitable alarms. Procedures to be followed when these alarms are activated will be specified in the Surry ISFSI operating procedures and are described in Section 4.3.7 of this SAR.

3.3.6 Fire and Explosion Protection

A backup diesel generator and its associated fuel tank are located within the ISFSI security fence. To prevent a postulated fire associated with a leaking fuel tank from propagating to the ISFSI, a collection trench is provided for the diesel fuel tank. There are no other significant combustible sources within the ISFSI security fence.

As indicated in Section 2.2.3.1, overpressure of less than 1 psi can be conservatively postulated to occur at the Surry ISFSI as a result of accidents involving explosive materials which are stored or transported near the site. Therefore, the SSSCs are designed to withstand a 1 psi external overpressure without any impairment of their safety functions. In addition, Section 2.2.3.2.1 indicates that an accidental release of fuel oil from the onsite fuel oil storage facility could result in an increase in the ambient temperature of about 8°F. As indicated in Section 3.2.1.1, the thermal analyses of the SSSCs assume an ambient temperature which exceeds the maximum temperature experienced at the site by about 10°F, and the maximum insolation during the summer solstice. These criteria provide sufficient margin to encompass the 8°F increase in ambient temperature that may be expected from the postulated oil fire.

3.3.7 Materials Handling and Storage

3.3.7.1 Spent Fuel Handling and Storage

The handling of spent fuel within the Surry Power Station is addressed as part of the facility license under 10 CFR Part 50. This includes the handling of the SSSCs within the spent fuel building and the loading of the casks with irradiated assemblies. Fuel that may be damaged to the extent of losing its cooling geometry or reasonable cladding integrity will be kept at the spent fuel pool and not considered for storage at the ISFSI.

Handling of the sealed casks outside of the power station in the process of emplacing them at the ISFSI will be done according to procedures that ensure that their safety functions and the power station capability for safe shutdown are not impaired. These operations are described in Chapters 5 and 9.

3.3.7.2 Radioactive Waste Treatment

The Surry ISFSI does not generate radioactive waste. However, cask loading and decontamination, while in the fuel and decontamination building, may generate very small amounts of waste. This waste is disposed of in accordance with the radioactive waste procedures described in Chapter 6, and is part of the 10 CFR Part 50 licensed activities.

3.3.7.3 Waste Storage Facilities

Waste storage facilities are neither required nor provided for the Surry ISFSI.

3.3.8 Industrial and Chemical Safety

No hazardous chemical are involved in the operation of the Surry ISFSI. Ion exchange resins are not used at the ISFSI, and no operations involving resins are anticipated.

Handling of the storage casks is the only operation which may be viewed as presenting a situation important to plant personnel safety, although equivalent loads are lifted and transported frequently during other industrial operations. Adherence to the ISFSI procedures will ensure that risks incurred during the handling of the SSSCs are minimized.

Table 3.3-1

DESIGN CRITERIA FOR DRY SEALED SURFACE STORAGE CASKS

The casks must meet the following criteria, assuming that the casks are loaded with the fuel described in Table 3.1-1.

1.	Maximum weight with yoke	125 tons
2.	Maximum length	16 feet with covers removed
3.	Criticality with single active or credible passive failure	k_{eff} <.95
4.	Capable of being lifted by mobile crane or lifting rig	
5.	Capable of being stored and transported in vertical or horizontal position	
6.	Adequate provisions to monitor performance of cask	
7.	Maximum surface dose	200 mrem/hr [a]
8.	Ambient temperature	-20°F to 115°F
9.	Direct exposure to sunlight	5 kW over 10-hr periods
10.	Ambient humidity	0 to 100%
11.	Tornado winds	300 mph rotational velocity, 60 mph translational velocity; or per Regulatory Guide 1.76, April 1974
12.	Tornado pressure drop	3 psi in 3 seconds
13.	Maximum winds (V_{30})	105 mph
14.	Gustiness factor	1.3
15.	Explosive peak overpressure	1 psi
16.	Design Earthquake peak acceleration	0.07 g

17. Withstand drop onto concrete slab without compromising cask integrity and without physical damage to fuel or loss of subcriticality
18. Capable of tipping over and rolling without exceeding expected damage for the cask drop onto concrete slab.
19. Designed, fabricated, delivered to site, and sealed according to recognized commercial codes and standards
20. Construction materials to be compatible with each other and with expected radiation levels
21. All surfaces contacting fuel assemblies to be free of burrs, sharp corners, edges, and weld beads that could mar or damage the fuel assembly surface or injure personnel
22. Permanent identification of each fuel assembly storage location to be provided
23. Leak tightness to be maintained under all operating conditions and credible events
24. Leak tightness to be maintained following cask drop onto ISFSI pad, Design Earthquake, and other postulated site hazards

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a. Doses for particular casks may vary, but dose due to total array of casks at the ISFSI must be enveloped by the analyses of Chapter 7 of this SAR.

Table 3.3-1 (continued)

DESIGN CRITERIA FOR DRY SEALED SURFACE STORAGE CASKS

25. All cutting and welding required for the handling of the casks not to result in damage to the fuel assemblies
26. All surfaces (external) wetted by fuel pool water to be epoxy coated to facilitate decontamination. This includes lifting yoke.

3.4 CLASSIFICATION OF STRUCTURE COMPONENTS AND SYSTEMS

3.4.1 General

The SSSCs are the only components of the Surry ISFSI which are important to safety. None of the other systems and structures comprising the Surry ISFSI (concrete slabs, fence, monitors, wiring, and lights) perform a safety function. The handling mechanisms (rigs, impact limiters, and transporter) are not considered important to safety because the SSSCs are designed to withstand their failure without jeopardizing the health and safety of the public.

The specific portions of the casks that are important to safety and a definition of the specific safety function are provided in the SSSC topical reports.

3.5 DECOMMISSIONING CONSIDERATIONS

3.5.1 General

No radioactive releases during normal operation or accidents resulting in radioactive releases are considered credible. Therefore, no means exist for the contamination of the outside surface of the casks, the concrete slabs, or any other part of the ISFSI. Even the accidents analyzed in Chapter 8 are postulated to result only in radioactive gaseous releases which will not contribute to the contamination of any component of the ISFSI. Thus, there is no need for any additional design criteria to explicitly facilitate decommissioning of the Surry ISFSI.

