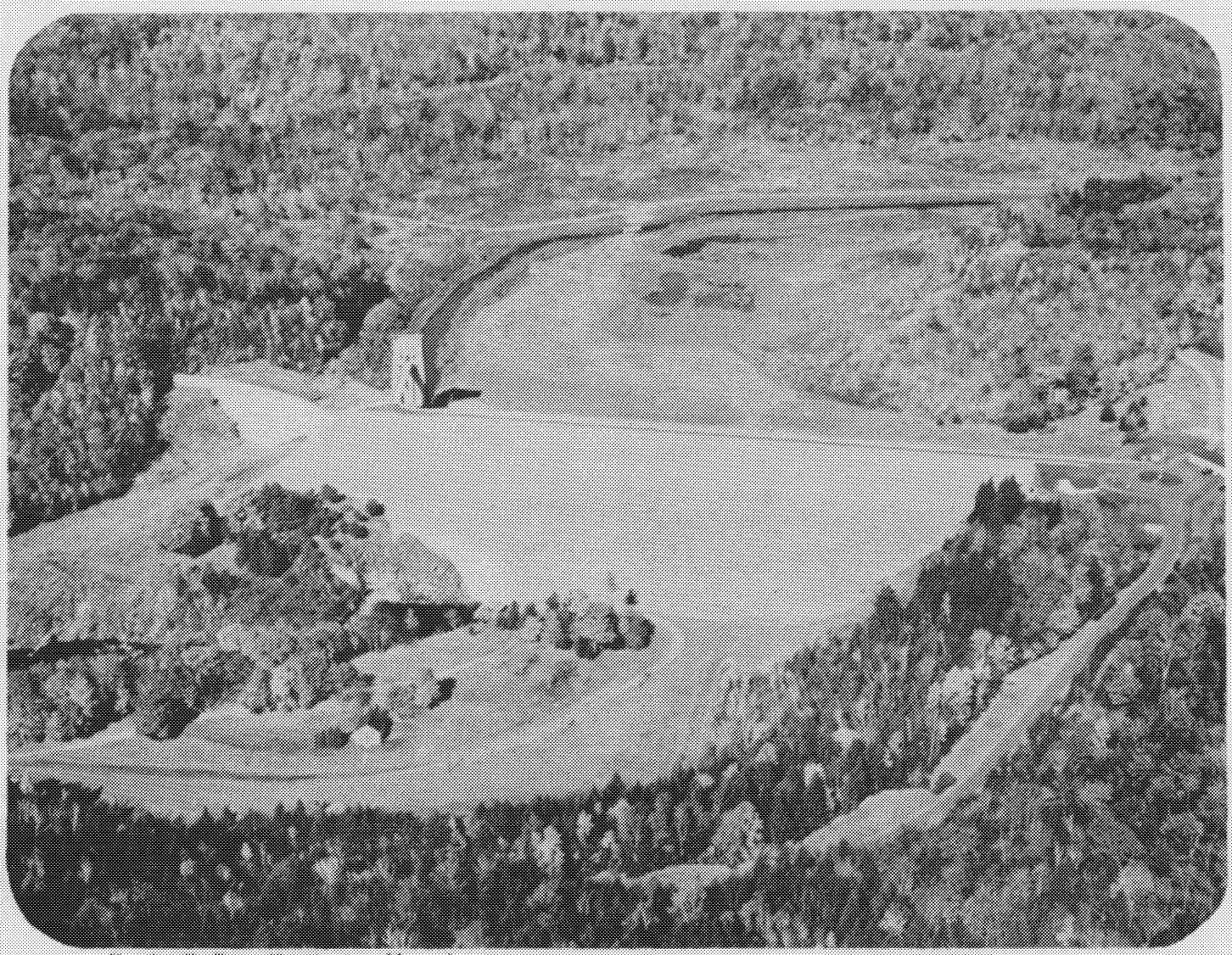
U.S. Army
Corps of Engineers
New England Division

DYNAMIC ANALYSIS OF KNIGHTVILLE DAM

APPENDIX TO FINAL REPORT

DECEMBER, 1982



Knightville Dam, Huntington, Massachusetts

Volume 2

DYNAMIC ANALYSIS OF KNIGHTVILLE DAM

APPENDIX TO FINAL REPORT

PREPARED FOR

U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

DECEMBER, 1982

VOLUME 2

APPENDIX TO FINAL REPORT

DYNAMIC ANALYSIS
OF KNIGHTVILLE DAM

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APPENDIX A

FIELD TESTING PROGRAM DATA

Sections

A1 - Subsurface Exploration Logs

A2 - Description of Undisturbed Sampling Procedures

A3 - Description of Inclinometer Casing Installation Procedures

A4 - Cross-hole, Down-hole, and Surface Wave Velocity Measurement Data

A5 - Typical Ocilloscope Photographs

Section A1

Subsurface Exploration Logs

Boring No.	Location	Sheets
K-1	Crest	5
K-2	Crest	5
K-3	Crest	5
K-4	Downstream Slope	2
K-5	Downstream Slope	3
K-6	Downstream Slope	3

DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM

HUNTINGTON, MASSACHUSETTS

LEGEND FOR SUBSURFACE EXPLORATION LOGS

SAMPLE SYMBOL	ABBRE-VIATION	DESCRIPTION
	S.S.	SPLIT SPOON SAMPLE (1-3/8" OR 3" O.D.)
	S.P.	2.8-INCH STATIONARY PISTON SAMPLE
	P.S.	2.8-INCH PITCHER-TYPE SAMPLE
	N.W.C.B.	NW-SIZE CORE BARREL
	psi	POUNDS/SQUARE INCH
	L.L.	LIQUID LIMIT
	P.L.	PLASTIC LIMIT
	N.P.	NONPLASTIC
	G_s	SPECIFIC GRAVITY
	WOR	WEIGHT OF RODS
		IMPERVIOUS ROLLED FILL
		HYDRAULIC CORE
		RIPRAP
		HYDRAULIC SHELL
		MICACEOUS SCHIST
		BITUMINOUS PAVEMENT AND/OR REFUSAL

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-1
SHEET 1 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+75.0, 3.2'L DATE 11/16 to 11/19/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Phillips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)	
0 (629.8)			BITUMINOUS PAVEMENT									
				Tri-Cone								
5	41-54-42 (66)	1	Brown to Gray-Brown, Very Dense Angular to Subrounded, SILTY SAND to SANDY SILT, with trace gravel (SM to ML)	2" S.S.								
				Tri-Cone								
10	9-10-11 (55)	2		2" S.S.	4"		14.0					
				Tri-Cone								
15	29-26-19 (22)	3		2" S.S.		None Recorded						
				Tri-Cone								
20	18-26-21 (66)	4		2" S.S.								
				Tri-Cone								
25	100*/2" (0)	5		2" S.S.	3"		13.6			NP	45	S#6 & S#7 Combined
	21-48-37 (72)	6										
	34-119-105 (72)	7					11.2					
	54-51-49-53 (63)	8										
30	44-52-37-47 (58)	9					15.4					
(597±)		10	Very Dense, Brown SILT & fine SAND				23.8					
35	17-23-21-27 (63)											

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-1
SHEET 2 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+75.0, 3.2' L DATE 11/16 to 11/19/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Phillips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
35				Tri-Cone							
	4-1-44 (67)		11 Dense Brown Sandy SILT (SM-ML)	2" S.S.			30.0				
(589±) 40				Tri-Cone							
	2-3-1-3 (100)		12	2" S.S.							
45			Very Loose to Loose, Brown SILT, some sand. (ML)	Tri-Cone							
	1-WOR-1-3 (100)		13	2" S.S.	3"	None Recorded					
50				Tri-Cone							
	2-1-1 (100)		14	2" S.S.						NP	82
55				Tri-Cone							
	4-3-4 (66)		15	2" S.S.							
60				Tri-Cone							
	2-1-1 (94)		16	2" S.S.							
(565±) 65				Tri-Cone							
	3-3-5 (100)		17	2" S.S.							
70											

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K - 1
SHEET 3 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+ 75.0, 3.2 L DATE 11/16 to 11/19/81
DRILL CME - 75 ANGLE Vertical REFERENCE ELEV 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling DRILLER J.C. Phillips INSPECTOR Albert DiBernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
70				Tri-Cone	3"	None Recorded					
	3-5-4 (100)		18	2" S.S.							
75				Tri-Cone							
	3-3-5 (100)		19 Loose to Medium Dense Brown SILT, some fine sand. (ML)	2" S.S.							
80				Tri-Cone							
	3-3-3 (94)		20	2" S.S.						NP	85
85				Tri-Cone							
	3-3-4 (100)		21	2" S.S.			39.0				
90				Tri-Cone							
	3-3-2 (78)		22	2" S.S.							
95				Tri-Cone							
	3-2-2 (66)		23	2" S.S.							
100				Tri-Cone							
105											

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-1
SHEET 4 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+75.0, 3.2 L DATE 11/16 to 11/19/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co DRILLER J.C. Phillips INSPECTOR Albert DiBernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
105											
	2-3-6 (100)		24	2½" S.S.							
110											
			Loose to Medium Dense, Brown SILT, some fine sand. (ML)	Tri-Cone							
115											
	5-5-6 (100)		25	2" S.S.					29	NP	
120					5"	None Recorded					
				Tri-Cone							
125											
	5-4-6 (100)		26	2" S.S.							
130				Tri-Cone							
	4-4-6 (94)		27	2" S.S.			40.7				
135				Tri-Cone							
	3-2-3 (83)		28	2" S.S.							
140											

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-1
SHEET 5 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+75.0, 3.2 L DATE 11/16 to 11/19/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Phillips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6" INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
140				Tri-Cone							
(487±)	7-4-9 (83)		29	2" S.S.			29.7				
145			Medium Dense Brown SANDY SILT (SM-ML)	Tri-Cone	3"	Recorded					
	9-7-9 (44)		30	2" S.S.		None					
150				Tri-Cone							
155 (474.3)	500psi (89)		1 Micaceous Schist	NW CB	OPEN						
160			Bottom of Hole Depth at 159'-1"								

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-2
SHEET 1 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+60.2, 3.0'L DATE 10/28 to 11/09/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Philips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)	
0 (629.8)			BITUMINOUS PAVEMENT									
	8-20-31 (44)		1 Very Dense Brown GRAVELLY SAND (SW)	2" S.S. / Tri-Cone								
5	36-54-39 (61)		2 Very Dense Brown SAND, some fine gravel, trace silt (SW)	2" S.S. / Tri-Cone	6"	None Recorded						
10	16-17-21 (55)		3 Medium Dense Brown CLAYEY SAND, trace fine gravel (SC)	2" S.S. / Tri-Cone			14.1					
15	15-22-19 (50)		4 Dense Gray SANDY CLAY (CL)	2" S.S. / Tri-Cone					20	2		S#4 & S#5 Combined
20	12-17-18 (55)		5 Medium Dense Gray SANDY CLAY, some gravel (CL)	2" S.S. / Tri-Cone	5"							
25	14-30-39 (50)		6 Very Dense Gray SANDY CLAY, some gravel (CL)	2" S.S. / Tri-Cone			11.5					
30	53-92-75 (50)		7 Very Dense Gray Brown GRAVEL, some sand, trace silt (GW)	2" S.S. / Tri-Cone								
(597±)												
35	19-21-		8 Dense Brown Silty	2"			24.1					

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K - 2
SHEET 2 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7 + 60.2, 3.0'L DATE 10/28 to 11/09/81
DRILL CME - 75 ANGLE Vertical REFERENCE ELEV. 629.8 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Philips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)	
35	19 (50)		fine SAND (SM)	S.S. Tri-Cone	5"	None Recorded						
40	3-4-3 (75)		9	2" S.S. Tri-								
	PUSH (78)		1	2.8" S.P.			31.0 26.6	84.7 90.3			50 65	G_s = 2.74
45				Tri-Cone								
	PUSH (53)		2	2.8" S.P.			36.1	85.0			65	
50				Tri-Cone								
	PUSH (63)		3	2.8" S.P.			30.3 31.8	95.0 90.7	24	2	71	G_s=2.81
55			Brown SANDY SILT to Brown SILT, some fine sand (SM-ML to ML)	Tri-Cone								
	PUSH (64)		4	2.8" S.P.			24.5 28.1	93.0 91.0	23	NP	51 55	
60				Tri-Cone								
	PUSH (89)		5	2.8" S.P.			31.0	90.6			40	
65				Tri-Cone								
	PUSH (55)		6	2.8" S.P.								
70												

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-2
SHEET 3 OF 5

SUBSURFACE EXPLORATION LOG

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LOCATION Huntington, Mass. COORDINATES Sta. 7+60.2, 3.0'L DATE 10/28 to 11/09/81
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SAMPLE HAMMER: WT. 140 lbs DROP 30 in. CASING HAMMER: WT 300 lbs DROP 30 in.

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)	
70				Tri-Cone	5"	None Recorded						
	PUSH (95)	7		2.8" S.P.			42.5 38.6 39.4	79.9 84.1 80.4	32	NP	71 85	G_s = 2.81
75				Tri-Cone								
	PUSH (90)	8		2.8" S.P.			37.7 35.7	84.4 87.5		NP	65 83	
80				Tri-Cone								
	PUSH (95)	9		2.8" S.P.			29.3 34.6	90.9 87.4			68 88	
85			Brown SANDY SILT to Brown SILT, some fine sand (SM-ML to ML)	Tri-Cone								
	PUSH (95)	10		2.8" S.P.			39.0 35.9	82.3 84.9			87 75	
90				Tri-Cone								
	PUSH (95)	11		2.8" S.P.			38.3 41.3 37.7	84.1 79.4 84.2	33	NP	82 91	G_s = 2.78
95				Tri-Cone								
	PUSH (95)	12		2.8" S.P.			37.7 38.9	84.7 83.3			91 91	G_s = 2.81
100				Tri-Cone								
105												

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-2
SHEET 4 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+60.2, 3.2' L DATE 10/28 to 11/09/81
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SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)	
105	PUSH (96)	13		2.8" S.P.			40.7 38.0	81.0 83.7			97 91	
110				Tri-Cone								
115	PUSH (88)	14		2.8" S.P.			32.7	84.6			92	G_s = 2.76
120			Brown SANDY SILT to Brown SILT, some fine sand (SM-ML to ML)	Tri-Cone	5"	None Recorded						
125	PUSH (93)	15		2.8" S.P.								
130				Tri-Cone								
135	PUSH (87)	16		2.8" S.P.			32.2 31.4 34.4	89.8 92.4 88.4	35	5	91 90	G_s=2.77
140												

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-2
SHEET 5 OF 5

SUBSURFACE EXPLORATION LOG

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DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS 200 (%)
140				Tri-Cone							
145	PUSH (94)	17	Brown SANDY SILT to Brown SILT, some fine sand (SM-ML to ML)	2.8" S.P.	5"	None Recorded	32	92	28	5	85
150				Tri-Cone							
155			Refusal		OPEN						
160			Bottom of Borehole at depth = 158'-3"								

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-3
SHEET 1 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+45.0, 2.8'L DATE 11/11 to 11/14/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.9 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Phillips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
0 (629.9)			BITUMINOUS PAVEMENT								
5				Tri-Cone	4"	Recorded					
10	26-10-12 (66)		1 Medium Dense Gray SAND and CLAY, trace fine gravel (SC)	2" S.S.							
15				Tri-Cone		None					
20	16-16-24 (55)		2 Medium Dense Gray SAND and CLAY, trace fine gravel (SC)	2" S.S.							
25				Tri-Cone	3"						
30	95-89 (100)		3 Very Dense SILTY SAND, some gravel (SM)	2" S.S.			14.7				
(597±)				Tri-Cone							
35											

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-3
SHEET 2 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7 + 45.0, 2.8'L DATE 11/11 to 11/14/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV 629.9 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J. C. Phillips INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS 200 (%)
35				Tri-Cone							
40	4-4-6 (100)	4	Loose to Medium Dense Brown SILT, some fine sand (SM)	2" S.S.			36.5				
45				Tri-Cone	5"	None Recorded					
50	2-1-3 (100)	5		2" S.S.							
55				Tri-Cone							
60	1-1-2 (100)	6		2" S.S.							
65				Tri-Cone							
70	4-6-8	7		2"							

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-3
SHEET 3 OF 5

SUBSURFACE EXPLORATION LOG

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DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
70	(89)			S.S.							
75				Tri-Cone							
80	3-1-3 (50)		8	2" S.S.							
			Loose Brown SILT some fine sand (ML)	Tri-Cone	5"	None Recorded					
85											
90	2-2-3 (61)		9	2" S.S.							
95				Tri-Cone							
100	3-1-3 (100)		10	2" S.S.							
				Tri-Cone							
105											

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-3
SHEET 4 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7 + 45.0, 2.8' L DATE 11/11 to 11/14/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV. 629.9 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co DRILLER J.C. Phillips INSPECTOR Albert DiBernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (psf)	LL (%)	PI (%)	MINUS #200 (%)
105				Tri-Cone							
110	3-WOH-8 (100)	11		2" S.S.							
115			Medium Dense Brown SILT, some fine sand (ML)	Tri-Cone	5"	None Recorded					
120	4-4-5 (89)	12		2" S.S.			39.4				
125				Tri-Cone							
130	10-12-13 (100)	13		2" S.S.			37.7				
135				Tri-Cone	OPEN						
140											

FORM NO. 150-3-60

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-3
SHEET 5 OF 5

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+45.0, 2.8' L DATE 11/11 to 11/14/81
DRILL CME-75 ANGLE Vertical REFERENCE ELEV 629.9 DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER J.C. Phillips INSPECTOR Albert DiBernardo
SAMPLE HAMMER: WT. 140 lbs DROP 30 in. CASING HAMMER: WT 300lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	DRY DENSITY (%)	LL (%)	PI (%)	MINUS #200 (%)
140											
145				Tri-Cone	OPEN						
150											
(475.4) 155			Refusal								
			Bottom of hole 156.5 ft.								

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K – 4
SHEET 1 OF 2

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Massachusetts COORDINATES Sta. 7+76.1; 180' L DATE 24 Nov to 03 Dec 1981
DRILL Joy ANGLE Vertical REFERENCE ELEV 571.8 ft. DATUM MSL
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert DiBernardo
SAMPLE HAMMER: WT. 300 lb DROP 18 inches CASING HAMMER: WT 300 lbs DROP 30 inches

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	PLUS #4 (%)	MINUS #200(%)
(571.8)			Dumped Riprap	Tri-Cone	4" Diameter	None Recorded			
5									
(564.1)	21-77-65-75 (66)		1	3" S.S.					
10	17-24-38 (50)		2	3" S.S.			8.4	23	12
	29-50-74-30 (50)		3	3" S.S.					
	14-25-33-37 (58)		4	3" S.S.					
15	11-12-14 (55)		5	3" S.S.			4.2	75	4
	14-14-14-17 (25)		6	3" S.S.					
			Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)	Tri-Cone					
20	48-99-47 (44)		7	3" S.S.					
				Tri-Cone					
25	34-14-12 (33)		8	3" S.S.			10.7	28	12
				Tri-Cone					
30	43-50-47 (72)		9	3" S.S.					
				Tri-Cone					
35	50-72-70-79 (58)		10	3" S.S.					

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K – 4
SHEET 2 OF 2

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+76,1; 180'L DATE 24 Nov to 03 Dec 81
DRILL Joy ANGLE Vertical REFERENCE ELEV 571.8 ft. DATUM MSL
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR A. DiBernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 inches CASING HAMMER: WT 300 lbs DROP 30 inches

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	PLUS # 4(%)	MINUS #200(%)
35				Tri-Cone	4" Diameter	None Recorded			
40	38-37 50-60 (58)		11	3" S.S.			7.4	41	10
				Tri-Cone					
	128-69-73-135 (50)		12	3" S.S.					
45			Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)	Tri-Cone					
	127-140-190-61 (42)		13	3" S.S.					
50				Tri-Cone					
	67-85-89-91 (71)		14	3" S.S.			7.8	31	11
				Tri-Cone					
55	79-91-128-137 (42)		15	3" S.S.					
	50-35-23-84 (71)		16	3" S.S.					
60				Tri-Cone					
	200-150/1" (100)		17	3" S.S.					
65				Tri-Cone					
(503.6)			Micaceous Schist		Open				
70			Bottom of Borehole (69 ft.)						

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K 5
SHEET 1 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta.7 + 59.6, 180' L DATE 11/11 to 11/23/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT(%)	PLUS #4 (%)	MINUS #200 (%)
(571.8)			Dumped Riprap						
(567.1) 5	10-17-28-35-(33)	1	Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)	3" S.S.	6" - Diameter	None Recorded			
				Tri-Cone					
	15-26 37-42 (96)	2		3" S.S.					
10	23-37-75-46 (66)	3							
	300psi (0)	1		NW-CB					
	14-20-20-26 (50)	4		3" S.S.					
15	20-25-26-20 (54)	5						30	10
				Tri-Cone					
	300psi (0)	2		NW-CB					
20	14-20-26-14 (58)	6		3" S.S.					
	14-16-17-15 (46)	7							
25	300psi (22)	1		2.8" P.S.					
				Tri					
	27-28-25-25 (54)	8		3" S.S.					
	22-39-38-44 (71)	9							
30	300 psi (0)	2		2.8" P.S.					
	300 psi (0)	3							
35		4							

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-5
SHEET 2 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+59.6, 180' L DATE 11/11 to 11/23/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT(%)	PLUS #4 (%)	MINUS #200 (%)	DRY UNIT WEIGHT (pcf)
35	300 psi (0)		4		6" Diameter	None Recorded				
	15-27-32-30 (54)		10	3" S.S.						
	17-30-32-40 (66)		11							
40	300 psi (0)		5							
	300 psi (62)		6	2.8" P.S.						
45	300 psi (0)		7							
	15-30-37-42 (83)		12	3" S.S.				45	9	
	27-33-31-47 (79)		13							
50	30-34-42-50 (66)		14							
	300 psi (8)		8	2.8" P.S.						
55	30-48-50-53 (71)		15	3" S.S.						
	30-75-82-91 (83)		16							
	60-45-39-76 (71)		17					30	12	
60	300 psi (62)		9	2.8" P.S.			8.4	0	50	143.8
	300 psi (62)		10							
65	30-34-36-81 (33)		18	3" S.S.						
(505.1)	500 psi (100)		3	NW-CB						
70										

Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)

Micaceous Schist

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-5
SHEET 3 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta.7+59.6, 180'L DATE 11/11 to 11/23/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT(%)	PLUS #4 (%)	MINUS #200 (%)		
70				Tri-Cone	6" Open						
(498.3)			Bottom of Borehole at Depth = 73.5'								
75											

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-6
SHEET 1 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta. 7+43.6; 180' L DATE 11/02 to 11/10/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT (%)	PLUS #4 (%)	MINUS #200 (%)
(571.8)			Dumped Riprap						
5									
(563.8)	23-67-72 (80)	1	Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)	2" S.S.	6" Diameter	None Recorded			
	30-24-100 (50)	2							
10				Tri-Cone					
	31-40-47 (61)	3		2" S.S.					
15				Tri-Cone					
	300 psi (10)	1		2.8" P.S.					
20	9-20-21 (0)	4		2" S.S.					
				Tri-Cone					
	24-26-22-30 (46)	5		2" S.S.					
25				Tri Cone					
	30-28-48-126 (50)	6		2" S.S.					
30				Tri-Cone					
	13-42-50-47 (33)	7		2" S.S.					
35									

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-6
SHEET 2 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta.7+43.6; 180'L DATE 11/02 to 11/10/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING	WATER CONTENT(%)	PLUS #4 (%)	MINUS #200 (%)	DRY UNIT WEIGHT (pcf)
35				Tri-Cone	6" - Diameter	None Recorded				
	47-60-100/2.5 (40)		8	2" S.S.						
40				Tri-Cone						
	275 psi (58)		2	2.8" P.S.						138.4
45			Brown to Gray-Brown, Dense to Very Dense, Angular to Sub-rounded GRAVELLY SAND to SANDY GRAVEL with numerous Cobbles, trace to some silt (SW to GW)	Tri-Cone						
	300 psi (10)		3	2.8" P.S.						
50				Tri-Cone						
	59-60-63 (56)		9	2" S.S.						
55				Tri-Cone						
	300 psi (29)		4	2.8" P.S.						
60				Tri-Cone						
	26-31-49 (78)		10	3" S.S.						
65				Tri-Cone						
(504.1)										
	500 psi (38)		1	NW-CB						
70			Micaceous Schist							

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS

BORING NO. K-6
SHEET 3 OF 3

SUBSURFACE EXPLORATION LOG

JOB NO. 1608 JOB TITLE Dynamic Stability Analysis of Knightville Dam
LOCATION Huntington, Mass. COORDINATES Sta.7+43.6; 180' L DATE 11/02 to 11/10/81
DRILL Joy ANGLE Vertical REFERENCE ELEV. 571.8 Ft. DATUM NGVD
DRILLING CONTRACTOR Guild Drilling Co. DRILLER John Texeira INSPECTOR Albert Di Bernardo
SAMPLE HAMMER: WT. 300 lbs DROP 18 in CASING HAMMER: WT 300 lbs DROP 30 in

DEPTH IN FEET (ELEVATION)	BLOWS ON SAMPLER FOR 6 INCHES (% RECOVERY)	SYMBOL	SAMPLE NUMBER AND DESCRIPTION OF MATERIAL	SAMPLER AND BIT	CASING TYPE	BLOWS/FOOT ON CASING
70						
	500 psi (61)		2	NW-CB	OPEN	
75						
(494.8)			Bottom of Borehole at Depth = 77 ft.			
80						

Section A2

Description of Undisturbed Sampling Procedures

UNDISTURBED STATIONARY PISTON SAMPLING

General. Samples were taken at various selected elevations to produce undisturbed specimens 24 inches long and 2.8 inches in diameter, which are suitable for laboratory examination and testing.

Sampler Description. The stationary piston sampler consisted of a head, thin-wall tube, and close fitting piston operated by a separate rod. The head was fitted with a spring piston rod check and a lefthand threaded connection to the piston rod and had an "A" rod connection to the drill rod. The thin wall tubes were teflon coated on the inside to minimize sample disturbance and to resist corrosion.

Sampling Procedure. Before each sample was taken, the casing or borehole was carefully cleaned out with a deflected jet or clean out auger. The inside of the sampler tube was thoroughly cleaned prior to taking the sample. The sampler was pushed into the soil a distance of twenty four (24) inches at a rate ranging between three (3) and six (6) inches per second by hydraulic pressure. The samples were removed from the borehole and prepared for storage and transport by TAMS' field representative. The sample from the shoe was extruded into a glass jar. The glass jars were capped and preserved for laboratory testing.

Section A3

Description of Inclinometer Casing Installation Procedures

Installation. Cased holes having a minimum I.D. of 5.0 inches were required for the installation of the slope inclinometer casing. The borings were advanced uncased using drilling mud, and a 5-inch-diameter casing was installed in the hole prior to installing the inclinometer casing. The installation of the inclinometer casing was as follows:

(1) After the drill casing had been driven to the final elevation of the bottom of the boring, the hole was cleaned out and flushed with clear water until clean of all cuttings.

(2) A six (6) to twelve (12) inch layer of pea gravel was placed at the bottom of the borehole to serve as a base for the inclinometer casing.

(3) The bottom of the first piece of inclinometer casing was plugged with a plastic bottom plug, glued and riveted in place according to the manufacturer's instructions. A non-telescoping coupling was installed on the top end of the inclinometer casing in the same manner. Any holes which may have been left in the casing were taped over to prevent grout from entering the casing. One piece of 0.5 inch I.D. PVC tubing pipe, of a length equal to at least the depth of the borehole was taped to the outside of the inclinometer casing and about six (6) inches from the bottom. This PVC tubing was plugged at the bottom and had holes cut through its wall as near to the bottom as possible such that the total area of all the holes combined was greater than the inside area of the tubing.

(4) The casing was lowered into the borehole and filled with clean water to prevent flotation. Additional lengths of inclinometer casing were added, using an aligning tool during the installation of each coupling of the inclinometer casing to assure that the grooves on the inside of the

casing were aligned properly. One pair of grooves was aligned in the direction of the linear array of borings and the second pair of grooves perpendicular to the boring array. This continued until the bottom of the hole was reached. The inclinometer casing was kept filled with water during the entire procedure and drill rods were added for weighting the casing as required to keep the casing firmly resting on the bottom of the hole.

(5) The top piece of the inclinometer casing was cut off so that the top was about six (6) inches above the ground surface. The top of this casing was temporarilly plugged so that no debris or grout entered during the subsequent procedures.

Grouting. Grout pumps having a capacity of not less than 40 gpm at a discharge pressure of 25 psi were used.

A grout mixer of the venturi type was used to obtain a thoroughly mixed, uniform highly plastic grout mixture. Precautions were taken to avoid clogging of the pump during the pumping of grout down the hole.

The grout mix consisted of Portland cement, pulverized bentonite, and water in proportions to produce an acceptable mixture. The water/cement ratio was varied, generally ranging between 7:1 and 12:1. All grout contained one to three pounds of pulverized bentonite per 94-lb sack of cement. The method of adding and mixing bentonite resulted in a uniform and thorough dispersion of the grout. A smooth slurry was first prepared by mixing about one pound of bentonite per sack of cement with the water, and then adding the cement. The remainder of the bentonite, if any, was added after the cement.

The annular space between the boring casing and inclinometer casing was grouted from the bottom of the borehole in increments of 10 feet. The procedure was as follows:

(1) Enough grout was mixed to fill the annular space between the inclinometer casing and the wall of the hole for a length of 15 feet assuming the five (5) inch casing was removed from the hole.

(2) The 5-inch-diameter casing was carefully pulled out 10 feet; the inclinometer casing was kept in place.

(3) The grout was pumped down the ½-inch-diameter PVC tubing in one continuous operation under a pressure of not more than 45 psig. Too high a pressure could have caused the pipe to move up about the surface of the grout. Within 20 feet of the ground surface, gauge pressures were reduced sufficiently ensuring that the grout continued to flow.

(4) The grout tube was pulled free from the tape holding it to the plastic casing and raised 5 feet.

(5) Steps 1 to 4 were repeated except that a volume of grout adequate for 10 feet of the annular space was supplied and the grout tube raised in 10-ft increments.

(6) After the final increment of 5-inch casing had been removed, grout was pumped into the hole until it appeared at the ground surface. Pumping continued until the grout tube was fully withdrawn.

(7) The inclinometer casing was washed with clean water using the drill rods that had been placed inside the casing for weighting purposes.

(8) The drill rods were left in place for sufficient time to allow the grout to cure and prevent flotation of the inclinometer casing.

(9) At some borehole locations, a 2.5-foot length of four inch schedule 90 PVC pipe with one end threaded to receive a side vented cap was installed over the casing for its protection.

(10) Upon completion of the work, the inside of the inclinometer casing at each borehole location was completely filled with grout from the bottom of the hole. After grouting operations, the inclinometer casing and PVC pipe were cut about 3 inches below the ground surface. The area around the top of the borehole was then restored to its original condition.

SECTION A4

CROSS-HOLE, DOWN-HOLE AND SURFACE WAVE VELOCITY MEASUREMENT DATA

Table No.	Title
A4(a)	In-situ Shear and Compression Wave Velocities from Cross-hole Tests; Crest Borings K-2 and K-3
A4(b)	In-situ Shear and Compression Wave Velocities from Cross-hole Tests; Downstream Slope Borings K-5 and K-6
A4(c)	In-situ Shear Wave Travel Times, Down-hole Tests; Crest Boring K-3
A4(d)	In-situ Shear and Compression Wave Travel Times, Down-hole Tests
A4(e)	In-situ Rayleigh (Surface) Wave Velocities on Crest and Downstream Slope

TABLE A4 (a)

IN-SITU SHEAR AND COMPRESSION WAVE VELOCITIES FROM CROSS-HOLE TESTS

CREST BORINGS K-2 AND K-3

DEPTH* ft.	TRAVEL PATH LENGTH ft.	TRAVEL TIME (msec) S-Wave	TRAVEL TIME (msec) P-Wave	WAVE VELOCITY (fps) S-Wave	WAVE VELOCITY (fps) P-Wave
5	15.1	19.0	11.0	800	1380
9	15.0	19.0	---	790	---
14	14.9	14.5	---	1030	---
19	14.8	14.5	5.0	1020	2960
24	14.7	12.5	---	1170	---
29	14.6	13.0	6.5	1120	2240
33	14.4	12.0	5.6	1200	2580
37.5	14.3	16.5	7.5	870	1910
42.5	14.2	19.5	10.0	730	1420
47.5	14.2	19.5	9.5	730	1490
59	14.2	18.0	10.5	790	1350
69	14.0	17.0	5.5	820	2540
79	13.9	16.0	3.0	870	4620
89	13.8	16.0	2.8	870	4940
99	13.8	16.0	2.8	860	4930
109	13.7	14.0	2.4	980	5710
119	13.7	15.0	2.8	920	4900
129	13.6	13.5	2.6	1010	5240
134	13.6	12.5	2.6	1080	5210
139	13.5	12.5	2.6	1080	5200
149	13.6	13.5	2.4	1010	5660
155	13.6	4.5	2.4	3050	5660
155-159	13.4	5.0	2.4	2680	5580

--- The travel time for P-waves could not be reliably estimated from data.

TABLE A4(b)

IN-SITU SHEAR AND COMPRESSION WAVE VELOCITIES FROM CROSS-HOLE TESTS

DOWNSTREAM SLOPE BORINGS K-5 AND K-6

DEPTH* ft.	TRAVEL PATH LENGTH ft.	TRAVEL TIME (msec)		WAVE VELOCITY (fps)	
		S-Wave	P-Wave	S-Wave	P-Wave
13.5	15.7	12.5	10.5	1260	1500
19.0	15.7	15.5	10.5	1010	1500
20.5	15.7	14.0	11.5	1120	1370
25.5	15.6	13.5	10.0	1160	1560
29.5	15.6	9.0	9.0	1730	1730
33.5	15.5	10.0	9.0	1550	1720
37.5	15.5	14.0	9.0	1110	1720
43.0	15.4	11.0	7.0	1400	2200
47.0	15.3	12.5	7.5	1220	2040
51.0	15.2	12.5	7.5	1220	2030
55.0	15.2	12.0	7.5	1270	2030
61.0	15.1	10.5	7.5	1440	2010
65.5	15.1	10.0	---	1510	----
70.3	15.0	3.0	---	5000	----

* As measured from El 574.1 ft (top of platform)

TABLE A4-(c)

IN-SITU SHEAR WAVE TRAVEL TIMES
DOWN-HOLE TESTS

CREST BORING K-3

DEPTH* ft.	TRAVEL PATH LENGTH** ft.	TRAVEL TIME S-WAVES (msec)
30	31.1	25.5
40	40.3	38.0
50	50.0	50.0
60	59.6	62.5
70	69.3	75.5
80	79.2	87
90	89.2	98
100	98.9	110
110	108.8	123
120	118.8	130

* As measured from El 629.8 (NGVD)

** Horizontal Distance from concrete block to K-3 = 13.5' (Block embedded 2')

- The data at depths not listed above were unreliable.

- The travel time for P-waves could not be reliably estimated from the data.

TABLE A4(d)

IN-SITU SHEAR AND COMPRESSION WAVE TRAVEL TIMES - DOWN-HOLE TESTS

SLOPE BORING K-6

DEPTH* ft.	TRAVEL PATH LENGTH** ft.	TRAVEL TIMES (msec) S-Wave	P-Wave
10.0	17.4	---	12.5
15.0	18.7	20.5	13.0
20.0	21.1	26.5	15.5
25.0	24.3	29.7	17.2
30.0	28.0	32.7	17.5
34.8	31.9	36.0	18.7
40.0	36.4	39.5	21.7
44.9	40.8	---	22.0
50.0	45.4	---	23.5
55.0	50.1	---	25.5
60.2	55.0	---	27.0
64.8	59.4	---	28.7
70.3	64.7	---	30.0

* As measured from El 574.1 ft (NGVD)

** Depth from Platform to Bottom of Concrete Block = 8 ft.
Horizontal Distance from Block to K-6 = 17.3 ft.

--- The travel time for S-waves could not be reliably estimated from data.

TABLE A4(e)

IN-SITU RAYLEIGH (SURFACE)
WAVE VELOCITIES ON CREST AND SLOPE

Location	Travel Path Length (ft)	Travel Time (msec)	Wave Velocity (fps)
Crest	29.6	37	800
Slope	17.3	19	910

Section A5

Typical Oscilloscope Photographs

Photo No.*	Seismic Wave Shown**	Boring No.	Depth (ft)	Page
7D	S	K-3	20	A5-1
21D	S,P	K-6	40	A5-2
28D	S,P	K-6	25	A5-3
35X	S	K-2, 3	99	A5-4
36X	P	K-2, 3	99	A5-5

* D = Down-hole; X = Cross-hole

** S = Shear Wave; P = Compression Wave

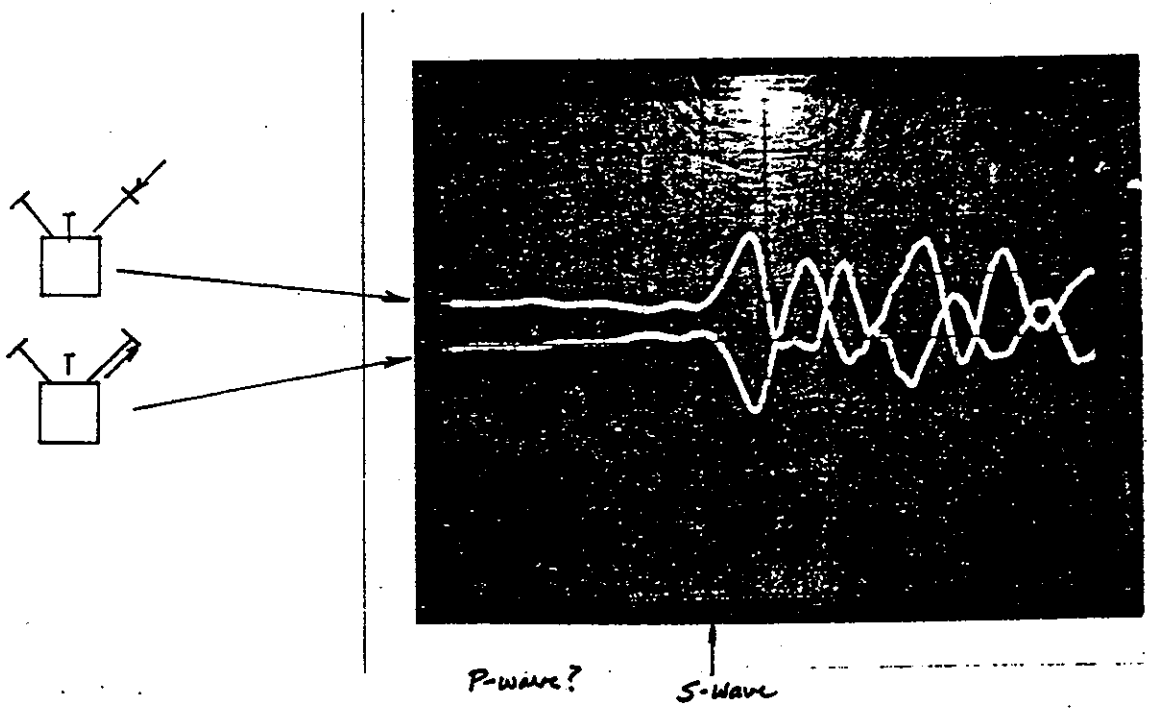
PHOTO NO. 7D

Downhole Method Crest Location

Boring K-3

Depth 20' Travel Distance 22.5'*

Vertical Setting (mV/div)	Geophone Orientation
2	Hor



Sweep rate 5 msec/div

*Horizontal distance from K-3 to edge of block = 13.5 ft
Block embedded 2 ft

Photo No. 21D

Downhole method - slope location (K-6)

Depth (below platform) = 40.0 ft Travel distance = 36.4 ft

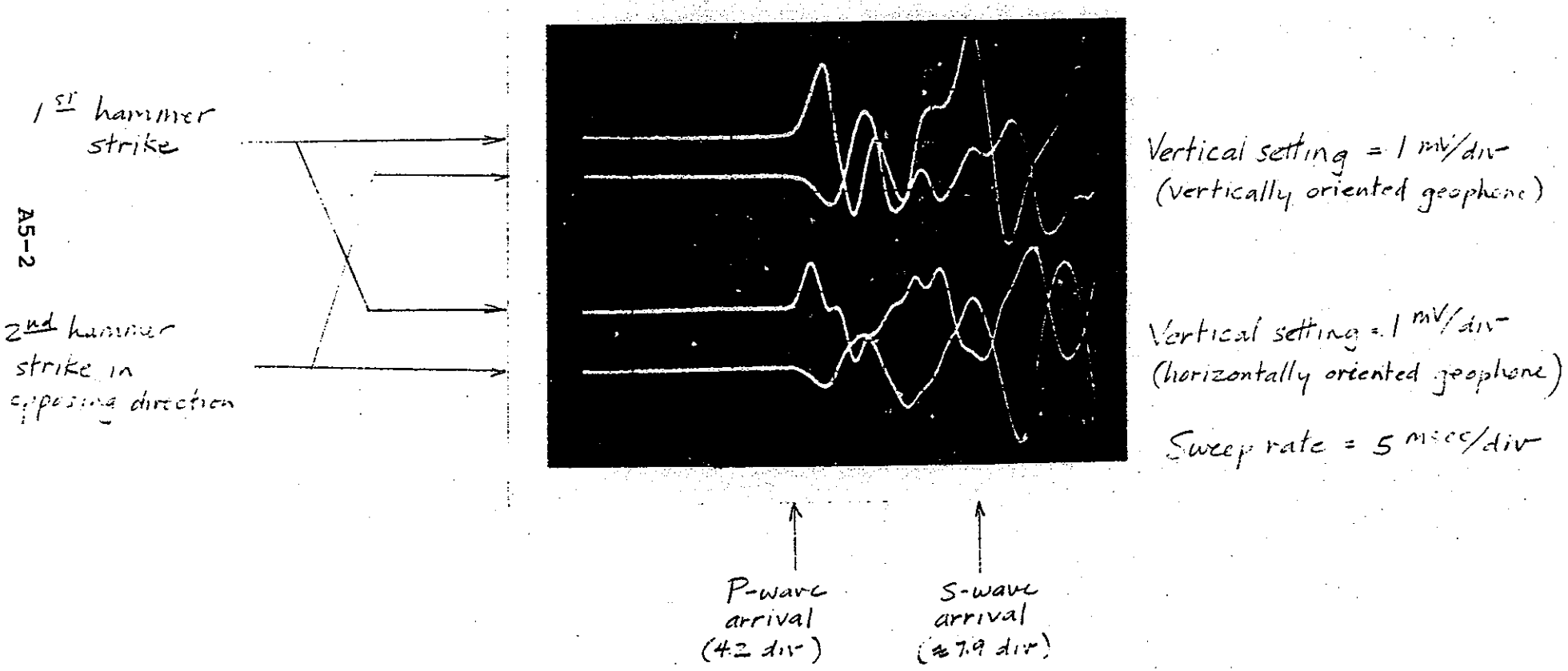


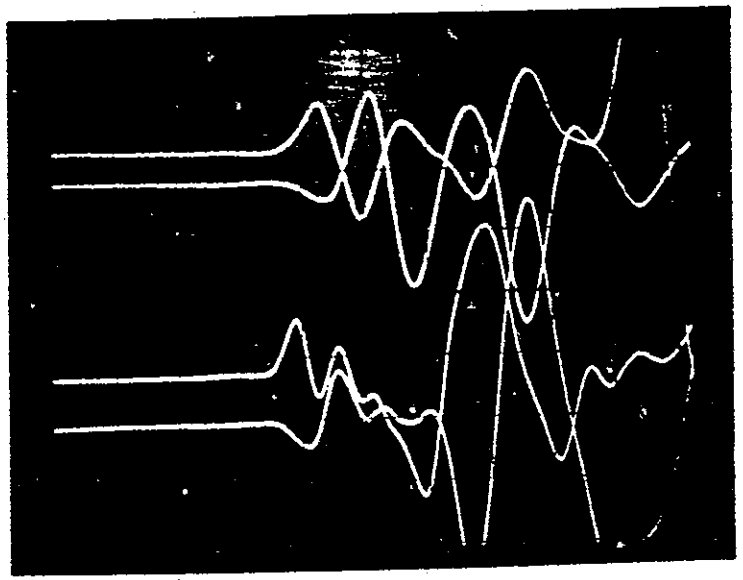
Photo No. 28D

Downhole method - slope location (K-6)

Depth (below platform) = 25.0 ft Travel distance = 24.2 ft

1st hammer strike

2nd hammer strike in opposing direction



Vertical setting = 2 mV/div (vertically oriented geophone)

Vertical setting = 2 mV/div (horizontally oriented geophone

Sweep rate = 5 msec/div

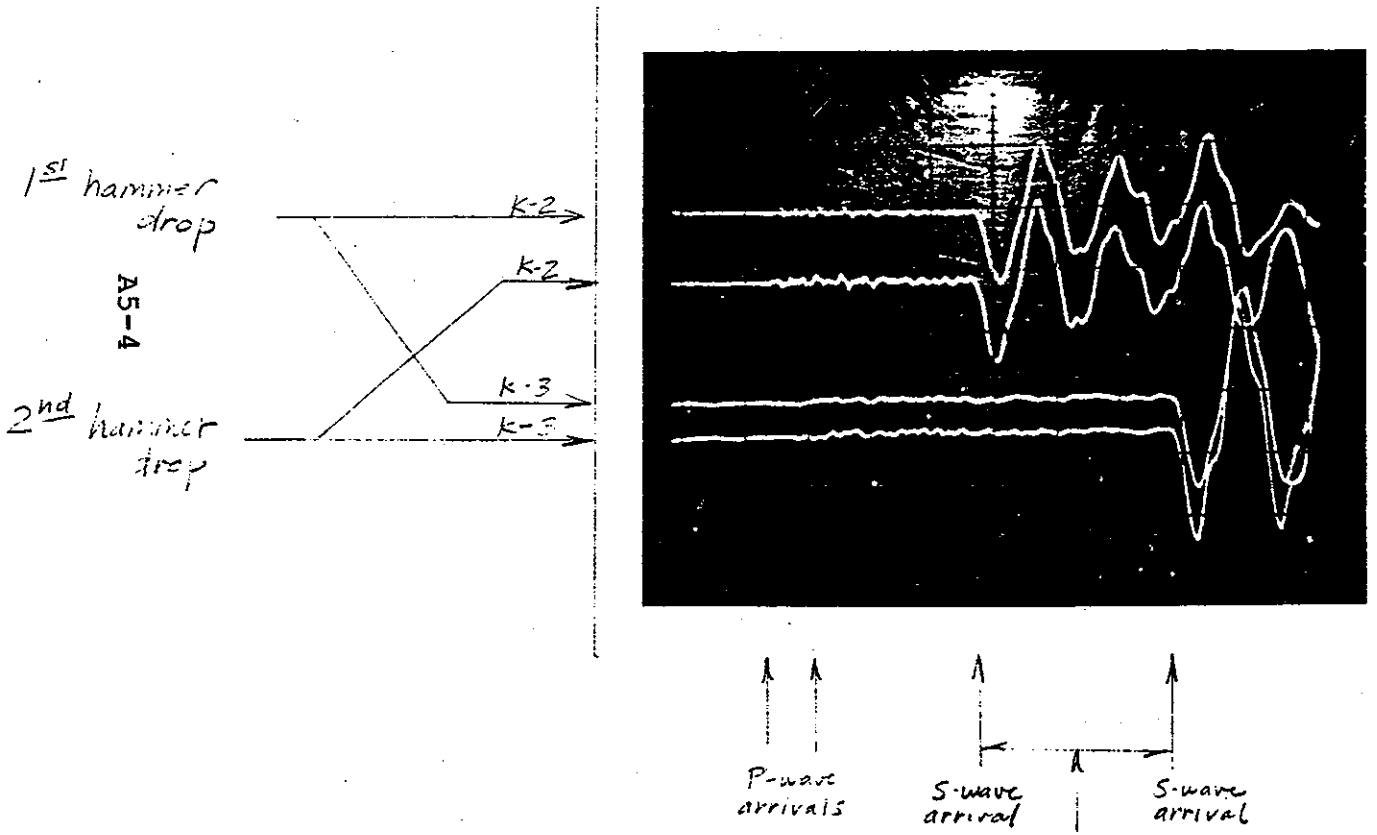
P-wave arrival (3.5 div)

S-wave arrival (6.0 div)

Photo No. 35 x

Crosshole method - Crest location (K-2 ↔ K-3)

Depth = 99 ft Travel distance = 13.8 ft



Vertical setting (K-2) = 10 mV/div

Vertical setting (K-3) = 5 mV/div

Sweep rate = 5 msec/div

3.2 div S-wave delay time (in div.)