Steps for decommissioning the casks are provided in the SSSC topical reports.

Appendix 3A

Appendix 3A
STRUCTURAL CONSIDERATIONS FOR THE ISFSI CONCRETE SLAB

3A.0 INTRODUCTION

The primary purpose of the concrete slab is to provide a well defined and level support surface for the casks. It also serves as an aid in preventing tip over of the casks in the event of a seismic occurrence in that it provides a hard and stable surface upon which the casks are supported. Section 3A.2.5 of this appendix provides a demonstration that the material stored in the cask creates no added hazard to public health and safety due to tip over. Therefore, the support slabs of the ISFSI have no function important to safety. As the ISFSI and the casks are of a totally passive design, there are no safe shutdown functions required for safety and the term Seismic Category I is not applicable. Analysis has been conducted to demonstrate that the slabs, fully loaded with casks, will withstand a design earthquake with no adverse effects either to the slab or to the casks. Further, the analysis has shown that the casks remain upright during and after the seismic event.

3A.1 ANALYSIS FOR DESIGN EARTHQUAKE

3A.1.1 Design Criteria

An analysis of the slab and casks was conducted for the design seismic event with the following design criteria:

1. Consistent with the results of Section 2.6 of the ISFSI SAR, the design earthquake shall have a peak free field acceleration of 0.07g.

2. The design spectrum shall be in accordance with Regulatory Guide 1.60, *Design Response Spectra for Seismic Design of Nuclear Power Plants,* Revision 1, December 1973.

3. Consistent with similar seismic analyses, which were conducted for Surry Power Station Units 1 and 2 as reported in its FSAR, the free field motion shall be applied at the ground surface.

4. Based on these input parameters, a dynamic analysis of the slab and casks shall be conducted to quantify the effects of the design earthquake both in regard to the slab and casks, but more importantly to evaluate the potential for cask tip over.

3A.1.2 Implementation of Criteria—Method of Analysis

A time-history analysis was conducted for the slab fully loaded with casks in accordance with the mathematical model shown in Figure 3A-1. The slab was modeled as a rigid mass connected to an equivalent vertical and two orthogonal horizontal soil springs and associated dampers. Since the casks are rigid with respect to earthquake exciting frequencies and no mechanism for dynamic interaction between casks is present, this combined inertia effect is represented by a single rigid mass added to the mass of the slab. Auxiliary analyses were conducted to evaluate cask rocking and the potential for tip over.

3A.1.2.1 **Design Time History**

Three statistically-independent synthetic time-history records shown in Figure 3A-2 were used to represent the vertical and two orthogonal horizontal time-history records. Figures 3A-3, 3A-4, and 3A-5 compare response spectra developed from these time history records with that specified by Regulatory Guide 1.60 normalized, i.e., adjusted upward for a 1.0g earthquake for various damping ratios. As indicated in the figures, each individual time history provides a response that is equal to or exceeds the Regulatory Guide 1.60 spectra at all frequencies. These three time histories were used to simultaneously excite the slab and casks. Although the duration of the design earthquake is expected to be much less, the time history records extend for 24 seconds.

3A.1.2.2 **Soil-Structure Interaction**

Soil-structure interaction is accounted for by elastic half space concepts, in accordance with the procedures outlined in Reference 1. To account for possible variations in soil, two analyses were conducted, using lower and upper bound soil properties that represent possible variations in representative properties of the composite soil.

Shear Modulus, G_s	13.7×10^5 psf (lower bound)
Shear Modulus, G_s	27.0×10^5 psf (upper bound)
Soil Density, γ_s	115 psf
Poisson's Ratio, μ	0.49

To provide additional conservatism, the computed radiation damping values were reduced to 75 percent of the values computed by Reference 1. Soil material damping was taken as 3 percent critical and added to the radiation damping.

3A.1.2.3 **Computer Code**

The analysis was conducted using the BSAP computer code (Reference 2), which is a linear analysis finite element program which has been reviewed previously by the NRC staff.

Overturning of the casks was evaluated by comparing the maximum kinetic energy of the casks (E_s) to the potential energy (E_o) required to cause overturning. The factor of safety against overturning is the ratio of potential energy to maximum kinetic energy, or:

$$F.S. = \frac{E_o}{E_s}$$

where $E_s = 1/2\, m_c\, (V^2{}_H + V^2{}_V)$

m_c is the mass of the cask and V_H and V_V are, respectively, the maximum values of the resultant horizontal and vertical velocities. This introduces a conservatism into the analysis, since at any given instant the sum of these velocity components are less than the maximum values.

3A.1.3 Results of Analysis

As indicated in the previous section, two dynamic analyses of the slab and casks were conducted to represent lower and upper bound limits of the composite soil. Natural frequencies of the slab loaded with 28 125-ton casks are as follows:

	Lower Bound Soil Properties	Upper Bound Soil Properties
N-S direction, Hz	5.51	7.74
E-W direction, Hz	5.25	7.38
Vertical direction, Hz	6.98	9.78

A peak g level of 0.093g was obtained on the slab for the lower bound soil properties. However, the variation in soil properties had little effect on the response since a maximum g level of 0.088 was obtained for the upper bound soil properties. This results in a maximum amplification of the slab with respect to the free field motion of approximately 1.33.

Since the natural frequencies of the fully loaded slab are associated with those expected to provide peak and near-peak response, as indicated by the results associated with variation of soil properties, the response of a slab less than fully loaded with casks and/or with lighter casks would be expected to be no greater and probably less than that presented.

Evaluation of cask tip over based on the results of the dynamic analysis and using the energy approach discussed in the previous section indicates that the factor of safety against tip over is at least 240 for the design earthquake. The kinetic energy developed in the casks represents no more than 1/240 of that necessary to cause tip over.

Seismically induced settlement, discussed in SAR Section 2.6, is of no consequence either to slab integrity or to cask tip over.