× 5 msec/div sweep rate

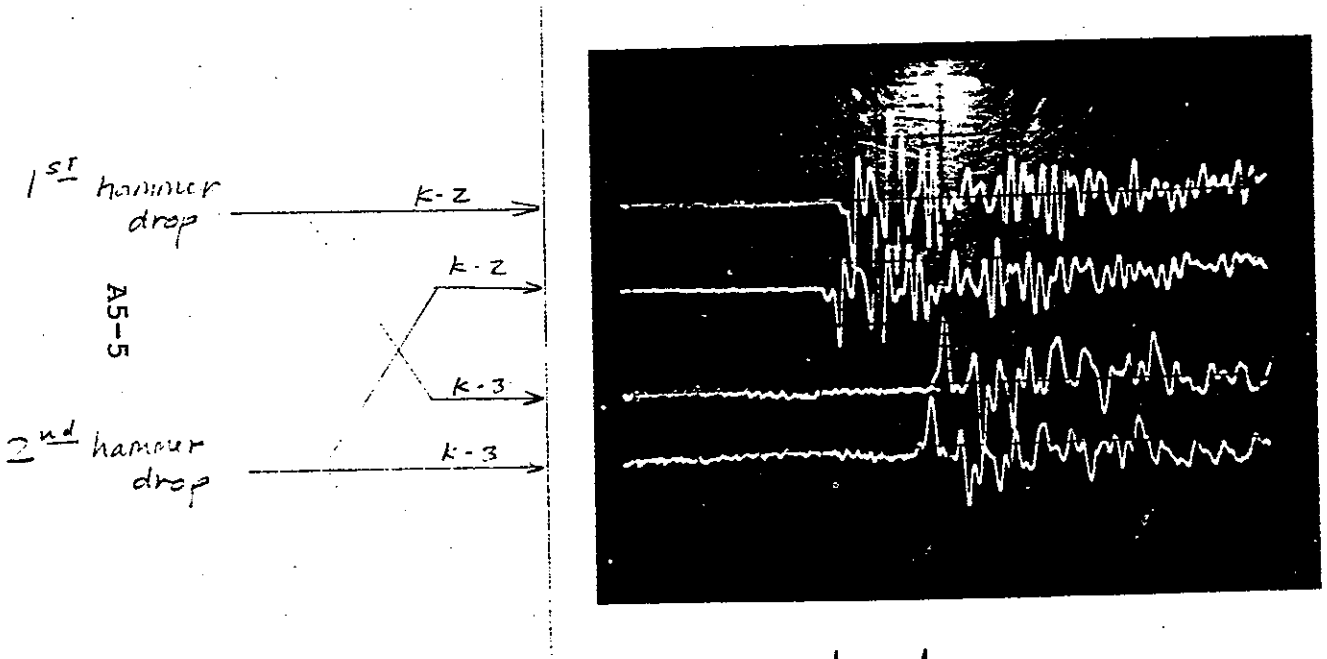
16.0 msec S-wave delay time (in msec)

$$V_s = \frac{13.8 \text{ ft}}{16.0 \text{ msec}} = 860 \text{ fps}$$

Photo No. 36x

Crosshole method - Crest location (K-2 ↔ K-3)

Depth = 99 ft Travel distance = 13.8 ft



Vertical setting (K-2) = 1 mV/div

Vertical setting (K-3) = 0.5 mV/div

Sweep rate = 2 msec/div

1.4 div P-wave delay time (div)

× 2 msec/div sweep rate

2.8 msec P-wave delay time (msec)

$$V_p = \frac{13.8 \text{ ft}}{2.8 \text{ msec}} = 4930 \text{ fps}$$

APPENDIX B

SEISMIC HAZARD ANALYSIS OF KNIGHTVILLE DAM REPORT

SEISMIC HAZARD ANALYSIS OF THE KNIGHTVILLE DAM

Huntington, MA

by

M. Nafi Toksoz

June 10, 1982

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INTRODUCTION

This report is concerned with the estimation of the earthquake hazards in the vicinity of the Knightville Dam at Huntington, MA. The estimation of the seismic hazard in any area of the northeastern U.S. is a significant problem because of the lack of instrumental data over a long period of time and our inability to define the geologic sources of the earthquakes, such as specific faults or fault zones. However, the earthquake history of this area spans almost 400 years, and the northeastern U.S. has the most complete record of earthquake activity of any area in the U.S. The vast majority of the earthquakes are small (Int. < V, M.M.). However, moderate to large events have occasionally occurred in this area, such as 1755 Cape Ann, MA, and 1940 Ossipee, NH. The task we undertake is to determine, on the basis of complete seismological analysis, the value of the peak ground acceleration experienced at the Knightville Dam site in the past, and the peak ground acceleration at the site due to the Maximum Credible Earthquake.

Throughout this report we will refer to the Knightville Dam "STUDY AREA". This STUDY AREA is defined in Figure 1 and includes the region between latitudes 40.5 and 44.0 degrees, and longitudes -75.0 and -70.0 degrees. A 161 km (100 mile) radius circle is also shown in Figure 1 for reference. The STUDY AREA extends beyond the 161 km (100 mile) radius in order to include some of the largest historic events that fall

just beyond the circle. These events contribute significantly to the earthquake hazard at the dam site.

We will also refer to both the historic and instrumental seismicity of the STUDY AREA. The definition of the time periods covered in each category deserves mention at this time. The historic seismicity begins with the first documented occurrence of an earthquake in this area, which is in 1568 and ends in October 1975. Starting in October 1975, the Weston Observatory (Weston Observatory, 1975-1981) began publishing the Bulletins of Seismicity for the Northeastern U.S. These bulletins are complete to March 1981 at this time. The bulletins tabulate the instrumentally determined earthquake hypocenters and magnitudes in this area. The locations are based on arrival time data from a network of nearly 100 stations in the northeastern U.S. Thus, the instrumental seismicity of the STUDY AREA covers the time period October 1975 through March 1981.

In the body of this report, we first discuss the regional seismicity and examine the seismicity of the STUDY AREA in detail. Next, the earthquake statistics will be tabulated and recurrence relations presented. Then we calculate the peak ground acceleration on average bedrock (igneous and metamorphic rocks) at the dam site due to both actual earthquakes and the maximum credible earthquakes in the STUDY AREA. In choosing the appropriate attenuation curves, we analysed the strong motion data from the Gaza, NH earthquake (m_b=4.7) of January 19, 1982, and these results are included in this study.

SEISMICITY

Regional Seismicity

The Knightville Dam is located in western Massachusetts in a relatively stable seismic region. The earthquake activity in Massachusetts, and the surrounding areas of the northeastern U.S. and eastern Canada, has been studied on the basis of both historical and instrumental data. The historic data goes back almost 400 years, the longest of any region in the U.S. A large number of small and an appreciable number of significant earthquakes with damage potential have been reported and tabulated in seismicity catalogs (Smith, 1962, 1966; Coffman and Von Hake, 1973). A map of historic earthquake epicenters covering the period 1534 to 1959 is shown in Figure 2. A list of all historic earthquakes, based on updated catalogs (Chiburis, 1981) for the STUDY AREA is given in Appendix A. A number of distinct areas of seismic activity stand out in Figure 2. These areas are the lower St. Lawrence River near La Malbaie, Quebec, the Adirondacks and western Quebec, southern New Hampshire to eastern Massachusetts, greater New York City and northern New Jersey, and south central Connecticut. There is a paucity of earthquake epicenters in western Massachusetts, the region surrounding the Knightville Dam area.

Historic data, although very important, cannot resolve the seismic characterization of a region. First, there is the question of the completeness of the data set. Since the

occurrence of most of these events is based on felt reports, the distribution of epicenters is greatly affected by the centers of population. Second, the epicenters, magnitudes and focal depths are estimated from intensity data and are subject to some uncertainties. Third, data will be biased toward larger events, since smaller events will not be felt as widely. Thus it is important to complement these historic data with instrumental network data.

Since 1975 a network of more than 100 seismic stations has been installed and operating in the northeastern U.S. As shown in Figure 3, this network provides an excellent coverage of the region for effective monitoring of earthquake activity. Note that there are several stations, all within a distance of about 70 km (43 miles) surrounding the dam site. The locations, distances from the Knightville Dam, and operating histories of these stations are listed in Table 1.

An earthquake epicenter map, based on the northeastern U.S. seismic network data, has been prepared for the period of October 1975 through March 1981, and is shown in Figure 4. The list of events occurring within the STUDY AREA with epicenter and magnitude information is tabulated in Appendix B.

Comparing the map of historical data (Figure 2) with that of the instrumental data (Figure 4), we note that in general the historical and instrumental seismicity data agree quite well. In some areas, such as Maine and parts of Canada where the population density is very low, the instrumental data

indicate greater seismic activity. A few areas that were active during historical times show less activity in the 1975-1981 time period. One important example is the Cape Ann, MA area, site of the November 18, 1755 earthquake (intensity VIII) which was the largest historic earthquake in the Massachusetts area. The present-day instrumental record does not show a great concentration of events in this region. This example illustrates the importance of a synthesis of both the historical and the current instrumental data in the determination of seismic risk.

Another observation that can be made from the instrumental data is the fact that epicenters, although accurately located, scatter and define regional patterns of seismicity rather than well-defined active fault zones. This is in contrast to the seismicity pattern in the Western U.S. Neither in the Knightville Dam region nor in the other parts of New England can we associate the earthquakes with well-defined active faults. Thus we cannot use the fault maps as the primary indicators of seismic risk.

In defining the "epicentral area" of the largest earthquakes we use the regional clustering of earthquakes. Although the seismotectonic implications of the clustering are not resolved, these clusters are defined on the basis of historic and recent instrumental data. We will investigate the sensitivity of acceleration to the radius of the epicentral areas by moving the largest earthquakes around the areas and calculating the resulting ground motions at the

dam site.

In Figure 5 we plot all regional epicenter data for the period 1534 through 1980. In this map, just as seen in Figures 2 and 4, western Massachusetts and the Knightville Dam site are seismically less active than the surrounding regions such as eastern Massachusetts, central New Hampshire, southern Connecticut, and the New York area. Thus in the evaluation of the earthquake risk, it is important to consider the contribution of these surrounding clusters as well as close-in regions.

Local Seismicity

For a detailed analysis of seismicity and the induced ground motion at the Knightville Dam site, we will include the events in the STUDY AREA shown in Figure 1, even though this includes earthquakes outside the region of 161 km (100 mile) radius around the dam. Our calculations of ground motions (site intensity or peak ground acceleration) at the dam site will show that some earthquakes outside the 161 km radius can contribute significantly to the ground motion at the dam site. Historic earthquake epicenters for the period 1568 to 1975 in the STUDY AREA are shown in Figure 6. Had we only considered the area inside the 161 km radius we would have ignored some of the seismically active zones of central New Hampshire, northeastern Massachusetts and New York. Furthermore, the largest historic earthquakes such as 1755 Cape Ann, MA, 1940 Ossipee, NH, 1931 Lake George, NY, and 1884 New York, NY, fall just outside the 161 km radius. For these reasons we defined the STUDY AREA as the whole region shown in Figure 6.

The instrumental seismicity in the STUDY AREA for the period October 1975 to March 1981 is shown in Figure 7. Again these events justify going outside the 161 km radius for estimating the ground motions at the dam site.

In looking at Figure 6, we observe a clustering of some events fairly close to the dam site. These events, although small, are important because of their proximity to the Knightville Dam. Thus we considered these, and all other events that fall within a radius of 50 km (31 miles) of the dam for detailed study. These events, their locations and epicentral intensities are listed in Table 2.

Finally, in Figure 8 we plot the locations of all known "significant" earthquakes in the STUDY AREA. A "significant" earthquake is defined as an event of at least intensity VII or magnitude 5.0 m_b. Epicentral data for these earthquakes are given in Table 3.

In the following sections we will calculate the frequency-magnitude statistics for events in the STUDY AREA, and then calculate the peak ground accelerations at the dam site.

EARTHQUAKE FREQUENCY AND RECURRENCE RELATIONS

To estimate the frequency of occurrence and return times for earthquakes in the STUDY AREA, we will use the relation

$$\log_{10} (N/yr) = A - B (M,I) \qquad (1)$$

where M is the magnitude, I is the intensity, N is the number of events in a catalog with magnitude (or intensity) greater

than or equal to M (or I), yr is the number of years covered in the catalog, and A and B are the constants to be determined. Normally the number of events, N, is for a fixed period for which catalogs or data are analyzed. We have applied this relationship to the historical and instrumental seismicity datasets for the Knightville Dam STUDY AREA.

Figure 9 shows the results of the $\log_{10}(N/yr)$ vs. intensity calculation for the historical dataset (1568 - October 1975) with aftershocks removed from the calculation. We see that there is a change in the slope of the data at intensity IV. This is to be expected in a sparsely populated area since the historical catalog will not be complete for events of intensity less than IV. Thus we have chosen to fit the relationship over the intensity range of IV to VII. The equation we obtained using a linear least-squares fit to the data is

$$\log_{10}(N/yr) = 2.179 - 0.587\ I \qquad (2)$$

Also shown in Figure 9 is yr/N for intensities of I to IX based on equation (2). On the figures the 95% confidence "standard deviation" levels are shown with dashed lines. The numerical values of N/yr and yr/N and their ranges are tabulated in Table 4. The results in Table 4 show that an intensity V event will occur in the STUDY AREA about every 5 years, an intensity VI every 20 years, an intensity VII every 80 years, intensity VIII every 330 years, and an intensity IX every 1300 years.

The slope of the curve as defined in equation 2 is 0.587. This is very close to the value of 0.59 obtained by Chinnery (1979) for southern New England and the 0.59 value for the whole U.S. (Cornell and Merz, 1975). The return period and the range of return periods given in Table 4 are consistent with those of Shakal and Toksoz (1977) and Chinnery (1979) for southern New England.

Figure 10 shows the results for the instrumental dataset (Oct. 1975 - Mar. 1981). In this case we have used magnitude rather than intensity as a measure of earthquake size. The equation obtained is:

$$\log_{10}(N/yr) = 3.365 - 1.064\ M \qquad (3)$$

where M in this case is the m_b value published in the quarterly catalogs. Also shown in Table 5 is a tabulation of N/yr and yr/N for magnitudes of 1.0 to 7.0. The results show that an event of magnitude 4.0 can be expected to occur in the STUDY AREA about every 8 years, a magnitude 5.0 every 90 years, and a magnitude 6.0 every 1000 years. Since the dataset covers only 5.5 years, the standard deviation is greater, and the extrapolation to the largest events is less reliable. The importance of the instrumental dataset is that the magnitudes are well quantified and subject to less error as compared to the more subjective intensity scale. Especially for the early historic events, intensity estimates were based on relatively sparse data. The favorable

comparison of the historic and instrumental results shown in Tables 4 and 5 assures that there are no gross errors in the historic dataset.

The consistency of the historical and instrumental estimates of frequency of occurrence versus size of events can be checked by comparing the slopes obtained from equations 2 and 3. The magnitude and intensity are generally related by $M = a_1 + a_2 I$ (Veneziano, 1975), where a_1 and a_2 are constants. The value of a_2 has been estimated to be 0.67 (Gutenberg and Richter, 1956) and 0.69 (Algermissen) on the basis of California data and 0.60 (Chinnery and Rogers, 1973; Howell, 1956) from the data in the eastern U.S. Using a factor of 0.60 for conversion, the magnitude slope of 1.06 corresponds to intensity slope of 0.63. Given the error bounds of both estimates, the intensity and magnitude slopes are consistent.

In Table 2, we listed all known earthquakes within a 50 km radius of the Knightville Dam. A fit of $Log_{10}(N/Yr)$ vs. I for this data yield a slope of 0.30 and an intercept of -0.35. Based on these results, an intensity V earthquake can be expected to occur close to the dam every 70 to 100 years. This considerably longer return time when compared to the results from Table 4 is simply a reflection of the much smaller area of consideration and the fact that the area within 50 km of the dam is characterized by a lower rate of seismic activity than that of the entire STUDY AREA.

ESTIMATION OF GROUND MOTION

Methods of Calculation

Given the history of earthquake activity in the STUDY AREA and the estimate of the return times for various earthquake intensities, the next step is to calculate the range of accelerations experienced at the dam site, and estimate the accelerations which would be produced by the Maximum Credible Earthquakes and Operating Bases Earthquakes. This can be accomplished in two ways. In the first method, we could model the ground motions produced by an earthquake of given fault dimensions, seismic moment, and stress drop. However, as we have seen, we cannot specify these quantities for New England earthquakes. In the second method, we can apply a number of empirical formulas which take on the general form:

$$g_s = G(M,R) \quad (4)$$

where g_s is the ground motion at the site (either intensity or acceleration) which is a function of the earthquake magnitude, M, and its distance from the site, R. The functional relationship G must take into account the attenuation of the ground motion g_s as a function of R, which is regionally dependent (Nuttli, 1973; Pulli and Aki, 1981; Herrmann, 1980). Expressions such as (4) are all subject to errors. We have used three different expressions to obtain a range of valid ground accelerations. We chose the final attenuation model based on the tests with the Gaza, NH earthquake data.

The first expression used is from Klimkiewicz (1980) which relates the Modified Mercalli intensity at a site to m_b and distance. His results were obtained via a multiple regression of individual intensity reports versus distance for four northeastern U.S. earthquakes with known m_b values. The expression is:

$$I(R) = 2.53 + 1.20m_b - 0.0027R - 1.84\mathrm{Log}_{10}(R) \qquad (5)$$

where R is the distance in kilometers. The intensity values obtained are for average foundation conditions. Results from (5) can then be correlated with various acceleration-intensity relations. A tabulation of results from (5) for various m_b and R values is given in Table 6.

The second expression used is from Nuttli and Herrmann (1981) which relates horizontal acceleration to m_b and distance. Their expression is:

$$\mathrm{Log}_{10}A = 0.55 + 0.5m_b - 0.83\mathrm{Log}_{10}(R) - 0.0019R \text{ for } R>15 \text{ km} \qquad (6)$$

For R<15 km, either the following expression:

$$A = \exp(0.933m_b) \text{ for } R<15 \text{ km} \qquad (7)$$

or the results from (6) should be used, whichever gives the smaller acceleration. A is in cm/sec^2 and R is in km. The advantage of these formulas is that they directly relate acceleration to m_b and distance and do not require conversion from intensity to acceleration. A tabulation of results from equations (6) and (7) is shown in Table 7.

The third expression is from Battis (1981) which again relates horizontal acceleration to m_b and distance. His expression is:

$$\ln A = 3.155 + 1.240 m_b - 1.244 \ln(R+25) \qquad (8)$$

where A is in cm/sec^2 and R is in km. (Note the use of natural logarithms in (8), whereas (6) uses common logarithms.) A tabulation of results from (8) is shown in Table 8.

The earthquake strength is defined in terms of the source intensity I_o for historic events and in terms of magnitudes for instrumentally recorded recent events. In this report we use the Modified Mercalli Intensity scale and body wave magnitude m_b. There are a number of different formulas to relate m_b and source intensity I_o. For the Eastern U.S. the empirical relationship established by different investigators (Chiburis, 1980; Chinnery and Rodgers, 1973; Howell, 1956) is

$$m_b = 1.0 + 0.60 I_o \qquad (9)$$

Other relationships that exist are from Nuttli (1973)

$$m_b = 2.607 + 0.341 I_o \qquad (10)$$

The average U.S. values (Gutenberg and Richter, 1956; Algermissen, 1969) obey the relation

$$m_b = 1.0 + 0.667 I_o \qquad (11)$$

Another question arises from the use of different magnitude scales. Chung and Bernreuter (1981) reviewed this topic extensively. Our primary interest is relating the local magnitude, M_L, and the well-defined body wave magnitude, m_b in New England. The relationship given by Chung and Bernreuter (1981) for the eastern U.S. is:

$$M_L = 0.54 + 0.88\ m_b \qquad (12)$$

Estimation of Maximum Credible Earthquakes

Given a measure of earthquake size, epicentral distance, and the above equations, we can calculate the peak ground acceleration at the dam site. However, problems arise since there are different attenuation curves, uncertainties in source intensities, and generally incomplete catalogs of earthquakes. Even in New England, where the historic record goes back almost 400 years, the available data may not contain the largest possible event in the area. The approaches used to cope with these problems vary depending on the tectonic setting of a region and the quality and quantity of the earthquake data (Stepp, 1972; TERA, 1980; Gutenberg and Richter, 1942; Algermissen and Perkins, 1976; Nuttli, 1973; Nuttli and Herrmann, 1978; Cornell, 1968; McGuire, 1977; McGuire and Shedlock, 1981; Murphy and O'Brien, 1977; Brazee, 1976; Hanks and Johnson, 1976, Battis, 1981).

In New England, we cannot associate the major earthquakes with known faults. Thus we cannot use deterministic fault models. Since surface faulting has not been observed, it is difficult to determine on a geologic basis the largest possible earthquake. The fault plane solutions (Pulli and Toksoz, 1981) show that although horizontal tectonic stresses are compressive, the stress directions are not uniform, indicating that local, rather than regional stresses predominate. Thus, very large regional tectonic events are unlikely. We will make some assumptions about the Maximum Credible Earthquakes:

1) The largest earthquakes will occur in the regions where there has been high seismic activity based on all historic and instrumental data.

2) In each region, the Maximum Credible Earthquake will have a source magnitude equal to that of the largest actual event ever observed plus one half m_b unit. In terms of source intensity, this generally translates to one intensity unit higher than the largest observed earthquake (equation 10). Since we have almost 450 years of data, our assumptions are reasonably conservative. If we assume a linear extension of the $Log_{10}(N)$ vs. I_o curve (Figure 9), an increase of one intensity unit for Cape Ann, MA would correspond to a return period of 1300 years.

Calculated Ground Motions

Using these equations we have carried out two sets of calculations. First, we have calculated the accelerations at the dam site for all events in the historical and instrumental earthquake catalogs and have compiled frequency-acceleration statistics. And second, we have calculated the accelerations at the dam site for hypothetical large events.

Figures 11a,b,c show the frequency-acceleration statistics for the historical data in the STUDY AREA using the acceleration-magnitude relations of Nuttli and Herrmann (1981), for the three intensity-magnitude relationships given in equations 9-11. The curves peak at about 2 cm/sec^2 since the historic data set is incomplete for small events. Otherwise the curves would increase monotonically toward zero acceleration. The largest acceleration observed is 41 cm/sec^2 which results from a small, close event (Leeds earthquake of 3-14-1893, I=IV). Other high values are due to large historic events in the distance range of 50-215 km.

Figures 12a,b,c show the same calculation using the attenuation formula from Battis (1981). Battis's formulas predict higher accelerations for distant events and smaller accelerations for close events. Thus the curves peak at about 4 cm/sec^2. The largest acceleration observed is 36 cm/sec^2 for the close event (Leeds earthquake).

These same calculations were carried out for the instrumental datasets. At this time period there are no events close to the dam site nor are there large events in the STUDY AREA. As a result the accelerations are all less than 5 cm/sec^2. These results are shown in Figures 13 and 14. Since the magnitudes are determined uniquely from the instrumental records, the two curves differ only because of different attenuations.

Maximum Credible Earthquakes

The next set of calculations is based on our estimates of the Maximum Credible Earthquakes to occur within the STUDY AREA. The locations of these earthquakes are listed in Table 9. They are based on the locations of large historical events in the STUDY AREA, and in one case, a close event of small magnitude. The maximum credible magnitudes have been selected by increasing the magnitudes of the largest known events by 1/2 m_b unit.

We have calculated the accelerations generated by these events at the dam site by using the three equations mentioned previously, i.e. (5), (7), and (8). Let us first consider the results of the application of Equation 5. Figures 15, 16, 17, and 18 show the intensity plots for the Maximum Credible

Earthquakes at Cape Ann, MA, Ossipee, NH, New York City, NY, and Lake George, NY. In addition, Table 9 lists the results for E. Haddam, CT and Torrington, CT, the sites of moderate events in the historical catalog. From these calculations, we find that the predicted intensities at the dam site are between intensity 4 3/4 and 5 1/2. By correlating these figures with the results of Nuttli (1979), we obtain a range of accelerations from 20 to 60 cm/sec^2. The largest accelerations are produced by the events at Cape Ann, MA, E. Haddam, CT, and Torrington, CT.

Similarly, we have used the equations of Nuttli and Herrmann (1981) and Battis (1981) to calculate the expected ground accelerations from these events. These results are also listed in Table 9. Using the equations of Nuttli and Herrmann (1981), the accelerations range from 17 to 42 cm/sec^2. The largest accelerations are produced by the close event at E. Haddam, CT. From Battis's (1981) equations, the accelerations range from 37 to 82 cm/sec^2. The largest accelerations are produced in this calculation by the Cape Ann earthquake, and next by the E. Haddam, CT event.

It is important to note that the values determined using Battis equations are the peak ground accelerations. Values calculated by Nuttli and Herrmann are sustained acceleration (or the average value for three cycles about the maximum). Sustained acceleration is lower than the peak acceleration, generally by a factor of 1.2 to 1.5. The discrepancy of about a factor of 2 between the two sets is partly due to different

models of attenuation. As we will discuss later, on the basis of Gaza, NH earthquake data, Battis's attenuation values are more applicable to New England than those of Nuttli and Hermann. Thus we will adopt the values based on Battis's curves for design purposes. Given the uncertainties, we should round the highest value to 80 cm/sec^2.

It is important to check the validity of the 80 cm/sec^2 peak ground acceleration and to obtain a measure of its variability such as the standard deviation or 95% confidence intervals. This can best be done by comparing our calculated value with the observed values from different earthquakes of comparable magnitude (m_b = 6.5). In the past decade a considerable amount of strong motion data have been collected in different parts of the world. In Appendix C we show some of these data in a collection of tables and figures. It is important to look not only at California data, but to other regions of the world where attenuation characteristics are different from those of California.

The San Fernando earthquake (9 February 1971, M_L = 6.4, m_b=6.2) and Borrego Mountain earthquake (9 April 1968, M_L = 6.6, m_b=6.1) are approximately the same magnitude as the Maximum Credible Earthquake near Cape Ann, MA. These two California earthquakes provided a large number of strong motion records (Shakal, 1979; Joyner and Boore, 1981). Looking at stations at epicentral distances of about 200 km, (see figures C-1 and C-2) we found a peak ground acceleration value of about 30 cm/sec^2 at station ANZ from the San Fernando

earthquake. A least squares fit to all the San Fernando data gives a mean value of about 9 cm/sec^2 for the peak acceleration (see Figure C-5). For the Borrego Mountain earthquake 9 and 16 cm/sec^2 peak ground accelerations were recorded at stations HSL and VER, respectively. Attenuation differences between California and eastern U.S. should increase the value of peak ground acceleration in the east by a factor of 8 at a distance of 200 km (Battis, 1981, Fig. 6). This means that we calculate the value of 80 cm/sec^2 for a Maximum Credible Earthquake of m_b = 6.5 at Cape Ann, MA which is consistent with those predicted from observed values from the Borrego Mountain earthquake, after adjusting for attenuation differences. Since most data in California come from strike-slip earthquakes, observed acceleration data are heavily weighted toward such events.

The San Fernando earthquake was a relatively high-stress drop, thrust type earthquake (Canitez and Toksoz, 1972; Heaton and Helmberger, 1979). Eastern U.S. earthquakes are also most likely to be shallow, high stress-drop, thrust type events (Pulli and Toksoz, 1981; Sbar and Sykes, 1973). If we use the mean value of 9 cm/sec^2 for San Fernando at 200 km and correct for attenuation by a factor of 8, we obtain an equivalent value of 70 cm/sec^2 for the northeastern U.S. The observed value of 30 cm/sec^2 at 200 km for San Fernando shows the extent of variations in peak ground acceleration as a function of azimuth at a given distance. Attenuation curves are

determined empirically by fitting observed values of peak ground acceleration as a function of distance. For an earthquake of given magnitude, source characteristics (such as stress drop) as well as attenuation affect the peak ground acceleration at a given distance. We do not have sufficient data to separate these effects, for estimating peak ground acceleration in New England, on the basis of calibration of data from California.

Next, we compare the calculated values for the Maximum Credible earthquake model for Cape Ann, MA (m_b = 6.5) with the strong motion data from Europe and other parts of the world. Figures C-3 and C-4 show the regression curves based on European data (Chiaruttini and Siro, 1981) for comparable magnitude and distances. Mean accelerations range from 30 cm/sec^2 for the Friuli earthquake data to 100 cm/sec^2 for the Alpide belt data.

The variations of strong motion at a given distance can be seen from Figures C-1, C-2, and C-3. In Figure C-5, the observed data and the fitted curves for the mean and plus/minus one and two standard deviations are illustrated. When we consider the worldwide data, after correction for magnitude differences we note that peaks vary by more than a factor of one hundred. Even for a single earthquake (San Fernando) the variation could be as much as a factor of 8.

As a result of these comparisons, we conclude that (1) the mean peak value for the peak ground acceleration for the

Knightville Dam site due to a Maximum Credible Earthquake at a regional distance range is about 80 cm/sec^2, and (2) if we assume the standard deviation to be similar to California earthquakes, the mean value plus one standard deviation can be as high as 125 cm/sec^2, and the mean value plus two standard deviations is 200 cm/sec^2. We recommend that in the analysis of the Knightville Dam a peak ground acceleration of 200 cm/sec^2 be used for the Maximum Credible Earthquake. The frequency at the peak acceleration is about 1 Hz if we use Borrego Mountain data, and about 4 Hz for the San Fernando model. The latter value is a more reasonable one given the low attenuation and high frequency radiation (high stress drop) earthquakes in New England. The duration of vibration will be longer than 10 seconds.

A point that needs to be discussed is why we recommend the use of the mean plus two standard deviations, instead of the more customary practice of using the mean plus one standard deviation, for calculating the maximum credible peak ground acceleration. When we look at the data for the San Fernando earthquake and the global acceleration data in Figure C-5, we find that a considerable number of observations fall outside the $\pm 1\ \sigma$ bands, and some outside the $\pm 2\ \sigma$ band. This significant scatter is also shown in Figure C-6 and in Tables C-1 and C-2. Since we expect the peak spectral frequencies to be higher in the eastern than in the western U.S., we will expect a greater scatter in the peak ground

accelerations at a given distance. Available strong motion data for the central U.S. (see Herrmann, 1981, Figure 2) show large scatter. For the Gaza, NH earthquake, nearly a factor of two variation in the peak accelerations between the "downstream" and the "abutment" sites is observed at the Franklin Falls Dam. This can be attributed to the scattering and focusing of high frequency (12 hz) seismic waves.

Another reason for the preference of the more conservative choice of plus 2 standard deviations is the uncertainties in the distance and magnitude of the maximum credible earthquake. An analysis of the changes in the magnitude and location (hence distance to the Knightville Dam) was carried out using equation 8. The magnitude of each maximum credible earthquake was varied by ± 0.50 m_b units. The radius of the epicentral area of each seismotectonic zone is estimated from the clustering of epicenters. Results are shown in Table 11. For the Cape Ann, MA and E. Haddam, CT seismotectonic zones, placing the maximum credible earthquake at the closest part of the zone and increasing the magnitude by 0.5 m_b will both produce peak ground accelerations of 205 cm/sec^2 at the Knightville Dam site. These values are close to the value of 200 cm/sec^2 recommended in this report for the regional event and 250 cm/sec^2 for the local event.

We calculated the Knightville site intensities for large earthquakes outside the STUDY AREA and found these to be small as compared to values listed in Table 9. For example, for the most active region of La Malbaie, PQ if we take a Maximum

Credible Earthquake of m_b = 7 1/4, the dam site intensity is only IV, lower than the values in Table 9.

Operating Basis Earthquake

The Operating Basis Earthquake is defined by the U.S. Nuclear Regulatory Commission as "the largest earthquake that reasonably could be expected to affect a structure during the life of the structure". We will assume a nominal life of 100 years for the Knightville Dam. From the frequency-intensity statistics (Table 4 or Figure 9) the largest event likely to occur in 100 years would have a source intensity of 7.5 or equivalent magnitude of m_b=5.5 . This event is most likely to occur in the Cape Ann, MA or Ossipee, NH seismotectonic zone. Placing such an event at the closest distance of 160 km and using Battis' (1981) attenuation curves, we obtain a peak ground acceleration of 32 cm/sec^2. Again, if we assume variations with azimuth due to source and lateral heterogeniety to be similar to those of the San Fernando earthquake (see Figure C-5), we obtain a mean plus one standard deviation of 52 cm/sec^2 and a mean plus two standard deviations of 84 cm/sec^2. For reasons elaborated in the discussion of the Maximum Credible Earthquake, we recommend the use of mean plus two standard deviations and a peak ground acceleration value of 84 cm/sec^2 for the operating basis earthquake. The peak frequency of such an event at the Knightville Dam is likely to be 3 - 5 Hz, slightly higher than that of the Maximum Credible Earthquake.

The earthquakes within the radius of 50 km of the dam (Table 2) present a problem. These are all historic events. They are generally small. There has been no detailed analysis of the reliability of their assigned intensities. The closest earthquake to the dam (distance = 12 km) is the Leeds, MA earthquake of 14 March 1893 with intensity of IV. To determine Maximum Credible Earthquake we applied the same criteria as before to this earthquake. We increased this intensity by one unit and calculated the dam site acceleration. These are also listed in Table 7. The peak ground acceleration is less than about 45 cm/sec^2. At such short distances however, earthquake source properties (finiteness, rupture propagation, asperities of stress drop across the fault plane) and local geologic heterogeneities can affect the maximum accelerations at a given site (see Shakal, 1979). Furthermore, the Gaza, NH earthquake of 19 January 1982 provided some strong motion data relevant to this problem. This is discussed in the next section.

Implications of the Strong Motion Data from the Gaza, NH Earthquake of January 19, 1982

On January 19, 1982 (Jan. 18 local time), an earthquake of m_b = 4.7 occurred in Gaza, NH (see Table 10 for epicentral information). This earthquake was recorded by the northeastern U.S. seismic network and is presently being studied in detail. The earthquake also triggered the strong

motion instruments located at six different sites at distances of 7 to 104 km. These instruments were installed and are operated by the Army Corp of Engineers. The stations, distances, and recorded accelerations are listed in Table 10. The importance of these data is that for the first time we have strong motion records in New England for an earthquake of known magnitude. Thus, we can test the applicability of the peak ground acceleration versus distance relations for a given m_b. These acceleration values are shown in Figures 19 and 20 along with the curves of Nuttli and Herrmann (1981) and Battis (1981).

We are interested in the free-field accelerations. For the plots we used the "down-stream" stations rather than the "dam-crest" or abutments. Further, to obtain the peak horizontal acceleration, we took the vector sum of the longitudinal and transverse components. The difference between this "resolved" horizontal value and the larger of the two components is not significant as can be seen in Table 10.

The Battis' curve is close to the observed values at far distances, but it underestimates the ground acceleration at close distances. At close distances, source properties play an important role in the resulting ground acceleration. The peak horizontal accelerations from the Gaza earthquake at close distances are comparable to values obtained from Imperial Valley Earthquake aftershock (15 October 1979, M_L = 5.0) and from Friuli aftershocks (see Appendix C, Tables C-1 and C-2). High acceleration (180 cm/sec^2) was observed at a

close-in distance from a magnitude 2.6 (m_b) earthquake in Blue Mountain Lake, NY in 1973 (Campbell, 1981, Figure 3). Thus the high accelerations recorded at the Franklin Falls Dam site are not unusual. New England earthquakes are generally shallow, high stress drop events. They generate abundant high frequency ground motion giving rise to high accelerations at near distances.

The Gaza, NH earthquake should be given great weight for estimating the peak ground acceleration at the Knightville Dam, since the Maximum Credible Earthquake close to the dam is based on the Leeds, MA event of March 14, 1893. Its size is comparable to the Gaza earthquake (m_b = 4.0 vs. 4.7) and the Gaza-Franklin Falls distance (7 km) is also similar to the Leeds-Knightville Dam distance (12 km).

We have made a new estimate of the peak ground acceleration at the dam site in the light of the Gaza, NH data. In Figure 21, the acceleration observations from the Gaza, NH earthquake have been superposed onto the decay curves of Battis (1981). We have chosen to roughly parallel Battis's curves for acceleration versus distance for the Gaza, NH data and a new curve for m_b = 4.0, which corresponds to the Maximum Credible Earthquake at Leeds, MA. For a distance of 12 km, we obtain an acceleration of about 70 cm/sec^2. As is shown in Table 10 and Appendix C (Tables C-1 and C-2), at close-in distances, accelerations can vary by a factor of 2 to 5 with azimuth, due both to source properties and local ground conditions. Given a 70 cm/sec^2 mean value, we could expect

peak accelerations of 200-300 cm/sec^2 due to the maximum credible local earthquake. The frequency at peak acceleration at such close distances is likely to be about 10-15 Hz. Duration of high acceleration is not likely to exceed about one second. Thus the effect is most likely to be on the stiff structures.

The frequency-magnitude statistics for the local events is poor due to few data points. However, the return period of an earthquake $m_b \approx 4.0$ is about 100 years. Thus, the $m_b = 4.0$ is also the Operating Basis Earthquake.

CONCLUSIONS

The seismic hazard and peak ground acceleration for the Knightville Dam site are based on the analyses of historic seismicity, instrumental data from the northeastern U.S. seismic network, and the strong ground motion records of the Gaza, NH earthquake of Jan. 19, 1982. The study emphasizes the physical and deterministic approach with great weight given to the data on geologic and tectonic characteristics of New England, and the source characteristics of earthquakes. All accelerations are for average bedrock conditions, which consist of igneous and metamorphic rocks. The primary results of the analysis are:

1) Both historic and network seismic data suggest that the Knightville Dam is in a relatively stable area. No significant earthquakes have occurred in the immediate vicinity of the dam. The earthquake hazards at the dam arise

from the accelerations due to regional major earthquakes and the small local events.

2) The peak ground accelerations calculated at the dam site using all historic and instrumental earthquakes in the STUDY AREA are equal to or less than 42 cm/sec^2. Different attenuation models affect the results, but the highest value does not change by more than 30 percent.

3) The peak ground accelerations due to the "Maximum Credible Earthquake" are calculated separately for regional events (that are outside the 50 km radius from the dam) and the local events (those inside the 50 km radius).

4) For "regional" model earthquakes, the calculated value of peak ground acceleration due to the Maximum Credible Earthquake is 82 cm/sec^2. This is due to the model earthquake at Cape Ann, MA with m_b = 6.5. The 82 cm/sec^2 is in good agreement with other estimates for the general area of New England (Algermissen and Perkin, 1976). It also compares favorably with European data, and after correction for attenuation differences, with peak ground acceleration values from California earthquakes of similar magnitudes (Borrego Mountain earthquake of 1968 and mean of the San Fernando earthquake of 1971). This 82 cm/sec^2 should be considered the mean value. We estimated the deviation from the mean, for a fixed distance, on the basis of global data. Mean plus one standard deviation gives 125 cm/sec^2 and mean plus two standard deviations gives a peak ground acceleration value of 200 cm/sec^2.

For the analysis of Knightville Dam, we recommend using the peak ground acceleration value of 200 cm/sec^2 for the Maximum Credible Earthquake. Our choice of the mean plus two standard deviations for the maximum credible peak ground acceleration (instead of mean plus one standard deviation) is based on both the expected scatter of the ground acceleration for a given earthquake, and the uncertainties associated with the distance and the magnitude of the maximum credible earthquake. Since this hypothetical event is distant and of magnitude 6.5, the frequency at peak acceleration will be between 1 and 5 Hz and duration of shaking at least 10 seconds.

For the Operating Basis Earthquake we calculate a peak ground acceleration of 32 cm/sec^2 at the Knightville Dam. The mean plus one standard deviation is 53 cm/sec^2 and the mean plus two standard deviations is 84 cm/sec^2. We recommend the value of 84 cm/sec^2 for the peak ground acceleration for the Operating Basis Earthquake. Peak frequency is likely to be between 2 and 6 Hz.

Calculated velocity and displacement values corresponding to peak ground accelerations at the nominal peak frequencies are given in Table 12.

5) For local events the picture is more complicated. The nearest known event to the Knightville Dam occurred near Leeds, MA at a distance of 12 km on March 14, 1893. A local Maximum Credible Earthquake based on the central U.S. attenuation models gives a peak ground acceleration of 30-60 cm/sec^2. However, if we use the results of the preliminary analysis of the strong motion data from the Gaza, NH earthquake of January 19, 1982 and a Maximum Credible

Earthquake near Leeds, MA, we obtain a peak ground acceleration of 70 cm/sec2 at the dam site. Given the variability of peak ground accelerations due to source properties and local heterogeneities, peak accelerations of 200 to 300 cm/sec2 cannot be ruled out. The durations of such motions will be short and the frequencies will be high (10-15 Hz). The return period of the local Maximum Credible Earthquake is less than 100 years. Thus, above values of peak ground acceleration apply for the Operating Basis Earthquake. Corresponding values of ground velocity and displacement are summarized in Table 12.

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Table 12. Summary of earthquake source characteristics and calculated ground motions at the Knightville Dam for Maximum Credible and Operating Basis Earthquakes.

Table 1

Seismic Stations Near the Knightville Dam Site

Code	Site	Lat	Long	Dist. in km	Operating history
WCM	Williams Coll., MA	42.710	-73.206	54	Operated during 1930's
QUA	Quabbin Reser., MA	42.457	-72.374	43	April 1975 - present
LNX	Lenox, MA	42.339	-73.272	35	June 1979 - present
MARL	Marlboro, VT	42.838	-72.801	60	July 1977 - present
ECT	Ellsworth, CT	41.835	-73.411	69	January 1976 - present
TMT	Talcott Mtn., CT	41.811	-72.799	55	Jan 1972 - June 1977
UCT	Storrs, CT	41.832	-72.251	72	Nov. 1974 - present

Table 2

Earthquakes within 50 km (31 miles)
of the Knightville Dam site.