3A.2 ADDITIONAL CONSIDERATIONS AND ANALYSIS

3A.2.1 Criteria

Analysis has been presented to demonstrate that adequate margins of safety are provided to ensure that cask tip over is not a viable consideration during the design seismic event. Additional analysis is presented in Section 3A.2.5 which ignores the above conclusions, but provides added assurance regarding the safety of the casks during a seismic event by evaluating the effects of a postulated tip over. It concludes that no adverse safety concerns exist if tip over occurs.

Further evidence regarding the extreme conservative design of these slabs and casks is obtained by evaluating the effects of an event even more severe than the design earthquake. The purpose of this additional analysis is to identify margins which exist above and beyond those necessary for the design earthquake.

To simulate the occurrence of substantial settlement, the following characteristics were considered:

1. Total uniform slab settlement of 14 inches. Although soil settlement may be induced by a seismic event, due to the time required for excess pore water pressure in the soil to dissipate, the actual settling of the ground would take place after the shaking has stopped. Therefore, it is not necessary to consider settlement or differential settlement in conjunction with a seismic event.

2. Accompanying the uniform settlement is a differential uniform settlement at the rate of 7 inches in 20 feet which is random in orientation and may occur in multiple directions.

3. Additionally, it has been assumed that the slab can sustain a loss of contact with the soil for a span of 15 feet at multiple locations randomly selected.

   These do not represent values determined by soil stability analysis but rather represent extreme assumptions much more severe than the design event selected only to demonstrate the additional safety margins which exist in the slab/cask system if influenced by a seismic event.

To evaluate the effect of settlement on the slab and casks, the following criteria were established:

1. As a result of the severe differential settlement conditions specified above, the concrete compressive strength shall be taken as equal to or less than the minimum specified design concrete strength of the slab.

   Reinforcing steel strain shall be no greater than 50 percent of the minimum specified ultimate strain. Results shall also show that the slab does not separate vertically due to shear loading.

2. The slab shall be considered acceptable for bridging a span of 15 feet if the concrete stresses remain below the minimum specified compressive stress and the reinforcing steel stresses do not exceed minimum specified yield stress.

3A.2.2 Method of Analysis

As previously discussed, the most critical effect of the dynamic response due to a seismic event is the potential for overturning. Considering the margins of safety associated with the overturning of a cask for the design earthquake and realizing that the kinetic energy will increase approximately with the square of the excitation level, it is evident that excitation levels in excess of ten times the design earthquake level are required to cause overturning of the casks. Thus, cask tip over due to dynamic events has substantial margins above the design earthquake excitation level. For this reason, further dynamic analysis of the slab loaded with casks need not be considered.

The effect of extreme seismic induced soil settlement may contain four possible separate components:

1. Uniform downward settlement
2. Uniform differential settlement
3. Differential settlement which is random in orientation and occurs in multiple directions
4. Loss of contact over a large area of the support surface

Uniform downward settlement causes no adverse effect on either the slab or the cask. The only effect such settlement has is to lower the final elevation of the slab/cask system. Likewise, uniform differential settlement of the slab causes no reduction in the structural integrity of the slab. It does, however, increase the chances of cask tip over. However, since the height of the cask center of gravity is approximately equal to its width, the differential settlement must cause the slab to be tilted in excess of 23° from the horizontal before this possibility is realized.

Multiple oriented differential settlement, if it is excessive, has the potential to cause permanent distress to the slab. Although such distress does not necessarily affect the functional requirements of the slab, as discussed previously, it is an issue that can be addressed to provide assurance that the slab remains continuous, and, therefore, maintains a sufficiently level and well defined resting place for the casks.

To evaluate the performance of the slab under these extreme conditions, two mathematical models of the slab were generated, representing two worst cases of randomly oriented settlement conditions. (See Figure 3A-6). The model represents the slab by two-dimensional elasto-plastic beam sections supported on a bed of special spring elements, which represents the elastic properties of the soil. The magnitude of the moments at the elastic limit of the beams was determined in accordance with the ultimate strength design methods included in ACI 318-83. The limit was assumed to occur when the tension reinforcing steel reaches its yield strain limit. As a result, the slab section was designed to be underreinforced and, therefore, yielding of the reinforcing steel will occur before crushing of the concrete. This ensures ductile behavior.

Maximum differential settlement was assumed to emanate from an arbitrary reference point on the slab in opposite directions such that the reference point either becomes a high point (see Figure 3A-6) or a low point as in Figure3A-6. This was accomplished by using special soil spring elements that have the capability of providing initial gaps at appropriate locations under the slab. Note that in Figure 3A-6 a slope equal to twice the maximum anticipated differential settlement is imposed on one side of the slab. This approach was necessary to initiate the mathematical solution and is valid in representing equal maximum settlement downward and away from an arbitrary reference point on the slab. Downward loading of the casks (along with the dead load of the slab) was enforced in accordance with the imposed spacing of the casks, but was oriented such as to represent a worse loading condition.

The analyses were conducted using the ANSYS computer code (Reference 3), which is a nonlinear finite element code that has been previously utilized in structural analysis of nuclear power plant structures.

The effect of loss of contact with the soil was considered by eliminating support under the slab for an infinitely long strip having a width of 15 feet. A study determined that the controlling location and orientation of this strip is most severe if it is either placed at the end of the slab, causing it to be cantilevered, or placed in the longitudinal direction of the slab such that either side of the slab is unsupported for a width of 15 feet. All other possible orientations produce less severe effects on the structure. Structural integrity of the slab was evaluated manually in accordance with ACI 318-83.

3A.2.3 Results

Maximum strain in the reinforcing steel occurs for the case where the arbitrary reference is the high point on the slab (Figure 3A-6). The computed strain is no more than 0.016 or 46 percent of the allowable. Shear capacity of the slab is computed to be no more than 36.5 percent of the ultimate capacity.

Utilizing a 3-foot-deep slab reinforced with No. 11 rebar at 12 O.C. each way, top and bottom, the reinforcing steel is stressed to approximately 85 percent of allowable due to loss of soil support. The allowable stress is 90 percent of yield stress of the reinforcing steel.