MODYYear	O.T.	Lat.	Long.	Mag.	Int	State	Area
01121807		42.30	-72.60	-	2	MA	Northampton
02151849		42.10	-72.60	-	3	MA	Springfield
10231870	1130	42.10	-72.60	-	3	MA	Springfield
12041884	0518	42.30	-72.70	-	2	MA	Northampton
01091886		41.90	-73.10	-	2	CT	Winsted
09031886		42.50	-73.40	-	2	NY	Lebanon Springs
09091886		42.50	-73.40	-	2	NY	Lebanon Springs
03141893		42.30	-72.70	-	4	MA	Leeds
06111898	0645	42.80	-72.60	-	4	VT	Brattleboro-Vernon
12081928	0412	41.80	-72.50	-	2	CT	Ellington
03271930	1930	42.10	-72.70	-	3	MA	W. Springfield
05041931	1017	42.40	-72.50	-	3	MA	Amherst
07201932	2330	42.20	-73.20	-	2	MA	Lake Garfield
03311943	1130	42.30	-72.60	-	2	MA	Northampton
02131954		42.20	-72.60	-	4	MA	Springfield
02131954		42.20	-72.60	-	4	MA	Springfield
04131959	2120	41.90	-73.30	3.4	0	CT	S. Canaan
06011963		42.60	-73.00	-	2	MA	North Adams

Table 3

Significant Earthquakes Within the Knightville Dam STUDY AREA (a significant earthquake is one of intensity at least VII or magnitude at least 5.0 m_b)

MODYYear	O.T.	Lat.	Long.	Dist in km	Mag. m_b	Int	State,	Area
11101727	0340	42.8	-70.6	193	-	7	MA	Cape Ann
12191737	0330	40.8	-74.0	192	-	7	NY,	New York
11181755	0912	42.7	-70.3	214	-	8	MA,	Cape Ann
08101884	1907	40.6	-74.0	212	-	7	NY,	New York
04201931	1954	43.4	-73.7	141	-	7	NH,	Lake George
12201940	0727	43.8	-71.3	209	5.4	7	NH,	Ossipee
12241940	1343	43.8	-71.3	209	5.4	7	NH,	Ossipee

Table 4

Earthquake Statistics for the
Knightville Dam STUDY AREA
Historical to October 1975

Intensity	min	N/yr	max	min	Yr/N	max
1.0	36.48	39.08	41.88	.02	.03	.03
1.5	17.93	19.88	22.05	.05	.05	.06
2.0	8.81	10.12	11.61	.09	.10	.11
2.5	4.33	5.15	6.12	.16	.19	.23
3.0	2.13	2.62	3.22	.31	.38	.47
3.5	1.05	1.33	1.70	.59	.75	.96
4.0	.51	.68	.89	1.12	1.48	1.95
4.5	.25	.34	.47	2.13	2.90	3.96
5.0	.12	.18	.25	4.04	5.70	8.05
5.5	.06	.09	.13	7.66	11.21	16.39
6.0	.03	.05	.07	14.55	22.03	33.34
6.5	.01	.02	.04	27.64	43.30	67.84
7.0	.01	.01	.02	52.48	85.11	138.04
7.5	.00	.01	.01	99.66	167.30	280.87
8.0	.00	.00	.01	189.23	328.85	571.48
8.5	.00	.00	.00	359.34	646.40	1162.79
9.0	.00	.00	.00	682.34	1270.57	2365.92

Table 5

Earthquake Statistics for the
Knightville Dam STUDY AREA
October 1975-March 1981

Magnitude	min	N/yr	max	min	Yr/N	max
1.0	177.83	199.99	224.91	.00	.01	.01
1.5	49.26	58.75	70.06	.01	.02	.02
2.0	13.65	17.26	21.83	.05	.06	.07
2.5	3.78	5.07	6.80	.15	.20	.26
3.0	1.05	1.49	2.12	.47	.67	.95
3.5	.29	.44	.66	1.52	2.29	3.45
4.0	.08	.13	.21	4.86	7.78	12.45
4.5	.02	.04	.06	15.61	26.48	44.93
5.0	.01	.01	.02	50.12	90.16	162.18
5.5	.00	.00	.01	160.88	306.90	585.46
6.0	.00	.00	.00	516.42	1044.72	2113.49
6.5	.00	.00	.00	1657.68	3556.31	7629.59
7.0	.00	.00	.00	5321.09	12105.95	27542.31

Table 6

MM Intensity vs. mb and distance
Klimkiewicz (1980) attenuation model for the NEUS
Intensities on Average Foundation Conditions

Distance (km)	mb 4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
1	7.3	7.9	8.5	9.1	9.7	10.3	10.9	11.5
2	6.8	7.4	8.0	8.6	9.2	9.8	10.4	11.0
3	6.4	7.0	7.6	8.2	8.8	9.4	10.0	10.6
4	6.2	6.8	7.4	8.0	8.6	9.2	9.8	10.4
5	6.0	6.6	7.2	7.8	8.4	9.0	9.6	10.2
6	5.9	6.5	7.1	7.7	8.3	8.9	9.5	10.1
7	5.8	6.4	7.0	7.6	8.2	8.8	9.4	10.0
8	5.6	6.2	6.8	7.4	8.0	8.6	9.2	9.8
9	5.5	6.1	6.7	7.3	7.9	8.5	9.1	9.7
10	5.5	6.1	6.7	7.3	7.9	8.5	9.1	9.7
20	4.9	5.5	6.1	6.7	7.3	7.9	8.5	9.1
30	4.5	5.1	5.7	6.3	6.9	7.5	8.1	8.7
40	4.3	4.9	5.5	6.1	6.7	7.3	7.9	8.5
50	4.1	4.7	5.3	5.9	6.5	7.1	7.7	8.3
60	3.9	4.5	5.1	5.7	6.3	6.9	7.5	8.1
70	3.7	4.3	4.9	5.5	6.1	6.7	7.3	7.9
80	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8
90	3.5	4.1	4.7	5.3	5.9	6.5	7.1	7.7
100	3.4	4.0	4.6	5.2	5.8	6.4	7.0	7.6
120	3.2	3.8	4.4	5.0	5.6	6.2	6.8	7.4
140	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2
160	2.8	3.4	4.0	4.6	5.2	5.8	6.4	7.0
180	2.7	3.3	3.9	4.5	5.1	5.7	6.3	6.9
200	2.6	3.2	3.8	4.4	5.0	5.6	6.2	6.8
220	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6
240	2.3	2.9	3.5	4.1	4.7	5.3	5.9	6.5
260	2.2	2.8	3.4	4.0	4.6	5.2	5.8	6.4
280	2.1	2.7	3.3	3.9	4.5	5.1	5.7	6.3
300	2.0	2.6	3.2	3.8	4.4	5.0	5.6	6.2
325	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0
350	1.7	2.3	2.9	3.5	4.1	4.7	5.3	5.9
375	1.6	2.2	2.8	3.4	4.0	4.6	5.2	5.8
400	1.5	2.1	2.7	3.3	3.9	4.5	5.1	5.7
425	1.3	1.9	2.5	3.1	3.7	4.3	4.9	5.5
450	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4
475	1.1	1.7	2.3	2.9	3.5	4.1	4.7	5.3
500	1.0	1.6	2.2	2.8	3.4	4.0	4.6	5.2

Table 7

Acceleration vs. mb and distance
Battis (1981)
Acceleration-Magnitude-distance relation
Acceleration in cm per second squared

Distance (km)	mb 4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
1	58.1	108.0	200.7	373.1	693.6	1289.4	2396.9	4455.6
2	55.4	103.0	191.5	356.0	661.8	1230.3	2287.0	4251.3
3	53.0	98.5	183.0	340.3	632.5	1175.8	2185.8	4063.2
4	50.7	94.3	175.2	325.7	605.5	1125.6	2092.4	3889.7
5	48.6	90.4	168.0	312.3	580.5	1079.1	2006.0	3729.1
6	46.7	86.8	161.3	299.8	557.3	1036.0	1925.8	3580.0
7	44.9	83.4	155.0	288.2	535.7	995.9	1851.3	3441.4
8	43.2	80.3	149.2	277.4	515.6	958.5	1781.7	3312.1
9	41.6	77.3	143.8	267.3	496.8	923.5	1716.8	3191.4
10	40.1	74.6	138.7	257.8	479.2	890.8	1656.0	3078.3
20	29.4	54.6	101.4	188.6	350.6	651.7	1211.4	2251.9
30	22.9	42.5	79.0	146.9	273.1	507.7	943.8	1754.4
40	18.6	34.5	64.2	119.4	221.9	412.4	766.7	1425.2
50	15.5	28.9	53.7	99.9	185.7	345.2	641.7	1192.8
60	13.3	24.7	46.0	85.5	158.9	295.4	549.1	1020.8
70	11.6	21.5	40.0	74.4	138.4	257.2	478.2	888.9
80	10.2	19.0	35.4	65.7	122.2	227.1	422.2	784.8
90	9.1	17.0	31.6	58.7	109.1	202.8	377.0	700.9
100	8.2	15.3	28.5	52.9	98.4	182.8	339.9	631.8
120	6.8	12.7	23.7	44.0	81.8	152.0	282.6	525.3
140	5.8	10.8	20.1	37.5	69.6	129.4	240.6	447.3
160	5.1	9.4	17.5	32.5	60.4	112.3	208.7	388.0
180	4.5	8.3	15.4	28.6	53.2	98.8	183.7	341.4
200	4.0	7.4	13.7	25.5	47.3	88.0	163.6	304.1
220	3.6	6.6	12.3	22.9	42.6	79.2	147.1	273.5
240	3.2	6.0	11.2	20.8	38.6	71.8	133.5	248.1
260	3.0	5.5	10.2	19.0	35.3	65.6	121.9	226.6
280	2.7	5.0	9.4	17.4	32.4	60.3	112.0	208.3
300	2.5	4.7	8.7	16.1	30.0	55.7	103.5	192.5
325	2.3	4.3	7.9	14.7	27.3	50.8	94.4	175.5
350	2.1	3.9	7.3	13.5	25.1	46.6	86.7	161.1
375	1.9	3.6	6.7	12.4	23.1	43.0	80.0	148.7
400	1.8	3.3	6.2	11.5	21.5	39.9	74.2	137.9
425	1.7	3.1	5.8	10.8	20.0	37.2	69.1	128.4
450	1.6	2.9	5.4	10.1	18.7	34.7	64.6	120.0
475	1.5	2.7	5.1	9.4	17.5	32.6	60.6	112.6
500	1.4	2.6	4.8	8.9	16.5	30.7	57.0	106.0

Table 8

Acceleration vs. mb and distance
Nuttli and Herrmann (1981)
Acceleration-Magnitude-distance relation
Acceleration in cm per second squared

Distance (km)	mb 4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5
1	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
2	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
3	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
4	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
5	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
6	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
7	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
8	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
9	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
10	41.8	66.6	106.2	169.3	269.9	430.3	686.1	1093.9
20	27.0	48.1	85.5	152.1	270.5	481.0	855.4	1521.1
30	18.5	32.9	58.5	104.0	184.9	328.8	584.8	1039.9
40	13.9	24.8	44.1	78.4	139.4	247.9	440.8	784.0
50	11.1	19.7	35.1	62.4	110.9	197.2	350.6	623.5
60	9.1	16.2	28.8	51.3	91.2	162.2	288.5	513.0
70	7.7	13.7	24.3	43.2	76.8	136.6	243.0	432.1
80	6.6	11.7	20.8	37.0	65.8	117.1	208.2	370.2
90	5.7	10.2	18.1	32.1	57.1	101.6	180.7	321.4
100	5.0	8.9	15.8	28.2	50.1	89.1	158.5	281.8
120	3.9	7.0	12.5	22.2	39.5	70.2	124.8	222.0
140	3.2	5.7	10.1	17.9	31.8	56.6	100.6	178.9
160	2.6	4.6	8.3	14.7	26.1	46.4	82.5	146.8
180	2.2	3.9	6.9	12.2	21.7	38.6	68.6	121.9
200	1.8	3.2	5.8	10.2	18.2	32.4	57.6	102.4
220	1.5	2.7	4.9	8.7	15.4	27.4	48.7	86.7
240	1.3	2.3	4.2	7.4	13.1	23.4	41.5	73.9
260	1.1	2.0	3.6	6.3	11.3	20.0	35.6	63.3
280	1.0	1.7	3.1	5.5	9.7	17.3	30.7	54.6
300	.8	1.5	2.7	4.7	8.4	14.9	26.5	47.2
325	.7	1.3	2.2	4.0	7.0	12.5	22.3	39.6
350	.6	1.1	1.9	3.3	5.9	10.6	18.8	33.4
375	.5	.9	1.6	2.8	5.0	8.9	15.9	28.3
400	.4	.8	1.3	2.4	4.3	7.6	13.5	24.0
425	.4	.6	1.2	2.0	3.6	6.5	11.5	20.5
450	.3	.6	1.0	1.7	3.1	5.5	9.8	17.5
475	.3	.5	.8	1.5	2.7	4.7	8.4	15.0
500	.2	.4	.7	1.3	2.3	4.1	7.2	12.9

Table 9

Area	dist. in km	max. cred. m_b	Int[1]	cm/sec2 Accel[2]	Accel[3]	Accel[4]
Ossipee, NH	205	6.0	5	20-50	17	46
Cape Ann, MA	213	6.5	5 1/2	30-60	29	82
NY City, NY	197	6.0	5	20-50	19	48
Lake George, NY	136	5.5	4 3/4	25-45	19	39
E. Haddam, CT	89	5.7	5 1/2	30-60	41	76
Torrington, CT	50	5.0	5 1/2	30-60	35	54
Leeds, MA	12	4.0	5 1/2	30-60	42	37

1) Intensity calculated via Klimkiewicz (1980) [Equation 4]

2) Acceleration from the acceleration-intensity curve of Nuttli (1979).

3) Acceleration calculated from the expressions of Nuttli and Herrmann (1981) [Equations 6 and 7].

4) Acceleration calculated from the expression of Battis (1981) [Equation 8].

Table 10

The Gaza, NH Earthquake of Jan. 19, 1982
Epicentral data: O.T. 00:14:42.5 UTC
Latitude 43.520
Longitude -71.610
Depth 5 km (approx)
Magnitude 4.7 (m_b)

Strong Motion Data

Peak Acceleration Values

	L		V		T		Resolved horiz	
	g	cm/s2	g	cm/s2	g	cm/s2	g	cm/s2
Franklin Falls Dam, NH								
Distance = 7 km								
Abutment	0.22	218	0.14	136	0.56	544	0.60	586
Crest	0.10	100	0.10	101	0.24	238	0.26	258
Downstream	0.08	77	0.14	136	0.31	307	0.32	317
North Hartland Dam, VT								
Distance = 62 km								
Crest	0.02	23	0.02	17	0.02	22	0.03	32
Union Village Dam, VT								
Distance = 61 km								
Abutment	0.01	6	0.01	6	0.01	5	0.01	8
Crest	0.02	22	0.02	22	0.03	26	0.03	34
Downstream	0.03	34	0.03	28	0.03	26	0.04	43
White River Jtn., VT								
Distance = 61 km								
Basement	0.02	15	0.02	17	0.02	21	0.03	26
North Springfield Dam, VT								
Distance = 75 km								
Downstream	0.03	31	0.02	15	0.02	23	0.03	39
Crest	0.03	31	0.02	15	0.02	15	0.03	34
Ball Mountain Dam, VT								
Distance = 104 km								
Crest	0.01	11	0.02	15	0.01	12	0.02	16

Table 11

Acceleration in cm/sec^2

Cape Ann, MA Seismic Zone
m_b

Dist km	6.00	6.25	6.50*	6.75	7.00
163	59.2	80.7	110.0	150.0	204.6
188	50.7	69.1	94.2	128.5	175.1
213*	44.1	60.2	82.1	111.9	152.6
238	39.0	53.2	72.5	98.8	134.7
263	34.8	47.5	64.7	88.3	120.3

Ossipee, NH Seismic Zone

Dist km	5.50	5.75	6.00*	6.25	6.50
155	33.6	45.8	62.5	85.2	116.2
180	28.6	39.0	53.2	72.5	98.8
205*	24.8	33.8	46.1	62.8	85.6
230	21.8	29.7	40.5	55.2	75.3
255	19.4	26.5	36.1	49.2	67.0

NY City, NY Seismic Zone

Dist km	5.50	5.75	6.00*	6.25	6.50
147	35.6	48.5	66.1	90.2	122.9
172	30.0	41.0	55.9	76.1	103.8
197*	25.9	35.3	48.1	65.6	89.5
222	22.7	30.9	42.2	57.5	78.4
247	20.1	27.4	37.4	51.0	69.5

Lake George, NY Seismic Zone

Dist km	5.00	5.25	5.50*	5.75	6.00
111	25.6	34.9	47.6	65.0	88.6
136*	20.8	28.3	38.6	52.7	71.8
161	17.4	23.7	32.3	44.0	60.0

E. Haddam, CT Seismic Zone

Dist km	5.25	5.50	5.75*	6.00	6.25
64	59.2	80.7	110.1	150.1	204.6
89*	43.5	59.3	80.9	110.3	150.4
114	34.0	46.4	63.2	86.2	117.5

* The nominal values used at each zone. Corresponding peak ground accelerations are underlined.

Table 12

	Regional		Local*
	Maximum Credible Earthquake	Operating Basis Earthquake	Maximum Credible Earthquake
Epicentral Area (Seismotectonic Zone)	Cape Ann, MA	Cape Ann, MA or Ossipee, NH	Leeds, MA
Nominal Distance to Knightville Dam (km)	213	213	12
Magnitude (m_b)	6.5	5.5	4.0
Peak Frequency (Hz)	3.0	4.0	12.0
Acceleration at Dam mean cm/sec^2	82.	32.	70.
Acceleration at Dam mean +1σ cm/sec^2	125.	52.	-
Acceleration at Dam mean +2σ cm/sec^2	200.	84.	250.**
(1) Velocity at Dam mean cm/sec	4.35	1.70	0.93
Velocity at Dam mean +1σ cm/sec	6.63	2.76	-
Velocity at Dam mean +2σ cm/sec	10.6	4.46	3.32
(1) Displacement at Dam mean cm	0.23	0.09	0.01
Displacement at Dam mean +1σ cm	0.35	0.15	-
Displacement at Dam mean +2σ cm	0.56	0.24	0.04

* Same value for Maximum Credible and Operating Basis earthquake. (see text)

** Estimate based on scatter of near field data. (see text)

(1) Velocity and displacement corresponding to peak acceleration at listed frequency (these are not peak velocity and peak displacement).

Figure Captions

Figure 1 Base map of the Knightville Dam STUDY AREA. The dam site is indicated by the diamond. The circle surrounding the dam has a radius of 161 km (100 miles). All references to the STUDY AREA in the text correspond to the area shown in this map.

Figure 2 Historical seismicity of the northeastern US -southeastern Canada from Smith (1962, 1966). The time period covered is 1534 to 1959.

Figure 3 Distribution of seismic stations in the New England area. The Knightville Dam site is indicated by the star (*). The three-letter codes of some of the nearby stations are also shown. The operating histories of these stations are given in Table 1.

Figure 4 Instrumental seismicity of the northeastern US -southeastern Canada. The period covered is October 1975 to March 1981.

Figure 5 Seismicity of the northeastern US -southeastern Canada for the period 1534 to 1980.

Figure 6 Historical seismicity of the Knightville Dam STUDY AREA for the period 1568 to October 1975.

Figure 7 Instrumental seismicity of the Knightville Dam STUDY AREA for the period October 1975 to March 1981.

Figure 8 Significant earthquakes within the Knightville Dam STUDY AREA. A significant earthquake is defined as an event of intensity at least VII (M.M.) or magnitude at least 5.0 (m_b).

Figure 9 Frequency-intensity statistics for the historical earthquakes in the STUDY AREA (those listed in Appendix A). Aftershocks have been removed. Also shown is the least squares fit to the data and 95% confidence limits. See Table 4 for tabulated statistics.

Figure 10 Frequency-magnitude statistics for the instrumental earthquake epicenters in the STUDY AREA (those listed in Appendix B). Aftershocks have been removed. Also shown is the least squares fit to the data and 95% confidence limits. See Table 5 for tabulated statistics.

Figure 11 Frequency-acceleration statistics for the historical earthquakes in the STUDY AREA, using the equations of Nuttli and Herrmann (1981). Three plots are shown. In 11a, intensity has been converted to magnitude using the equation M=1+0.60I; in 11b, the equation M=2.607+0.341I was used; in 11c, the equation M=1+0.66I was used.

Figure 12 Frequency-acceleration statistics for the historical earthquakes in the STUDY AREA, using the equation of Battis (1981). Three plots are shown. In 12a, intensity has been converted to magnitude using the equation M=1+0.60I; in 12b, the equation M=2.607+0.341I was used; in 12c, the equation M=1+0.66I was used.

Figure 13 Frequency-acceleration statistics for the instrumental earthquake epicenters in the STUDY AREA (those listed in Appendix B), using the equations of Nuttli and Herrmann (1981).

Figure 14 Frequency-acceleration statistics for the instrumental earthquake epicenters in the STUDY AREA, using the equation of Battis (1981).

Figure 15 Intensity distribution for the Maximum Credible Earthquake at Cape Ann, MA, based on the equation of Klimkiewicz (1980).

Figure 16 Intensity distribution for the Maximum Credible Earthquake at Ossipee, NH, based on the equation of Klimkiewicz (1980).

Figure 17 Intensity distribution for the Maximum Credible Earthquake at New York City, NY, based on the equation of Klimkiewicz (1980).

Figure 18 Intensity distribution for the Maximum Credible Earthquake at Lake George, NY, based on the equation of Klimkiewicz (1980).

Figure 19 Acceleration-distance-magnitude relation from Nuttli and Herrmann (1981). Also shown are the actual values observed at three sites for the Gaza, NH earthquake of 19 Jan 82. The data plotted are resolved horizontal components (sqrt[L2+T2]).

Figure 20 Acceleration-distance-magnitude relation from Battis (1981). Also shown are the actual values observed for the Gaza, NH earthquake of 19 Jan 82. The data plotted are resolved horizontal components (sqrt[L2+T2]).

Figure 21 Acceleration values for the Gaza, NH earthquake superposed onto the acceleration curves of Battis (1981). The curves have been re-scaled to account for the Gaza data. Shown are our acceleration curves for m_b = 4.7 (Gaza) and m_b = 4.0 (Leeds).

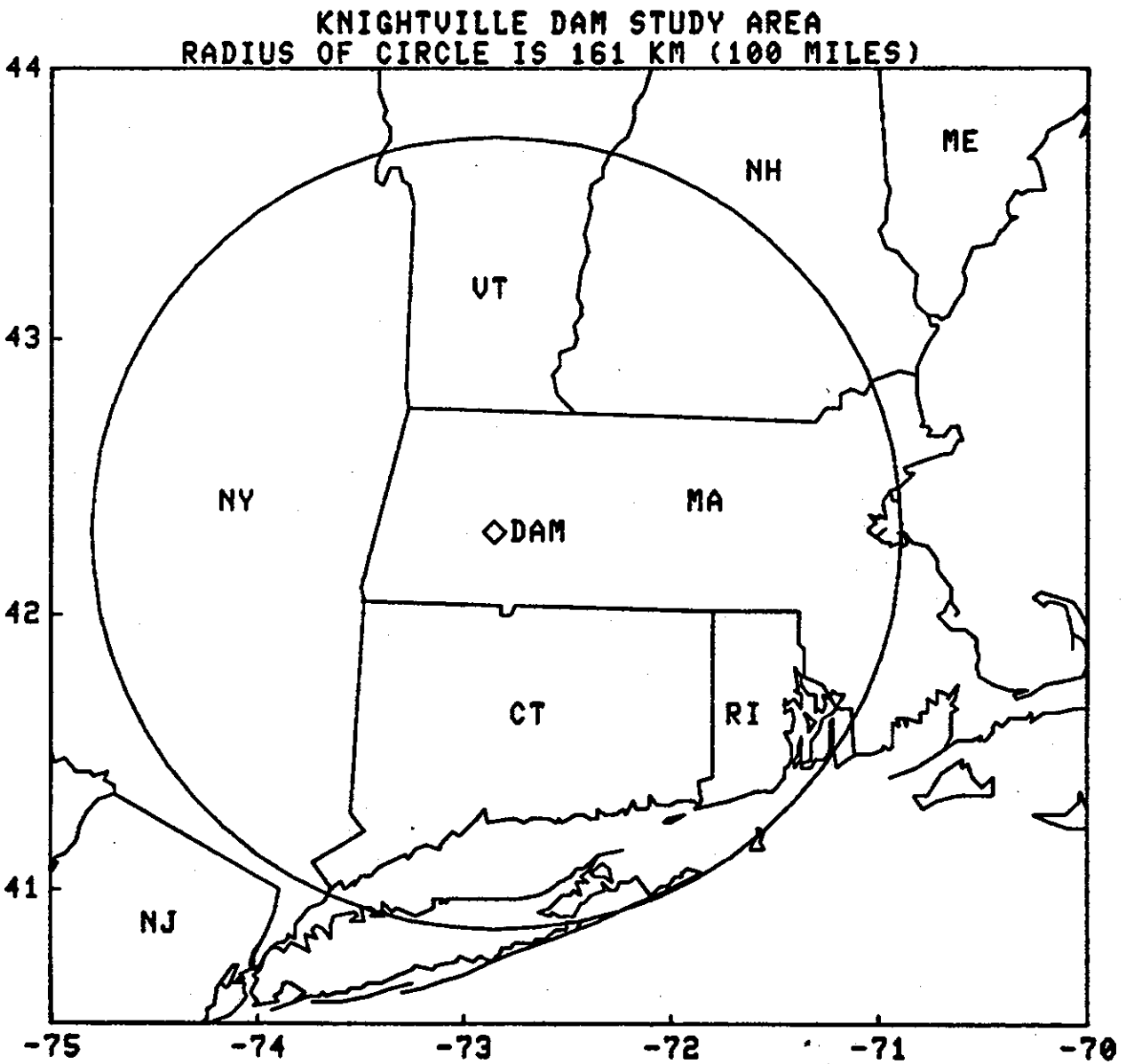

KNIGHTVILLE DAM STUDY AREA
RADIUS OF CIRCLE IS 161 KM (100 MILES)
44
43
42
41
-75
-74
-73
-72
-71
-70
ME
NH
VT
NY
MA
DAM
CT
RI
NJ

Figure 1

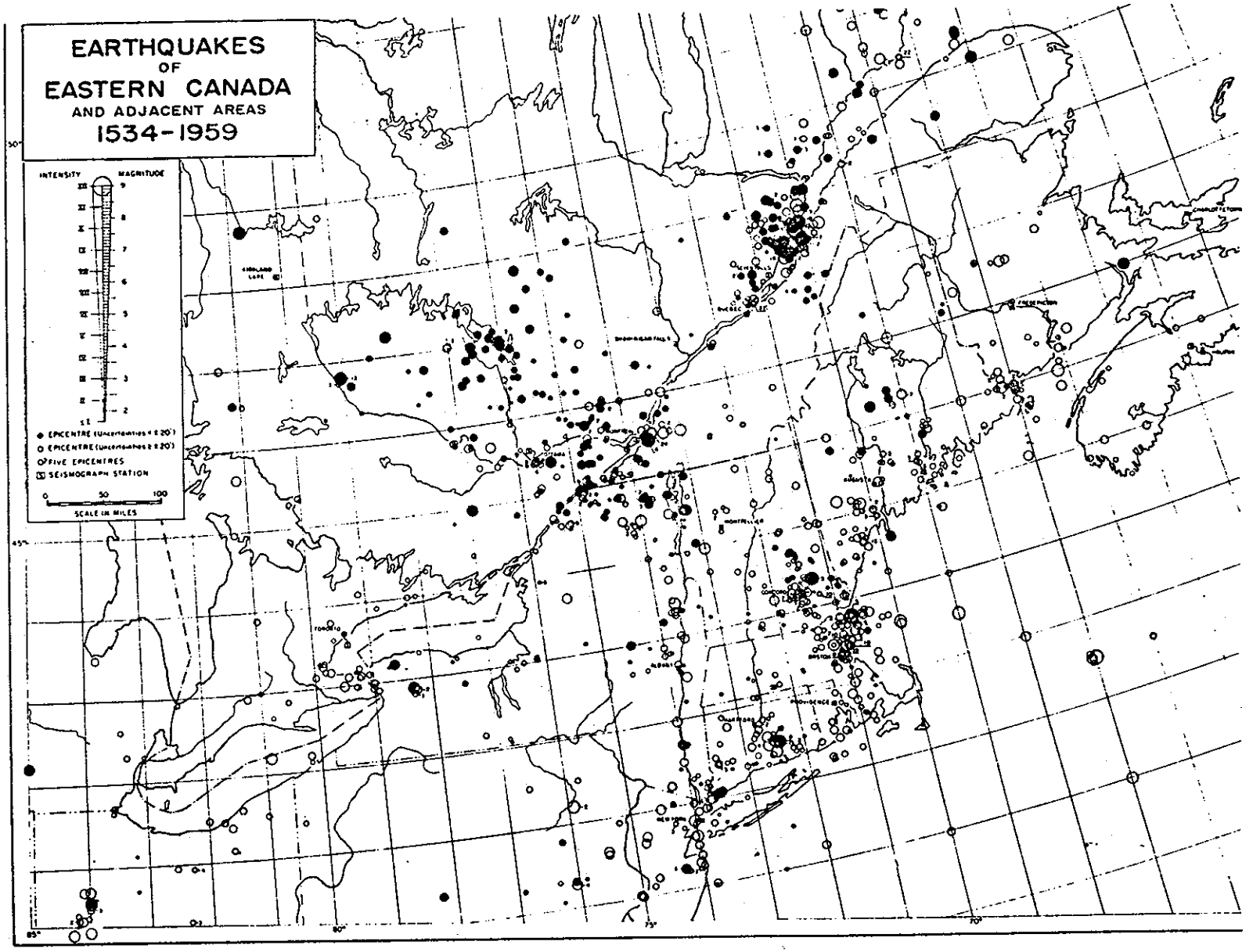

EARTHQUAKES
OF
EASTERN CANADA
AND ADJACENT AREAS
1534-1959
INTENSITY
MAGNITUDE
EPICENTRE
EPICENTRE
FIVE EPICENTRES
SEISMOGRAPH STATION
0
50
100
SCALE IN MILES
50°
45°
85°
80°
75°
70°

Figure 2

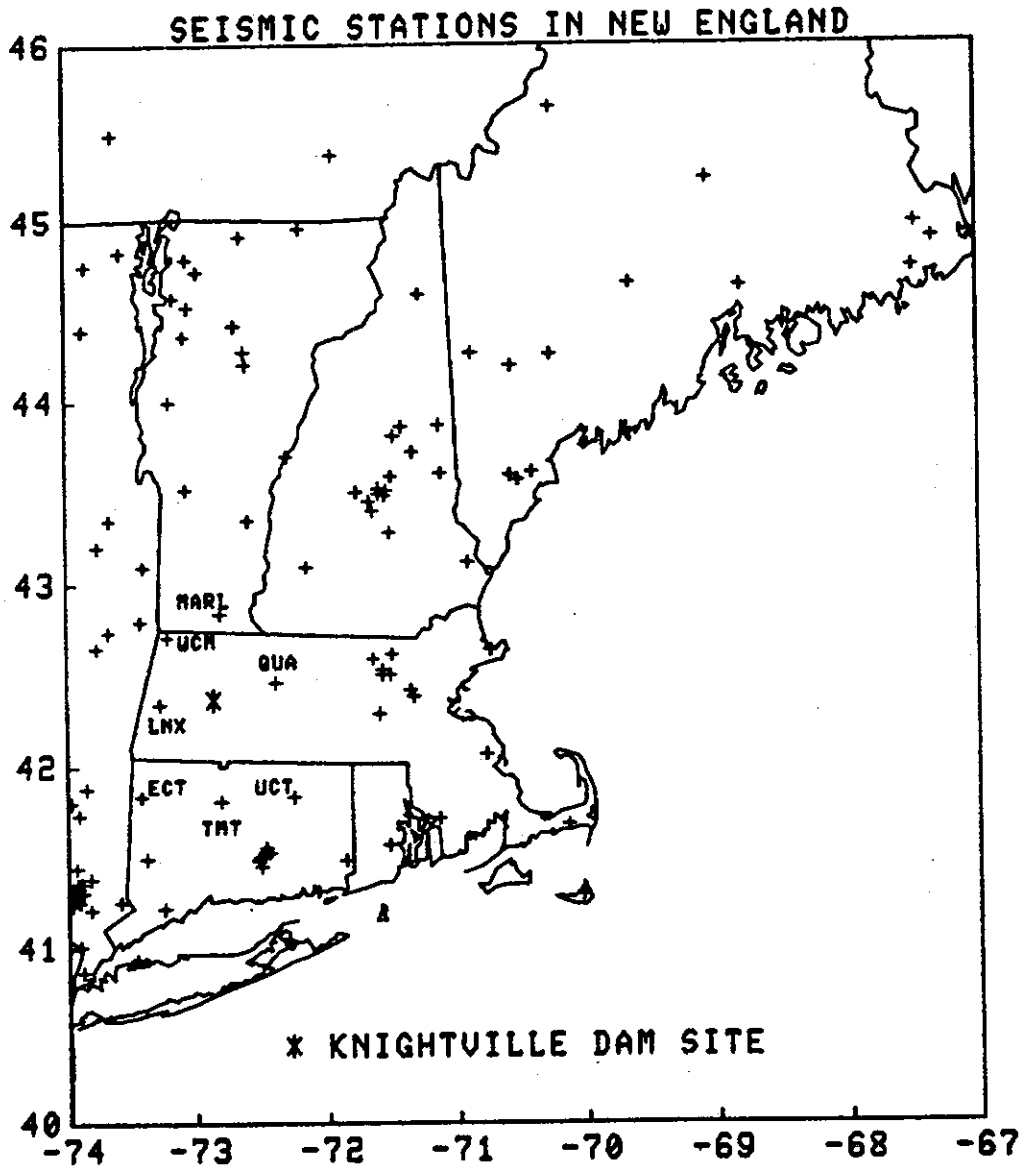

SEISMIC STATIONS IN NEW ENGLAND
46
45
44
43
42
41
40
-74
-73
-72
-71
-70
-69
-68
-67
MARL
UCH
QUA
LNX
ECT
UCT
TMT
* KNIGHTVILLE DAM SITE

Figure 3

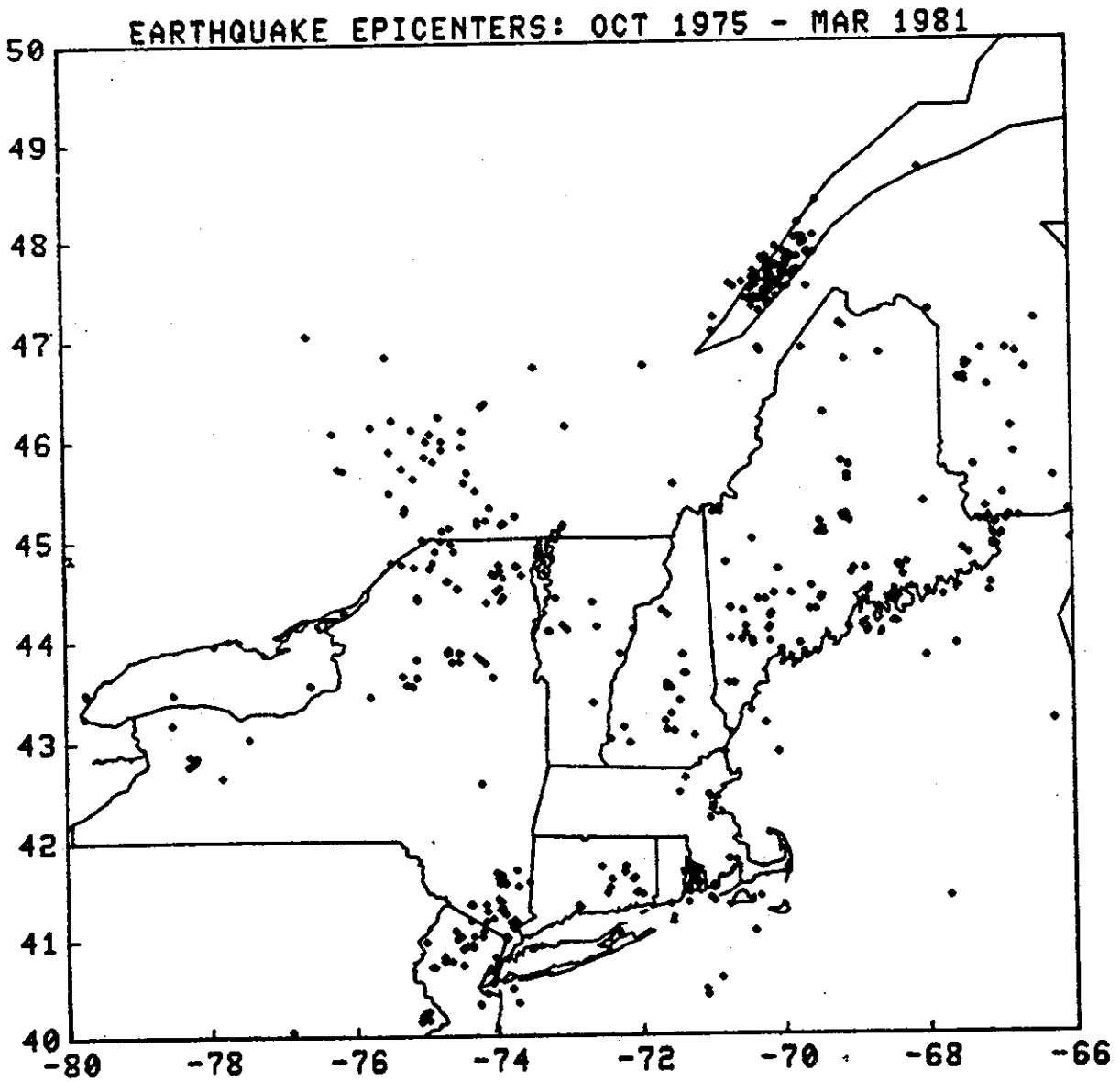

EARTHQUAKE EPICENTERS: OCT 1975 - MAR 1981
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 4

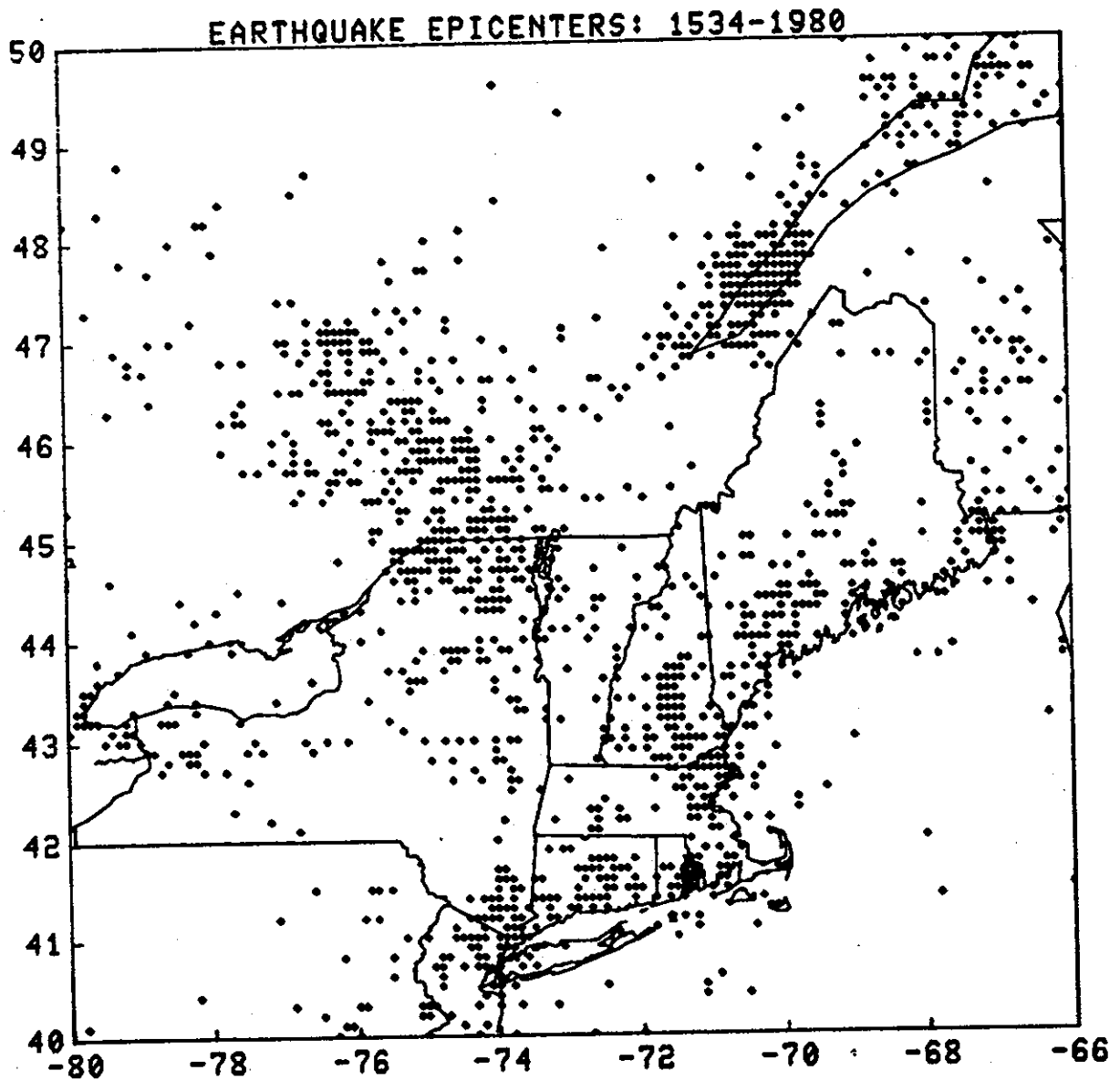

EARTHQUAKE EPICENTERS: 1534-1980
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 5

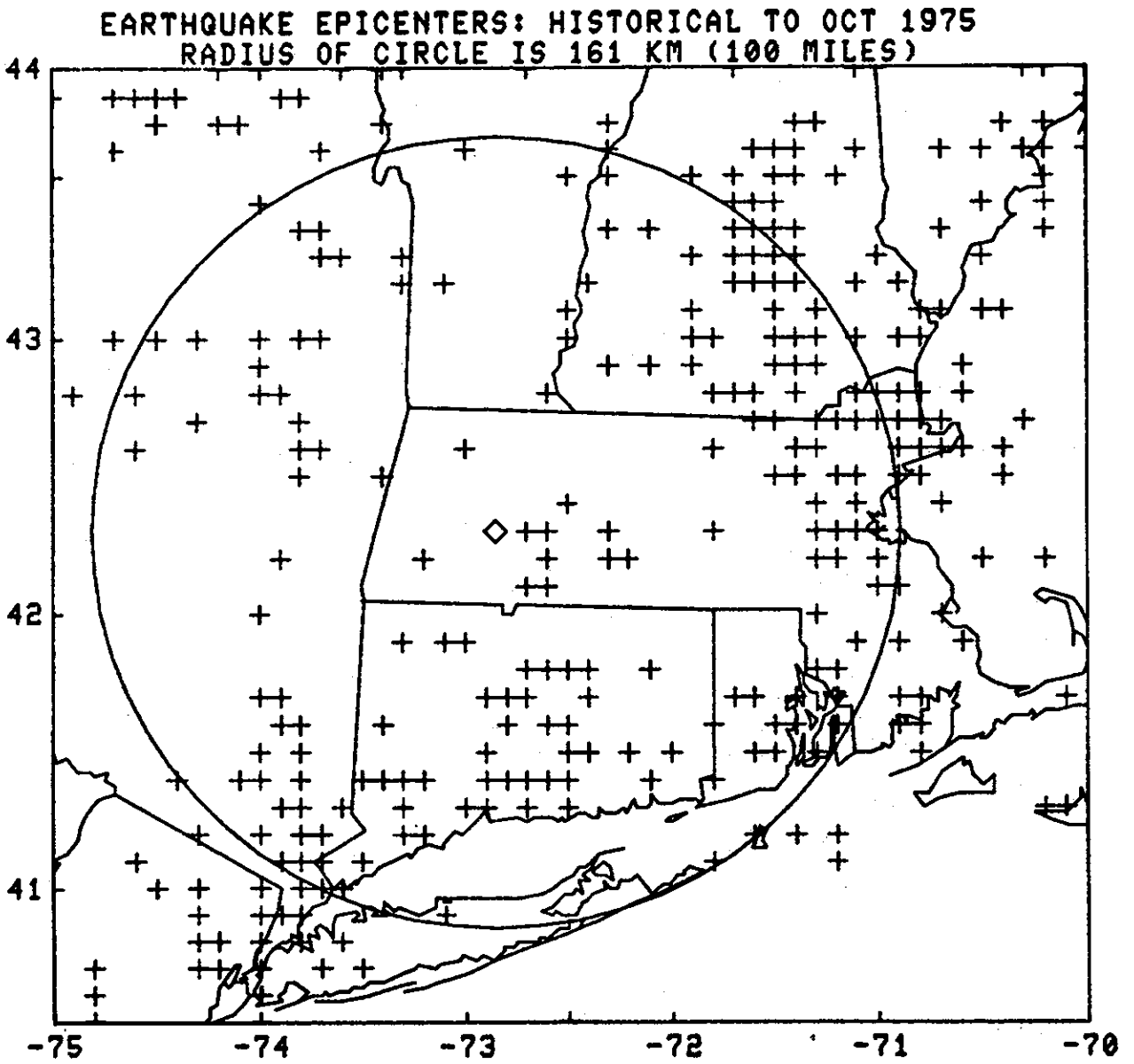

EARTHQUAKE EPICENTERS: HISTORICAL TO OCT 1975
RADIUS OF CIRCLE IS 161 KM (100 MILES)
44
43
42
41
-75
-74
-73
-72
-71
-70

Figure 6

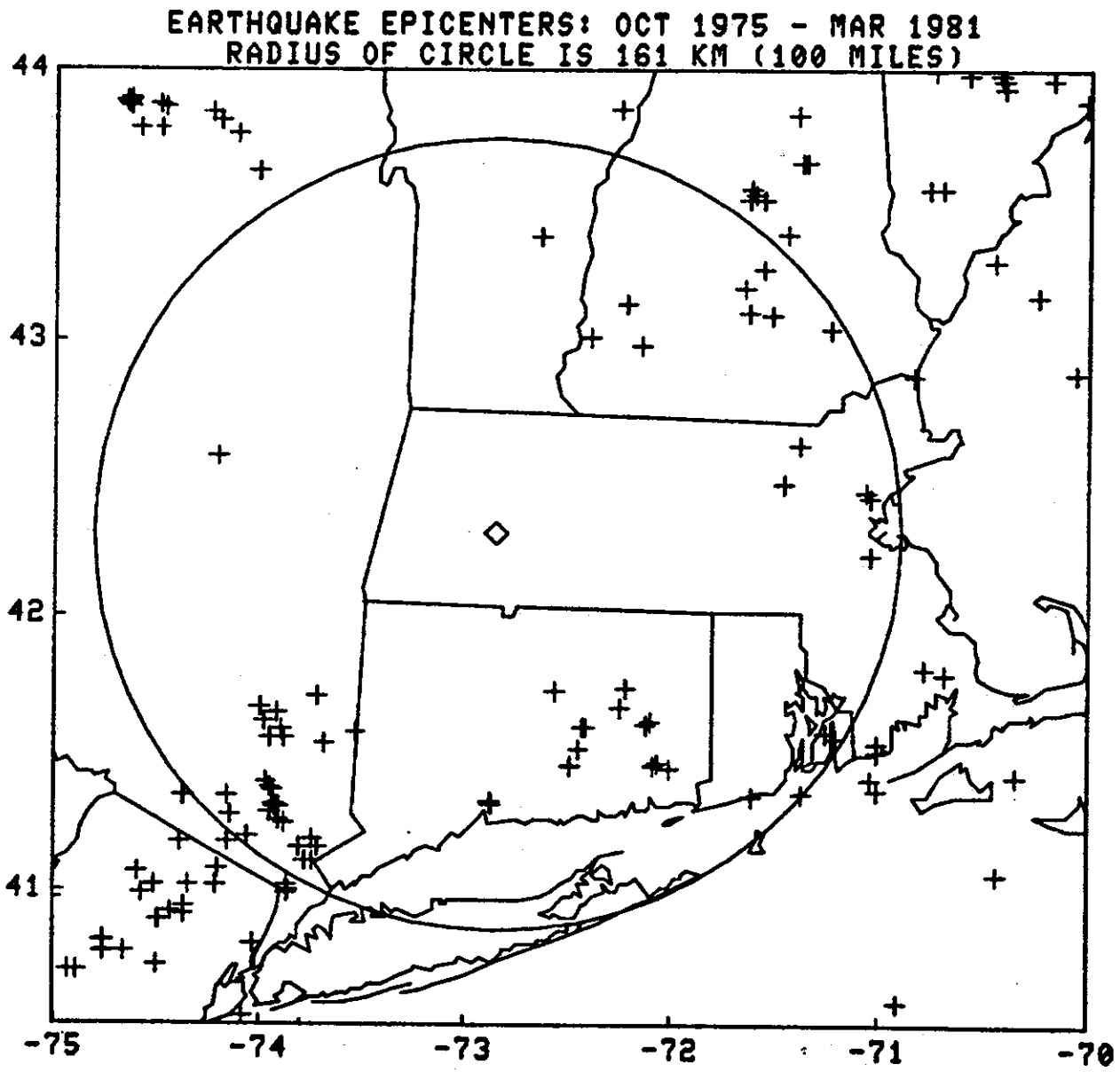

EARTHQUAKE EPICENTERS: OCT 1975 - MAR 1981
RADIUS OF CIRCLE IS 161 KM (100 MILES)
44
43
42
41
-75
-74
-73
-72
-71
-70

Figure 7

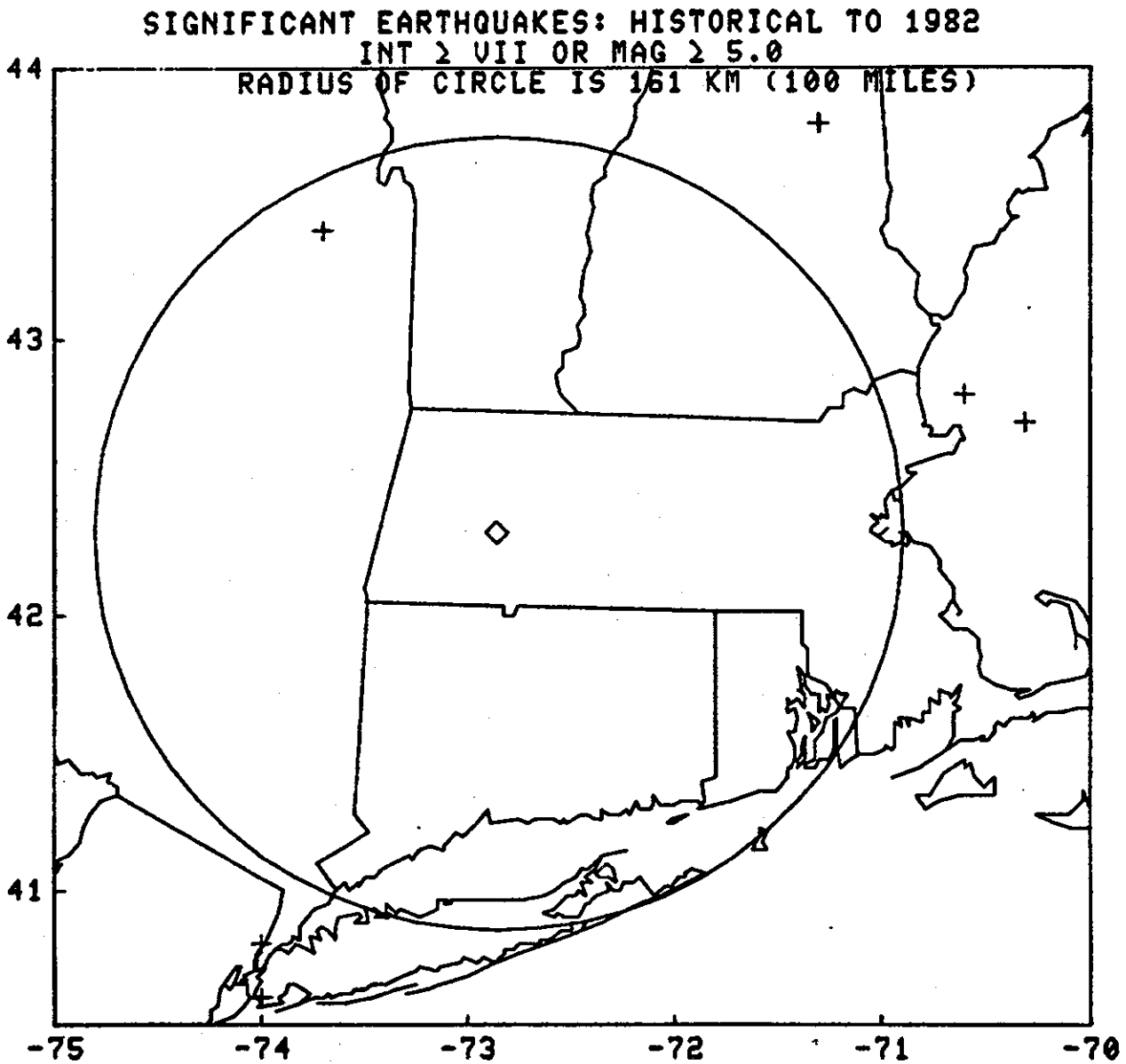

SIGNIFICANT EARTHQUAKES: HISTORICAL TO 1982
INT ≥ VII OR MAG ≥ 5.0
RADIUS OF CIRCLE IS 161 KM (100 MILES)
44
43
42
41
-75
-74
-73
-72
-71
-70

Figure 8

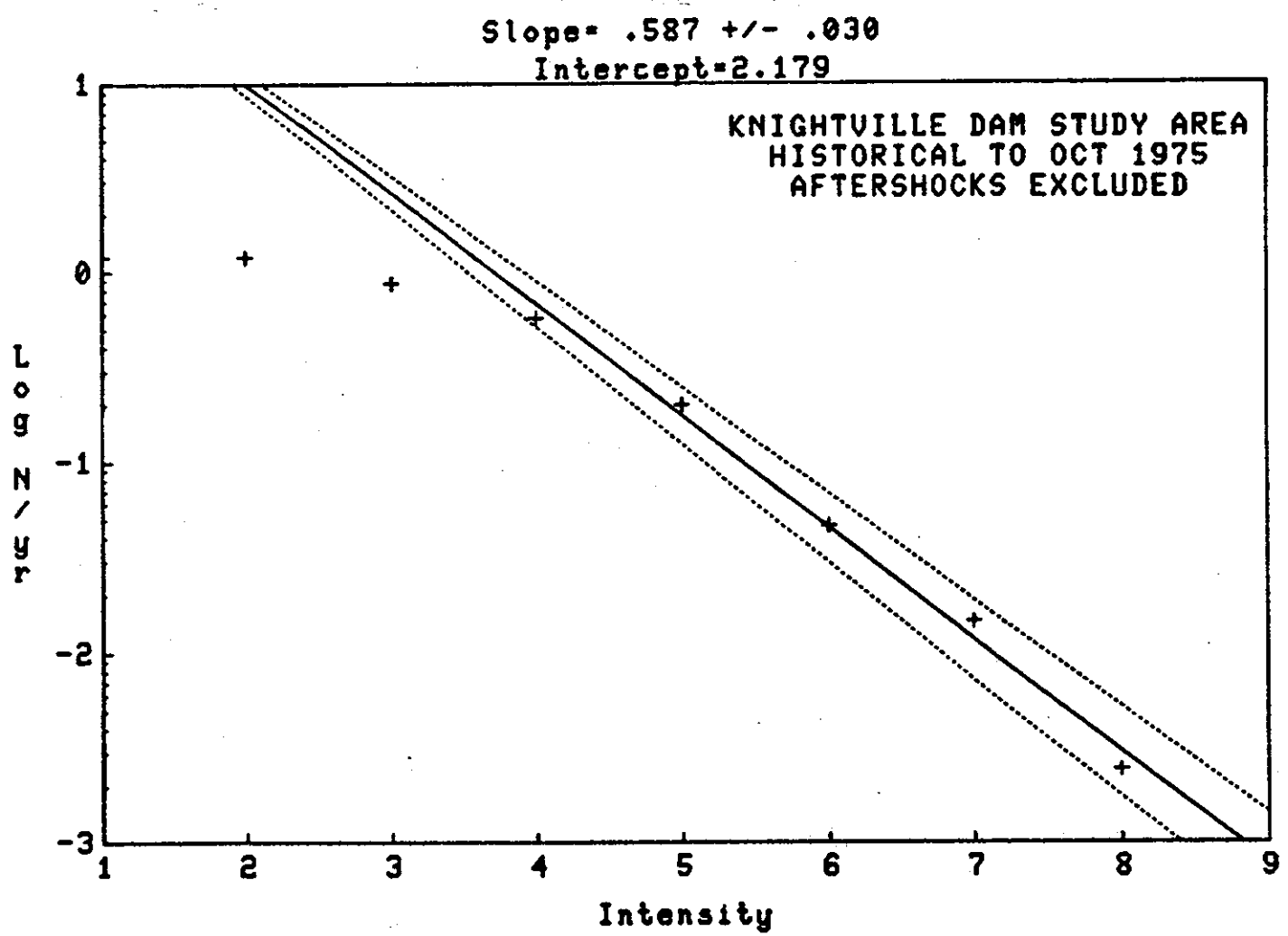

Slope= .587 +/- .030
Intercept=2.179
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
AFTERSHOCKS EXCLUDED
Log N/yr
1
0
-1
-2
-3
1
2
3
4
5
6
7
8
9
Intensity

Figure 9

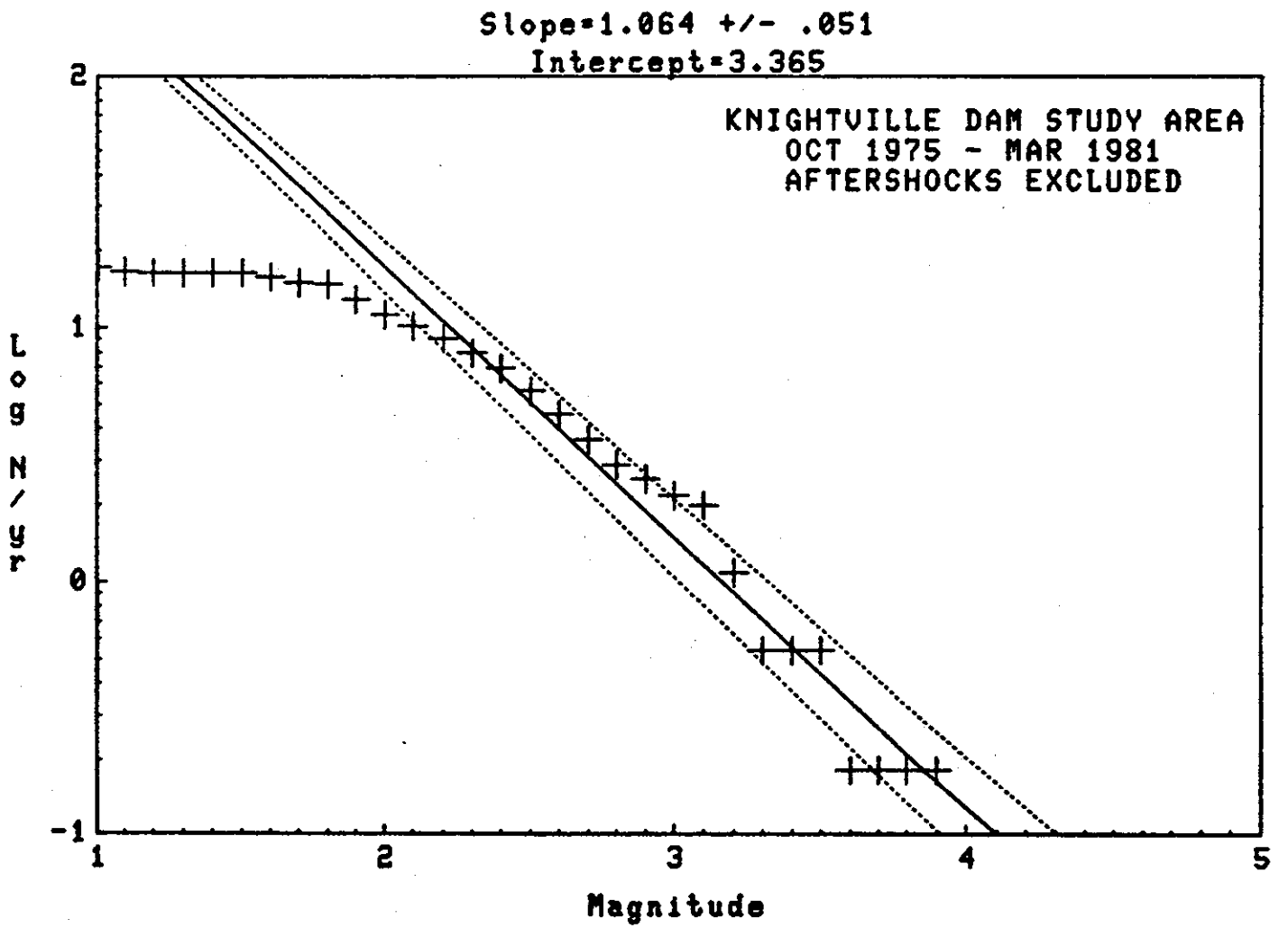

Slope=1.064 +/- .051
Intercept=3.365
KNIGHTVILLE DAM STUDY AREA
OCT 1975 - MAR 1981
AFTERSHOCKS EXCLUDED
Log N/yr
2
1
0
-1
1
2
3
4
5
Magnitude

Figure 10

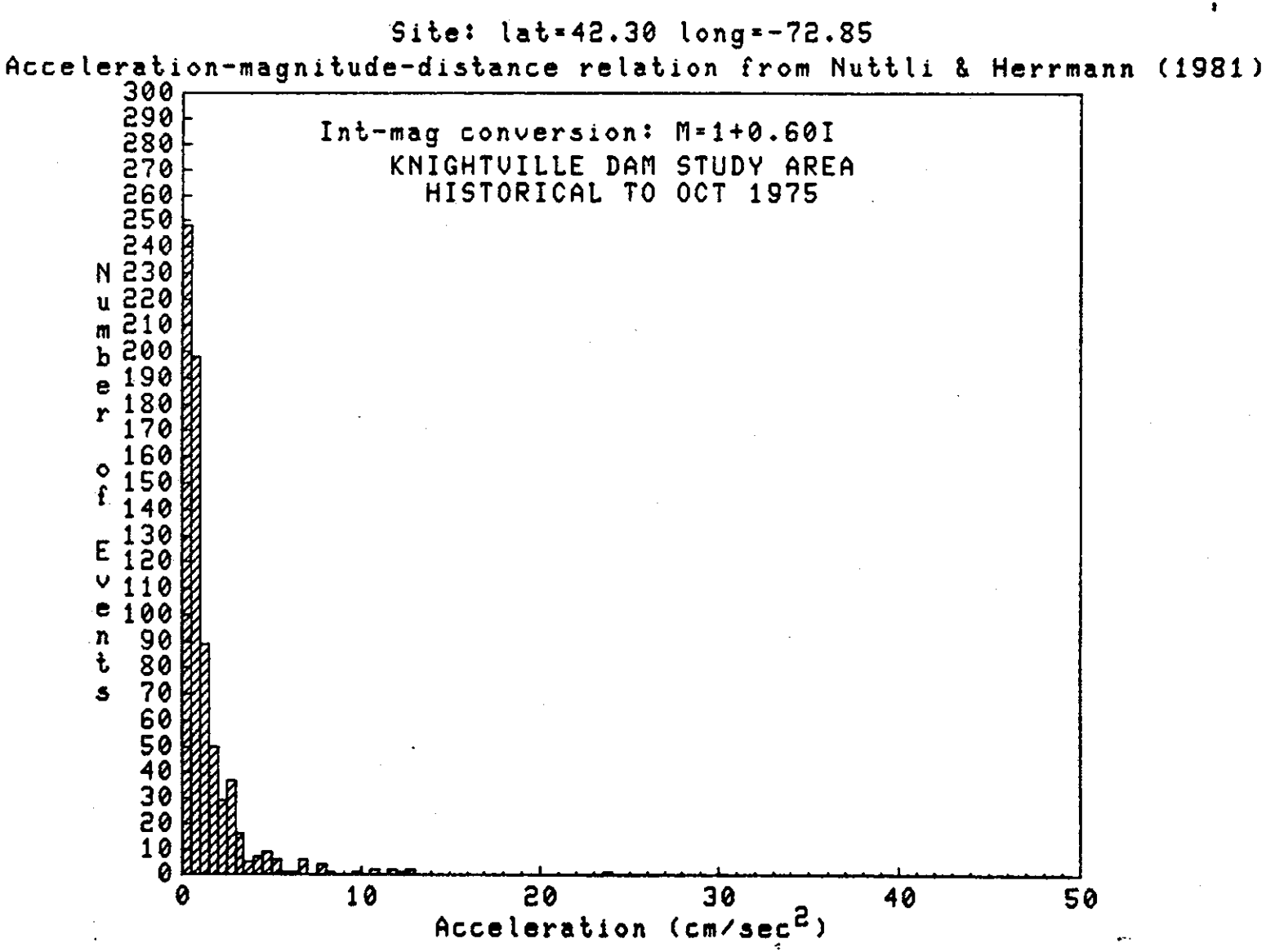

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Nuttli & Herrmann (1981)
Int-mag conversion: M=1+0.60I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
300
290
280
270
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 11a

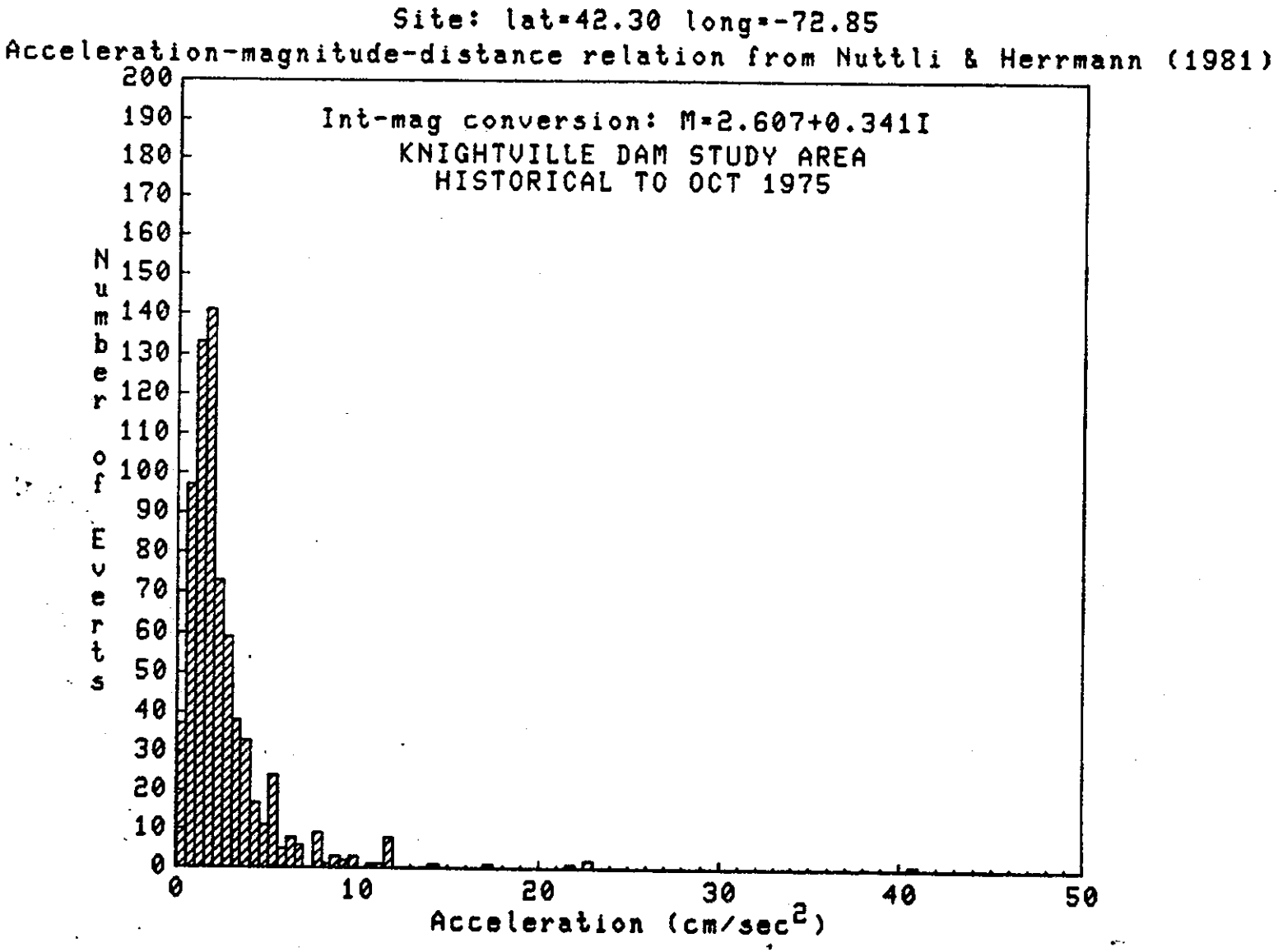

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Nuttli & Herrmann (1981)
Int-mag conversion: M=2.607+0.341I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
Acceleration (cm/sec2)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200

Figure 11b

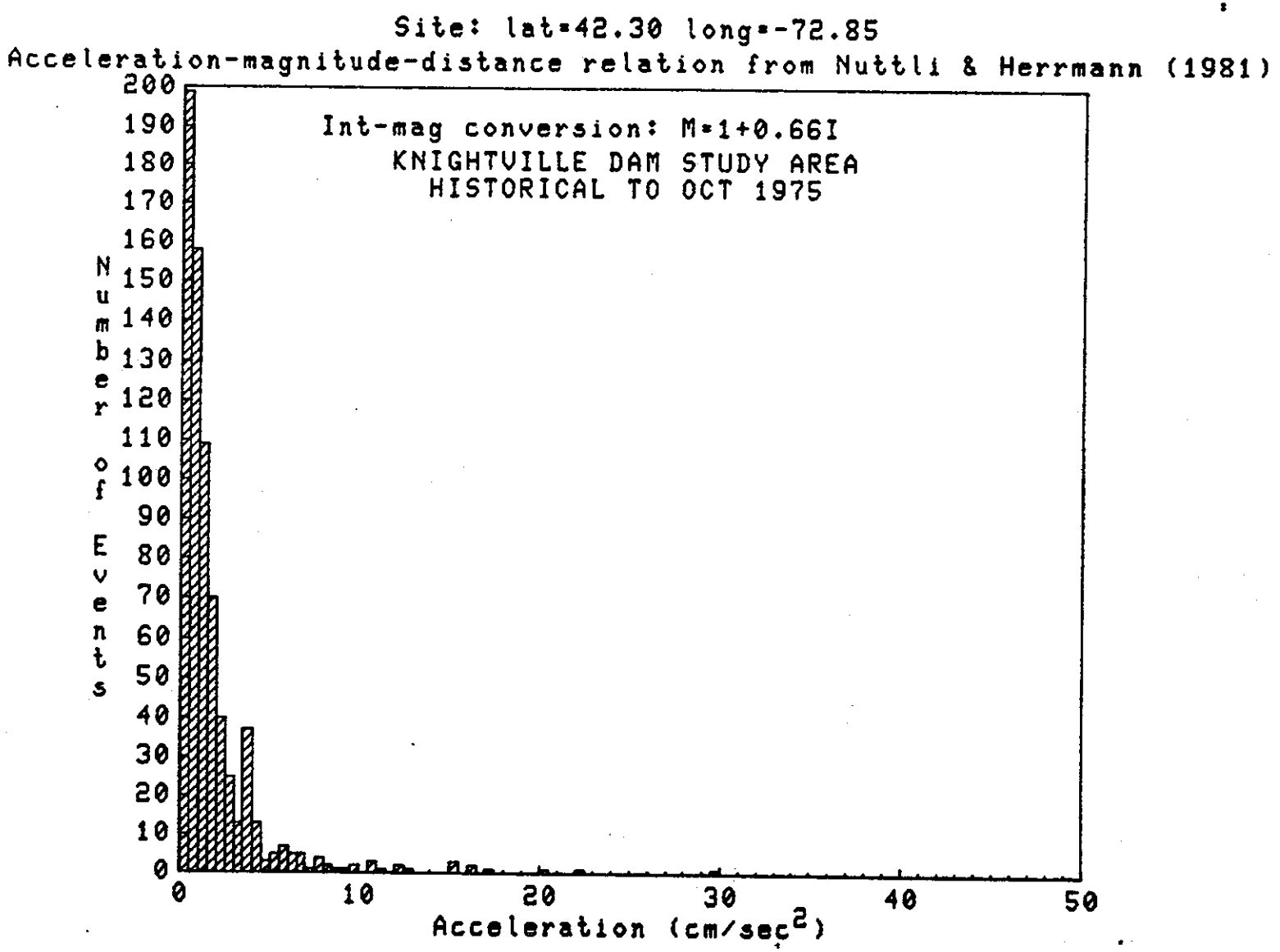

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Nuttli & Herrmann (1981)
Int-mag conversion: M=1+0.66I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 11c

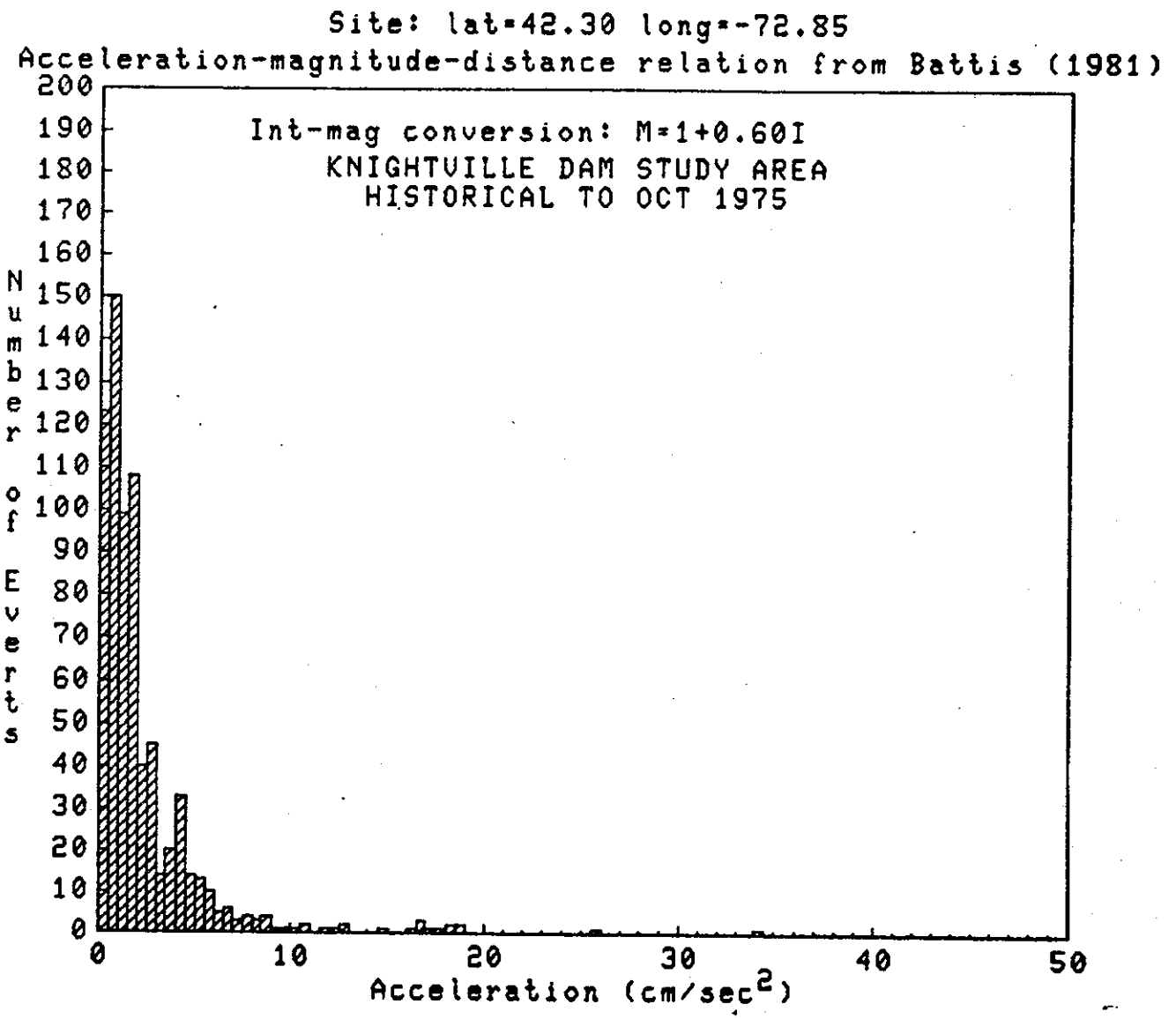

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Battis (1981)
Int-mag conversion: M=1+0.60I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 12a

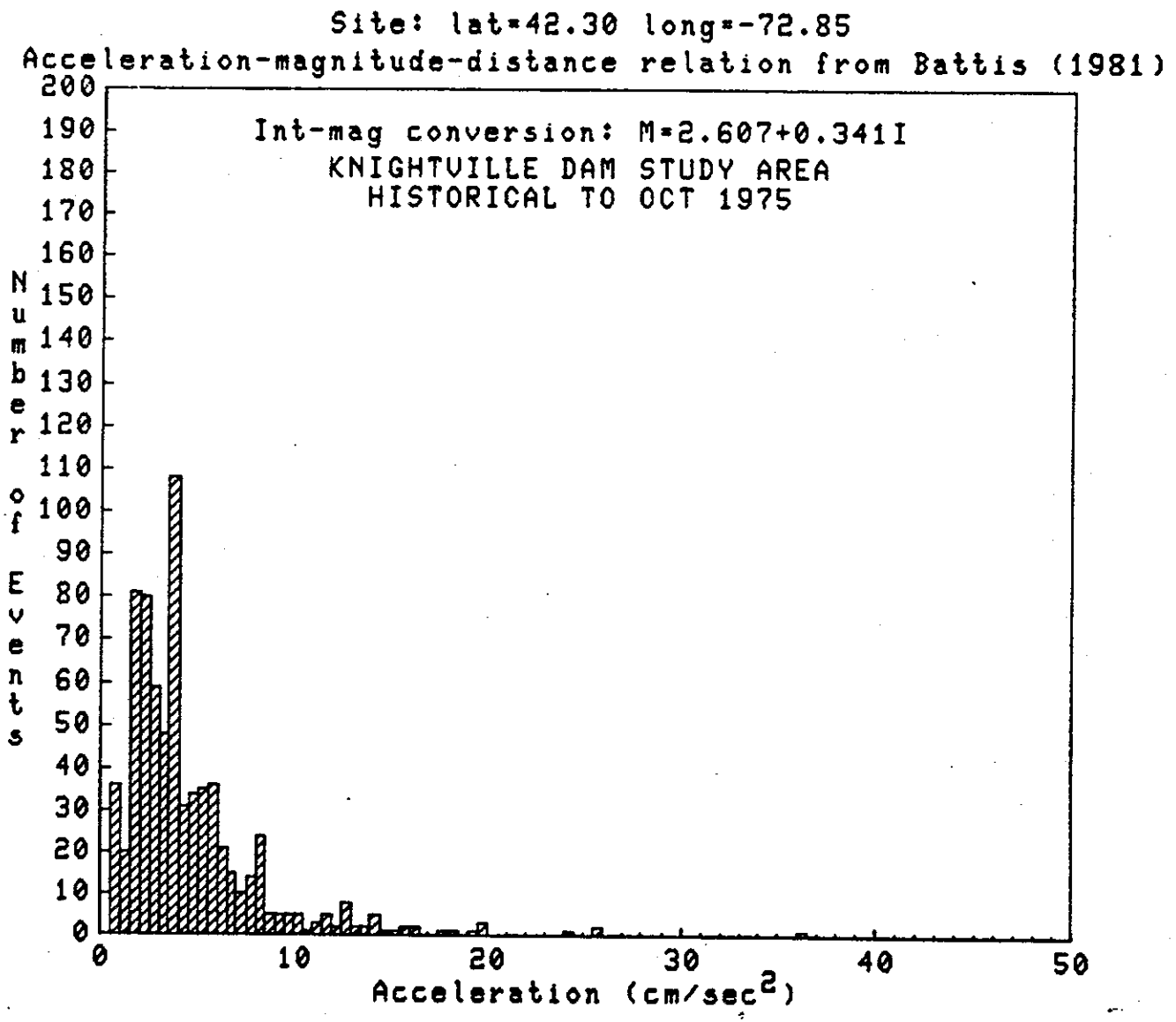

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Battis (1981)
Int-mag conversion: M=2.607+0.341I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 12b

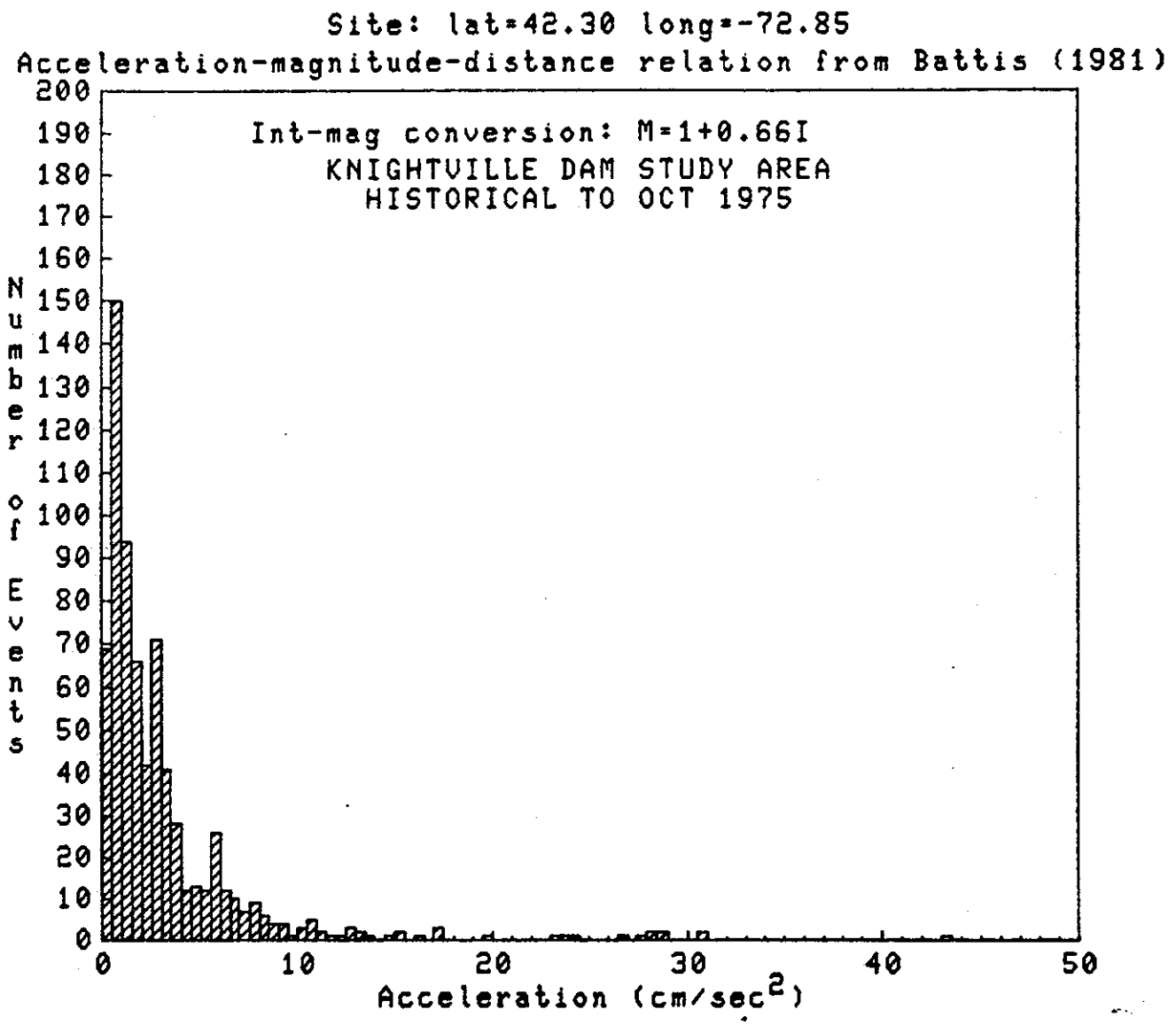

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Battis (1981)
Int-mag conversion: M=1+0.66I
KNIGHTVILLE DAM STUDY AREA
HISTORICAL TO OCT 1975
Number of Events
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 12c

Site: lat=42.30 long=-72.85

Acceleration-magnitude-distance relation from Nuttli & Herrmann (1981)

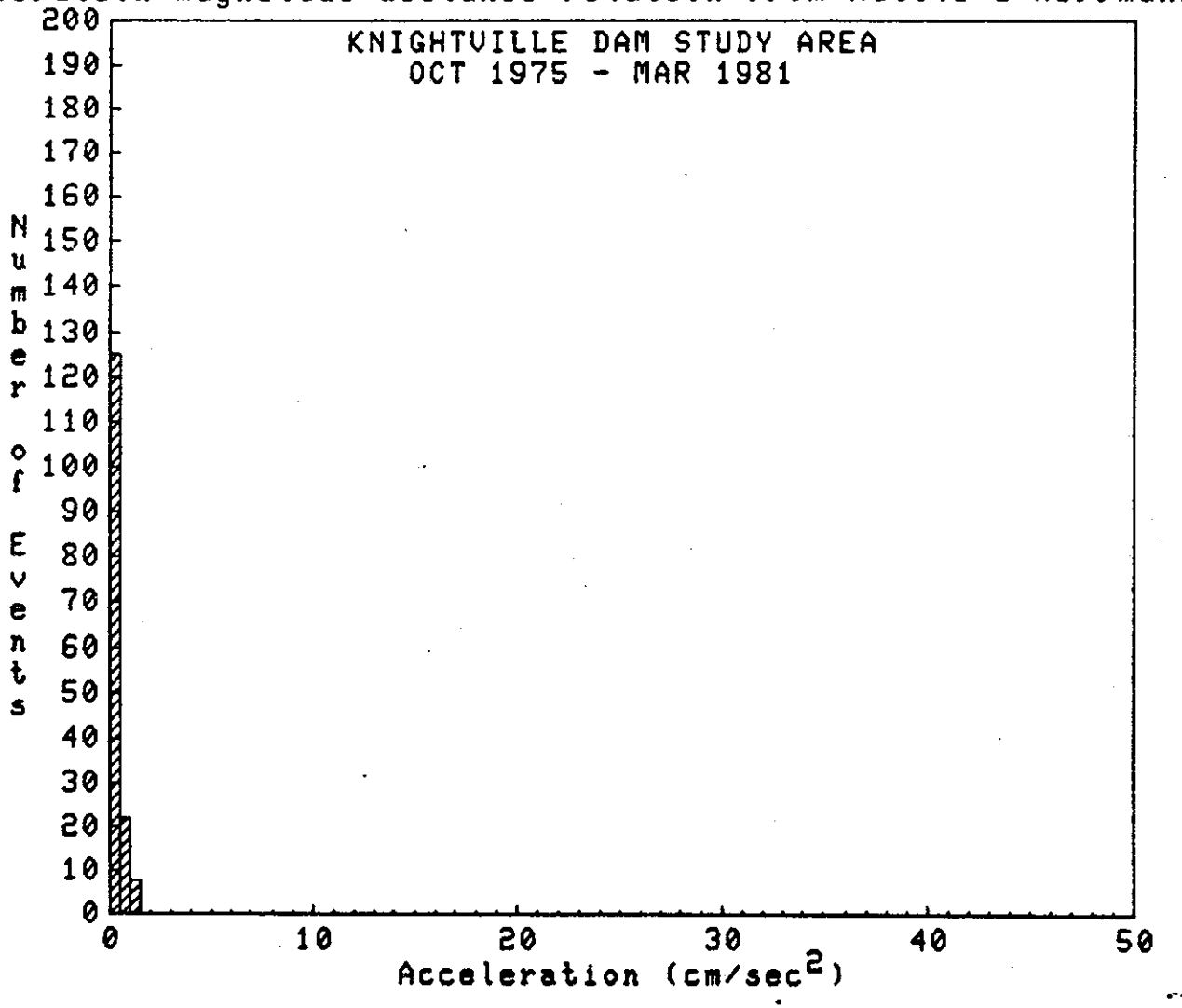


Figure 13

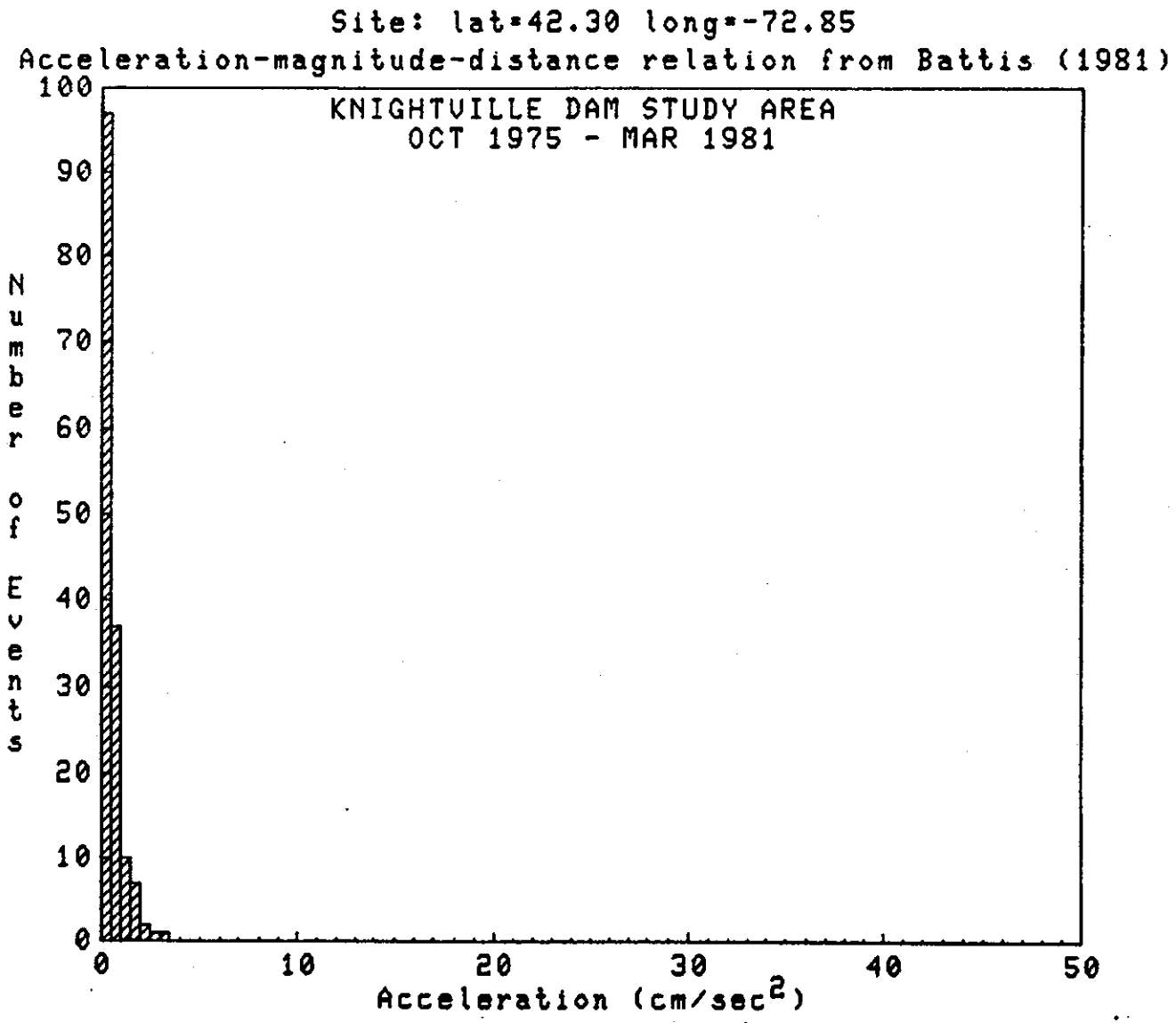

Site: lat=42.30 long=-72.85
Acceleration-magnitude-distance relation from Battis (1981)
KNIGHTVILLE DAM STUDY AREA
OCT 1975 - MAR 1981
Number of Events
0
10
20
30
40
50
60
70
80
90
100
0
10
20
30
40
50
Acceleration (cm/sec2)