3A.2.4 Criteria to Evaluate Acceptability of the Concrete Slab Following a Design Earthquake

In the unlikely event that the design earthquake were to occur at the site, assessment of potential damage would address the following three concerns:

1. Structural integrity of the concrete slab
2. Stability of the casks as it is affected by potential differential settlement
3. Stability of the foundation material

Although the system can be exposed to much more severe seismic conditions without jeopardizing the overall stability of the casks, continued use of the slab after a design earthquake will be based on meeting such criteria.

Meeting these criteria ensures the slab will remain within its elastic limit and that foundation stability is maintained.

Structural integrity of the slab is influenced by the strain in the reinforcing steel since the slab is underreinforced. This strain can be evaluated by the change in curvature of the slab caused by the seismic event.

Differential settlement which would cause instability of the casks is not a controlling concern. Based on the geometry of the cask, the slab could experience a differential settlement at the rate of 105 inches over 20 feet before cask instability would occur.

Stability of the foundation materials can be ensured if differential settlement is within limits to maintain the structural integrity of the concrete slab.

Utilizing the mathematical models shown in Figure 3A-6 to evaluate the slab, it has been determined that a vertical relative displacement caused by a seismic event of 1/2 inch between any two points on the slab 14 feet apart can be tolerated before slab replacement or a detailed structural evaluation is required. If the relative settlement of the slab is within these limits, the slab may be safely used with assurance that integrity will be maintained during a future design seismic event. These relative displacement limits are based on postulated differential settlement of 3 inches in 20 feet occurring in opposite directions from an arbitrary reference point on the slab.

3A.2.5 Cask Tip-Over Accidents

As previously discussed in Section 3A.1.3, adequate margins of safety exist to ensure against cask tip-over resulting from the ISFSI design earthquake.

The cask tip-over analyses are described in the SSSC topical reports and include an evaluation of the following concerns:

1. Criticality must be within acceptable limits.
2. Cask integrity must be maintained (no loss of confinement).
3. Any damage must be limited so as not to preclude the removal of fuel assemblies (i.e., basket integrity must be maintained).

3A.2.6 Conclusions

Based on the results of the site specific investigations and analyses for the Surry ISFSI, the following conclusions can be made:

1. Based on the criteria established in 10 CFR 72.66(b) and using a building code approach for determining the seismic design level, a conservative value of 0.07g was determined for the design earthquake.
2. The soil stability analysis under static loading indicated that the factor of safety against a bearing failure is greater than 3.0.
3. The minimum factor of safety against the potential of liquefaction using the simplified procedure is 1.5.
4. The analyses that were performed for the concrete slab indicated the slab would remain continuous and without loss of integrity during the design earthquake. Additional analyses indicated the concrete slab could withstand, without loss of integrity, uniform downward

settlement of 14 inches, differential settlement of 7 inches in 20 feet, or loss of soil contact for a span of 15 feet.

5. Analyses performed regarding the potential for cask tip over indicated a factor of safety to be over 240 under design earthquake conditions. Therefore, it can be concluded that the cask will not tip over during a design seismic event.

3A.3 References

1. Whitman, R. V., Richard, E. F., *Design Procedure for Dynamically Loaded Foundations,* Journal of the Soil Mechanics and Foundation Division Proceedings of the ASCE, 1967, pp. 169 to 193.

2. *BSAP, Bechtel Structural Analysis Program,* CE800, Version E13-47.

3. *ANSYS Program,* CE798, Rev. 3, Update 67H.