Figure 14

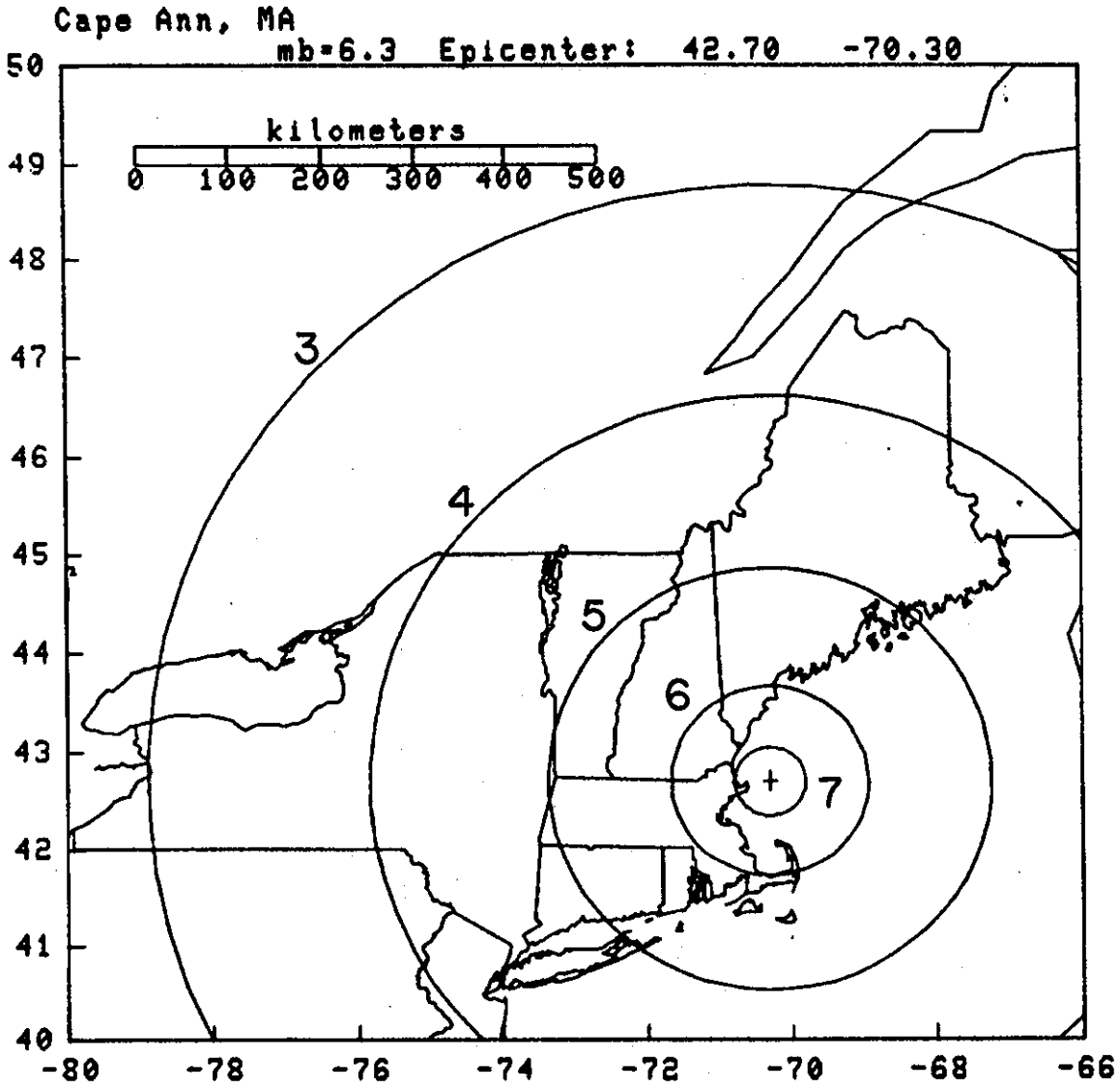

Cape Ann, MA
mb=6.3 Epicenter: 42.70 -70.30
kilometers
0 100 200 300 400 500
3
4
5
6
7
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 15

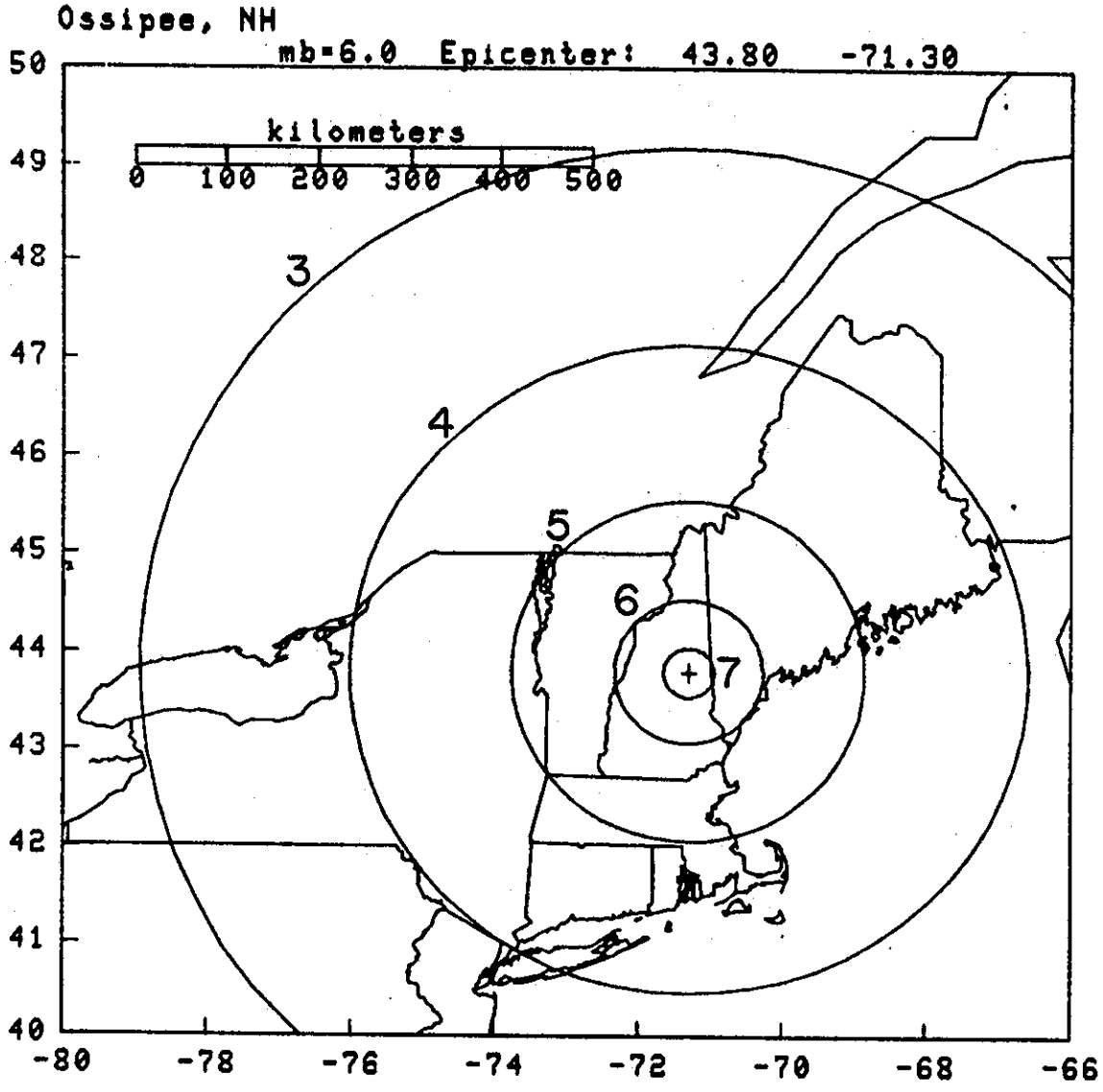

Ossipee, NH
mb=6.0 Epicenter: 43.80 -71.30
kilometers
0 100 200 300 400 500
3
4
5
6
7
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 16

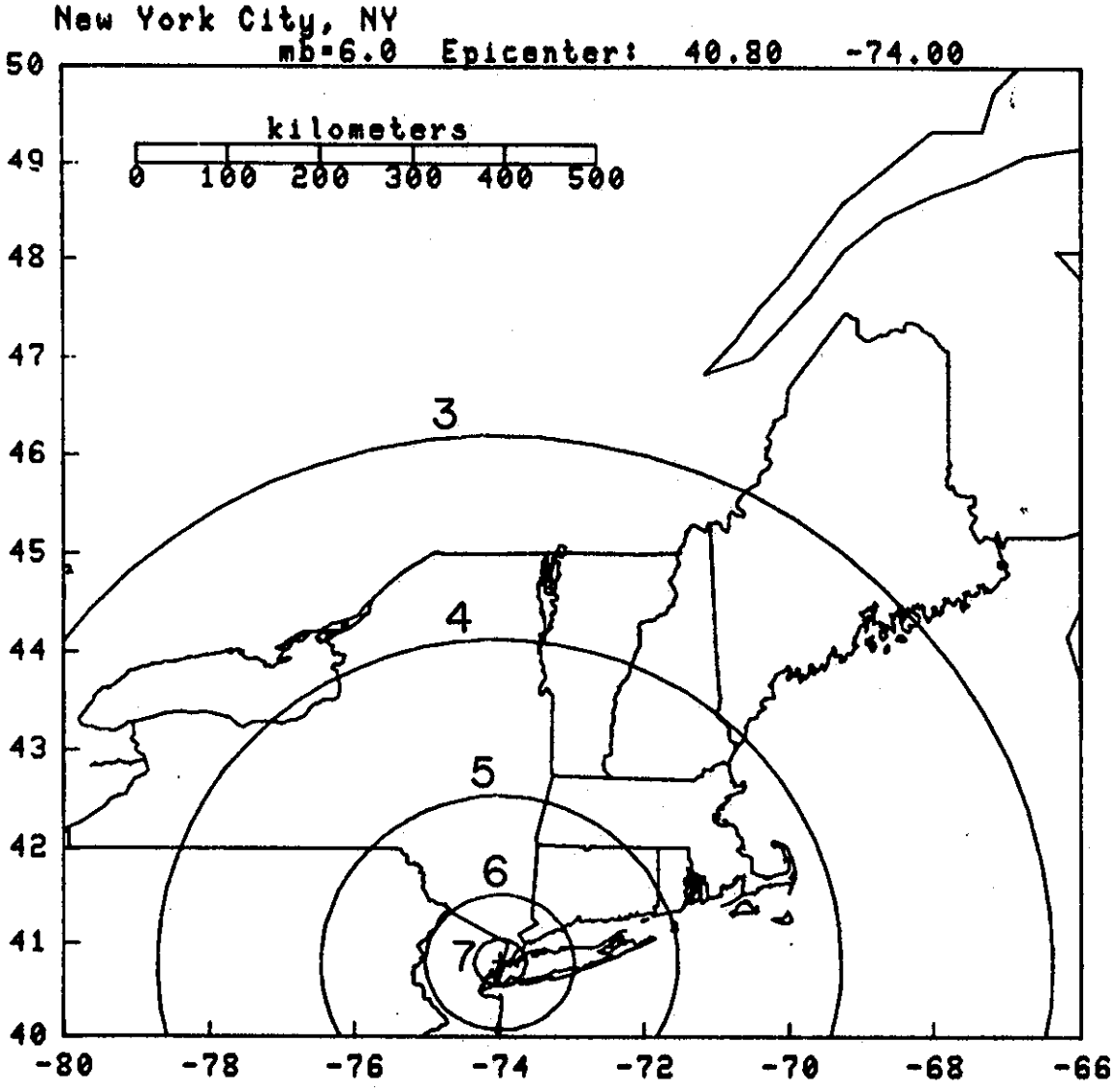

New York City, NY
mb=6.0 Epicenter: 40.80 -74.00
kilometers
0 100 200 300 400 500
3
4
5
6
7
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 17

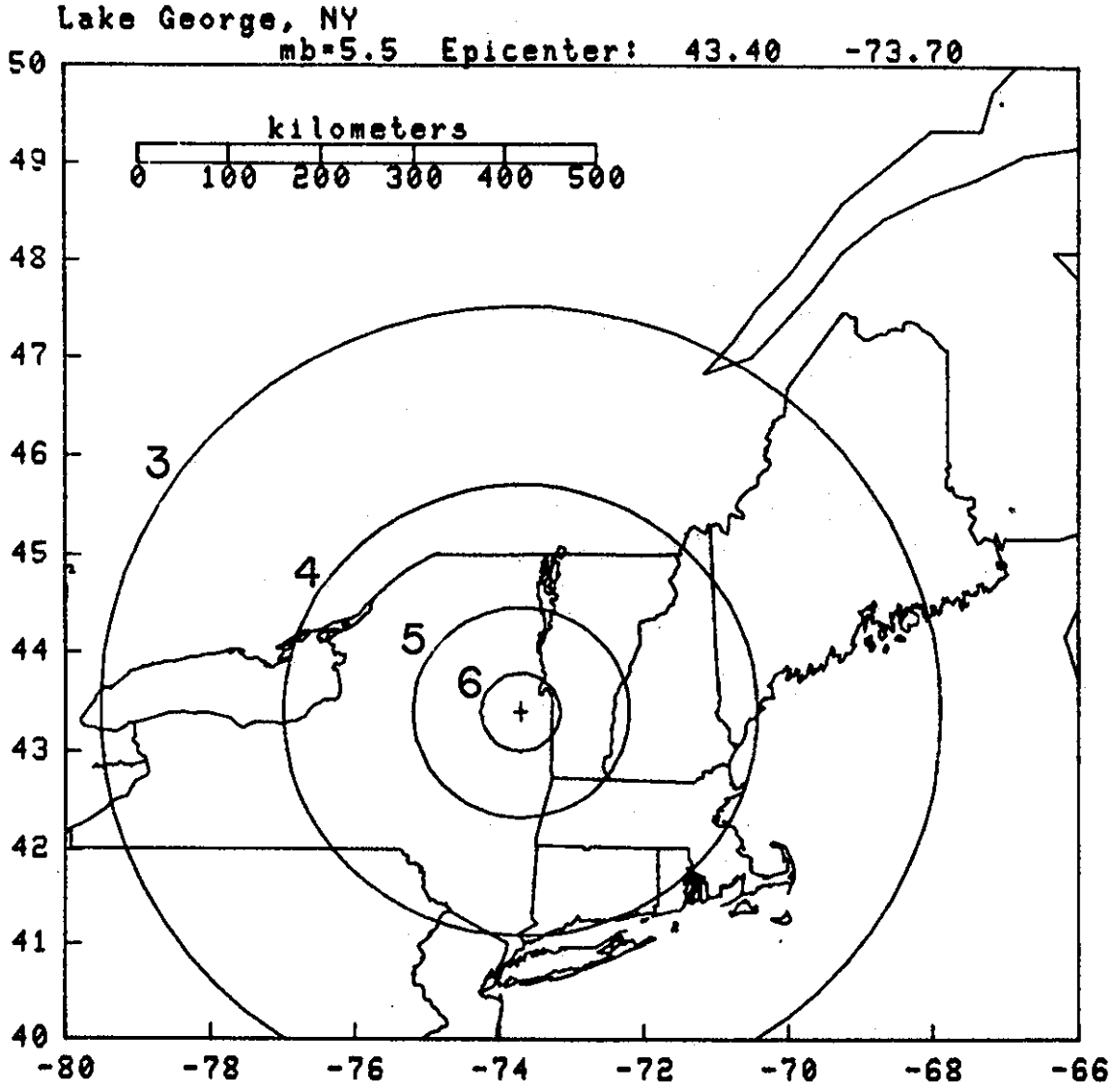

Lake George, NY
mb=5.5 Epicenter: 43.40 -73.70
kilometers
0 100 200 300 400 500
3
4
5
6
50
49
48
47
46
45
44
43
42
41
40
-80
-78
-76
-74
-72
-70
-68
-66

Figure 18

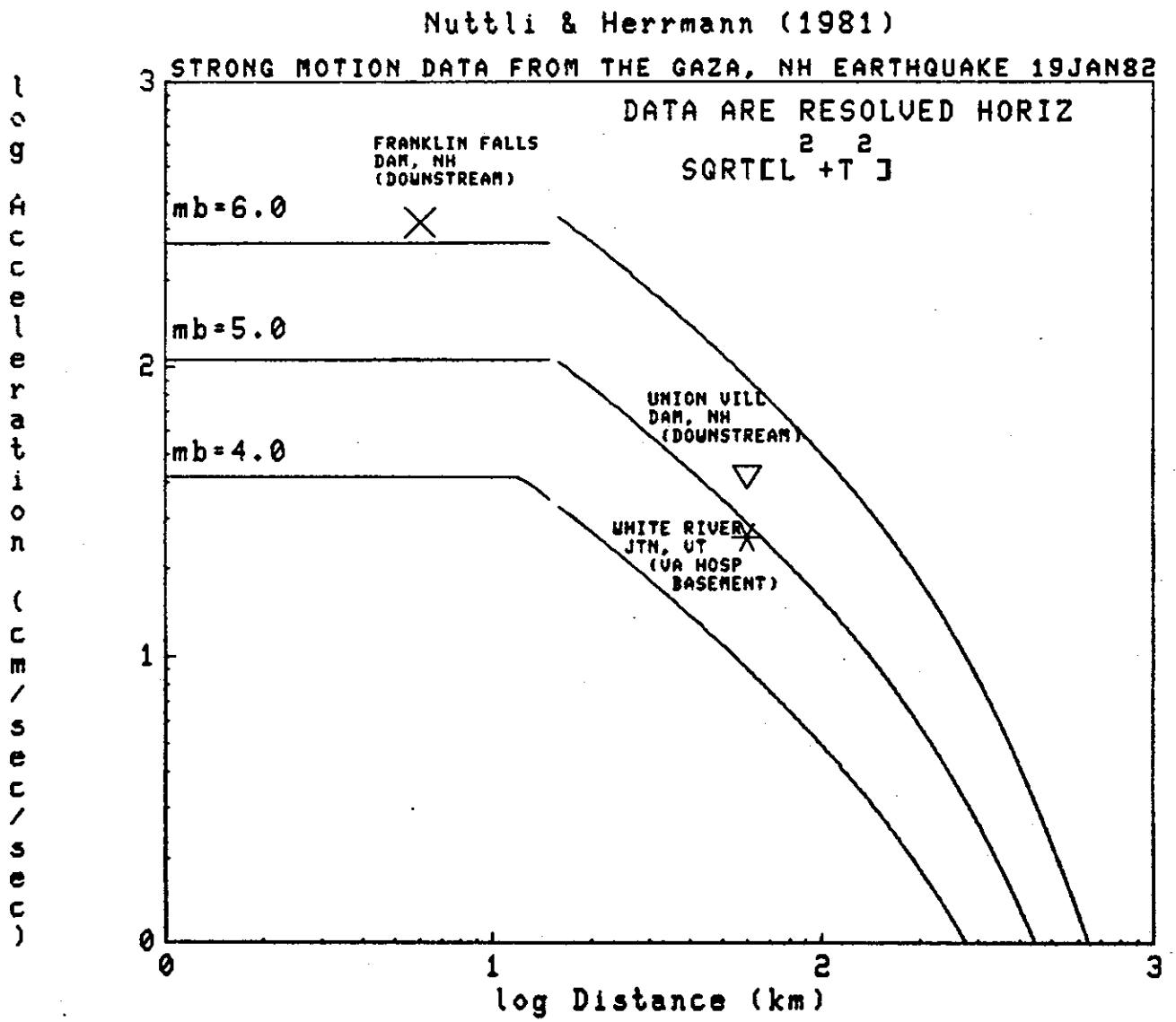

Nuttli & Herrmann (1981)
STRONG MOTION DATA FROM THE GAZA, NH EARTHQUAKE 19JAN82
DATA ARE RESOLVED HORIZ
SQRT[L2 +T2]
FRANKLIN FALLS DAM, NH (DOWNSTREAM)
mb=6.0
mb=5.0
mb=4.0
UNION VILL DAM, NH (DOWNSTREAM)
WHITE RIVER JTN, VT (VA HOSP BASEMENT)
log Acceleration (cm/sec/sec)
log Distance (km)
0
1
2
3

Figure 19

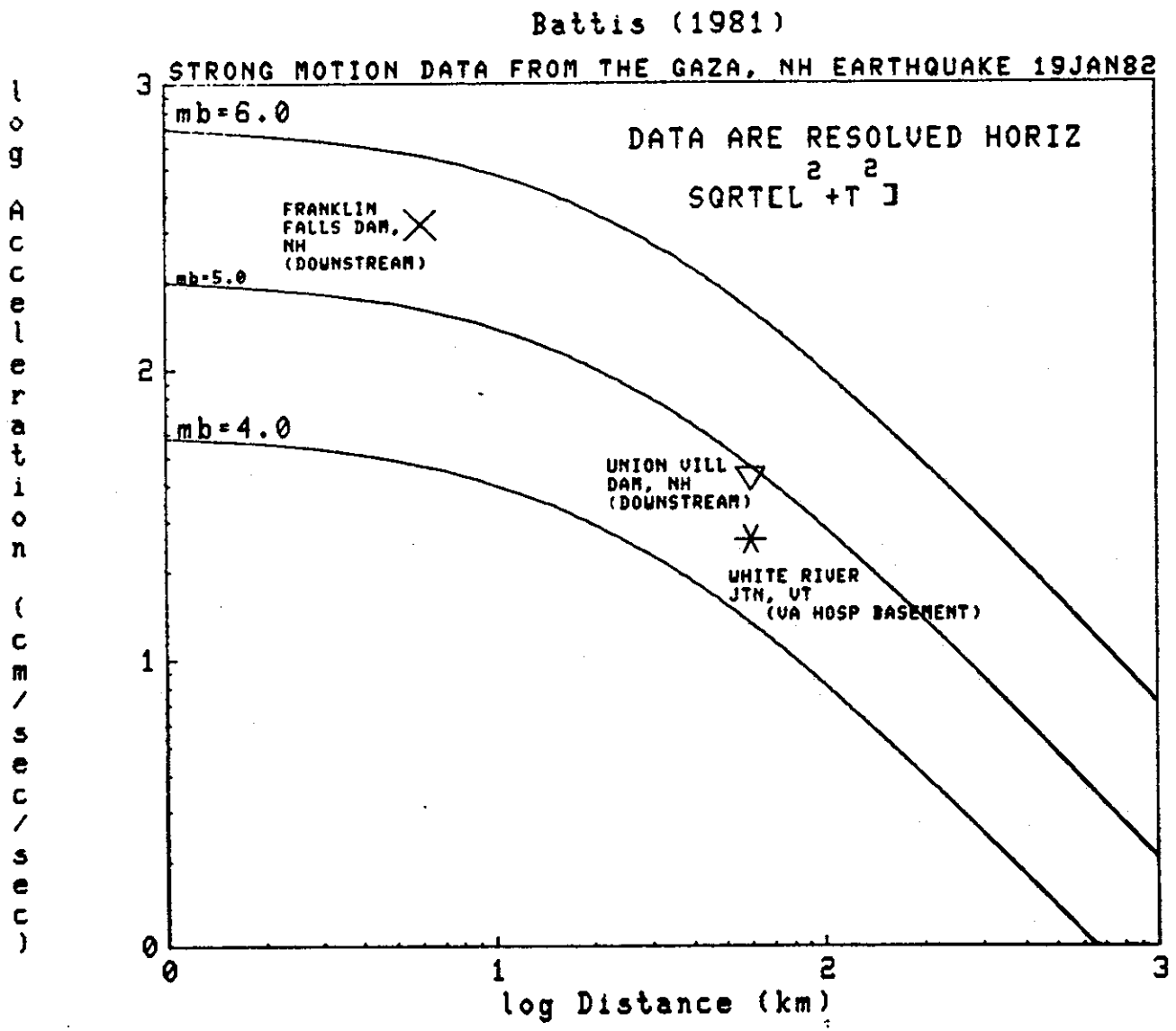

Battis (1981)
STRONG MOTION DATA FROM THE GAZA, NH EARTHQUAKE 19JAN82
DATA ARE RESOLVED HORIZ
SQRT[L2 +T2]
mb=6.0
mb=5.0
mb=4.0
FRANKLIN FALLS DAM, NH (DOWNSTREAM)
UNION VILL DAM, NH (DOWNSTREAM)
WHITE RIVER JTN, VT (VA HOSP BASEMENT)
log Acceleration (cm/sec/sec)
log Distance (km)
0
1
2
3

Figure 20

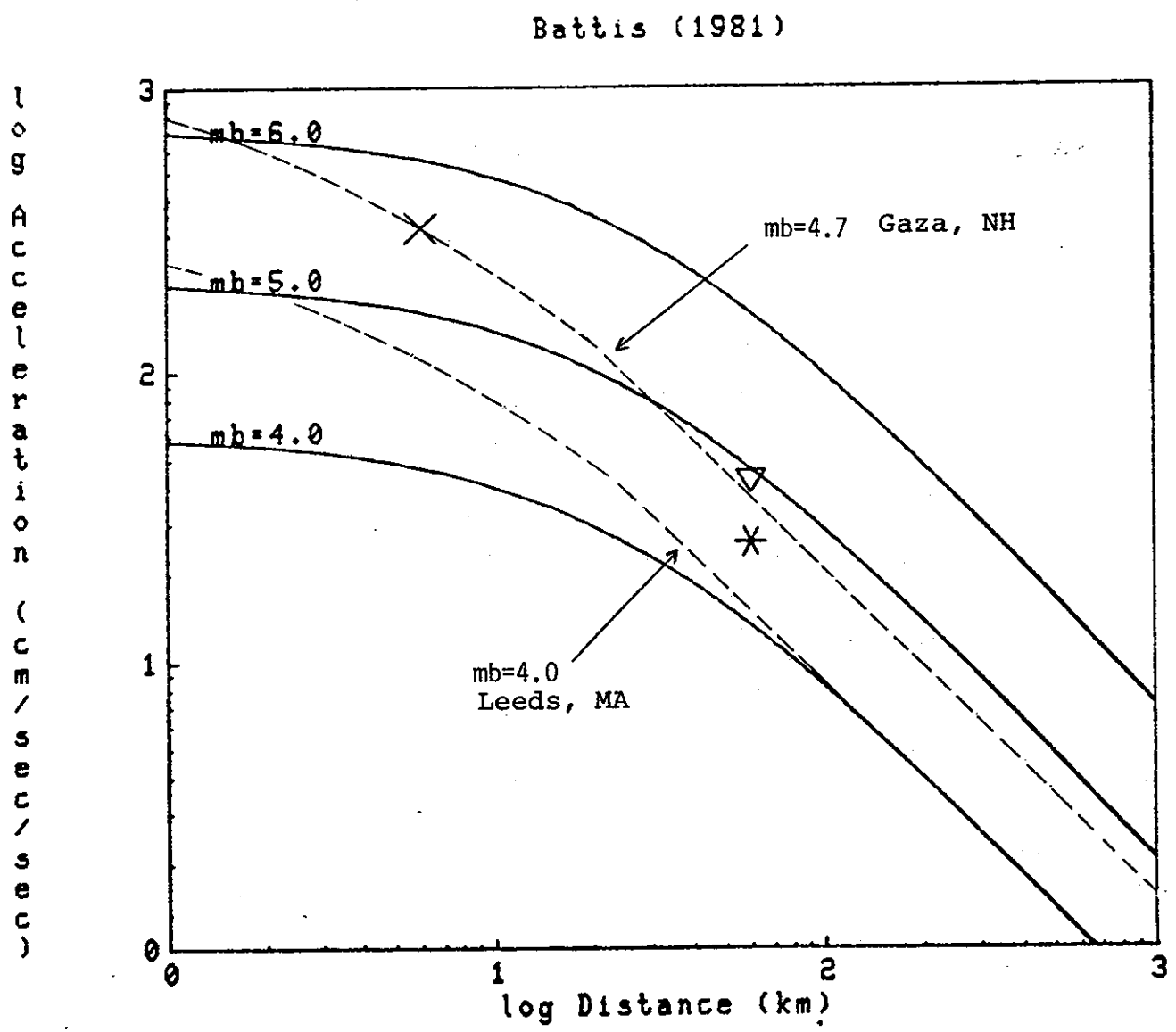

Battis (1981)
log Acceleration (cm/sec/sec)
log Distance (km)
mb=6.0
mb=5.0
mb=4.0
mb=4.7 Gaza, NH
mb=4.0
Leeds, MA
0
1
2
3

Figure 21

APPENDIX A

List of historical earthquake epicenters in the Knightville Dam STUDY AREA. The time period covered in 1568 through October 1975.

Code:	Column 1	Date in Month-Day-Year
	Column 2	Origin Time in Hr-Min-Sec (UTC)
	Column 3	Latitude
	Column 4	Longitude
	Column 5	Magnitude if known
	Column 6	Intensity (M.M.)
	Column 7	X indicates aftershock
	Column 8	State
	Column 9	City or local area

1568		41.50	-72.50	0.0	6 CT MOODUS-E.HADDAM
1574		41.50	-72.50	0.0	5 CT MOODUS-E.HADDAM
1584		41.50	-72.50	0.0	5 CT MOODUS-E.HADDAM
1592		41.50	-72.50	0.0	5 CT MOODUS-E.HADDAM
1627		42.60	-70.80	0.0	6 MA ESSEX
07011638		42.50	-70.90	0.0	3 MA SALEM
01251639		42.50	-70.90	0.0	3 MA LYNN
03151643	1200	42.80	-70.80	0.0	4 MA NEWBURY
06111643	1800	42.80	-70.80	0.0	4 MA NEWBURY
03141644		41.90	-70.60	0.0	2 MA PLYMOUTH
11081653		42.60	-70.90	0.0	4 MA DANVERS
04141658		42.50	-70.90	0.0	5 MA LYNN
02051662	2300	41.90	-70.60	0.0	2 MA PLYMOUTH
04031668	1400	42.30	-71.10	0.0	4 MA BOSTON
06261668		42.30	-71.10	0.0	2 MA ROXBURY
11301669		42.30	-71.10	0.0	2 MA BOSTON
1670		42.30	-71.10	0.0	2 MA BOSTON
12131677		41.10	-73.50	0.0	4 CT STAMFORD
02181685		42.70	-70.80	0.0	4 MA DANVERS
09071688		41.70	-72.90	0.0	2 CT N.BRISTOL
1698		41.40	-73.50	0.0	4 CT DANBURY
02101701		42.60	-70.90	0.0	3 MA DANVERS
03081701		42.60	-70.90	0.0	3XMA DANVERS
1702		41.40	-73.50	0.0	4 CT DANBURY
06271705		42.40	-71.10	0.0	4 MA BOSTON
1706		42.30	-71.10	0.0	2 MA BOSTON
1711		41.40	-73.50	0.0	4 CT DANBURY
01191721		42.30	-71.10	0.0	2 MA BOSTON
06231724		42.30	-71.10	0.0	2 MA BOSTON
11101727	0340	42.80	-70.60	0.0	7 MA CAPE ANN
11101727	0435	42.80	-70.60	0.0	4XMA CAPE ANN
11101727	0715	42.80	-70.60	0.0	4XMA CAPE ANN
11141727	2200	42.80	-70.60	0.0	5XMA CAPE ANN
11181727	1620	42.80	-70.60	0.0	4XMA CAPE ANN
11231727	2030	42.80	-70.60	0.0	2XMA CAPE ANN
11231727	2130	42.80	-70.60	0.0	2XMA CAPE ANN
11241727	1000	42.80	-70.60	0.0	4XMA CAPE ANN

12011727		42.80	-70.60	0.0	4XMA CAPE ANN
12161727		42.80	-70.60	0.0	4XMA CAPE ANN
12191727	1500	42.80	-70.60	0.0	4XMA CAPE ANN
12291727	0330	42.80	-70.60	0.0	4XMA CAPE ANN
12291727	0900	42.80	-70.60	0.0	2XMA CAPE ANN
01051728	0300	42.80	-70.60	0.0	4XMA CAPE ANN
01121728		43.60	-71.70	0.0	3 NH NEW HAMPTON
01151728	0200	42.80	-70.60	0.0	3XMA CAPE ANN
01181728	0200	42.80	-70.60	0.0	4XMA CAPE ANN
02051728	0230	42.80	-70.60	0.0	4XMA CAPE ANN
02081728	1130	42.80	-70.60	0.0	4XMA CAPE ANN
02091728		42.80	-70.60	0.0	2XMA CAPE ANN
02101728	2030	42.80	-70.60	0.0	5XMA CAPE ANN
03031728	0530	42.80	-70.60	0.0	3XMA CAPE ANN
03111728		42.80	-70.60	0.0	3XMA CAPE ANN
03281728	0800	42.80	-70.60	0.0	3XMA CAPE ANN
03311728	1840	42.80	-70.60	0.0	3XMA CAPE ANN
04011728	0200	42.80	-70.60	0.0	2XMA CAPE ANN
05091728	2200	42.80	-70.60	0.0	2XMA CAPE ANN
05161728		42.80	-70.60	0.0	4XMA CAPE ANN
05241728	0240	42.80	-70.60	0.0	4XMA CAPE ANN
05291728	0100	42.80	-70.60	0.0	4XMA CAPE ANN
06021728	1500	42.80	-70.60	0.0	4XMA CAPE ANN
06051728	0400	42.80	-70.60	0.0	3XMA CAPE ANN
06171728	0800	42.80	-70.60	0.0	2XMA CAPE ANN
06191728	0800	42.80	-70.60	0.0	2XMA CAPE ANN
06221728	1400	42.80	-70.60	0.0	2XMA CAPE ANN
07141728	0700	42.80	-70.60	0.0	2XMA CAPE ANN
07301728	1500	42.80	-70.60	0.0	4XMA CAPE ANN
08021728	0315	42.80	-70.60	0.0	4XMA CAPE ANN
09251728		42.80	-70.60	0.0	2XMA CAPE ANN
02101729	1400	42.80	-70.60	0.0	5XMA CAPE ANN
03301729		41.40	-73.50	0.0	2 CT DANBURY
03301729	1900	42.80	-70.60	0.0	4XMA CAPE ANN
08061729		41.40	-73.50	0.0	4 CT DANBURY
09191729	2030	42.80	-70.60	0.0	4XMA CAPE ANN
10101729	2130	42.80	-70.60	0.0	4XMA CAPE ANN
11101729	0340	42.80	-70.60	0.0	3XMA CAPE ANN

11251729	1300	42.80	-70.60	0.0	4XMA CAPE ANN
12091729	0100	42.80	-70.60	0.0	4XMA CAPE ANN
02201730	0100	42.80	-70.60	0.0	4XMA CAPE ANN
02201730	0500	42.80	-70.60	0.0	4XMA CAPE ANN
03091730	1845	42.80	-70.60	0.0	4XMA CAPE ANN
04241730	0100	42.80	-70.60	0.0	4XMA CAPE ANN
08081730	1400	42.80	-70.60	0.0	3XMA CAPE ANN
08261730	1300	42.80	-70.60	0.0	3XMA CAPE ANN
11171730		42.80	-70.60	0.0	3XMA CAPE ANN
11251730	1400	42.80	-70.60	0.0	2XMA CAPE ANN
12071730	0120	42.80	-70.60	0.0	4XMA CAPE ANN
12181730	0345	42.80	-70.60	0.0	3XMA CAPE ANN
12221730	2345	42.80	-70.60	0.0	3XMA CAPE ANN
12241730	0330	42.80	-70.60	0.0	4XMA CAPE ANN
01131731	0000	42.80	-70.60	0.0	4XMA CAPE ANN
01191731	0000	42.80	-70.60	0.0	4XMA CAPE ANN
01231731	0500	42.80	-70.60	0.0	4XMA CAPE ANN
03181731	2200	42.80	-70.60	0.0	2XMA CAPE ANN
06061731	1400	42.80	-70.60	0.0	2XMA CAPE ANN
07161731	1000	42.80	-70.60	0.0	4XMA CAPE ANN
09031731	0200	42.80	-70.60	0.0	2XMA CAPE ANN
10131731	0400	42.80	-70.60	0.0	4XMA CAPE ANN
02191732	0000	42.80	-70.60	0.0	4XMA CAPE ANN
12 1732		42.80	-70.60	0.0	3XMA CAPE ANN
01101733	A.M.	42.80	-70.60	0.0	3 MA CAPE COD
03121733		42.80	-70.60	0.0	2XMA CAPE ANN
10101733		42.80	-70.60	0.0	2XMA CAPE ANN
10301733	P.M.	42.80	-70.60	0.0	2XMA CAPE ANN
06281734	0320	42.80	-70.60	0.0	2XMA CAPE ANN
07101734	2015	42.80	-70.60	0.0	2XMA CAPE ANN
10201734	1520	42.80	-70.60	0.0	3XMA CAPE ANN
11231734	0500	42.80	-70.60	0.0	4XMA CAPE ANN
11271734	1100	42.80	-70.60	0.0	3XMA CAPE ANN
02131736	2245	42.80	-70.60	0.0	4XMA CAPE ANN
04011736	1530	42.80	-70.60	0.0	2XMA CAPE ANN
07241736	1445	42.80	-70.60	0.0	3XMA CAPE ANN
10121736	0630	42.80	-70.60	0.0	4XMA CAPE ANN
11231736	0700	42.80	-70.60	0.0	4XMA CAPE ANN

11231736	1100	42.80	-70.60	0.0	3XMA CAPE ANN
02171737	2115	42.80	-70.60	0.0	4 MA CAPE ANN
09201737	1520	42.80	-70.60	0.0	4 MA CAPE ANN
12191737	0330	40.80	-74.00	0.0	7 NY NY CITY
08131739	0730	42.80	-70.60	0.0	4 MA CAPE ANN
12251740	1135	42.80	-70.60	0.0	2XMA CAPE ANN
01291741	0900	42.80	-70.60	0.0	2XMA CAPE ANN
02051741	2050	42.80	-70.60	0.0	4XMA CAPE ANN
06241741	1535	42.20	-71.20	0.0	3 MA BOSTON
12171741	1300	42.30	-71.20	0.0	4 MA BOSTON
06131744		42.30	-71.20	0.0	2 MA CAMBRIDGE
06141744		42.60	-70.90	0.0	2 MA CAPE ANN
06141744		42.60	-70.90	0.0	2 MA CAPE ANN
06141744	1515	42.50	-70.90	0.0	6 MA CAPE ANN
06141744	2200	42.50	-70.90	0.0	4XMA SALEM
06151744		42.60	-70.90	0.0	2XMA CAPE ANN
07011744		42.50	-70.90	0.0	4 MA SALEM
07091744		42.50	-70.90	0.0	3XMA SALEM
12231744	1700	42.80	-70.60	0.0	2XMA CAPE ANN
01031745	1700	42.80	-70.90	0.0	3 MA NEWBURY
06121745		42.30	-71.10	0.0	2 MA BOSTON
02031746	0200	42.30	-71.10	0.0	2 MA BOSTON
02141746	0200	42.30	-71.10	0.0	3 MA BOSTON
08251747	A.M.	43.20	-70.90	0.0	3 NH DOVER
07211751	A.M.	43.20	-70.90	0.0	3 NH DOVER
11181755	0912	42.70	-70.30	0.0	8 MA OFF CAPE ANN
11181755	1029	42.70	-70.30	0.0	4XMA OFF CAPE ANN
11231755	0127	42.70	-70.30	0.0	5XMA OFF CAPE ANN
12201755	0115	42.70	-70.30	0.0	4XMA OFF CAPE ANN
01021756		42.30	-71.10	0.0	3 MA BOSTON
11161756	0900	42.30	-71.10	0.0	3 MA BOSTON
12051756	0300	42.30	-71.10	0.0	3XMA BOSTON
07081757	1915	42.30	-71.10	0.0	4 MA BOSTON
02021759	0700	42.30	-71.00	0.0	4 MA BOSTON
02031760		42.30	-71.10	0.0	2 MA BOSTON
11091760		42.30	-71.10	0.0	3 MA BOSTON
02 1761		42.30	-71.10	0.0	3XMA BOSTON
03121761	0715	42.50	-70.90	0.0	5 MA BOSTON

03161761		42.30	-71.10	0.0	4XMA BOSTON
11021761	0100	43.10	-71.50	0.0	4 NH S.OF CONCORD
01231766	1000	43.70	-70.30	0.0	4 ME PORTLAND
01241766		43.70	-70.30	0.0	2XME PORTLAND
06141766		42.70	-70.90	0.0	3 MA ESSEX
08251766		41.50	-71.30	0.0	4 RI NEWPORT
12171766	1148	43.10	-70.80	0.0	4 NH PORTSMOUTH
10191769	A.M.	43.70	-70.30	0.0	4 ME PORTLAND
10191769	1700	43.70	-70.30	0.0	4XME PORTLAND
02071776		41.70	-71.40	0.0	2 RI SOUTHERN
09141777		43.00	-71.50	0.0	2 NH MANCHESTER
11291780		42.50	-70.90	0.0	4 MA LYNN
11241783		41.00	-74.50	0.0	4XNJ MORRIS CO.
11301783	0200	41.00	-74.50	0.0	4XNJ MORRIS CO.
11301783	0350	41.00	-74.50	0.0	6 NJ MORRIS CO.
11301783	0700	41.00	-74.50	0.0	4XNJ MORRIS CO.
11291786	2100	42.40	-71.10	0.0	3 MA CAMBRIDGE
02251787	0600	42.40	-71.10	0.0	3XMA CAMBRIDGE
05161791	1300	41.50	-72.50	0.0	6 CT MOODUS-E.HADDAM
05191791	0300	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
01101792		42.50	-70.90	0.0	2 MA SALEM
08291792	0300	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
10241792	0600	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
01111793	1300	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
07061793	1100	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
03061794	1900	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
03071794	0400	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
03091794	1900	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
03101794	0400	41.50	-72.50	0.0	4XCT MOODUS-E.HADDAM
11111800		42.30	-71.10	0.0	3 MA BOSTON
12201800		43.70	-72.30	0.0	4 NH NW OF NEWPORT
12251800		41.90	-71.10	0.0	4 MA WAREHAM-TAUNTON
03011801	2030	43.10	-70.80	0.0	4 NH PORTSMOUTH
01181803	1450	42.50	-70.90	0.0	4 MA SALEM
02081804		42.50	-70.90	0.0	2 MA SALEM
05181804		40.80	-74.00	0.0	3 NY NY CITY
04061805	1915	42.50	-70.90	0.0	4 MA LYNN
04251805		42.50	-70.90	0.0	4 MA SALEM

05121805		42.80	-70.80	0.0	2	MA NEWBURY
08121805	0000	41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
12301805	1100	41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
01121807		42.30	-72.60	0.0	2	MA NORTHAMPTON
01141807	0400	43.00	-71.10	0.0	4	NH NEAR EXETER
02221807	1900	43.70	-70.50	0.0	3	ME WINDHAM
05061807	1800	43.50	-70.50	0.0	4	ME SACO RIVER
11101810	0215	43.00	-70.80	0.0	5	NH PORTSMOUTH
07 1811		41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
02091812	1400	41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
07051812	1300	41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
12281813	2100	41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
11291814	0014	43.70	-70.30	0.0	5	ME WINDHAM
09071817		42.50	-70.90	0.0	3	MA LYNN
10051817	1645	42.50	-71.20	0.0	6	MA WOBURN
03071823	1500	43.90	-70.00	0.0	4	ME BRUNSWICK
07231823	1155	42.90	-70.60	0.0	5	NH OFF HAMPTON
08231827		41.40	-72.70	0.0	4	CT NW OF NEW LONDON
07251828	1100	43.90	-70.00	0.0	4	ME BRUNSWICK
01011829		43.10	-70.80	0.0	4	NH PORTSMOUTH
08271829	0200	43.90	-70.00	0.0	3X	ME BRUNSWICK
08271829	0215	43.90	-70.00	0.0	3X	ME BRUNSWICK
12021830	0100	42.50	-70.90	0.0	3	MA LYNN
01151837	0700	42.50	-70.90	0.0	4	MA LYNN
04121837		41.70	-72.70	0.0	5	CT HARTFORD
01161840	2000	43.00	-75.00	0.0	6	NY HERKIMER
08091840	2030	41.50	-72.90	0.0	5	CT HARTFORD
01251841	A.M.	40.70	-74.00	0.0	3	NY NY CITY
10241843		41.10	-71.20	0.0	4	MA CANTON
06 1844	0100	41.50	-72.40	0.0	3	CT MOODUS-E.HADDAM
01011845		41.50	-72.40	0.0	3	CT MOODUS-E.HADDAM
10261845	2315	41.20	-73.30	0.0	5	CT BRIDGEPORT
11 1845		43.60	-72.30	0.0	4	NH LEBANON
05301846	1830	42.70	-70.30	0.0	4	MA CAPE ANN
07101846		43.10	-71.30	0.0	3	NH DEERFIELD
08251846	0945	42.50	-70.80	0.0	5	MA MARBLEHEAD
09121846	2330	43.10	-71.30	0.0	3	NH DEERFIELD
10301846	0200	43.10	-71.30	0.0	3X	NH DEERFIELD

10311846	P.M.	43.10	-71.30	0.0	3XNH DEERFIELD
11131846	0040	43.10	-71.30	0.0	3XNH DEERFIELD
12021846		43.10	-71.30	0.0	3XNH DEERFIELD
01121847	0430	42.60	-73.70	0.0	2 NY ALBANY
02021847		43.10	-71.30	0.0	3XNH DEERFIELD
02141847		43.10	-71.30	0.0	3XNH DEERFIELD
02211847		43.10	-71.30	0.0	3XNH DEERFIELD
04021847	0200	43.70	-70.70	0.0	3 ME LIMINGTON
07091847	A.M.	43.30	-73.70	0.0	3 NY GLENS FALLS
08081847	1500	41.70	-70.10	0.0	6 MA BREWSTER
09291847		40.50	-74.00	0.0	5 NY NY CITY
02041849		41.50	-71.60	0.0	3 RI NEWPORT
02151849		42.10	-72.60	0.0	3 MA SPRINGFIELD
10081849	P.M.	42.50	-71.40	0.0	4 MA MIDDLESEX CO.
07201850	P.M.	43.70	-70.30	0.0	3 ME PORTLAND
10121851	0230	43.10	-71.30	0.0	3 NH DEERFIELD
12251851	1245	44.00	-73.30	0.0	3 VT BRIDGEPORT
01101852	1140	41.20	-71.40	0.0	4 RI OFF COAST
06301852		43.40	-72.30	0.0	3 NH CLAREMONT
08011852		41.40	-72.10	0.0	3 CT GROTON
08111852	P.M.	43.10	-71.30	0.0	3 NH DEERFIELD
11281852	0445	43.00	-70.90	0.0	5 NH EXETER
06171853		43.70	-70.30	0.0	3 ME PORTLAND
06201853		43.70	-70.00	0.0	3XME PORTLAND
07171853	1030	43.50	-70.20	0.0	4XME PORTLAND
07201853	P.M.	43.70	-70.30	0.0	3XME PORTLAND
08171853		41.60	-70.90	0.0	3 MA NEW BEDFORD
09081853	0410	41.60	-70.90	0.0	3XMA NEW BEDFORD
11211853		43.00	-71.90	0.0	3 NH ANTRIM
11281853		43.00	-71.90	0.0	4XNH ANTRIM
01241854	1200	42.20	-72.30	0.0	3 MA PALMER
01271854	1200	42.20	-72.30	0.0	3XMA PALMER
02231854	0500	42.50	-71.10	0.0	3 MA READING
10011854		42.90	-72.30	0.0	2XNH KEENE
10251854	0300	42.90	-72.30	0.0	4 NH KEENE
12111854	1530	43.00	-70.80	0.0	5 NH NORTH HAMPTON
01161855	2300	44.00	-71.00	0.0	5 ME OTISFIELD
01171855		40.80	-73.60	0.0	2 NY NASSAU CO.