Figure 3A-1
BSAP MODEL OF SLAB, CASKS AND SOIL SPRINGS

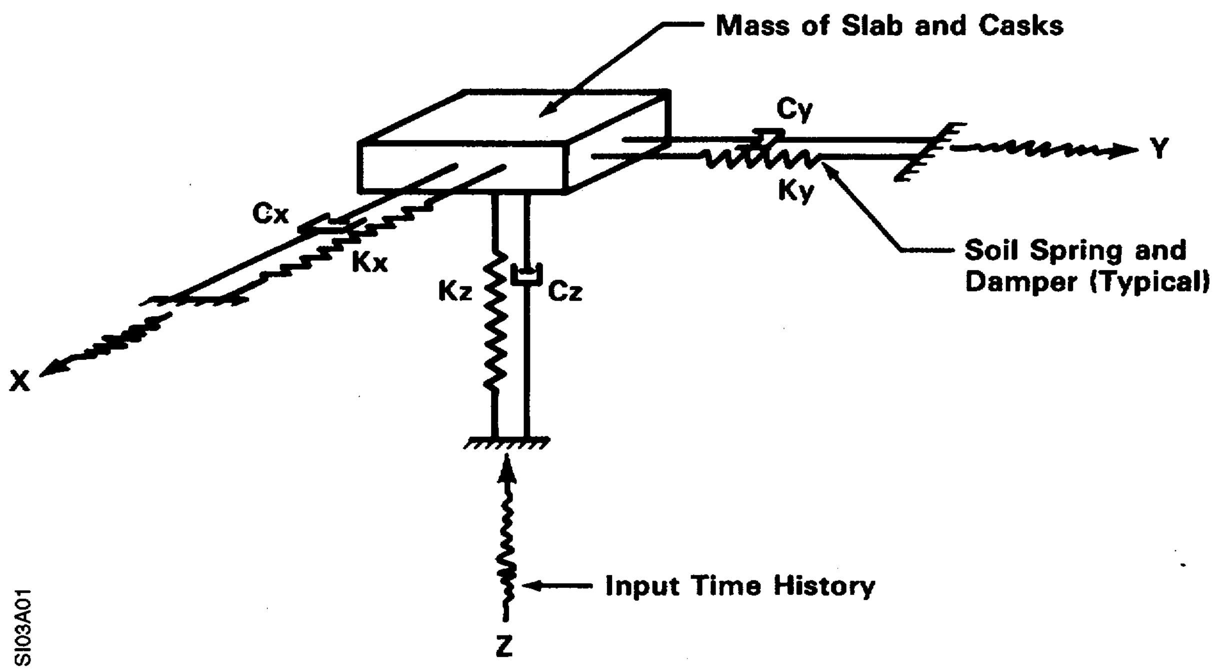


Figure 3A-2
SYNTHETIC TIME HISTORY MOTION OF THE DESIGN EARTHQUAKE

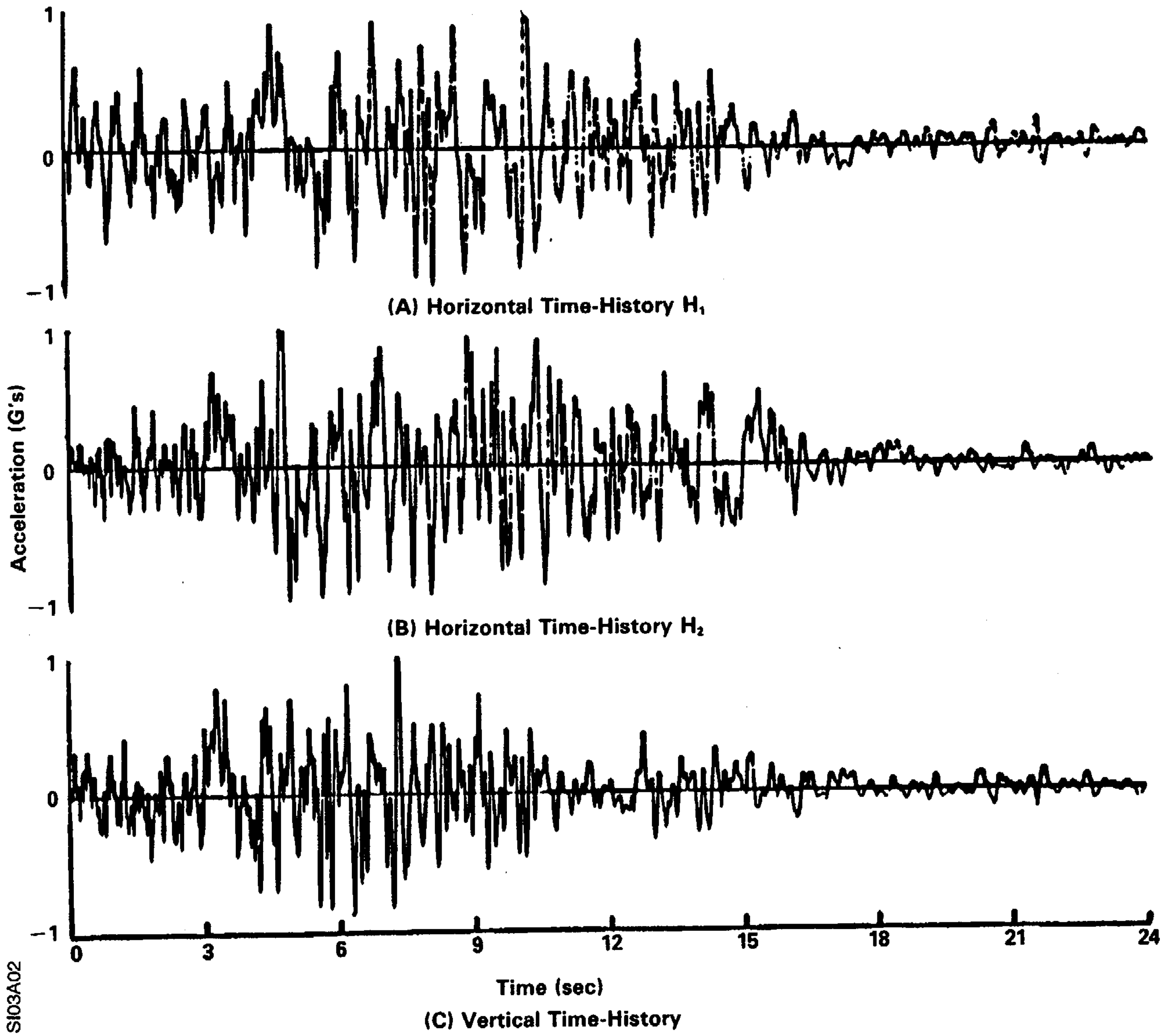


Figure 3A-3

COMPARISON OF THE ACCELERATION RESPONSE SPECTRA OF HORIZONTAL TIME HISTORY H_1 WITH THE HORIZONTAL DESIGN SPECTRA FOR 2 PERCENT, 5 PERCENT, AND 10 PERCENT CRITICAL DAMPING