01171855	0020	44.00	-71.00	0.0	4XME OTISFIELD
01191855	1600	43.70	-70.30	0.0	3 ME PORTLAND
01201855	0100	43.70	-70.30	0.0	3XME PORTLAND
01231855	2000	42.60	-70.40	0.0	3 MA NEWBURY
02071855	0430	42.00	-74.00	0.0	6 NY HUDSON VALLEY
12171855	1900	43.30	-73.70	0.0	4 NY WARREN
03131856	0300	41.40	-72.60	0.0	4 CT HADDAM
06101856		43.10	-72.50	0.0	2 VT BELLOWS FALLS
07011857	0345	41.50	-72.50	0.0	4 CT MOODUS-E.HADDAM
06271858		41.40	-72.80	0.0	4XCT NORTH HAVEN
07011858	0345	41.30	-73.00	0.0	4 CT NEW HAVEN
03121860	A.M.	41.50	-72.50	0.0	3 CT MOODUS-E.HADDAM
03171860	0230	42.20	-70.50	0.0	4 MA OFF PROVINCETOWN
03171860	0315	42.20	-70.50	0.0	4XMA OFF PROVINCETOWN
03011861		42.40	-71.10	0.0	3 MA BOSTON
03051861	1700	40.70	-74.20	0.0	3 NJ NEWARK
02031862	0100	41.50	-72.50	0.0	4 CT MOODUS-E.HADDAM
02041862	1230	42.50	-71.20	0.0	3 MA CAMBRIDGE
10231870	1130	42.10	-72.60	0.0	3 MA SPRINGFIELD
07201871		43.20	-71.50	0.0	4 NH CONCORD
07111872	1025	40.90	-73.80	0.0	4 NY NEW ROCHELLE
11181872	1900	43.20	-71.60	0.0	5 NH CONCORD
01111873	1000	43.90	-70.00	0.0	3 ME BRUNSWICK
07161873	A.M.	42.30	-71.80	0.0	2 MA WORCESTER
10051873	0730	42.90	-71.30	0.0	3 NH DERRY
01061874		43.60	-71.20	0.0	4 NH WOLFEBORO
01251874	1700	42.60	-71.40	0.0	4 MA LOWELL
01261874	0700	43.00	-71.50	0.0	4 NH MANCHESTER
01261874	1000	43.00	-71.50	0.0	3XNH MANCHESTER
02121874	1130	43.50	-70.50	0.0	2 ME SACO
11241874		42.70	-70.90	0.0	4 MA SALEM-NEWBURY
12111874	0325	40.90	-73.80	0.0	5 NY NY CITY
12131874		40.90	-73.80	0.0	5XNY NY CITY
02091875		41.50	-72.00	0.0	2 CT PRESTON
05061875		43.60	-71.20	0.0	2 NH WOLFEBORO
05151875	1515	42.40	-71.10	0.0	2 MA CAMBRIDGE
07281875	0910	41.90	-73.00	0.0	5 CT NW OF TORRINGTON
09261875	0200	41.30	-73.30	0.0	2 CT STEPNEY

11011875	0218	42.40	-71.10	0.0	2	MA CAMBRIDGE
12011875	0900	42.90	-72.30	0.0	4	NH KEENE
12011875	1100	42.90	-72.30	0.0	2X	NH KEENE
01071876		43.30	-71.70	0.0	2	NH WARNER + CONTOOCOOK
09221876	0430	41.50	-71.30	0.0	5	RI NEWPORT
02181877	1920	43.70	-70.30	0.0	3	ME PORTLAND
04231877	1600	43.00	-71.30	0.0	2	NH AUBURN
05141877	P.M.	42.80	-73.90	0.0	2	NY SCHENECTADY
09101877	0700	42.40	-71.10	0.0	3	MA CAMBRIDGE
03121878		42.70	-71.60	0.0	2	VT MILFORD
10041878	0730	41.50	-74.00	0.0	5	NY HUDSON VALLEY
12251878	0200	40.80	-73.80	0.0	2	NY FLUSHING
12291878	0232	42.70	-74.30	0.0	3	NY SCHOHARIE
10241879	2312	41.30	-72.90	0.0	2	CT NEW HAVEN
10261879	0330	43.00	-71.50	0.0	4	NH MANCHESTER
11031879	1215	43.20	-71.70	0.0	2	NH CONTOOCOOK
03291880		43.40	-70.70	0.0	3	ME SANFORD
05121880	1245	42.70	-71.00	0.0	5	MA BOXFORD
07131880	0400	43.20	-71.60	0.0	2	NH CONCORD
07211880	0000	43.00	-71.50	0.0	3	NH MANCHESTER
08211880		43.20	-71.10	0.0	2	NH BARRINGTON
01211881	0240	44.00	-70.00	0.0	4	ME BATH
02021881	0900	42.30	-71.10	0.0	2	MA BOSTON
02031881	0900	42.00	-70.70	0.0	2	MA PLYMOUTH
02041881		43.00	-70.80	0.0	2	NH GREENLAND
02121881		43.00	-70.80	0.0	2	NH PORTSMOUTH
03191881	0230	42.80	-73.90	0.0	3	NY SCHENECTADY
04031881	0925	43.00	-71.90	0.0	3	NH ANTRIM
04211881	1630	40.90	-73.10	0.0	3	NY PORT JEFFERSON
05181881	0520	43.20	-71.70	0.0	3	NH CONTOOCOOK
05181881	0830	43.20	-71.70	0.0	3X	NH CONTOOCOOK
06191881	0825	42.80	-70.90	0.0	4	MA NEWBURY
08131881	A.M.	43.20	-71.70	0.0	3	NH CONTOOCOOK
10061881	0503	43.20	-71.60	0.0	4	NH BRISTOL
10311881	0640	43.20	-71.70	0.0	4	NH CONTOOCOOK
12161881	2100	42.30	-71.10	0.0	3	MA DORCHESTER
04021882	A.M.	43.00	-74.30	0.0	2	NY AMSTERDAM
04171882	1900	43.20	-71.70	0.0	4	NH HOPKINTON

05011882		41.60	-71.40	0.0	2	RI	E.GREENWICH
05081882	0900	43.20	-71.60	0.0	3	NH	CONCORD
12191882	2224	43.20	-71.40	0.0	5	NH	CONCORD
02041883	2005	43.60	-71.20	0.0	4	NH	WOLFEBORO
02281883	0330	41.50	-71.30	0.0	5	RI	NEWPORT
01181884	0700	43.20	-71.70	0.0	4	NH	CONTOOCOOK
08081884		41.30	-70.20	0.0	2	MA	NANTUCKET I.
08101884	1907	40.60	-74.00	0.0	7	NY	NY CITY
08111884		40.60	-74.00	0.0	5X	NY	NY CITY
10101884	A.M.	42.30	-71.10	0.0	2	MA	ROXBURY
10271884	0100	42.80	-71.40	0.0	2	NH	NASHUA
11131884	0050	43.20	-71.60	0.0	4	NH	CONCORD
11231884	1730	43.20	-71.70	0.0	5	NH	CONCORD
12041884	0518	42.30	-72.70	0.0	2	MA	NORTHAMPTON
12171884	0700	43.70	-71.50	0.0	4	NH	CENTER HARBOR
01031885	0700	43.50	-71.50	0.0	2	NH	LACONIA
01041885	1106	41.30	-73.90	0.0	3	NY	PEEKSKILL
01311885	1005	41.30	-73.80	0.0	3	NY	YORKTOWN
03181885	1700	43.20	-71.70	0.0	2	NH	CONTOOCOOK
04281885	2210	41.30	-72.70	0.0	3	CT	GUILFORD
01061886	0010	42.90	-71.50	0.0	4	NH	MERRIMACK
01091886		41.90	-73.10	0.0	2	CT	WINSTED
01171886	2214	42.80	-71.40	0.0	4	NH	NASHUA
01251886		41.60	-73.80	0.0	4	NY	HOPEWELL JCT
02031886		41.20	-73.20	0.0	2	CT	BRIDGEPORT
08031886		43.50	-71.50	0.0	2	NH	MAYFIELD
09031886		42.50	-73.40	0.0	2	NY	LEBANON SPRINGS
09051886		41.50	-72.50	0.0	4	CT	MOODUS-E.HADDAM
09091886		42.50	-73.40	0.0	2	NY	LEBANON SPRINGS
07011887	0200	43.20	-71.50	0.0	4	NH	CONCORD
01181888		43.20	-71.70	0.0	2	NH	CONTOOCOOK
01301888		41.70	-71.20	0.0	2	MA	FALL RIVER
03081889		43.50	-71.60	0.0	4	NH	FRANKLIN
04111889		43.00	-71.50	0.0	2	NH	MANCHESTER
08101889		43.40	-73.70	0.0	4	NY	LAKE GEORGE
03291890		43.20	-71.50	0.0	2	NH	CONCORD
01151891		42.60	-71.80	0.0	2	MA	FITCHBURG
05021891	0010	43.20	-71.60	0.0	5	NH	NEAR CONCORD

05301891	0000	43.10	-71.50	0.0	4XNH	NEAR CONCORD
05011892		43.20	-71.50	0.0	2 NH	CONCORD
03091893	0530	40.60	-74.00	0.0	5 NY	NY CITY
03141893		42.30	-72.70	0.0	4 MA	LEEDS
06251893		41.90	-70.90	0.0	2 MA	MIDDLEBORO
07011893		43.10	-71.90	0.0	2 NH	ANTRIM
07021893		42.90	-72.10	0.0	2 NH	DUBLIN
08021893		41.70	-70.90	0.0	2 MA	NEW BEDFORD
04101894	A.M.	41.60	-72.50	0.0	4 CT	MOODUS-E.HADDAM
09031894		43.20	-72.40	0.0	2 NH	ALSTEAD
11231894	1230	41.40	-72.10	0.0	3 CT	NEW LONDON
12171894		42.50	-73.80	0.0	4 NY	S.OF ALBANY
05281895	1615	43.00	-72.50	0.0	3 VT	PUTNEY
09011895	1109	40.70	-74.80	0.0	6 NJ	HIGH BRIDGE
07011897	0920	43.70	-71.60	0.0	4 NH	MEREDITH
09051897		41.50	-72.50	0.0	4 CT	MOODUS-E.HADDAM
06111898	0645	42.80	-72.60	0.0	4 VT	BRATTLEBORO-VERNON
07251898		43.30	-71.60	0.0	2 NH	CONCORD-CANTERBURY
05161899		40.90	-74.00	0.0	2 NY	NY CITY
05171899	0115	41.60	-72.60	0.0	5 CT	MOODUS-E.HADDAM
04031900		41.70	-70.90	0.0	2 MA	NEW BEDFORD
03091901		43.20	-71.50	0.0	2 NH	CONCORD
05271902		40.80	-74.20	0.0	2 NJ	BAYONNE-WAYNE
07191902		43.60	-71.90	0.0	2 NH	GRAFTON
08111902		40.80	-74.20	0.0	2 NJ	BAYONNE-WAYNE
01211903	A.M.	42.10	-70.90	0.0	5 MA	WHITMAN
01221903		42.00	-71.30	0.0	4 MA	ATTLEBORO
04241903	1230	42.70	-71.00	0.0	4 MA	MERRIMAC VALLEY
02051905		42.80	-70.80	0.0	2 MA	NEWBURY
03051905	0225	43.60	-72.30	0.0	5 NH	LEBANON
08301905	1040	43.10	-70.70	0.0	5 NH	ROCKINGHAM CO.
11261905	0030	41.50	-71.30	0.0	4 RI	NEWPORT
03041906		43.80	-70.20	0.0	2 ME	PORTLAND-LEWISTON
05081906	1330	41.50	-72.50	0.0	4 CT	MOODUS-E.HADDAM
05141906		41.20	-73.20	0.0	2 CT	BRIDGEPORT
10191906	P.M.	43.50	-70.50	0.0	3XME	SACO
10201906	1415	43.50	-70.50	0.0	4 ME	SACO
10211906		43.70	-70.30	0.0	3XME	PORTLAND

01241907	1130	42.80	-74.00	0.0	4	NY SCHENECTADY
06291907		43.50	-70.50	0.0	4	ME BIDDEFORD
07111907		43.10	-70.80	0.0	2X	ME ME-NH COAST
10161907	0010	42.80	-71.00	0.0	5	MA NEWBURY
02051908	0700	42.30	-71.20	0.0	3	MA NEEDHAM
02051908	0820	41.40	-73.20	0.0	4	CT HOUSATONIC VALLEY
11231908	1300	43.50	-71.70	0.0	4	NH FRANKLIN
08161909	0130	42.30	-71.20	0.0	3	MA NEEDHAM
01231910	0130	43.80	-70.40	0.0	4	ME WINDHAM
05011910		40.70	-73.50	0.0	2	NY NASSAU CO.
08211910	1845	42.70	-71.10	0.0	4	MA MERRIMAC VALLEY
08301910	1430	43.40	-72.10	0.0	4	NH LAKE SUNAPEO
02061911	1136	42.40	-71.10	0.0	2	MA CAMBRIDGE
03021911	2130	43.20	-71.50	0.0	4	NH CONCORD
03311913	1600	42.30	-71.80	0.0	2	MA WORCESTER
08101913	0515	44.00	-74.00	0.0	4	NY LAKE PLACID
11031913	1430	41.50	-71.50	0.0	4	RI KINGSTOWN
11151913		41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
01141914	0000	42.30	-71.20	0.0	3	MA NEEDHAM
02211915	0203	42.80	-71.10	0.0	4	MA MERRIMAC VALLEY
01051916	1356	43.70	-73.70	0.0	5	NY LAKE GEORGE
02021916	1626	42.90	-74.00	0.0	5	NY MOHAWK VALLEY
02031916	0420	43.00	-74.00	0.0	5	NY MOHAWK VALLEY
06081916	2115	41.00	-73.80	0.0	4	NY WESTCHESTER CO.
11021916	0232	43.30	-73.70	0.0	5	NY GLENS FALLS
12021916	0900	41.50	-72.50	0.0	3X	CT MOODUS-E.HADDAM
02161917	0900	41.50	-72.50	0.0	4	CT MOODUS-E.HADDAM
03111917		41.50	-72.50	0.0	3X	CT MOODUS-E.HADDAM
10021917	0214	43.30	-73.60	0.0	3	NY GLENS FALLS
07111919	0140	43.90	-70.00	0.0	4	ME BRUNSWICK
07231919	1150	43.70	-70.30	0.0	4	ME PORTLAND
08111919		41.50	-72.50	0.0	3	CT MOODUS-E.HADDAM
05231920	0800	43.10	-71.50	0.0	4	NH CONCORD
06071920	0800	43.50	-70.50	0.0	4	ME SACO
01191921	1000	43.30	-73.70	0.0	4	NY GLENS FALLS
01271921	A.M.	43.30	-73.70	0.0	4	NY GLENS FALLS
07291921	2114	42.50	-70.40	0.0	4	MA CAMBRIDGE
05071922	2240	43.40	-71.40	0.0	4	NH PITTSFIELD

1923		42.80	-71.00	0.0	2	MA GROVELAND
01071925	1307	42.60	-70.60	0.0	5	MA CAPE ANN
03091925		42.90	-71.50	0.0	4	NH GOFF^S FALLS
04241925	0756	41.70	-70.80	0.0	5	MA WAREHAM
05041925	1751	42.50	-70.90	0.0	4	MA LYNN
10091925	1355	43.70	-71.10	0.0	6	NH OSSIPEE
10241925	0130	41.40	-73.30	0.0	3	CT NEWTOWN
10301925	A.M.	41.50	-72.50	0.0	4X	CT MOODUS-E.HADDAM
11011925		41.50	-72.50	0.0	2X	CT MOODUS-E.HADDAM
11141925	1304	41.70	-72.40	0.0	5	CT N.OF HEBRON
11161925	0620	41.80	-72.70	0.0	4	CT HARTFORD
11221925	0600	41.80	-71.30	0.0	3	MA FALL RIVER
01041926		41.60	-71.80	0.0	4	CT VOLUNTOWN
01221926	1957	42.40	-71.10	0.0	2	MA CAMBRIDGE
03041926	2100	42.50	-70.90	0.0	2	MA LYNN
03181926	2109	42.80	-71.80	0.0	5	NH NEW IPSWICH
05121926	0330	40.90	-73.90	0.0	5	NY NEW ROCHELLE
05151926	1100	43.70	-70.20	0.0	3	ME PORTLAND
05221926		41.70	-73.90	0.0	2	NY POUGHKEEPSIE
10251926	0152	42.10	-71.00	0.0	3	MA BROCKTON
03091927	0408	43.30	-71.40	0.0	4	NH CONCORD
03301927	P.M.	41.70	-72.80	0.0	4	CT NEW BRITAIN
08201927		42.30	-71.00	0.0	4	MA QUINCY
01131928	1950	41.20	-71.60	0.0	4	RI BLOCK I.
04281928	2207	43.20	-71.50	0.0	4	NH CONCORD
05221928	0024	43.20	-71.50	0.0	2X	NH CONCORD
05261928		43.20	-71.70	0.0	2	NH CONTOOCOOK
10171928	0030	42.80	-71.60	0.0	3	NH WILTON
11051928	0400	43.30	-71.00	0.0	2	NH ROCHESTER
12011928		43.30	-71.00	0.0	2	NH ROCHESTER
12081928	0412	41.80	-72.50	0.0	2	CT ELLINGTON
01131929		43.30	-71.00	0.0	2X	NH ROCHESTER
01151929	0245	43.30	-71.00	0.0	3	NH ROCHESTER
02051929	1710	43.30	-71.70	0.0	2	NH WEARE
02051929	1909	44.00	-70.30	0.0	4	ME AUBURN
09171929	0445	42.20	-71.00	0.0	2	MA HOLBROOK
10081929	1220	44.00	-70.20	0.0	3	ME LEWISTON
02141930	0615	43.40	-71.70	0.0	4	NH FRANKLIN

03111930	2330	44.00	-70.00	0.0	2	ME	TOPSHAM
03191930	0015	43.30	-71.60	0.0	4	NH	CONCORD
03271930	1930	42.10	-72.70	0.0	3	MA	W.SPRINGFIELD
08011930	0200	41.50	-70.80	0.0	3	MA	NEW BEDFORD
04201931	1954	43.40	-73.70	5.0	7	NY	LAKE GEORGE
05041931	1017	42.40	-72.50	0.0	3	MA	AMHERST
07011931	0245	41.60	-73.40	0.0	4	CT	NEW MILFORD
07201932	2330	42.20	-73.20	0.0	2	MA	LAKE GARFIELD
10151932	0310	43.60	-71.50	0.0	3	NH	MEREDITH
10161932	1912	42.90	-72.30	0.0	2	NH	KEENE
11041932	0500	43.20	-71.50	0.0	2	NH	CONCORD
01171933	0530	41.60	-70.90	0.0	4	MA	NEW BEDFORD
06261933	1410	41.00	-73.80	0.0	3	NY	SCARSDALE
10291933	P.M.	43.00	-74.70	0.0	4	NY	ST.JOHNSVILLE
01301934	1030	41.80	-72.60	0.0	4	CT	S.WINDSOR
04111934	0300	44.00	-72.70	0.0	3	VT	RUTLAND-MONTPELIER
04111934	0324	44.00	-72.70	0.0	3X	VT	RUTLAND-MONTPELIER
08021934	1458	42.60	-70.70	0.0	4	MA	CAPE ANN
08021934	1459	43.70	-70.30	0.0	4	ME	PORTLAND
08021934	1750	43.70	-70.30	0.0	3X	ME	PORTLAND
08031934	0230	43.70	-70.30	0.0	4X	ME	PORTLAND
01301935	2020	42.60	-71.30	0.0	2	MA	BILLERICA
04241935	0124	42.20	-70.20	0.0	4	MA	OFF CAPE COD
08091935	0730	41.40	-72.10	0.0	2	CT	NEW LONDON
09131935	0349	43.20	-71.50	0.0	3	NH	CONCORD
11011935	0630	42.60	-74.60	0.0	2	NY	RICHMONDVILLE
06141936	0540	43.50	-71.50	0.0	3	NH	LACONIA
06151936		43.80	-71.40	0.0	3	NH	CENTER SANDWICH
11101936	0246	43.60	-71.40	0.0	5	NH	LACONIA
07191937	0351	40.70	-73.70	0.0	4	NY	NY CITY
07271937	0910	41.80	-72.40	0.0	4	CT	MANCHESTER
09301937	2208	40.80	-74.30	0.0	3	NJ	VERONA
10111937	2200	41.20	-73.80	0.0	2	NY	WESTCHESTER CO.
10121937	0100	41.20	-73.80	0.0	2X	NY	WESTCHESTER CO.
10121937	0600	43.30	-70.50	0.0	2	ME	KENNEBUNKPORT
04011938	2215	43.30	-71.00	0.0	3	NH	ROCHESTER
04021938	0213	43.30	-71.00	0.0	3X	NH	ROCHESTER
04031938		43.30	-71.00	0.0	2X	NH	ROCHESTER

04131938	0100	43.20	-73.10	0.0	2 VT MANCHESTER
05161938	1925	40.80	-74.30	0.0	2 NJ VERONA
06141938	0402	41.40	-73.40	0.0	2 CT BETHEL
06141938	1930	41.40	-73.40	0.0	1XCT BETHEL
06231938	0357	42.60	-71.40	0.0	4 MA CHELMSFORD
07291938	0744	41.00	-73.70	0.0	3 NY WESTCHESTER CO.
08021938	0902	41.10	-73.70	0.0	4 CT GREENWICH
08231938	0518	41.20	-73.70	0.0	3 NY WESTCHESTER CO.
08231938	0711	41.20	-73.70	0.0	3XNY WESTCHESTER CO.
09201938		41.50	-72.20	0.0	3 CT NORWICH
10211938	0718	41.20	-73.70	0.0	2 NY DUTCHESS CO.
12061938	1938	40.80	-74.30	0.0	3 NJ VERONA
02011939	1037	42.60	-71.40	0.0	2 MA CHELMSFORD
08121939		41.50	-72.50	0.0	2 CT MOODUS-E.HADDAM
09131939	0122	40.80	-74.00	0.0	2 NJ UNION CITY
09211939	2030	41.40	-74.10	0.0	2 NY ORANGE CO.
10101939	A.M.	43.40	-71.60	0.0	3 NH TILTON
10111939	1849	42.90	-71.40	0.0	3 NH DERRY
10211939	0859	43.30	-73.30	0.0	2 NY GLENS FALLS
10251939	1446	42.20	-73.90	0.0	2 NY HUDSON
01021940	0205	42.50	-71.50	0.0	3 MA LITTLETON
01031940	0130	41.20	-71.60	0.0	2 RI BLOCK ISLAND
01281940	2311	41.60	-70.80	2.6	5 MA BUZZARDS BAY
03021940	0415	41.50	-72.50	0.0	3 CT MOODUS-E.HADDAM
03131940	0129	41.50	-72.50	0.0	3XCT MOODUS-E.HADDAM
04121940	0158	42.80	-74.60	0.0	2 NY SE OF ST.JOHNSVILLE
12201940	0727	43.80	-71.30	5.8	7 NH OSSIPEE
12241940	1300	43.80	-71.30	0.0	2XNH OSSIPEE
12241940	1343	43.80	-71.30	5.8	7XNH OSSIPEE
12241940	1432	43.80	-71.30	0.0	3XNH OSSIPEE
12241940	1812	43.80	-71.30	0.0	3XNH OSSIPEE
12251940	0503	43.80	-71.30	4.0	4XNH OSSIPEE
12271940	1956	43.80	-71.30	3.9	4XNH OSSIPEE
01021941	0342	43.80	-71.30	0.0	3XNH OSSIPEE
01041941	1110	43.80	-71.30	0.0	3XNH OSSIPEE
01181941	2325	43.80	-71.30	0.0	3XNH OSSIPEE
01211941	0227	43.80	-71.30	3.6	4XNH OSSIPEE
01231941	0014	43.80	-71.30	0.0	3XNH OSSIPEE

02121941	2223	43.80	-71.30	0.0	3XNH OSSIPEE
04291941	1405	40.50	-72.50	2.5	0 NY S.OF LONG ISLAND
05191941	1159	43.80	-72.30	2.0	0 VT N.OF HANOVER NH
07011941	1659	43.40	-70.20	2.0	0 ME OFF KENNEBUNKPORT
07291941	0024	41.10	-73.80	0.0	3 NY WHITE PLAINS
10111941	0815	42.30	-72.30	3.0	4 MA STURBRIDGE
04231942	2040	41.40	-72.90	2.0	0 CT NEW HAVEN
06141942	1104	42.40	-70.70	0.0	2 MA OFF BOSTON
06141942	1630	42.40	-70.70	0.0	2XMA OFF BOSTON
06141942	1952	42.70	-70.70	0.0	2XMA OFF BOSTON
10011942	2058	44.00	-73.60	2.5	0 NY LAKE CHAMPLAIN
10021942	2229	42.60	-73.80	3.0	0 NY ALBANY
12091942	1800	41.80	-72.70	0.0	2 CT HARTFORD
02101943	0945	43.70	-70.30	0.0	2 ME PORTLAND
03141943	1402	43.70	-71.60	3.9	0 NH MEREDITH
03311943	1130	42.30	-72.60	0.0	2 MA NORTHAMPTON
06111943	2251	41.10	-71.80	0.0	2 RI BLOCK ISLAND SOUND
03061944	0546	43.20	-71.60	0.0	2 NH CONCORD
03061944	1215	43.20	-71.60	0.0	2XNH CONCORD
04111944	2025	44.00	-71.70	0.0	3 NH WOODSTOCK
12141944	0314	41.60	-72.80	0.0	4 CT MERIDEN
03221945	0803	43.20	-71.60	0.0	3 NH CONCORD
08051945	1720	43.60	-72.50	0.0	3 VT WOODSTOCK
12281945	1023	43.80	-71.30	0.0	2 NH S.TAMWORTH
11281946	2200	43.90	-73.80	0.0	3 NY SCHROON LAKE
01041947	1851	41.00	-73.60	0.0	4 CT GREENWICH
04011947	1325	41.00	-74.30	0.0	3 NJ POMPTON LAKES
05041948	0223	41.40	-71.80	0.0	4 RI WESTERLY
06041948	0900	41.30	-72.50	0.0	3 CT WESTBROOK
08071948		44.00	-74.00	0.0	2 NY SANFORD LAKE
04171949	0015	41.60	-71.50	0.0	4 RI N.KINGSTON
09021949	0548	43.80	-71.30	0.0	3 NH S.TAMWORTH
02241950	1304	43.00	-71.80	0.0	3 NH SW OF CONCORD
03291950	1443	41.00	-73.60	0.0	4 CT GREENWICH
01261951	0327	41.50	-72.50	0.0	4 CT MOODUS-E.HADDAM
03311951	0350	42.20	-72.20	0.0	4 MA PALMER
06101951	1720	41.50	-71.50	4.6	4 RI KINGSTOWN
09031951	2126	41.20	-74.30	4.4	5 NY ROCKLAND CO.

09211951	1723	41.30	-70.10	0.0	2	MA	NANTUCKET
12081951	0437	41.60	-73.80	0.0	2	NY	NEWBURGH
08251952	0007	43.00	-74.50	0.0	5	NY	JOHNSTOWN
10081952	2140	41.70	-74.00	0.0	5	NY	POUGHKEEPSIE
10261952	0905	43.60	-71.20	0.0	2	NH	WOLFEBORO
03271953	0850	41.10	-73.50	0.0	5	CT	STAMFORD
03311953	0250	43.70	-73.00	0.0	3X	VT	BRANDON
03311953	1258	43.70	-73.00	4.0	5	VT	BRANDON
05111953	0613	44.00	-71.10	0.0	4	NH	CONWAY
08171953	0422	41.00	-74.00	0.0	4	NJ	BERGEN CO.
02131954		42.20	-72.60	0.0	4	MA	SPRINGFIELD
02131954		42.20	-72.60	0.0	4	MA	SPRINGFIELD
07291954	1956	42.70	-70.70	4.0	5	MA	CAPE ANN
10071954		42.70	-71.30	0.0	3	NH	PELHAM
01211955	0840	43.00	-73.80	0.0	5X	NY	MALTA
01211955	1220	43.00	-73.70	0.0	5	NY	MALTA
09211956	1700	41.80	-71.20	0.0	2	MA	SWANSEA
03231957	1902	40.60	-74.80	4.8	6	NJ	SCHOOLEY^S MTN
05061958	1900	42.70	-73.80	0.0	4	NY	ALBANY
09191958	1745	43.60	-70.20	0.0	5	ME	CAPE ELIZABETH
11211958	2330	44.00	-71.70	0.0	4	NH	WOODSTOCK
04131959	2120	41.90	-73.30	3.4	0	CT	S.CANAAN
12271961	1706	40.50	-74.80	0.0	5	NJ	FLEMINGTON
08171962		41.70	-71.70	0.0	2	RI	E.GREENWICH
10131962		41.00	-74.30	0.0	2	NJ	POMPTON LAKE
11271962	0415	41.50	-73.80	0.0	2	NY	POUGHKEEPSIE
12201962		41.00	-74.30	0.0	2	NH	POMPTON LAKE
12291962	0619	42.80	-71.70	0.0	5	NH	NASHUA
05191963		43.20	-73.30	0.0	3	NY	HUDSON FALLS
06011963		42.60	-73.00	0.0	2	MA	NORTH ADAMS
07011963	1959	42.60	-73.80	3.3	0	NY	ALBANY
10161963	1531	42.50	-70.40	4.3	5	MA	MARBLEHEAD
10171963	1245	42.70	-71.50	0.0	3	MA	DUNSTABLE
10181963	1543	42.50	-70.40	3.0	2X	MA	OFF CAPE ANN
10301963	2236	42.70	-70.80	3.2	4X	MA	OFF CAPE ANN
11051963	0946	42.70	-70.30	2.7	2	MA	OFF CAPE ANN
12041963	2132	43.70	-71.40	3.6	4	NH	LACONIA
04011964	1121	43.40	-71.50	2.4	4	NH	LACONIA

06161964		40.90	-74.30	0.0	4	PA EASTERN
06261964	1104	43.30	-71.50	3.5	5	NH CONCORD
06261964	1250	43.30	-71.90	0.0	0	NH CONCORD
09291964	0016	41.20	-73.70	0.0	3X	NY MT.KISCO
09291964	2026	41.20	-73.70	0.0	3X	NY MT.KISCO
11171964	1708	41.20	-73.70	0.0	5	NY MT.KISCO
11301964		41.30	-73.90	0.0	2X	NY MT.KISCO
11301964	0034	42.80	-74.90	2.6	0	NY N.OF COOPERSTOWN
01031965	1705	43.50	-71.50	3.4	3	NH LACONIA
09291965	2057	41.40	-74.40	0.0	4	NY GOSHEN-MIDDLETOWN
10241965	1745	41.30	-70.10	0.0	5	MA NANTUCKET
10241965	1900	41.30	-70.10	0.0	0	MA NANTUCKET
12081965	0302	41.70	-71.40	0.0	4	RI WARWICK
05211966		41.20	-74.00	0.0	2	NY SOUTHEASTERN
06301966	0029	44.00	-73.40	2.8	2	NY LAKE CHAMPLAIN
07111966	0236	42.40	-71.30	0.0	0	MA NEAR WESTON
10231966	2305	43.00	-71.40	2.7	4	NH MANCHESTER
02021967	1340	41.60	-71.20	3.1	5	RI NARRAGANSETT BAY
11221967	2210	41.20	-73.80	0.0	5	NY WESTCHESTER CO.
11031968	0833	41.40	-72.50	3.3	5	CT MOODUS-E.HADDAM
04251969	0014	40.70	-74.30	0.0	0	NJ PRINCETON-HOPEWELL
05111969	0303	43.10	-70.50	2.1	0	ME OFF SOUTHWESTERN MA
08061969	1602	43.80	-71.40	2.6	5	NH OSSIPEE
08241969	0151	43.10	-70.50	2.4	0	ME OFF SOUTHWESTERN MA
08241969	0259	43.10	-70.40	2.1	0X	ME OFF SOUTHWESTERN MA
10061969		41.10	-74.60	0.0	4	NJ OGDENSBURG
09191970	1335	42.90	-71.90	2.6	4	NH GREENFIELD
05231971	0624	43.80	-74.50	3.9	5	NY BLUE MTN LAKE
05231971	0929	43.90	-74.50	3.6	5X	NY BLUE MTN LAKE
06211971	0248	43.90	-74.50	3.3	4X	NY BLUE MTN LAKE
07101971	0814	43.90	-74.50	3.4	5X	NY BLUE MTN LAKE
08151971	1011	43.90	-74.50	2.0	0X	NY BLUE MTN LAKE
10211971	0054	42.70	-71.20	2.3	5	MA LAWRENCE
12201971	1144	43.90	-74.60	2.0	0X	NY BLUE MTN LAKE
02151972	2352	41.30	-73.60	2.6	0	NY POUND RIDGE
03151972	1210	43.70	-74.70	2.6	0	NY OLD FORGE
06161972	0901	42.80	-73.90	2.0	0	NY SCHENECTADY
11051972	2229	43.90	-74.40	2.3	0	NY BLUE MTN LAKE

01101973	0241	41.40	-74.00	1.5	0	NY	PEEKSKILL
01141973	0808	41.80	-72.10	1.0	0	CT	CHAPLIN
03071973	0405	43.80	-74.50	1.9	0	NY	BLUE MTN LAKE
03071973	0504	43.90	-75.00	2.4	0	NY	N.OF OLD FORGE
06111973	1008	43.90	-73.90	2.8	0	NY	E.OF BLUE MTN LAKE
07151973	0820	43.90	-74.40	3.4	0	NY	BLUE MTN LAKE
07151973	1032	43.90	-74.40	3.2	0XNY		BLUE MTN LAKE
07161973	0841	43.80	-74.50	3.3	0XNY		BLUE MTN LAKE
08241973	0417	43.80	-72.30	2.7	0	VT	EAST CENTRAL
10211973	0925	43.80	-74.50	2.2	0XNY		BLUE MTN LAKE
04081974	2208	41.20	-74.00	2.1	0	NY	STONY POINT
06071974	1945	41.60	-73.90	3.3	6	NY	WAPPINGERS FALLS
09111974	2054	43.80	-74.20	2.2	0	NY	BLUE MTN LAKE
09151974	1401	43.90	-73.90	1.7	0	NY	SCHROON LAKE
09181974	0623	43.40	-73.80	2.5	0	NY	SW OF LAKE GEORGE
10011974	0636	41.70	-71.60	2.5	2	RI	WEST WARWICK
11191974	0923	43.50	-74.00	2.3	0	NY	STONY CREEK
12271974	0429	42.30	-71.30	2.5	0	MA	NEEDHAM
12271974	1451	42.20	-71.30	2.2	0XMA		NEEDHAM
01271975	1140	43.80	-73.40	1.7	0	NY	NEAR VT BORDER
03301975	1602	44.00	-74.20	2.0	0	NY	NEWCOMB
04291975	0951	41.60	-73.90	2.3	0	NY	WAPPINGERS FALLS
06151975	0808	41.60	-73.90	2.0	0	NY	WAPPINGERS FALLS
06221975	1130	43.60	-75.00	1.9	0	NY	OLD FORGE
07021975	0531	42.20	-71.30	.9	0	MA	HARDING
07191975	2059	41.40	-73.80	2.3	3	NY	MAHOPAC
07261975	1126	42.70	-70.70	1.1	0	MA	CAPE ANN
08031975	0103	42.70	-70.90	2.4	3	MA	IPSWICH
08041975	0458	43.80	-74.10	2.1	0	NY	BLUE MTN LAKE
08221975	1749	41.10	-73.90	2.3	0	NY	LAKE DEFOREST
08261975	2218	41.20	-71.20	2.1	0	RI	RHODE ISLAND SOUND
09111975	1654	43.90	-74.70	1.6	0	NY	RAQUETTE LAKE
09231975	1511	43.90	-74.60	1.6	0XNY		RAQUETTE LAKE

Appendix B

List of instrumental earthquake epicenters in the Knightville Dam STUDY AREA. The time period covered is October 1975 through March 1981.

Code:

Column 1	Date in Month-Day-Year
Column 2	Origin Time in Hr-Min-Sec (UTC)
Column 3	Latitude
Column 4	Longitude
Column 5	Focal depth in km if known
Column 6	Magnitude
Column 7	Intensity (M.M.)
Column 8	X indicates foreshock or aftershock
Column 9	State
Column 10	City or local area

10241975	070846.4	41.62	-73.98	5	2.0	0 NY, WAPPINGER FALLS
10241975	074312.4	41.59	-73.90	3	2.2	0XNY, WAPPINGER FALLS
10281975	214547.6	41.71	-73.72	0	0.0	0XNY, WAPPINGER FALLS
11021975	040914.2	41.67	-74.00	0	0.0	0XNY, WAPPINGER FALLS
11031975	205455.9	43.87	-74.64	3	3.9	0 NY, RACQUETTE LAKE
11031975	210640.8	43.89	-74.65	1	0.0	0XNY, RACQUETTE LAKE
11051975	115311.2	43.89	-74.65	1	0.0	0XNY, RACQUETTE LAKE
11101975	030229.2	41.18	-74.38	5	1.8	0 NY, GREENWOOD LAKE
11161975	111333.7	41.41	-71.04	0	2.1	0 MA, BUZZARDS BAY
11171975	223641.1	43.90	-74.64	3	0.0	0XNY, RACQUETTE LAKE
11181975	210013.9	43.89	-74.64	3	0.0	0XNY, RACQUETTE LAKE
02051976	080252.6	43.97	-70.17	0	0.0	0 ME, LEWISTON
02061976	091505.6	41.74	-72.22	0	1.9	0 CT, MANSFIELD
03041976	162036.2	41.42	-70.34	0	2.7	0 NANTUCKET SOUND
03061976	041405.8	41.16	-73.81	6	0.0	0 NY, OSSINING
03111976	082932.2	41.56	-71.21	0	3.5	0 RI, PORTSMOUTH
03111976	083511.7	41.58	-71.26	0	2.4	0XRI, PORTSMOUTH
03111976	210720.3	40.91	-74.36	1	2.4	0 NJ, RIVERDALE
03121976	102856.4	40.94	-74.36	0	0.0	0XNJ, RIVERDALE
04061976	210536.9	41.46	-72.49	0	1.8	0XCT, EAST HADDAM
04131976	153912.9	40.80	-74.03	0	3.1	0 NJ, RIDGEFIELD
04241976	102222.1	41.46	-72.49	0	2.2	4 CT, EAST HADDAM
04301976	195059.0	41.46	-72.49	1	1.8	0XCT, EAST HADDAM
04301976	204009.2	41.46	-72.49	0	1.9	0XCT, EAST HADDAM
05091976	135603.5	43.88	-74.65	2	1.0	0 NY, RACQUETTE LAKE
05101976	013420.5	41.54	-71.01	0	2.7	0 MA, NEW BEDFORD
05101976	0319	43.88	-74.65	0	0.0	0XNY, RACQUETTE LAKE
07281976	020434.8	43.16	-70.24	0	2.3	0 ME, OFF SW COAST
08191976	154753.6	43.89	-74.64	4	0.0	0 NY, RACQUETTE LAKE
08201976	220814.3	41.11	-73.75	6	2.5	0 NY, MT PLEASANT
08211976	140012.5	43.89	-74.66	1.4	0.0	0 NY, RACQUETTE LAKE
09181976	011523.0	43.82	-74.20	0	1.6	0 NY, SE OF BLUE MTN LAK
09221976	090444.9	41.29	-73.95	8	1.8	0 NY, INDIAN PT
10281976	0110	40.89	-74.49	0	0.0	0XNJ, DENVILLE FORESHOCK
10281976	011331.4	40.89	-74.49	0	0.0	0 NJ, DENVILLE
10281976	0647	40.89	-74.49	0	0.0	0XNJ, DENVILLE
11051976	180748.7	43.79	-74.59	8.7	0.0	0 NY, RAQUETTE LAKE

11151976	152037.5	43.56	-71.62	0	0.0	0XNH, LACONIA
11221976	044313.4	41.00	-73.86	5	1.9	2 NY, YONKERS
11221976	224928.0	43.54	-71.60	0	1.8	0 NH, LACONIA
12051976	163206.9	40.77	-74.76	3.4	1.8	2 NJ, SCHOOLEY'S MTN
12071976	045507.2	40.77	-74.76	5.1	1.7	0XNJ, SCHOOLEY'S MTN
12171976	103057.6	41.47	-72.07	0	2.2	-1 CT, SOUTHEASTERN
01071977	000512.8	41.02	-74.51	19	0.0	0 NJ, NR GREEN POND
02071977	025650.0	41.60	-72.43	0	2.1	0 CT, MARLBOUROUGH
03071977	094444.0	41.60	-72.42	0	1.8	0 CT, MARLBOROUGH
03101977	162225.2	41.18	-74.15	6	2.2	0 NY, SUFFERN
04061977	203157.8	41.06	-70.43	0	2.5	0 ATLANTIC OCEAN
05051977	083931.8	43.86	-72.26	0	2.1	0 VT, LAKE FAIRLEE
06101977	124804.8	40.70	-74.89	6	1.1	0 NJ, NR HIGH BRIDGE
07011977	155319.5	42.88	-70.06	0	2.6	0 MA, NE OF CAPE ANN
07021977	111321.6	40.70	-74.93	7	2.3	0 NJ, HAMPTON
09021977	055356.5	41.31	-73.92	3	2.4	0 NY, PEEKSKILL
09021977	130943.9	41.31	-73.92	0	0.0	0XNY, PEEKSKILL AFTSHOCK
09021977	222259.9	41.31	-73.92	0	0.0	0XNY, PEEKSKILL AFTSHOCK
09031977	000421.8	41.31	-73.92	0	0.0	0XNY, PEEKSKILL AFTSHOCK
09031977	000852.8	41.31	-73.92	0	0.0	0XNY, PEEKSKILL AFTSHOCK
09081977	235916.0	43.56	-70.69	0	1.8	0 ME, LITTLE OSSIPEE PON
09141977	110607.1	41.31	-73.92	0	0.0	0XNY, PEEKSKILL AFTSHOCK
09171977	184712.7	41.20	-74.06	1	0.0	0 NY, HAVERSTRAW
09291977	184403.0	41.31	-73.93	2	0.0	0XNY, PEEKSKILL AFTSHOCK
10141977	000954.2	41.56	-73.95	0	2.2	0 NY, N OF NEWBURGH
10271977	092210.9	41.07	-74.59	6	1.5	0 NJ, SPARTA
11271977	135703.0	41.02	-74.21	5	1.8	0 NJ, OAKLAND
12041977	235044.0	40.81	-74.76	0	1.8	2XNJ, SCHOOLEY'S MTN
12041977	235056.0	40.81	-74.76	0	2.1	2 NJ, SCHOOLEY'S MTN
12061977	1751	40.81	-74.76	0	1.6	2XNJ, SCHOOLEY'S MTN
12071977	0834	40.81	-74.76	0	1.7	2XNJ, SCHOOLEY'S MTN
12091977	173307.9	41.56	-73.88	5	2.3	0 NY, HOPEWELL JCT
12201977	174423.8	41.79	-70.68	0	3.1	5 MA, WAREHAM
12201977	224444.5	41.81	-70.78	0	2.0	3XMA, WAREHAM AFTSHOCK
12231977	045511	40.81	-74.76	0	2.3	2 NJ, SCHOOLEY'S MTN
12231977	162022	40.81	-74.76	0	1.4	2XNJ, SCHOOLEY'S MTN
12241977	1025	40.81	-74.76	0	1.6	2XNJ, SCHOOLEY'S MTN
12251977	153553.8	43.19	-71.65	0	3.2	4 NH, HOPKINTON

12251977	153934	40.81	-74.76	0	1.5	2XNJ, SCHOOLEY'S MTN
12261977	165446	40.81	-74.76	0	1.7	2XNJ, SCHOOLEY'S MTN
01041978	192853.5	43.99	-70.57	0	2.4	0XME, OTISFIELD AFTSHOCK
01131978	051503.0	41.40	-73.97	0	0.0	0 NY, WEST PT.
01141978	184731.0	41.37	-73.94	4	0.0	0 NY, WEST PT.
01151978	074106.4	41.38	-73.95	3	0.0	0XNY, WEST PT.
01181978	002857.3	42.98	-72.15	0	1.9	0 NH, E OF KEENE
02151978	052841.0	40.92	-74.43	6	1.6	0 NJ, BOUTON
03051978	075325.5	41.35	-74.15	5	2.1	2 NY, HIGHLAND MILLS
03201978	081654.1	43.09	-71.52	0	2.4	0 NH, NW OF MANCHESTER
03311978	142757.0	43.10	-71.63	0	2.7	-1 NH, DUNBARTON
04031978	235758.0	40.53	-74.08	0	2.0	0 NJ, OFF SANDY HOOK
04051978	144549.5	43.85	-74.24	11	2.6	0 NY, NEWCOMB
04261978	093418.9	41.35	-71.61	0	1.7	0 RI, BLOCK ISLAND SOUND
05181978	012937.9	41.02	-74.34	6	1.5	2 NJ, BLOOMINGDALE
06161978	045950.6	40.99	-74.57	0	0.0	0 NJ, SPARTA
06211978	183110.0	43.66	-71.38	0	1.8	0 NH, L.WINNIPESAUKEE
06301978	201343.6	41.08	-74.20	5	2.9	4 NJ, MAHWAH--OAKLAND
06301978	223949.7	41.08	-74.20	6	2.2	0XNJ, MAHWAH--OAKLAND
07231978	230217.3	41.32	-73.93	2	2.1	0 NY, PEEKSKILL
07281978	144447.4	40.58	-70.91	0	2.4	0XRI, CONTINENTAL SHELF
08061978	104910.6	42.45	-71.06	0	1.6	0XMA, WAKEFIELD
08061978	130108.8	42.43	-71.04	0	1.8	0 MA, WAKEFIELD
08171978	214108.1	43.52	-71.56	0	1.9	0 NH, WINNISQUAM LAKE
08251978	200130.5	42.87	-70.83	0	2.3	2 NH, SEABROOK
09011978	033343.6	42.48	-71.46	0	2.0	2 MA, BOXBOROUGH-ACTON
09131978	124114.4	41.36	-71.37	0	2.8	0 RI, RI SOUND
09191978	041146.6	40.99	-73.87	5	1.8	0 NY, YONKERS
10271978	235226.7	41.51	-71.01	0	1.9	0 MA,S OF DARTMOUTH
10291978	235942.8	43.94	-70.40	0	2.5	0 ME,CRESCENT LAKE
10301978	063213.5	43.97	-70.41	0	1.6	0XME,CRESCENT LAKE
10311978	031647.6	44.00	-70.73	0	1.9	0XME,CRESCENT LAKE
10311978	044928.9	43.99	-70.42	0	2.0	0XME,CRESCENT LAKE
01011979	041347.4	43.52	-71.63	0	1.9	0 NH,SE OF BRISTOL
01071979	092248.8	41.73	-72.57	0	0.0	0 CT,E. GLASTONBURY
01311979	214200.6	43.38	-72.64	5	0.0	0 VT,LUDLOW
02021979	022613.3	40.77	-74.66	0	1.9	0 NJ,CHESTER
02111979	072121.8	41.19	-73.75	11	2.0	0 NY,NR CROTON RESERVOIR

03101979	044939.7	40.72	-74.50	3	3.1	2 NJ,BERNARDSVILLE
04231979	000545.7	43.04	-71.24	0	3.1	3 NH,CANDIA
07091979	054232.2	43.39	-71.45	7	2.4	0 NH,S OF LACONIA
07281979	232912.3	43.29	-70.44	11	3.5	2 ME,SE OF KENNEBUNKPORT
09041979	073856.0	41.58	-73.53	8	0.0	0 CT,NY BORDER
10011979	195016.3	41.65	-73.92	5	0.0	0 NY,POUGHKEEPSIE
11081979	011316.7	43.88	-74.50	2	0.0	0 NY,BLUE MTN LAKE
11201979	061732.8	42.22	-71.04	0	2.2	0 MA,QUINCY
11291979	025844.5	43.79	-74.49	0	2.5	0 NY,BLUE MTN LAKE
12011979	161406.9	41.35	-74.36	0	2.0	0 NY,PINE ISLAND
12121979	083422.0	41.45	-72.01	0	0.0	0 CT,SE OF NORWICH
12301979	141523.8	41.16	-73.72	3.8	2.2	3 NY,MT KISCO
01021980	031230.6	41.62	-72.10	0	1.0	0 CT,NEAR NORWICH
01171980	101225.4	41.26	-73.91	5	2.4	2XNY,PEEKSKILL
01171980	101316.1	41.31	-73.93	3.3	2.9	2 NY,PEEKSKILL
02091980	131136.0	43.56	-70.76	0	2.4	2 ME,W OF BIDDEFORD
02291980	055356.1	42.58	-74.20	12.2	3.1	0 NY,CATSKILL
04011980	044836.4	43.84	-71.40	0	2.0	0 NH,NE OF SQUAM LAKE
04071980	093600.4	43.13	-72.22	0	2.7	0 NH,W OF HIGHLAND LK
04251980	002335.0	41.35	-74.37	0.0	2.4	0 NY,FLORIDA
05071980	043249.3	41.02	-73.87	0.0	2.6	0 NY,ARDSLEY
05121980	013856.3	41.28	-74.14	4.9	2.4	0 NY,HARRIMAN
05201980	213323.0	41.35	-74.37	2.3	2.6	0 NY,FLORIDA
05211980	192217.9	43.01	-72.40	0	1.6	0 NH,NW OF KEANE
06291980	032519.8	41.61	-72.12	0	1.5	0XCT,N OF NORWICH
06291980	032910.1	41.46	-72.09	0	1.8	0 CT,N OF NORWICH
07281980	094309.3	41.52	-72.45	6.	.8	0 CT, N OF MOODUS
09041980	043055.8	41.11	-73.78	0.	3.2	2 NY, THORNWOOD
09051980	235948.2	41.67	-72.25	0.	1.0	0 CT, SW OF WILLIMANTIC
09211980	205245.1	43.63	-74.02	1.	3.2	0 NY, BAKERS MILLS
09271980	004830.5	41.54	-73.69	6.	2.5	0 NY, PAWLING
09281980	222005.4	43.77	-74.12	0.	3.0	0 NY, SE BLUE MT LAKE
10141980	081935.6	43.87	-74.47	1.	1.0	0 NY, BLUE MTN LAKE
10151980	170250.9	41.25	-73.88	2.	1.5	0 NY, W CROTON RESEVOIR
10241980	172738.2	41.32	-72.87	7.	2.8	2 CT, NE OF NEW HAVEN
10251980	004128.3	41.33	-72.88	6.	2.7	2XCT, NE OF NEW HAVEN, A
11051980	224001.4	43.66	-71.36	5.	1.9	0 NH, N LAKE WINNIPES
11151980	172303.0	41.37	-71.01	2.	1.6	0 MA, CUTTY HUNK ISLAND

11231980	003932.4	42.62	-71.39	1.	2.5	2	MA, S OF LOWELL
12121980	230453.3	41.31	-73.91	15.	2.3	0	NY, ANNSVILLE
01049181	091710.2	43.89	-70.01	0.	2.6	2	ME, N OF BRUNSWICK
02091981	031410.4	43.26	-71.56	7.	1.9	0	NH, NW OF CONCORD

Appendix C

Examples of observed strong motion data, peak ground acceleration values from selected regions of the world.