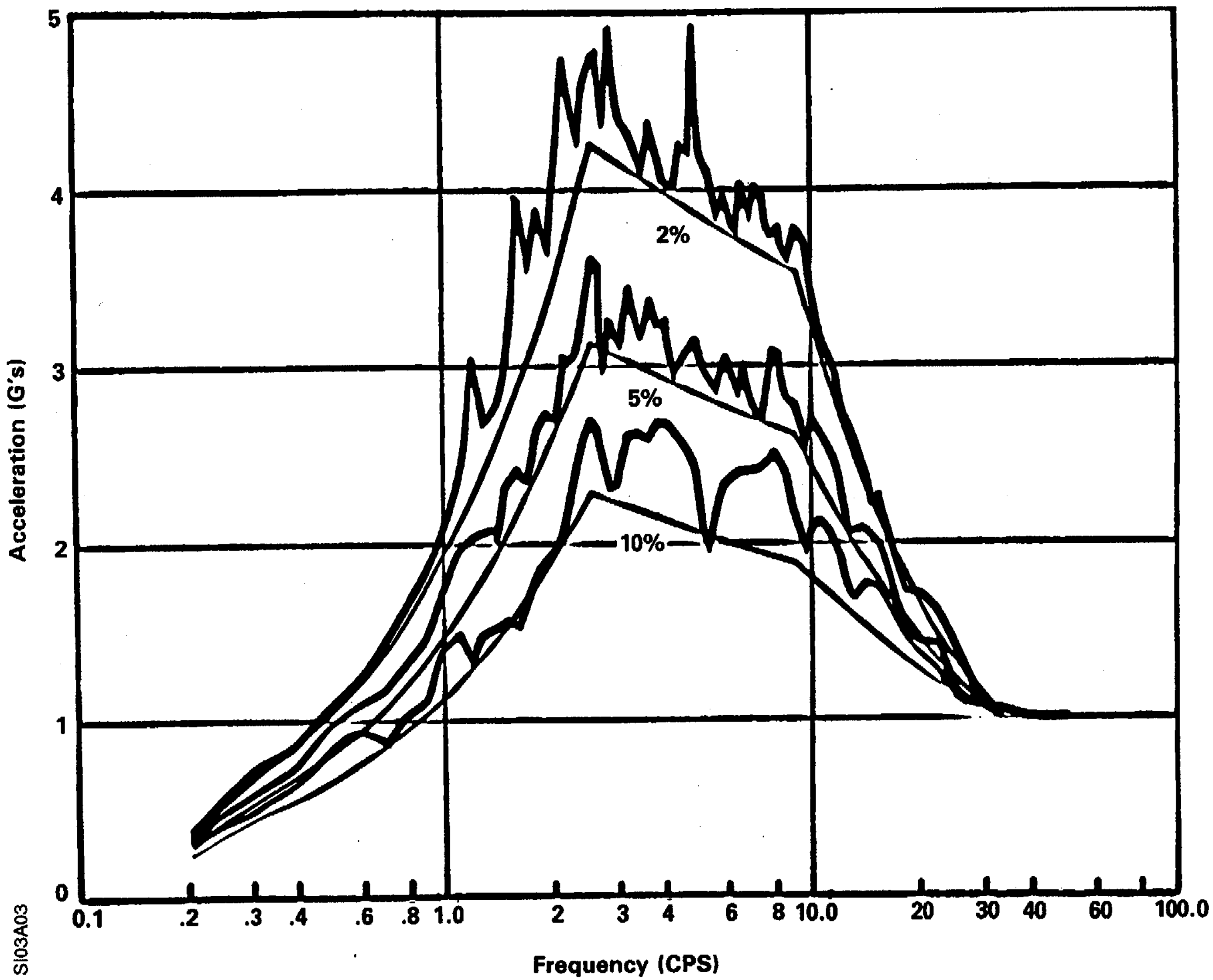


Figure 3A-4
COMPARISON OF THE ACCELERATION RESPONSE SPECTRA OF HORIZONTAL TIME HISTORY H_2 WITH THE HORIZONTAL DESIGN SPECTRA FOR 2 PERCENT, 5 PERCENT, AND 10 PERCENT CRITICAL DAMPING

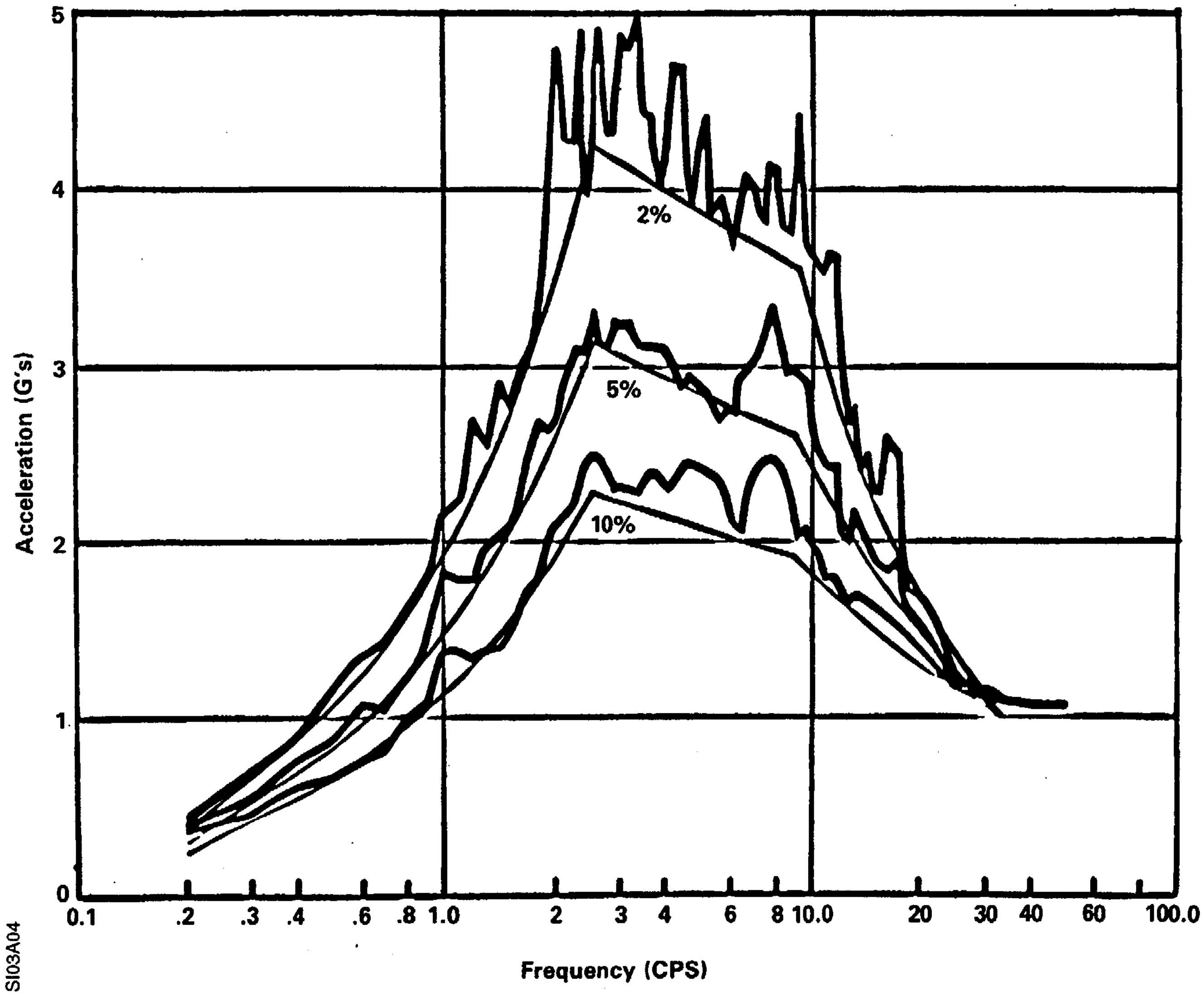


Figure 3A-5
COMPARISON OF THE ACCELERATION RESPONSE SPECTRA OF THE VERTICAL TIME HISTORY WITH THE VERTICAL DESIGN SPECTRA FOR 2 PERCENT, 5 PERCENT, AND 10 PERCENT CRITICAL DAMPING