Appendix C - Figure Captions

Fig. C-1. Acceleration records from the San Fernando earthquake of 9 Feb 1971 (M_L = 6.4) at selected stations. Radial components except at COS, LBI, SJC and WRI where transverse component is plotted. The circles indicate distances of 40, 80, 120 and 160 km from the epicenter (from Shakal, 1979, p. 59, Fig. 2-2).

Fig. C-2. Acceleration records from the Borrego Mountain earthquake of 9 April 1968 (M_L = 6.6). Radial components only. The ELC record is plotted at 1/4 gain relative to others (from Shakal, 1979, p. 58, Fig. 2-1).

Fig. C-3. Attenuation laws in the Alpide belt, and Friuli and their comparison with the Circumpacific belt (shaded region). The three curves (solid and two dashed lines) are referred to in the body of the report (from Chiaruttini and Siro, 1981, p. 2005, Fig. 6).

Fig. C-4. Attenuation laws and 95 percent confidence bands at two different magnitudes, for three data sets (Friuli, Ancona, and Alpide Belt) (from Chiaruttini and Siro, 1981, p. 2001, Fig. 3).

Fig. C-5. Peak acceleration-attenuation relations derived from worldwide earthquakes (left) and San Fernando earthquake of 1971 (from Hays, 1980, p. 24, Fig. 19).

Fig. C-6. Residuals (ratios of observed to predicted) of peak horizontal acceleration with respect to a prediction equation whose coefficients are determined by regression analysis using California strong motion data (Table C-1) (Joyner and Boore, 1981, p. 2024, Fig. 5). The prediction equation is

$$\log A = -1.02 + 0.249M - \log r - 0.00255r + 0.26P$$

where A = peak acceleration, r = hypocentral distance [$r=(d2 + 7.3^2)^{1/2}$], and P = 0 for 50 percent probability, p = 1 for 84 percent probability that prediction will exceed the real value.

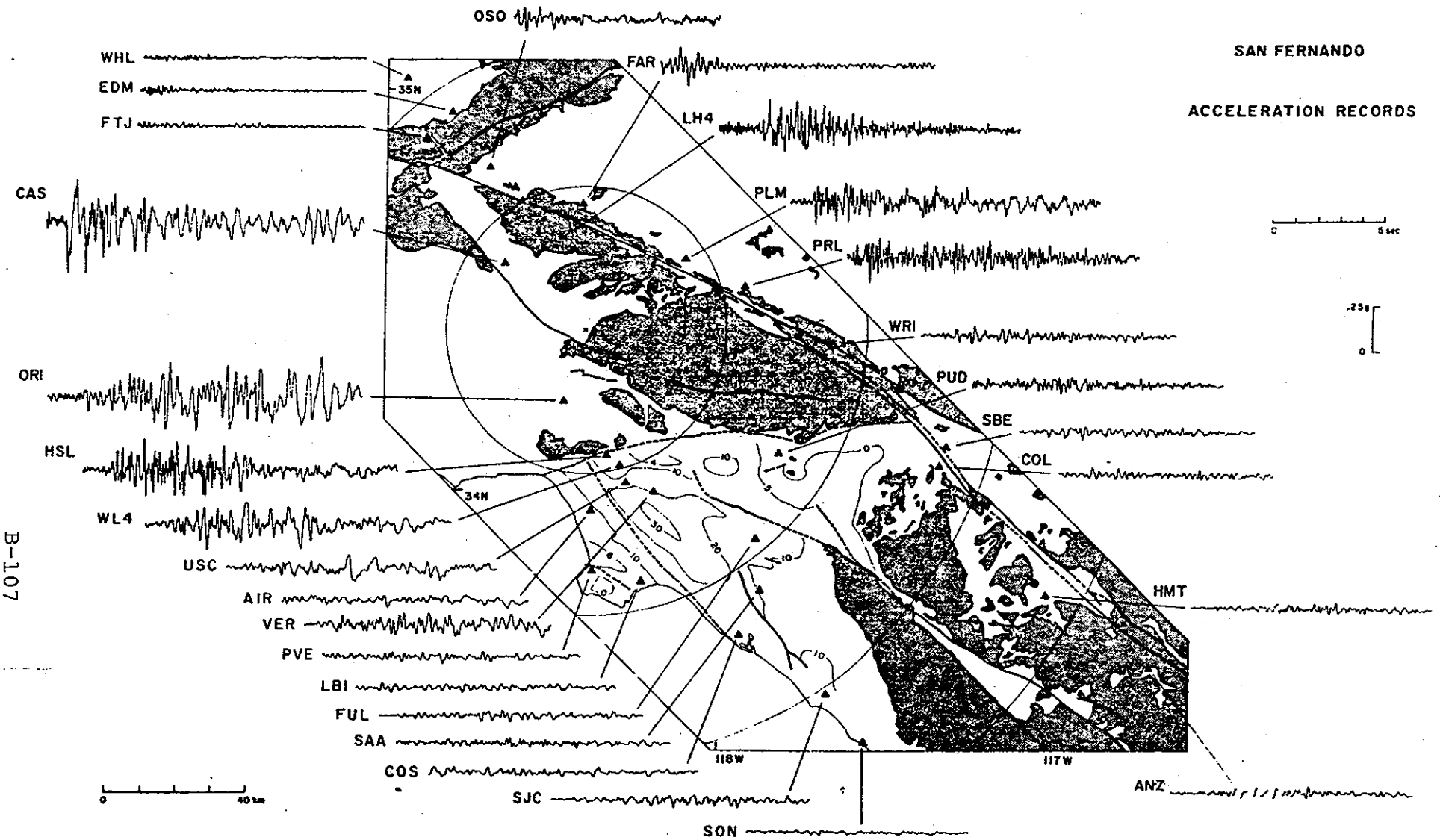

SAN FERNANDO
ACCELERATION RECORDS
OSO
WHL
EDM
FTJ
CAS
ORI
HSL
WL4
USC
AIR
VER
PVE
LBI
FUL
SAA
COS
SJC
SON
FAR
LH4
PLM
PRL
WRI
PUD
SBE
COL
HMT
ANZ
35N
34N
118W
117W
0
5 sec
.25g
0
40 km

Figure C-1

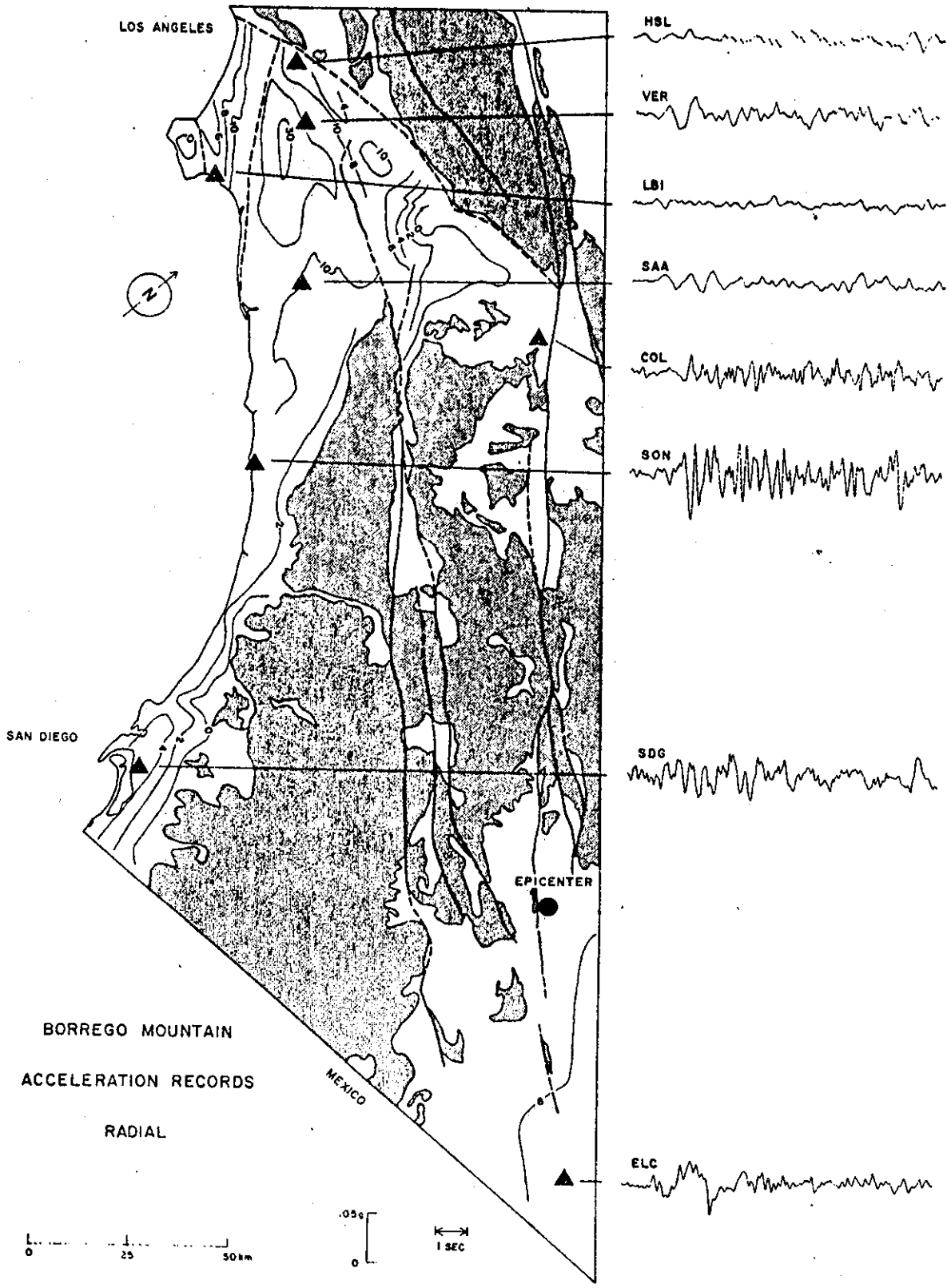

LOS ANGELES
HSL
VER
LBI
SAA
COL
SON
SAN DIEGO
SDG
EPICENTER
BORREGO MOUNTAIN
ACCELERATION RECORDS
RADIAL
MEXICO
ELC
0
25
50km
.05g
0
1 SEC

Figure C-2

C-2

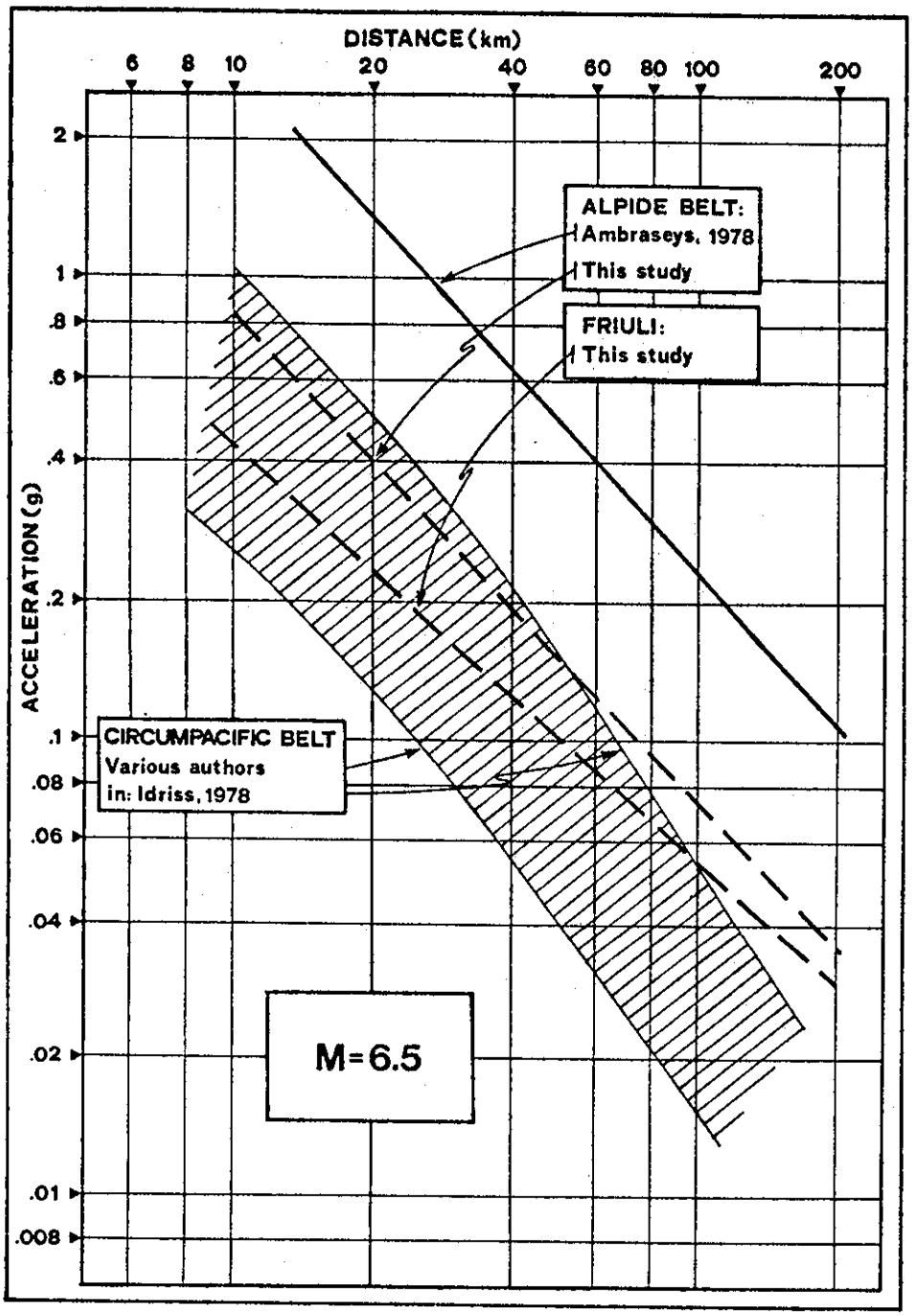

DISTANCE (km)
6
8
10
20
40
60
80
100
200
ACCELERATION (g)
2
1
.8
.6
.4
.2
.1
.08
.06
.04
.02
.01
.008
ALPIDE BELT:
Ambraseys, 1978
This study
FRIULI:
This study
CIRCUMPACIFIC BELT
Various authors
in: Idriss, 1978
M=6.5

Figure C-3

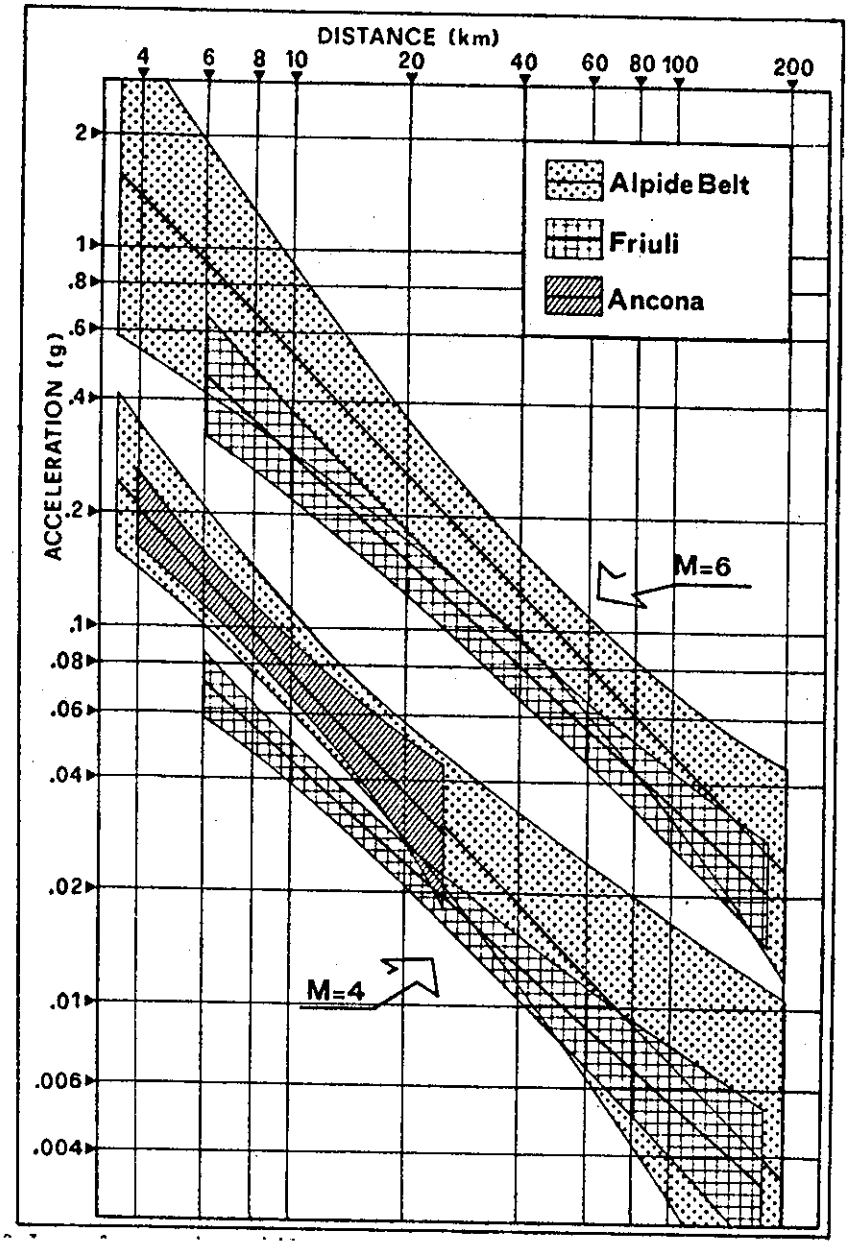

DISTANCE (km)
4
6
8
10
20
40
60
80
100
200
ACCELERATION (g)
2
1
.8
.6
.4
.2
.1
.08
.06
.04
.02
.01
.006
.004
Alpide Belt
Friuli
Ancona
M=6
M=4

Figure C-4

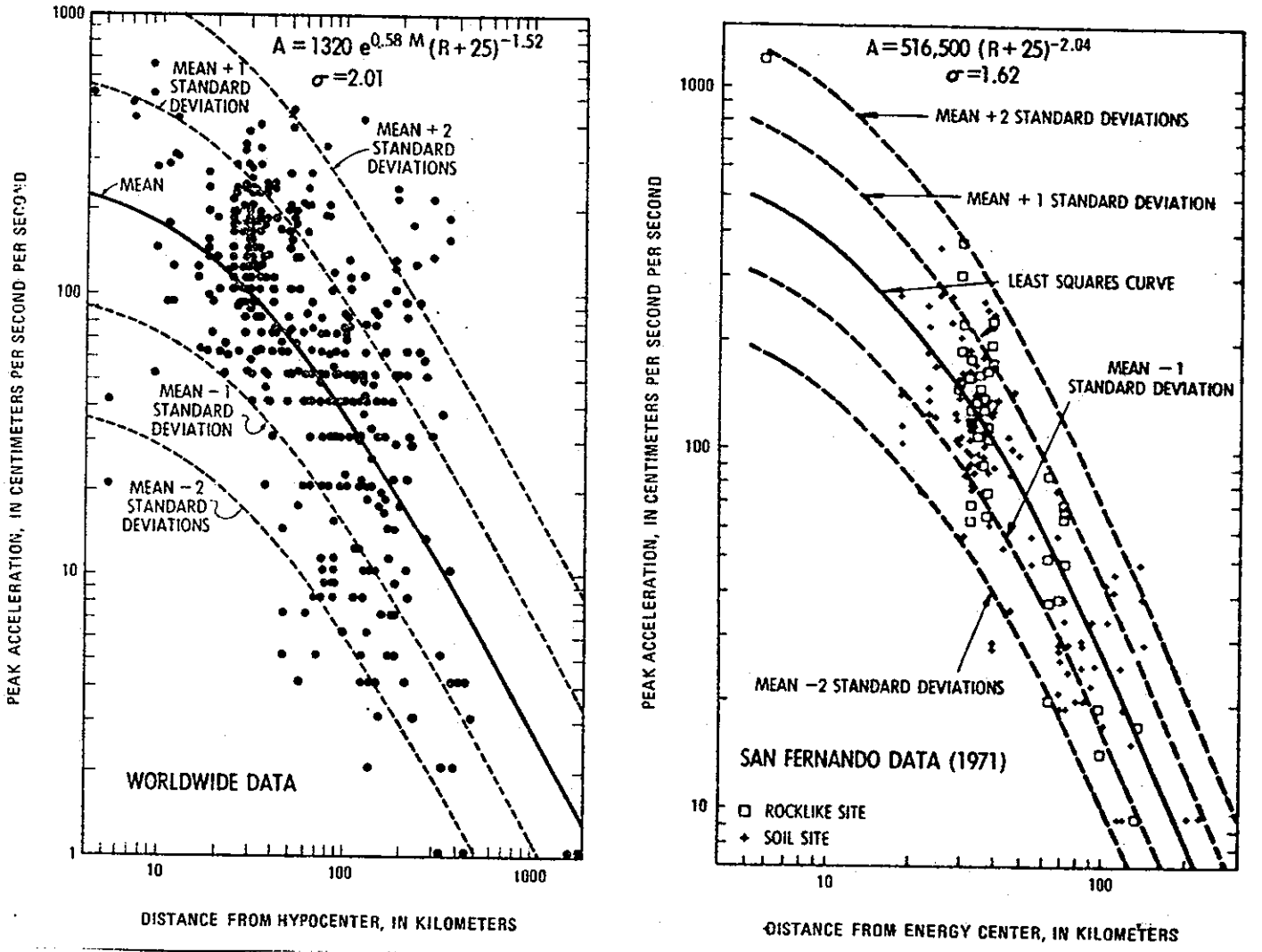

A = 1320 e^0.58 M (R+25)^-1.52
σ = 2.01
MEAN + 1 STANDARD DEVIATION
MEAN + 2 STANDARD DEVIATIONS
MEAN
MEAN − 1 STANDARD DEVIATION
MEAN − 2 STANDARD DEVIATIONS
WORLDWIDE DATA
PEAK ACCELERATION, IN CENTIMETERS PER SECOND PER SECOND
DISTANCE FROM HYPOCENTER, IN KILOMETERS
1000
100
10
1
10
100
1000
A = 516,500 (R+25)^-2.04
σ = 1.62
MEAN + 2 STANDARD DEVIATIONS
MEAN + 1 STANDARD DEVIATION
LEAST SQUARES CURVE
MEAN − 1 STANDARD DEVIATION
MEAN −2 STANDARD DEVIATIONS
SAN FERNANDO DATA (1971)
ROCKLIKE SITE
SOIL SITE
PEAK ACCELERATION, IN CENTIMETERS PER SECOND PER SECOND
DISTANCE FROM ENERGY CENTER, IN KILOMETERS
1000
100
10
10
100

Figure C-5

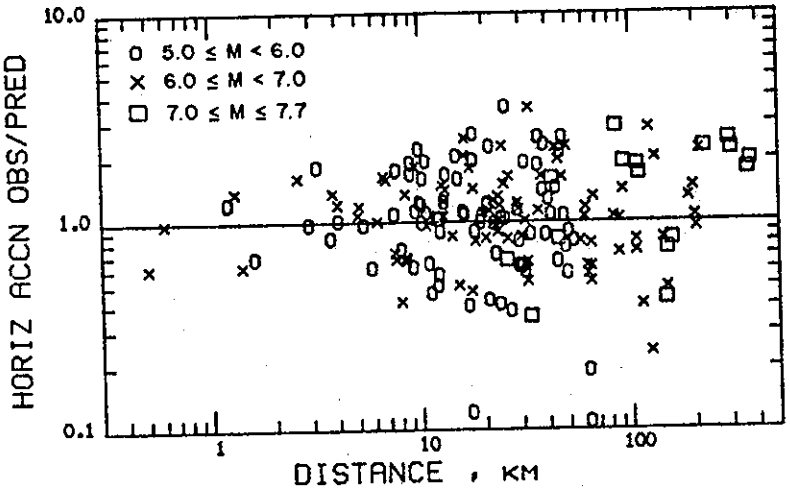

10.0
1.0
0.1
HORIZ ACCN OBS/PRED
O 5.0 ≤ M < 6.0
X 6.0 ≤ M < 7.0
□ 7.0 ≤ M ≤ 7.7
1
10
100
DISTANCE , KM

Figure C-6

Table Captions

Table C-1. Strong motion data for California earthquakes. Part 1 of the table lists the earthquake parameters, part 2 the strong motion data for each earthquake (from Joyner and Boore, 1981, Tables 1 and 2, pp. 2017-2022).

Table C-2. Peak acceleration data for Friuli, Ancona, and Alpide belt. M_L = local magnitude, D = distance (from epicenter (km), SITE = station, A = acceleration cm/sec2, IMM = Intensity (Modified Mercalli), (from Chiaruttini and Siro, 1981, pp. 2008-2009).

SOURCES OF DATA USED IN ASSIGNING MAGNITUDES AND STATION DISTANCES

Earthquake	M	M_L	Date (GMT) Month	Day	Year	Sources
Imperial Valley, California	7.0	6.4	5	19	40	Trifunac and Brune (1970); Trifunac (1972); Richter (1958); Hanks *et al.* (1975).
Kern County, California	7.4	7.2	7	21	52	Richter (1958); Page *et al.* (1972); Bolt (1978); Dunbar *et al.* (1980); Hanks *et al.* (1975); Boore and Kanamori (unpublished data).
Daly City, California	(5.3)	5.3	3	22	57	Tocher (1959); Cloud (1959).
Parkfield, California	6.1	5.5	6	28	66	McEvilly *et al.* (1967); Lindh and Boore (1981); Trifunac and Udwadia (1974); Tsai and Aki (1969).
Borrego Mountain, California	6.6	6.7	4	9	68	Kanamori and Jennings (1978); Hamilton (1972); Hanks and Wyss (1972); Swanger and Boore (1978); Hanks *et al.* (1975).
Santa Rosa, California (2 events)	(5.6) (5.7)	5.6 5.7	10	2	69	Bolt and Miller (1975); Unger and Eaton (1970); J. D. Unger and J. P. Eaton (written communication, 1976).
Lytle Creek, California	5.3	5.4	9	12	70	T. C. Hanks (written communication, 1971); Hanks *et al.* (1975).
San Fernando, California	6.6	6.4	2	9	71	Allen *et al.* (1973); Heaton and Helmberger (1979).
Bear Valley, California	5.3	5.1	2	24	72	Bolt and Millter (1975); Ellsworth (1975); Johnson and McEvilly (1974).
Sitka, Alaska	7.7		7	30	72	Page and Gawthrop (1973); Page (oral communication, 1976); Purcaru and Berckhemer (1978).
Managua, Nicaragua	(6.2)	6.2	12	23	72	Jennings and Kanamori (1979); Plafker and Brown (1973); Ward *et al.* (1973); Knudson and Hansen A. (1973); U.S. Dept. of Commerce (1973).
Point Mugu, California	5.6	6.0	2	21	73	Ellsworth *et al.* (1973); Boore and Stierman (1976); Stierman and Ellsworth (1976).
Hollister, California	(5.2)	5.2	11	28	74	Cloud and Stifler (1976); W. H. K. Lee (written communication, 1976).
Oroville, California	6.0	5.7	8	1	75	Fogleman *et al.* (1977); Bufe *et al.* (1976); Lahr *et al.* (1976); Langston and Butler (1976); Hart *et al.* (1977).
Santa Barbara, California	5.1	5.1	8	13	78	Wallace and Helmberger (1979); Lee *et al.* (1978).
St. Elias, Alaska	7.6		2	28	79	Hasegawa *et al.* (1980); C. D. Stephens (written communication, 1979); J. Boatwright (oral communication, 1979).

Table C-1

Earthquake	M	M_L	Date (GMT)			Sources
			Month	Day	Year	
Coyote Lake, California	5.8	5.9	8	6	79	Uhrhammer (1980); Lee *et al.* (1979).
Imperial Valley, California	6.5	6.6	10	15	79	Kanamori (oral communication, 1981); C. E. Johnson (oral communication, 1979); Boore and Porcella (1981).
Imperial Valley, California aftershock	(5.0)	5.0	10	15	79	C. E. Johnson (oral communication, 1979).
Livermore Valley, California	5.8	5.5	1	24	80	Bolt *et al.* (1981); R. A. Uhrhammer (oral communication, 1981); J. Boatwright (oral communication, 1980).
Livermore Valley, California	5.5	5.6	1	27	80	Bolt *et al.* (1981); R. A. Uhrhammer (oral communication, 1981); J. Boatwright (oral communication, 1980); Cockerham *et al.* (1980).
Horse Canyon, California	(5.3)	5.3	2	25	80	L. K. Hutton (written communication, 1980).

Table C-1 cont.

STRONG-MOTION DATA

Earthquake	Station*	Distance (km)	Peak Horizontal Acceleration (g)	Peak Horizontal Velocity (cm/sec)	Site Condition
Imperial Valley, 1940	117	12.0	0.359	36.9	soil
Kern County, 1952	1083	148.0	0.014		rock
	1095	42.0	0.196	17.7	soil
	283	85.0	0.135	19.3	soil
	135	107.0	0.062	8.9	soil
	475	109.0	0.054	9.1	soil
	113	156.0	0.014		soil
	1008	224.0	0.018		soil
	1028	293.0	0.010		soil
	2001	359.0	0.004		soil
	117	370.0	0.004		soil
Daly City, 1957	1117	8.0	0.127	4.9	rock
Parkfield, 1966	1438	16.1	0.411	22.5	rock
	1083	63.6	0.018	1.1	rock
	1013	6.6	0.509	78.1	soil
	1014	9.3	0.467	25.4	soil
	1015	13.0	0.279	11.8	soil
	1016	17.3	0.072	8.0	soil
	1095	105.0	0.012	2.2	soil
	1011	112.0	0.006		soil
	1028	123.0	0.003		soil
Borrego, Mountain, 1968	270	105.0	0.018		rock
	280	122.0	0.048		rock
	116	141.0	0.011		rock
	266	200.0	0.007		rock
	117	45.0	0.142	25.8	soil
	113	130.0	0.031		soil
	112	147.0	0.006		soil
	130	187.0	0.010		soil
	475	197.0	0.010		soil
	269	203.0	0.006		soil
	135	211.0	0.013		soil
Santa Rosa, 1969 first event	1093	62.0	0.005		soil
Santa Rosa, 1969 second event	1093	62.0	0.003		soil
Lytle Creek, 1970	111	19.0	0.086	5.6	rock
	116	21.0	0.179		rock
	290	13.0	0.205	9.6	soil
	112	22.0	0.073		soil
	113	29.0	0.045		soil
San Fernando, 1971	128	17.0	0.374	14.6	rock
	126	19.6	0.200	8.6	rock
	127	20.2	0.147	4.8	rock
	141	21.1	0.188	20.5	rock
	266	21.9	0.204	11.6	rock
	110	24.2	0.335	27.8	rock
	1027	66.0	0.057	2.8	rock
	111	87.0	0.021		rock
	125	23.4	0.152	18.0	soil
	135	24.6	0.217	21.1	soil
	475	25.7	0.114	14.3	soil

Table C-1 cont.

Earthquake	Station*	Distance (km)	Peak Horizontal Acceleration (g)	Peak Horizontal Velocity (cm/sec)	Site Condition
	262	28.6	0.150	14.2	soil
	269	37.4	0.148	5.4	soil
	1052	46.7	0.112	8.5	soil
	411	56.9	0.043	5.0	soil
	290	60.7	0.057	3.8	soil
	130	61.4	0.030	10.4	soil
	272	62.0	0.027	7.3	soil
	1096	64.0	0.028	1.4	soil
	1102	82.0	0.034	2.5	soil
	112	88.0	0.030		soil
	113	91.0	0.039		soil
Bear Valley, 1972	1028	31.0	0.030		soil
Sitka, 1972	2714	45.0	0.110		rock
	2708	145.0	0.010		rock
	2715	300.0	0.010		soil
Managua, 1972	3501	5.0	0.390		soil
Point Mugu, 1973	655	50.0	0.031		rock
	272	16.0	0.130		soil
Hollister, 1974	1032	17.0	0.011		rock
	1377	8.0	0.120		soil
	1028	10.0	0.170		soil
	1250	10.0	0.140		soil
Oroville, 1975	1051	8.0	0.110	5.0	rock
	1293	32.0	0.040		rock
	1291	30.0	0.070		soil
	1292	31.0	0.080		soil
Santa Barbara, 1978	283	2.9	0.210		
	885	3.2	0.390		
	Goleta substation†	7.6	0.280		
St. Elias, 1979	2734	25.4	0.160		
	Munday Creek‡	32.9	0.064		
	2728	92.2	0.090		
Coyote Lake, 1979	1413	1.2	0.420	43.8	rock
	1445	1.6	0.230	20.5	rock
	1408	9.1	0.130	10.3	rock
	1411	3.7	0.260	32.2	soil
	1410	5.3	0.270	29.4	soil
	1409	7.4	0.260	31.9	soil
	1377	17.9	0.110		soil
	1492	19.2	0.120		soil
	1251	23.4	0.038		soil
	1422	30.0	0.044		soil
	1376	38.9	0.046		soil
Imperial Valley, 1979	Cerro Prieto§	23.5	0.170		rock
	286	26.0	0.210	9.0	rock
	Meloland Overpass¶	0.5	0.320		soil
	5028	0.6	0.520	110.0	soil
	942	1.3	0.720	110.0	soil
	Aeropuerto§	1.4	0.320		soil
	5054	2.6	0.810	44.0	soil
	958	3.8	0.640	53.0	soil
	952	4.0	0.560	87.0	soil
	5165	5.1	0.510	68.0	soil
	117	6.2	0.400		soil

Table C-1 cont.

Earthquake	Station*	Distance (km)	Peak Horizontal Acceleration (g)	Peak Horizontal Velocity (cm/sec)	Site Condition
	955	6.8	0.610	78.0	soil
	5055	7.5	0.260	48.0	soil
	Imperial Co. Center¶	7.6	0.240		soil
	Mexicali SAHOP§	8.4	0.460		soil
	5060	8.5	0.220	37.0	soil
	412	8.5	0.230	44.0	soil
	5053	10.6	0.280	19.0	soil
	5058	12.6	0.380	39.0	soil
	5057	12.7	0.270	46.0	soil
	Cucapah§	12.9	0.310		soil
	5051	14.0	0.200	17.0	soil
	Westmoreland¶	15.0	0.110		soil
	5115	16.0	0.430	31.0	soil
	Chihuahua§	17.7	0.270		soil
	931	18.0	0.150	19.0	soil
	5056	22.0	0.150	15.0	soil
	5059	22.0	0.150	15.0	soil
	5061	23.0	0.130	15.0	soil
	Compuertas§	23.2	0.190		soil
	5062	29.0	0.130		soil
	5052	32.0	0.066		soil
	Delta§	32.7	0.350		soil
	724	36.0	0.100		soil
	Victoria§	43.5	0.160		soil
	5066	49.0	0.140		soil
	5050	60.0	0.049		soil
	2316	64.0	0.034		soil
Imperial Valley, 1979 aftershock	5055	7.5	0.264		
	942	8.8	0.263		
	5028	8.9	0.230		
	5165	9.4	0.147		
	952	9.7	0.286		
	958	9.7	0.157		
	955	10.5	0.237		
	117	10.5	0.133		
	412	12.0	0.055		
	5053	12.2	0.097		
	5054	12.8	0.129		
	5058	14.6	0.192		
	5057	14.9	0.147		
	5115	17.6	0.154		
	5056	23.9	0.060		
	5060	25.0	0.057		
Livermore Valley, 1980 24 January	1030	10.8	0.120		
	1418	15.7	0.154		
	1383	16.7	0.052		
	1308	20.8	0.045		
	1298	28.5	0.086		
	1299	33.1	0.056		
	1219	40.3	0.065		
Livermore Valley, 1980 27 January	Fagundes Ranch¶	4.0	0.259		
	Morgan Terrace Park¶	10.1	0.267		
	1030	11.1	0.071		

Table C-1 cont.

Earthquake	Station*	Distance (km)	Peak Horizontal Acceleration (g)	Peak Horizontal Velocity (cm/sec)	Site Condition
	1418	17.7	0.275		
	1383	22.5	0.058		
	Antioch Contra Loma¶	26.5	0.026		
	1299	29.0	0.039		
	1308	30.9	0.112		
	1219	37.8	0.065		
	1456	48.3	0.026		
Horse Canyon, 1980	5045	5.8	0.123		
	5044	12.0	0.133		
	5160	12.1	0.073		
	5043	20.5	0.097		
	5047	20.5	0.096		
	C168	25.3	0.230		
	5068	35.9	0.082		
	C118	36.1	0.110		
	5042	36.3	0.110		
	5067	38.5	0.094		
	5049	41.4	0.040		
	C204	43.6	0.050		
	5070	44.4	0.022		
	C266	46.1	0.070		
	C203	47.1	0.080		
	5069	47.7	0.033		
	5073	49.2	0.017		
	5072	53.1	0.022		

* Station numbers preceded by the letter *C* are those assigned by the California Division of Mines and Geology. Other numbers are those assigned by the U.S. Geological Survey (1977; the stations not necessarily being USGS stations).

† Station operated by the Southern California Edison Company.

‡ Station operated by the Shell Oil Company.

§ Station operated by the Universidad Nacional Autonoma de Mexico and the University of California at San Diego.

¶ Station operated by the California Division of Mines and Geology.

Table C-1 cont.