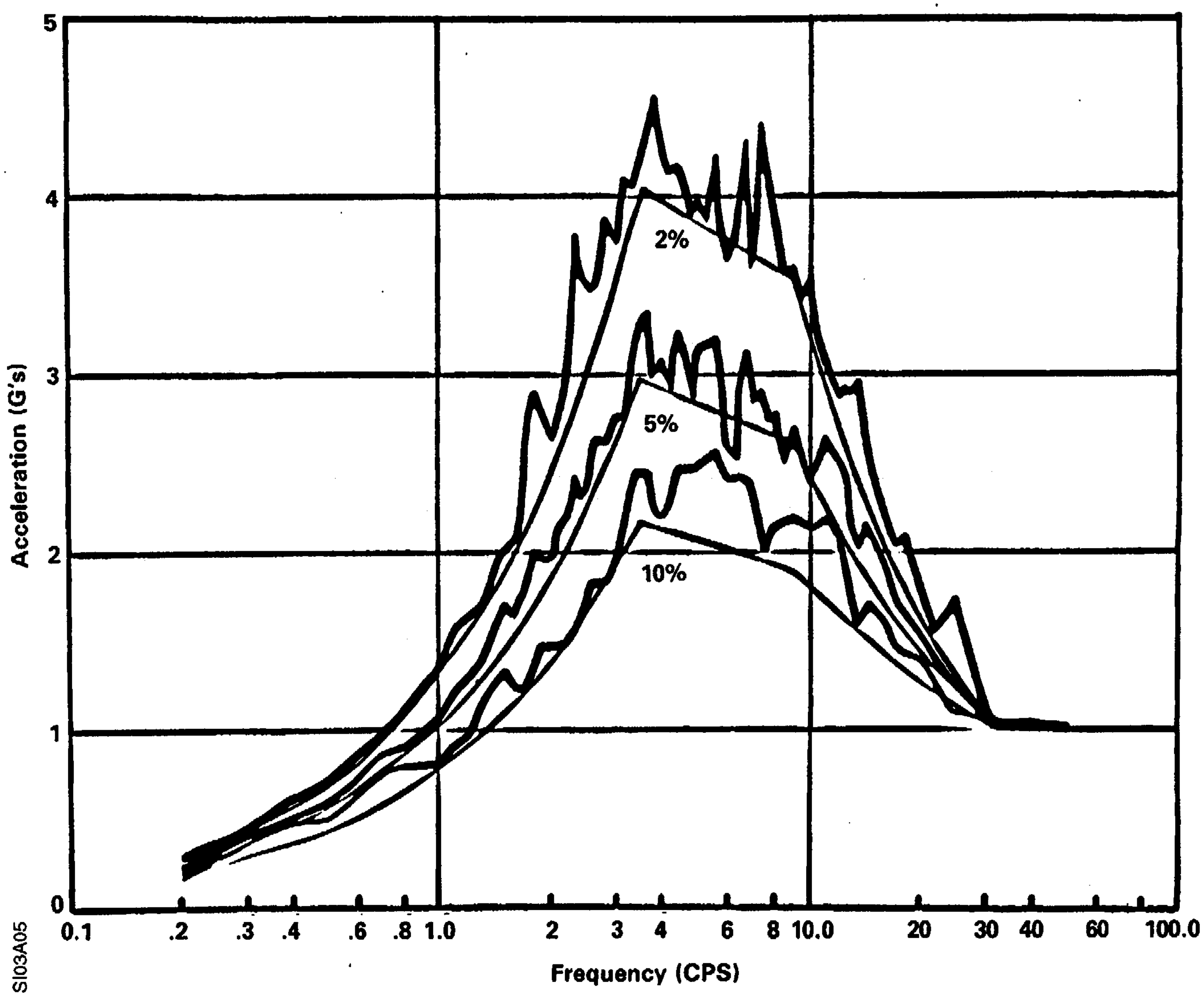


Figure 3A-6 (SHEET 1 OF 2)
ANSYS MODEL OF SLAB, CASKS AND SOIL SPRINGS

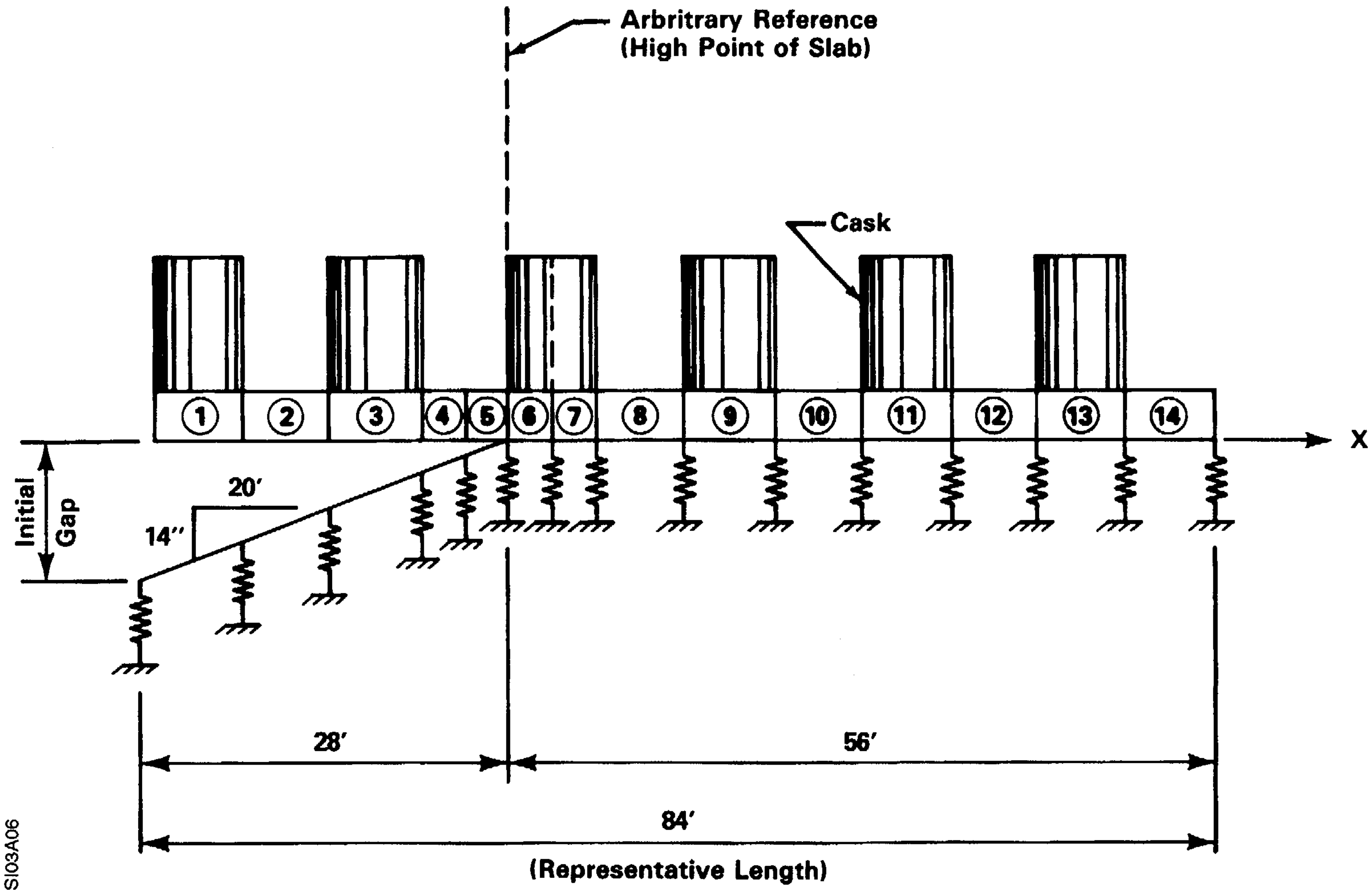
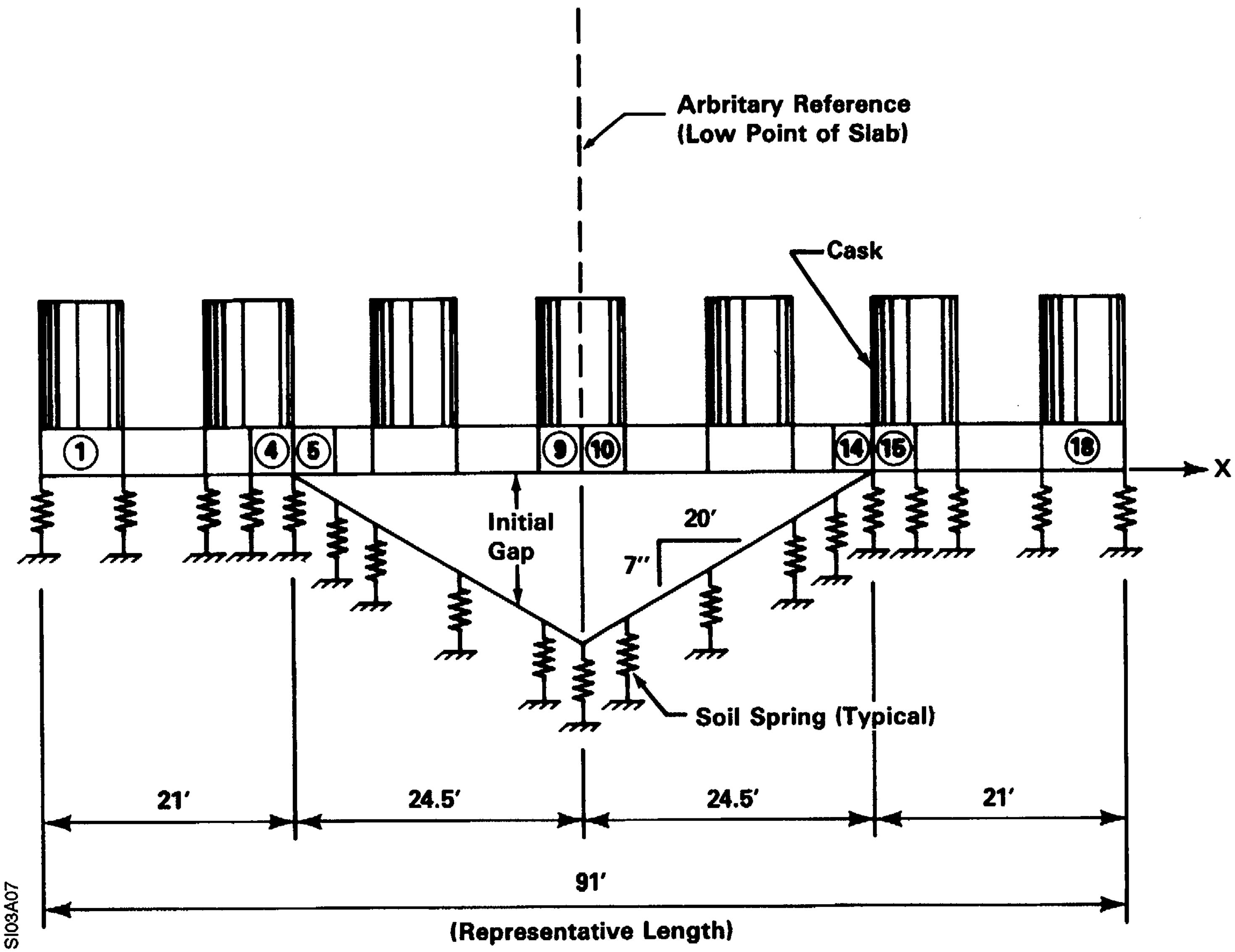


Figure 3A-6 (SHEET 2 OF 2)
ANSYS MODEL OF SLAB, CASKS AND SOIL SPRINGS



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