FILE RUSM (CREATION DATE = 81/10/13.) (EUROPEAN PEAK ACCELERATION SET)

CASE-NO	ML	D	AREA	SOIL	DATE	TIME	SITE	A	IMM	WEIGHT
1	5.4	29.0	FRIULI	THIN ALL	760506	1959	TOLMEZZO	15.80	6.0	1.00
2	6.2	24.0	FRIULI	THIN ALL	760506	2000	TOLMEZZO	36.70	7.5	1.00
3	6.2	34.0	FRIULI	THICKALL	760506	2000	CODROIPO	8.70	6.5	1.00
4	6.2	40.0	FRIULI	THIN ALL	760506	2000	BARCIS	3.40	6.5	1.00
5	6.2	80.0	FRIULI	THICKALL	760506	2000	CONEGLIA	7.80	6.5	1.00
6	6.2	103.0	FRIULI	ROCKLIKE	760506	2000	FELTRE	4.80	4.5	1.00
7	6.2	110.0	FRIULI	THICKALL	760506	2000	LJUBLJAN	4.00	6.0	1.00
8	6.2	110.0	FRIULI	THICKALL	760506	2000	LJUBLJAN	2.20	6.0	1.00
9	6.2	118.0	FRIULI	THICKALL	760506	2000	CASTELFR	3.10	5.0	1.00
10	6.2	137.0	FRIULI	ROCKLIKE	760506	2000	ASIAGO	3.00	4.5	1.00
11	6.2	158.0	FRIULI	ROCKLIKE	760506	2000	RONSELIC	2.30	4.5	1.00
12	6.2	191.0	FRIULI	ROCKLIKE	760506	2000	MALCESIN	3.60	4.5	1.00
13	4.6	30.0	FRIULI	THIN ALL	760507	0023	TOLMEZZO	12.80	0	1.00
14	4.1	26.0	FRIULI	THIN ALL	760507	1342	TOLMEZZO	3.00	0	1.00
15	4.1	23.0	FRIULI	THIN ALL	760508	0310	TOLMEZZO	4.60	4.5	1.00
16	5.3	24.0	FRIULI	THICKALL	760509	0054	MAJANO	8.40	6.0	1.00
17	2.3	31.0	FRIULI	THIN ALL	760509	0054	FORGARIA	4.30	6.0	1.00
18	4.3	34.0	FRIULI	THIN ALL	760509	0054	TOLMEZZO	4.10	6.0	1.00
19	4.4	17.0	FRIULI	THICKALL	760510	0435	MAJANO	4.80	5.0	1.00
20	4.4	20.0	FRIULI	THIN ALL	760510	0435	FORGARIA	3.10	0	1.00
21	4.8	6.0	FRIULI	THIN ALL	760511	2244	FORGARIA	30.80	8.5	1.00
22	4.8	9.0	FRIULI	THICKALL	760511	2244	MAJANO	8.20	5.5	1.00
23	4.8	16.0	FRIULI	THIN ALL	760511	2244	TARCENTO	7.70	5.0	1.00
24	4.9	16.0	FRIULI	THIN ALL	760511	2244	TOLMEZZO	3.20	0	1.00
25	3.7	4.0	FRIULI	THIN ALL	760513	1304	FORGARIA	7.40	5.0	1.00
26	3.7	14.0	FRIULI	THIN ALL	760513	1304	TARCENTO	1.90	0	1.00
27	3.7	18.0	FRIULI	THIN ALL	760513	1304	TOLMEZZO	1.90	0	1.00
28	3.7	15.0	FRIULI	THIN ALL	760514	0426	FORGARIA	2.80	4.0	1.00
29	3.7	11.0	FRIULI	THIN ALL	760518	0130	FORGARIA	6.20	4.0	1.00
30	3.7	11.0	FRIULI	ROCKLIKE	760518	0130	S.ROCCO	4.90	4.0	1.00
31	3.3	14.0	FRIULI	THIN ALL	760518	0239	FORGARIA	2.60	3.5	1.00
32	3.7	13.0	FRIULI	THIN ALL	760601	1721	FORGARIA	2.70	3.0	1.00
33	3.7	8.0	FRIULI	THIN ALL	760601	1721	TOLMEZZO	1.80	3.5	1.00
34	4.3	20.0	FRIULI	THICKALL	760608	1214	MAJANO	2.00	0	1.00
35	4.3	22.0	FRIULI	THIN ALL	760608	1214	FORGARIA	4.70	0	1.00
36	4.3	14.0	FRIULI	THIN ALL	760608	1214	BREGINJ	10.30	4.5	1.00
37	4.0	6.0	FRIULI	THIN ALL	760609	1848	FORGARIA	7.20	4.5	1.00
38	4.0	6.0	FRIULI	ROCKLIKE	760609	1848	S.ROCCO	3.80	4.5	1.00
39	4.0	10.0	FRIULI	THICKALL	760609	1848	MAJANO	1.50	4.0	1.00
40	4.0	17.0	FRIULI	THIN ALL	760609	1849	TOLMEZZO	3.40	0	1.00
41	4.0	14.0	FRIULI	ROCKLIKE	760609	1848	SOMPLAGO	1.50	4.5	1.00
42	4.2	7.0	FRIULI	THIN ALL	760611	1716	FORGARIA	10.40	5.0	1.00
43	4.2	7.0	FRIULI	ROCKLIKE	760611	1714	S.ROCCO	6.90	4.0	1.00
44	4.2	12.0	FRIULI	THICKALL	760611	1716	MAJANO	1.80	0	1.00
45	4.2	12.0	FRIULI	THICKALL	760611	1716	MAJANO	1.30	0	1.00
46	4.2	14.0	FRIULI	THIN ALL	760611	1716	TOLMEZZO	2.90	0	1.00
47	4.4	15.0	FRIULI	THIN ALL	760617	1428	FORGARIA	5.30	5.0	1.00
48	4.4	15.0	FRIULI	ROCKLIKE	760617	1428	S.ROCCO	7.00	0	1.00
49	4.4	19.0	FRIULI	THICKALL	760617	1428	MAJANO	1.80	2.5	1.00
50	4.4	19.0	FRIULI	THICKALL	760617	1429	MAJANO	1.90	3.5	1.00
51	4.3	16.0	FRIULI	THIN ALL	760626	1113	TOLMEZZO	1.60	0	1.00
52	4.2	15.0	FRIULI	THIN ALL	760714	0539	TARCENTO	3.90	4.5	1.00
53	4.2	16.0	FRIULI	ROCKLIKE	760714	0539	SOMPLAGO	1.70	0	1.00
54	3.8	11.0	FRIULI	THIN ALL	760715	1258	TARCENTO	4.50	4.0	1.00
55	3.7	13.0	FRIULI	THIN ALL	760906	1924	FORGARIA	2.20	4.0	1.00
56	3.7	12.0	FRIULI	THICKALL	760906	1928	BUJA	3.20	0	1.00
57	3.7	10.0	FRIULI	THICKALL	760906	1928	RAPPDANO	8.70	0	1.00
58	3.8	16.0	FRIULI	THIN ALL	760907	1108	TOLMEZZO	1.50	4.0	1.00
59	3.8	7.0	FRIULI	ROCKLIKE	760907	1109	SOMPLAGO	1.70	4.0	1.00
60	3.8	9.0	FRIULI	THIN ALL	760907	1108	FORGARIA	3.30	4.0	1.00
61	5.1	10.0	FRIULI	THIN ALL	760911	1631	TARCENTO	26.40	6.5	1.00
62	5.1	19.0	FRIULI	THIN ALL	760911	1631	BREGINJ	17.10	6.0	1.00
63	5.1	18.0	FRIULI	ROCKLIKE	760911	1631	S.ROCCO	7.10	0	1.00
64	5.1	18.0	FRIULI	THIN ALL	760911	1631	FORGARIA	11.50	0	1.00
65	5.1	31.0	FRIULI	THIN ALL	760911	1631	KOBARID	10.00	5.5	1.00
66	5.1	12.0	FRIULI	THICKALL	760911	1631	BORDANO	13.80	0	1.00
67	5.1	15.0	FRIULI	THICKALL	760911	1631	MOLINIS	7.60	0	1.00
68	5.1	18.0	FRIULI	ROCKLIKE	760911	1631	CORNINO	2.40	0	1.00
69	5.1	12.0	FRIULI	THICKALL	760911	1631	BUJA	4.90	0	1.00
70	5.6	13.0	FRIULI	THICKALL	760911	1635	BUJA	23.30	0	1.00
71	5.6	18.0	FRIULI	THIN ALL	760911	1635	FORGARIA	23.50	0	1.00
72	5.6	18.0	FRIULI	ROCKLIKE	760911	1635	S.ROCCO	9.50	0	1.00
73	5.6	22.0	FRIULI	THIN ALL	760911	1635	BREGINJ	12.30	0	1.00
74	5.6	24.0	FRIULI	ROCKLIKE	760911	1635	ROBIC	5.70	0	1.00
75	5.6	34.0	FRIULI	THIN ALL	760911	1635	KOBARID	9.90	0	1.00
76	5.6	43.0	FRIULI	THICKALL	760911	1635	CODROIPO	2.60	0	1.00
77	5.6	48.0	FRIULI	THIN ALL	760911	1635	BARCIS	3.50	0	1.00
78	5.6	16.0	FRIULI	THICKALL	760911	1635	MOLINIS	24.00	0	1.00
79	5.6	21.0	FRIULI	ROCKLIKE	760911	1635	CORNINO	6.60	0	1.00
80	3.6	13.0	FRIULI	THICKALL	760912	0814	BORDANO	3.60	0	1.00
81	4.3	14.0	FRIULI	THICKALL	760912	1955	BORDANO	5.90	7.0	1.00
82	4.3	8.0	FRIULI	THICKALL	760913	1854	BUJA	6.80	4.5	1.00
83	4.3	17.0	FRIULI	THIN ALL	760913	1854	FORGARIA	2.60	4.5	1.00
84	4.3	19.0	FRIULI	ROCKLIKE	760913	1854	ROBIC	5.00	0	1.00
85	4.3	12.0	FRIULI	THICKALL	760913	1854	BORDANO	8.20	0	1.00
86	3.7	17.0	FRIULI	THICKALL	760914	0825	BORDANO	3.00	0	1.00
87	5.8	42.0	FRIULI	THICKALL	760915	0315	CODROIPO	3.00	6.0	1.00
88	5.8	82.0	FRIULI	THICKALL	760915	0315	CONEGLIA	2.00	5.5	1.00
89	5.8	17.0	FRIULI	THIN ALL	760915	0315	FORGARIA	26.30	8.5	1.00
90	5.8	16.0	FRIULI	ROCKLIKE	760915	0315	S.ROCCO	12.30	8.5	1.00
91	5.8	13.0	FRIULI	THICKALL	760915	0315	BUJA	11.00	8.0	1.00
92	5.8	22.0	FRIULI	THIN ALL	760915	0315	BREGINJ	52.50	7.5	1.00
93	5.8	34.0	FRIULI	THIN ALL	760915	0315	KOBARID	12.60	7.0	1.00
94	5.8	23.0	FRIULI	ROCKLIKE	760915	0315	ROBIC	10.50	7.5	1.00
95	5.8	15.0	FRIULI	THICKALL	760915	0315	MOLINIS	14.30	8.0	1.00
96	5.8	18.0	FRIULI	ROCKLIKE	760915	0315	CORNINO	7.50	8.5	1.00
97	4.7	14.0	FRIULI	THICKALL	760915	0439	BUJA	3.90	4.0	1.00
98	4.7	19.0	FRIULI	THIN ALL	760915	0439	FORGARIA	5.80	0	1.00
99	4.7	22.0	FRIULI	THIN ALL	760915	0439	BREGINJ	7.10	0	1.00
100	6.1	14.0	FRIULI	THIN ALL	760915	0921	TARCENTO	13.90	8.0	1.00
101	6.1	14.0	FRIULI	THICKALL	760915	0921	BUJA	10.10	8.0	1.00
102	6.1	19.0	FRIULI	ROCKLIKE	760915	0921	S.ROCCO	23.80	8.5	1.00
103	6.1	20.0	FRIULI	THIN ALL	760915	0921	FORGARIA	35.30	8.5	1.00
104	6.1	22.0	FRIULI	THIN ALL	760915	0921	BREGINJ	41.90	7.5	1.00
105	6.1	24.0	FRIULI	ROCKLIKE	760915	0921	ROBIC	8.80	7.5	1.00
106	6.1	34.0	FRIULI	THIN ALL	760915	0921	KOBARID	14.30	7.0	1.00
107	6.1	43.0	FRIULI	THICKALL	760915	0921	CODROIPO	5.40	6.0	1.00
108	5.1	47.0	FRIULI	THIN ALL	760915	0921	BARCIS	2.70	7.0	1.00
109	6.1	81.0	FRIULI	THICKALL	760915	0921	CONEGLIA	3.10	5.5	1.00
110	6.1	103.0	FRIULI	ROCKLIKE	760915	0921	FELTRE	2.70	0	1.00
111	6.1	190.0	FRIULI	ROCKLIKE	760915	0921	MALCESIN	2.60	0	1.00
112	6.1	20.0	FRIULI	THICKALL	760915	0921	MOLINIS	10.70	8.0	1.00
113	3.4	17.0	FRIULI	THIN ALL	760915	0937	TOLMEZZO	1.80	4.0	1.00
114	4.3	19.0	FRIULI	THIN ALL	760915	0945	FORGARIA	4.40	4.5	1.00
115	4.3	14.0	FRIULI	THIN ALL	760915	0945	TOLMEZZO	2.90	4.0	1.00
116	4.5	16.0	FRIULI	THIN ALL	760915	1111	TOLMEZZO	2.90	5.0	1.00
117	4.5	21.0	FRIULI	THIN ALL	760915	1111	FORGARIA	4.40	0	1.00
118	4.1	16.0	FRIULI	THIN ALL	760915	1931	TOLMEZZO	3.80	0	1.00
119	3.9	18.0	FRIULI	THIN ALL	760915	2024	TOLMEZZO	1.50	0	1.00
120	3.7	14.0	FRIULI	THIN ALL	760916	0130	TOLMEZZO	2.20	0	1.00

Table C-2

ANCONA SAMPLE OF PEAK ACCELERATION DATA 81/10/14. 14.53.42. PAGE 3

FILE EUSM (CREATION DATE = 81/10/13.) (EUROPEAN PEAK ACCELERATION SET)

CASE-NO	ML	D	AREA	SOIL	DATE	TIME	SITE	A	IPM	WEIGHT
1	4.7	6.0	ANCONA	ALL.LIKE	720614	1855	ANCONA	43.80	7.5	1.00
2	4.7	11.0	ANCONA	ALL.LIKE	720614	1855	ANCONA	16.60	7.5	1.00
3	4.7	6.0	ANCONA	ROCKLIKE	720614	1855	ANCONA	61.00	7.5	1.00
4	4.2	7.0	ANCONA	ALL.LIKE	720614	2101	ANCONA	24.40	6.0	1.00
5	4.2	4.0	ANCONA	ROCKLIKE	720614	2101	ANCONA	51.00	6.0	1.00
6	4.2	4.0	ANCONA	ALL.LIKE	720614	2101	ANCONA	42.20	6.0	1.00
7	3.5	5.0	ANCONA	ROCKLIKE	720615	1010	ANCONA	6.30	4.5	1.00
8	4.0	4.0	ANCONA	ALL.LIKE	720621	1506	ANCONA	41.50	5.5	1.00
9	4.0	7.0	ANCONA	ROCKLIKE	720621	1506	ANCONA	22.00	5.5	1.00
10	4.0	10.0	ANCONA	ROCKLIKE	720621	1506	ANCONA	3.00	5.5	1.00
11	4.0	7.0	ANCONA	ALL.LIKE	720621	1506	ANCONA	9.70	5.5	1.00
12	3.4	5.0	ANCONA	ALL.LIKE	720627	0246	ANCONA	5.20	4.0	1.00
13	3.5	8.0	ANCONA	ALL.LIKE	720204	0530	ANCONA	2.90	4.0	1.00
14	4.2	12.0	ANCONA	ALL.LIKE	720205	0704	ANCONA	3.80	6.5	1.00
15	3.4	8.0	ANCONA	ALL.LIKE	720205	1347	ANCONA	2.90	4.5	1.00
16	4.2	13.0	ANCONA	ALL.LIKE	720205	1514	ANCONA	6.30	6.5	1.00
17	4.2	9.0	ANCONA	ALL.LIKE	720206	0134	ANCONA	22.50	6.5	1.00
18	4.2	9.0	ANCONA	ALL.LIKE	720206	0134	ANCONA	6.80	6.5	1.00
19	3.9	13.0	ANCONA	ALL.LIKE	720206	2144	ANCONA	4.50	5.5	1.00
20	3.9	9.0	ANCONA	ALL.LIKE	720208	1219	ANCONA	11.20	5.5	1.00
21	3.9	9.0	ANCONA	ALL.LIKE	720208	1219	ANCONA	13.40	5.5	1.00
22	3.9	9.0	ANCONA	ALL.LIKE	720208	1219	ANCONA	7.90	5.5	1.00
23	3.7	16.0	ANCONA	ALL.LIKE	720314	2303	ANCONA	5.60	4.0	1.00
24	3.7	24.0	ANCONA	ALL.LIKE	720314	2303	ANCONA	2.50	4.0	1.00
25	3.5	6.0	ANCONA	ALL.LIKE	720404	0826	ANCONA	6.50	4.5	1.00
26	3.5	5.0	ANCONA	ALL.LIKE	720404	0826	ANCONA	9.00	4.5	1.00
27	3.7	8.0	ANCONA	ALL.LIKE	720530	2356	ANCONA	4.80	5.5	1.00
28	4.0	21.0	ANCONA	ALL.LIKE	720125	2022	ANCONA	4.00	6.5	1.00
29	3.8	9.0	ANCONA	ALL.LIKE	720125	2322	ANCONA	7.60	6.0	1.00
30	3.5	12.0	ANCONA	ALL.LIKE	720126	1050	ANCONA	4.70	6.0	1.00
31	4.4	5.0	ANCONA	ALL.LIKE	720204	0242	ANCONA	13.10	7.5	1.00
32	4.4	13.0	ANCONA	ALL.LIKE	720204	0918	ANCONA	12.80	7.5	1.00
33	4.4	25.0	ANCONA	ALL.LIKE	720204	1719	ANCONA	6.20	6.5	1.00
34	4.0	8.0	ANCONA	ALL.LIKE	720204	1817	ANCONA	9.30	6.0	1.00
35	3.8	10.0	ANCONA	ALL.LIKE	720204	1902	ANCONA	5.00	5.0	1.00
36	4.2	6.0	ANCONA	ALL.LIKE	720205	0126	ANCONA	20.30	7.0	1.00
37	3.6	8.0	ANCONA	ALL.LIKE	720205	0349	ANCONA	10.00	5.0	1.00
38	3.6	8.0	ANCONA	ALL.LIKE	720205	0349	ANCONA	4.70	5.0	1.00
39	3.9	5.0	ANCONA	ALL.LIKE	720205	0505	ANCONA	9.60	6.0	1.00
40	3.9	5.0	ANCONA	ALL.LIKE	720205	0505	ANCONA	8.60	6.0	1.00

ALPIDE BELT SAMPLE OF PEAK ACCELERATION DATA 81/10/14. 12.07.22. PAGE 3

FILE EUSM (CREATION DATE = 81/10/13.) (EUROPEAN PEAK ACCELERATION SET)

CASE-NO	ML	D	AREA	SOIL	DATE	TIME	SITE	A	IPM	WEIGHT
1	3.6	6.0	USR	ALL.LIKE	660507	2110	TASHKENT	4.00	6.0	.13
2	5.2	8.0	USR	ALL.LIKE	660425	2322	TASHKENT	17.70	7.5	.13
3	4.1	7.0	USR	ALL.LIKE	660509	1843	TASHKENT	8.60	6.5	.13
4	4.1	7.0	USR	ALL.LIKE	660509	1850	TASHKENT	35.90	6.0	.13
5	4.1	7.0	USR	ALL.LIKE	660509	1850	TASHKENT	5.00	6.0	.13
6	3.5	4.0	USR	ALL.LIKE	660524	0749	TASHKENT	3.70	6.0	.13
7	3.6	3.0	USR	ALL.LIKE	660604	2111	TASHKENT	3.50	6.0	.13
8	3.6	3.0	USR	ALL.LIKE	660604	2111	TASHKENT	70.00	6.0	.13
9	3.4	4.0	USR	ALL.LIKE	660629	0900	TASHKENT	16.00	6.0	.13
10	3.4	4.0	USR	ALL.LIKE	660629	0900	TASHKENT	6.50	6.0	.13
11	4.0	4.0	USR	ALL.LIKE	660704	1422	TASHKENT	23.20	6.5	.13
12	4.0	4.0	USR	ALL.LIKE	660704	1422	TASHKENT	4.70	6.5	.13
13	3.5	6.0	USR	ALL.LIKE	661013	1110	TASHKENT	3.30	5.5	.13
14	3.7	5.0	USR	ALL.LIKE	670124	0704	TASHKENT	5.90	5.5	.13
15	2.9	5.0	USR	ALL.LIKE	670515	1643	TASHKENT	2.40	5.5	.13
16	5.4	36.0	YUG	ROCKLIKE	671202	1244	DEBAR	7.40	5.5	1.00
17	3.7	11.0	USR	ROCKLIKE	680626	0208	KADJARAN	14.00	4.5	.40
18	3.7	7.0	USR	ROCKLIKE	680726	2322	KADJARAN	41.00	6.0	.40
19	3.3	5.0	USR	ROCKLIKE	680901	1643	KADJARAN	19.00	4.5	.40
20	3.4	5.0	USR	ROCKLIKE	680903	0138	KADJARAN	29.30	4.5	.40
21	3.5	4.0	USR	ROCKLIKE	680904	1913	KADJARAN	37.00	5.0	.40
22	3.3	6.0	ITL	ALL.LIKE	701121	0850	MIGNANO	1.70	5.5	1.00
23	4.0	11.0	ITL	ALL.LIKE	710214	0004	VALFABRI	6.00	6.5	1.00
24	4.5	30.0	PAK	ROCKLIKE	720417	0225	GEIGMEHR	22.60	4.5	1.00
25	4.6	30.0	IRN	ROCKLIKE	720424	1441	GHIR	5.90	4.5	1.00
26	6.6	10.0	GRC	ROCKLIKE	720917	1407	ARGOSTOL	17.20	6.0	1.00
27	5.3	30.0	GRC	ROCKLIKE	721030	1432	ARGOSTOL	16.00	4.5	1.00
28	3.0	2.0	YUG	ROCKLIKE	730118	0806	RIJEKA	10.80	5.0	1.00
29	4.5	22.0	YUG	ROCKLIKE	730606	2112	HERCEGNO	5.90	6.0	1.00
30	3.7	11.0	YUG	ALL.LIKE	730603	0639	BANJALUK	3.00	5.0	1.00
31	3.5	15.0	YUG	ALL.LIKE	730921	0138	JAJCE	2.50	5.0	1.00
32	6.4	16.0	GRC	ALL.LIKE	731104	1352	LEUKAS	52.30	7.0	1.00
33	4.5	12.0	GRC	ALL.LIKE	731104	1611	LEUKAS	7.80	5.0	1.00
34	3.5	14.0	PRT	ROCKLIKE	731123	1336	AZORES	27.00	5.0	1.00
35	4.9	22.0	YUG	ALL.LIKE	731129	1647	SLAVBROD	10.40	5.0	1.00
36	5.2	13.0	PRT	ALL.LIKE	731211	0010	AZORES	21.00	6.5	1.00
37	4.0	10.0	GRC	ROCKLIKE	740129	1512	PATRAS	4.90	5.0	1.00
38	2.7	2.0	YUG	ALL.LIKE	740505	1824	SKOPJE	2.00	5.5	1.00
39	4.8	15.0	YUG	ROCKLIKE	740523	1951	INOTSKI	17.80	6.0	1.00
40	4.4	41.0	YUG	ALL.LIKE	740620	0928	JAJCE	3.00	5.5	1.00
41	5.1	53.0	YUG	ALL.LIKE	740620	1708	LASKO	4.00	5.5	1.00
42	6.8	135.0	PAK	ALL.LIKE	741228	1211	TARBELA	3.50	3.5	1.00
43	6.8	135.0	PAK	ALL.LIKE	741228	1211	TARBELA	1.50	3.5	1.00
44	3.7	21.0	YUG	ALL.LIKE	750217	1425	BANJALUK	4.00	4.5	1.00
45	6.1	45.0	IRN	ROCKLIKE	750307	0704	BANDARAB	14.10	7.0	1.00
46	5.6	60.0	GRC	ROCKLIKE	750404	0516	PATRAS	5.00	4.5	1.00
47	4.5	40.0	GRC	ALL.LIKE	750513	0025	XYLOKAST	7.90	4.5	1.00
48	4.6	42.0	ITL	ROCKLIKE	750610	1011	SM-LAMIS	3.00	4.0	1.00
49	5.1	16.0	GRC	ALL.LIKE	751012	0423	CORINTH	3.10	4.5	1.00
50	4.0	10.0	IRN	ALL.LIKE	760410	0130	SHIRAZ	7.40	5.0	1.00
51	7.5	22.0	USR	ROCKLIKE	760517	0258	KARAKYR	130.00	9.0	1.00
52	5.5	16.0	IRN	ALL.LIKE	761107	0400	GHAIN	21.90	7.5	1.00
53	5.3	18.0	IRN	ALL.LIKE	761109	1759	GHAIN	57.20	6.5	1.00
54	6.8	55.0	IRN	ROCKLIKE	761124	1222	NAKU	10.10	5.0	1.00
55	7.8	120.0	ROM	ROCKLIKE	770304	1921	VRINCOAI	32.00	7.0	1.00
56	7.9	109.0	ROM	ALL.LIKE	770304	1921	BUCHARES	20.60	7.0	1.00
57	7.8	490.0	YUG	ROCKLIKE	770304	1921	NISH	4.30	5.5	1.00
58	6.6	57.0	IRN	ALL.LIKE	770321	2118	BANDARAB	15.90	6.5	1.00
59	5.9	25.0	IRN	ROCKLIKE	770406	1336	NAGHAN	108.10	7.5	1.00
60	7.2	72.0	YUG	ALL.LIKE	780620	2004	GEVGELIJ	4.60	5.0	1.00
61	5.2	20.0	GRC	ALL.LIKE	780704	2223	THESSALO	24.00	6.0	1.00
62	5.2	20.0	GRC	ALL.LIKE	780704	2223	THESSALO	11.00	6.0	1.00
63	5.2	20.0	GRC	ROCKLIKE	780704	2223	THESSALO	10.00	6.0	1.00
64	5.2	25.0	GRC	ALL.LIKE	780704	2223	SCHOLARI	5.00	5.5	1.00

Table C-2 cont.

APPENDIX C

STRONG MOTIONS FOR USE IN ANALYSIS OF KNIGHTVILLE DAM REPORT

STRONG MOTIONS FOR USE IN ANALYSIS OF KNIGHTVILLE DAM HUNTINGTON, Mass.

prepared by

Robert V. Whitman
Cambridge, Massachusetts

for

Tippetts-Abbett-McCarthy-Stratton

June 1982

INTRODUCTION

The report Seismic Hazard Analysis of the Knightville Dam by M. Nafi Toksoz has recommended that two events be considered as a basis for safety evaluation of the dam:

* A regional event: an earthquake of $m_b = 6.5$ in the region of Cape Ann, Massachusetts, about 200 km from the site.

* A local event: an earthquake of $m_b = 4.0$ centered 10 to 15 km from the site.

Toksoz includes that the Maximum Credible ground shaking from both these events has a peak acceleration of 200 cm/s^2 while the Operating Basis shaking for the regional event is characterized by 84 cm/s^2 and the local event by 200 cm/s^2.

This report recommends time series of acceleration corresponding to these two events, for the use in dynamic analysis of the dam.

APPROACH

The best approach to the designation of time series of acceleration is to select actual recordings of ground motion made during events having approximately the same magnitude and epicentral distance by instruments located at sites with more-or-less the proper geological conditions. This approach should, in general, ensure that the time series as originally recorded have about the peak acceleration sought for purposes of analysis, and also the proper predominant frequencies and duration. Time series selected in this way may then be scaled to the designated peak accelerations with reasonable confidence.

There are, of course, various difficulties in the application of this approach in practice. One is that different time series recorded for similar conditions can cause quite different dynamic response in a structure, owing to the importance of the fine detail of the time series. Hence, if only one time series is used as input for dynamic analysis, there is a risk of underestimating response seriously. On the other hand, performing multiple analyses can be expensive. Several methods have been used to resolve this dilemma.

* Use of an artificially-generated time series in which all frequencies are represented strongly. This approach, which once was popular, is now recognized to be potentially too conservative.

* Selecting a time-series with a predominant period equal to the fundamental period of the structure to be analyzed. (This period is estimated as about 0.5s for Knightville Dam.) While this may provide conservative results when the structure remains linear, it is not certain to give a realistic result when significant non-linearity occurs.

* Arbitrarily increasing the scale of a single time series in the hope that this will lead to conservative results.

It is far better to use several time series as input to repetive analyses; however, in accordance with the requirements of the contract for this project only one time series is specified.

Another difficulty is locating enough different records that fulfill the ideal criteria. This problem has now been somewhat alleviated, at least for earthquakes of moderate magnitude, by the rapid growth in the number of actual recordings.

Still another problem is the influence of local site conditions. Knightville Dam is underlain by schist. The right portion of the dam rests directly on this rock, while under the left side there is intervening glacial till. This may be characterized as a "hard" site, and it is hence appropriate to use records from instruments located upon rock or hard ground.

For this present task, a major problem is the lack of strong motion recordings from the Eastern United States, together with compelling indirect evidence that attenuation of motions with distance may be much slower in the east than in California from which most of the recordings come. At present, this difficulty can only be handled through judgment.

REGIONAL EVENT

The difficulties in applying the ideal criteria immediately become evident in this case. Among the available records, those for appropriate magnitude, epicentral distance and site conditions all have peak accelerations much less than 100 cm/s^2. This is because the available records all come from California, where the attenuation of peak acceleration apparently is much more rapid than in New England.

Given this dilemma, the approach used has been to select records which resulted from about the correct magnitude, which were obtained for the most correct site conditions and which had about the peak acceleration predicted for the Knightville site. Thus, the following time series is selected:

* The N55E component of the record at Puddingstone Reservoir during the San Fernando earthquake of 9 February 1971. This is record P223 in the library complied by the California Institute of Technology.

This time series is reproduced in Figure 1 and its response spectra for 5% damping is shown in Figure 3.

The San Fernando earthquake had $m_b = 6.4$. The recording stations were between 65 and 75 km from the epicenter. The peak acceleration in the record at Puddingstone Reservoir was 70 cm/s^2. This value approximate the 84 cm/s^2 predicted at Knightville Dam for the operating regional event. The predominant frequencies in the record, as indicated by the peaks to the response spectrum, are in the range from 1 to 5 Hz. The duration of the strongest part of the shaking is nearly 10 seconds. The site at Puddingstone Reservoir consists of volcanic clastics and intrusions with associated shales. This record can reasonably be normalized to 84 cm/s^2 or 200 cm/s^2.

LOCAL EVENTS

The time series recommended for use in representing ground motions from such an event is:

* The S80E component of the record at Golden Gate Park during the San Francisco Earthquake of 22 March 1957. This is record A015 in the library compiled by the California Institute of Technology.

The time series for the Golden Gate Park record is reproduced in Figure 2, and the corresponding 5% damped response spectrum appears in Figure 3.

The San Francisco earthquake had $m_b = 4.9$. While these magnitudes are larger than the postulated event with $m_b = 4.0$, the durations and frequency contents lie within the range that might be expected from a local event. The peak acceleration at Golden Gate Park was 103 cm/s^2. The instrument at Golden Gate Park was founded upon chert with thin interbedded shale, and was about 12 km from the epicenter.

This time series can reasonably be normalized either to 100 cm/s^2 or to 200 cm/s^2. These ground motions will cause less dynamic response of the dam than the motions, with the same peak acceleration, from the regional event.

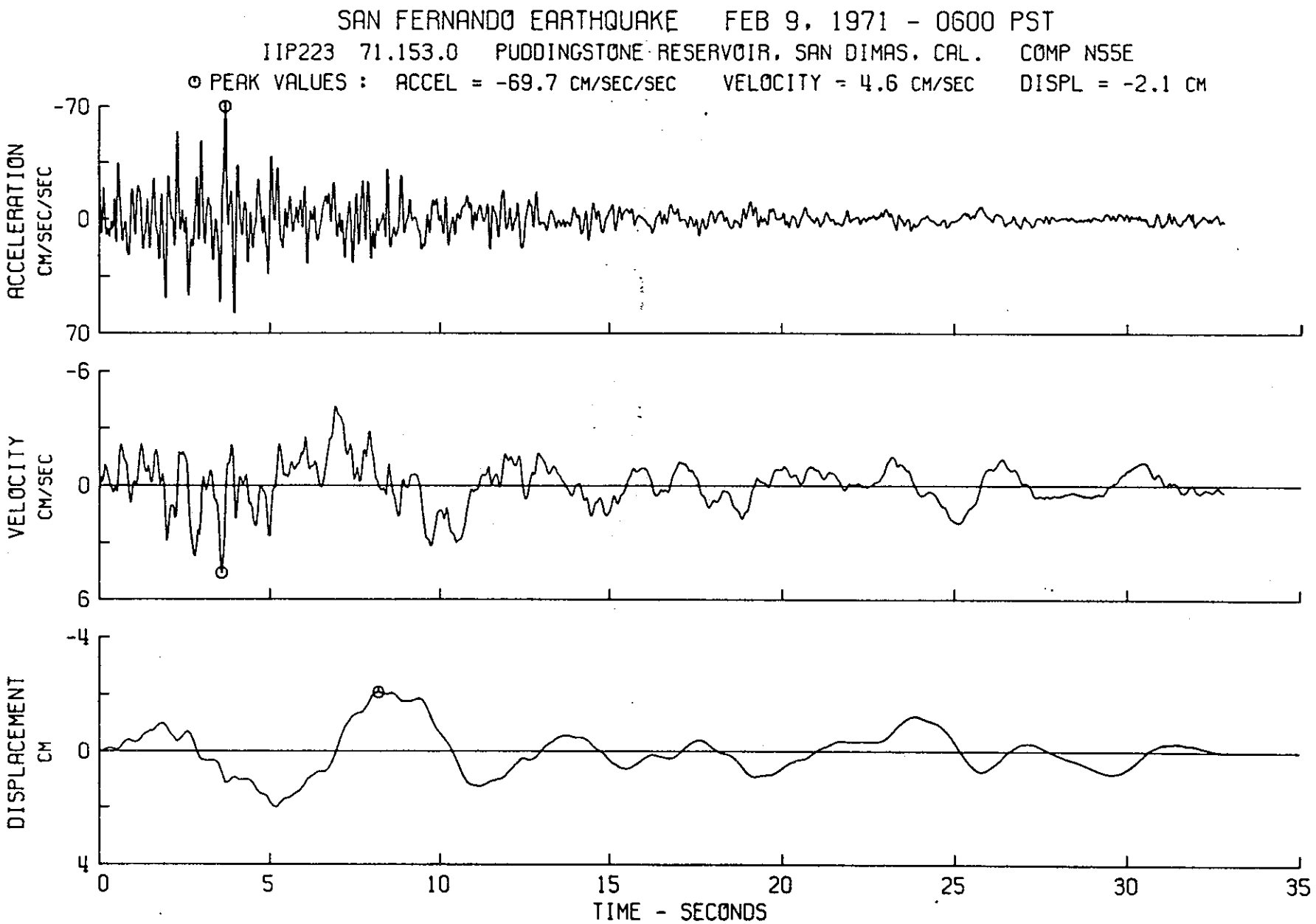

SAN FERNANDO EARTHQUAKE FEB 9, 1971 - 0600 PST
IIP223 71.153.0 PUDDINGSTONE RESERVOIR, SAN DIMAS, CAL. COMP N55E
⊙ PEAK VALUES : ACCEL = -69.7 CM/SEC/SEC VELOCITY = 4.6 CM/SEC DISPL = -2.1 CM
ACCELERATION CM/SEC/SEC
-70
0
70
VELOCITY CM/SEC
-6
0
6
DISPLACEMENT CM
-4
0
4
0
5
10
15
20
25
30
35
TIME - SECONDS

FIGURE 1

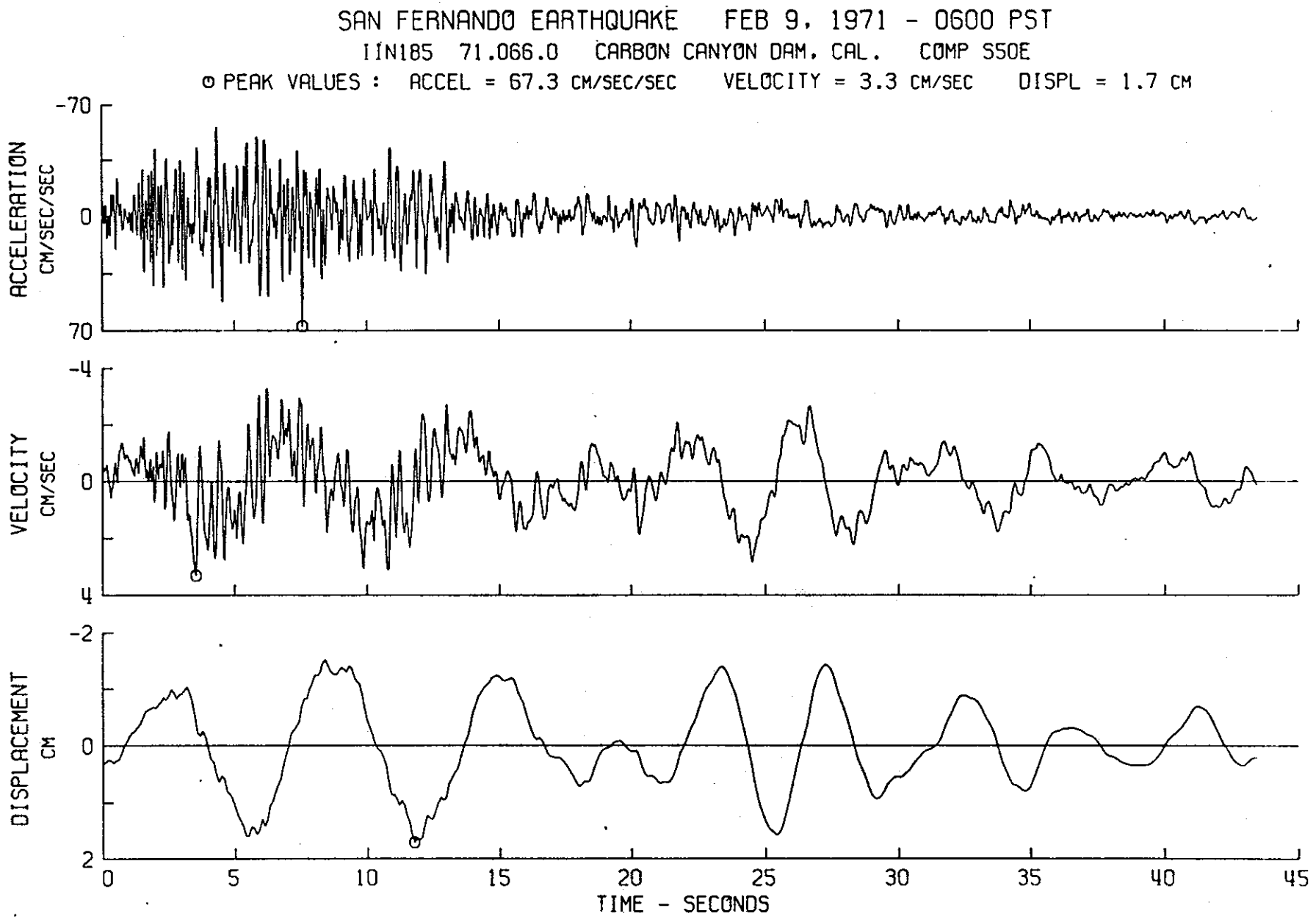

SAN FERNANDO EARTHQUAKE FEB 9, 1971 - 0600 PST
IIN185 71.066.0 CARBON CANYON DAM, CAL. COMP S50E
O PEAK VALUES : ACCEL = 67.3 CM/SEC/SEC VELOCITY = 3.3 CM/SEC DISPL = 1.7 CM
ACCELERATION CM/SEC/SEC
-70
0
70
VELOCITY CM/SEC
-4
0
4
DISPLACEMENT CM
-2
0
2
0
5
10
15
20
25
30
35
40
45
TIME - SECONDS

FIGURE 2

Carbon Canyon Dam — Comp S 50 E ----

Puddingstone Reservoir — Comp N 55 E ———

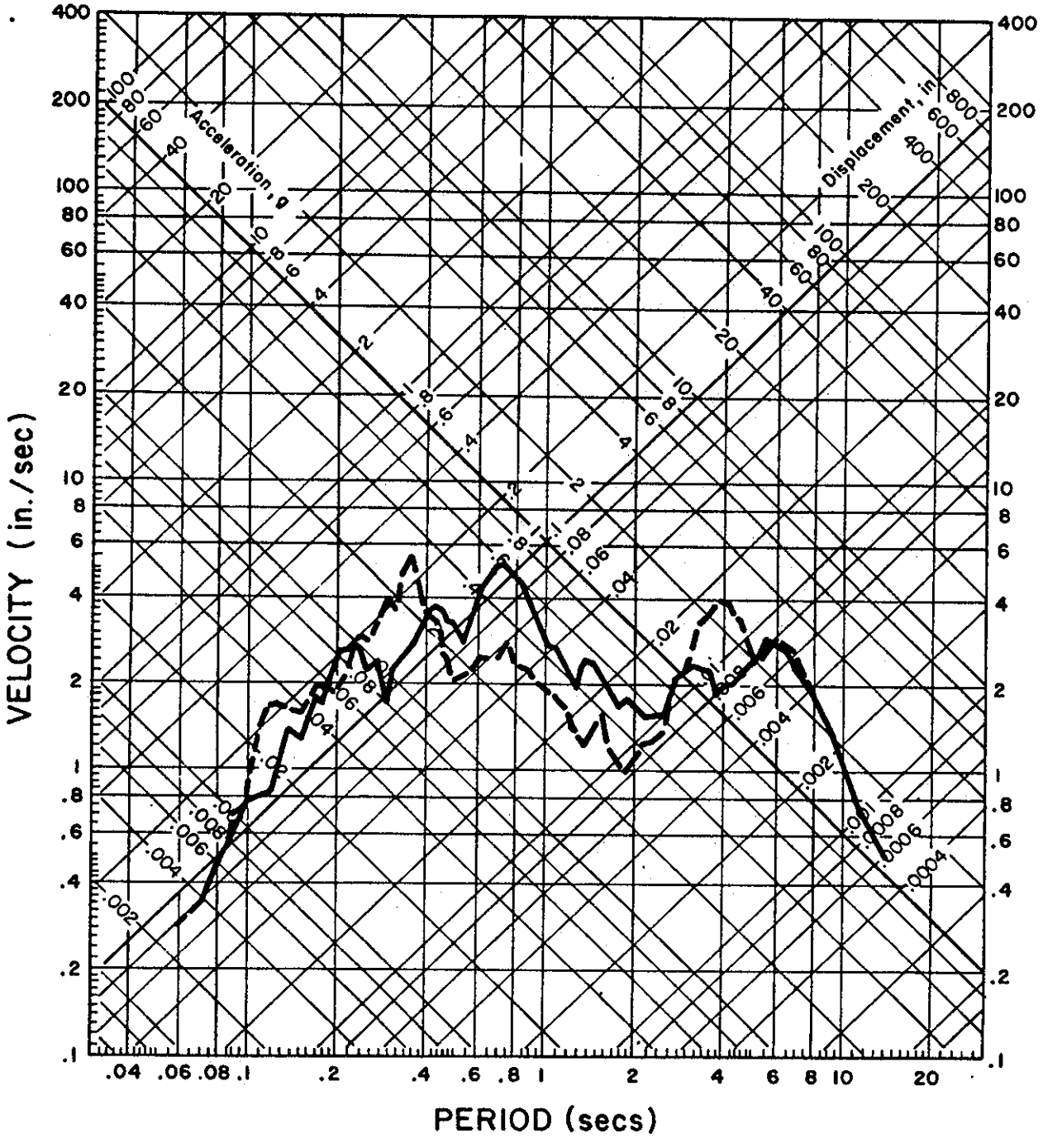


FIGURE 3

APPENDIX D

STATIC LABORATORY TESTING PROGRAM DATA

Sections

D1 - Summary of Laboratory Test Results Sheets

D2 - Triaxial Compression Test Results

D3 - Consolidation Test Results

D4 - Compaction Test Results

Section D1

Summary of Laboratory Test Results Sheets

Table D1	Sheet 1 of 7
Table D2	Sheet 2 of 7
Table D3	Sheet 3 of 7
Table D4	Sheet 4 of 7
Table D5	Sheet 5 of 7
Table D6	Sheet 6 of 7
Table D7	Sheet 7 of 7

Table D1

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content - %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	% Sand	% Fines	Hydrometer Analysis	Compaction	Unconfined Compression	Consolidation	Triaxial
K-1	S-1	Brown GRAVEL and SAND, some silt (GW)													
K-1	S-2	Brown-gray SAND and SILT, tr. of gravel (SW,ML)	14.0												
K-1	S-4	Brown Sandy SILT, trace of gravel (ML)													
K-1	S-6	Brown SAND and SILT, tr.+ gravel, slightly	13.6		NP										
K-1	S-7	Micaceous (Combined 6 and 7)(SM-ML,ML)	11.2		NP			10	45	45					
K-1	S-8	Brown SAND & SILT, tr.+ gravel, slightly micac.													
K-1	S-9	Brown SAND & SILT, tr.+ gravel, slightly micac.	15.4												
K-1	S-10	Brown SILT and fine SAND (ML,SM)	23.8												
K-1	S-11	Brown SANDY SILT (SM-ML)	30.0												
K-1	S-12	Brown SILT, some sand (ML)													
K-1	S-13	Brown SILT, some sand (ML)													
K-1	S-14	Gray-Brown SILT, some+ fine sand (ML)			NP			0	18	82					
K-1	S-15	Brown SILT, some fine sand (ML)													
K-1	S-16	Brown SILT, some fine sand (ML)													
K-1	S-17	Brown SILT, some fine sand (ML)													
K-1	S-18	Brown SILT, some fine sand (ML)													
K-1	S-19	Brown Sandy SILT (ML)													
K-1	S-20	Gray-Brown SILT, some fine sand (ML)													
K-1	S-21	Brown SILT, some fine sand (ML)	39.0												
K-1	S-22	Brown SILT, some clay, some- fine sand (ML)													
K-1	S-23	Brown SILT, some fine sand (ML)													

Note: Asterisk indicates that test results are shown on separate sheets.

FORM NO 170-

Table D2

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	% Sand	% Fines	Hydrometer Analysis	Compaction	Unconfined Compression	Consolidation	Triaxial
K-1	S-24	Brown SILT, some clay, some fine sand (ML)													
K-1	S-25	Gray-Brown SILT, trace fine sand (ML)		29	NP										
K-1	S-26	Brown SILT, some fine sand (ML)													
K-1	S-27	Brown SILT, some fine sand (ML)	40.7												
K-1	S-28	Brown SILT, some fine sand (ML)													
K-1	S-29	Brown Sandy SILT (SM-ML)	29.7												
K-1	S-30	Brown Sandy SILT (SM-ML)													

Note Asterisk indicates that test results are shown on separate sheets.

FORM NO 170-10-6

Table D3

SUMMARY OF LABORATORY TEST RESULTS
prepared for:
NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	SIEVE ANALYSIS % Sand	SIEVE ANALYSIS % Fines	Hydrometer Analysis	Consolidation	Static Triaxial	Dynamic Triaxial	Resonant Column
K-2	S-1	Brown Gravelly SAND (SW)													
K-2	S-2	Brown SAND, sm.f. gravel (SW)													
K-2	S-3	Brown Clayey SAND, tr.f. gravel (SC)	14.1												
K-2	S-4	Gray Sandy CLAY (CL)		20	2										
K-2	S-5	Gray Sandy CLAY, sm.-gravel (CL)													
K-2	S-6	Gray Sandy CLAY, sm.-gravel (CL)	11.5												
K-2	S-7	Gray Brown GRAVEL, sm. sand, tr. silt (GW)													
K-2	S-8	Brown Silty f. SAND (SM)	24.1												
K-2	S-9	Brown Clayey SILT, sm.f. sand (ML)													
K-2	SP1	Brown SILT and f. SAND (upper) (ML,SM)	31.0			2.74	84.7		50	50	*			*	
K-2	SP1	Brown Silty f. SAND (upper middle) (SM)	26.6				90.3		35	65					*
K-2	SP1	Brown SILT, sm.f. sand (lower middle) (ML)	----				----		33	67	*				
K-2	SP2	Brown SILT and f. SAND (ML,SM)	36.1				85.0		35	65	*			*	
K-2	SP3	Brown Sandy SILT (upper) (SM-ML)	30.3	24	2	2.81	95.0		29	71			*		
K-2	SP3	Brown SILT, sm.f. sand (lower) (ML)	31.8				90.7					*			
K-2	SP4	Brown SILT and f. SAND (upper) (ML,SM)	24.5	23	NP		93.0		49	51			*		
K-2	SP4	Brown SILT and f. SAND (lower) (ML,SM)	28.1				91.0		45	55			*		
K-2	SP5	Brown SILT and f. SAND (middle) (ML,SM)	31.0				90.6		60	40	*			*	
K-2	SP7	Brown SILT, sm.f. sand (upper) (ML)	39.4			2.81	80.4					*			
K-2	SP7	Brown Sandy SILT (middle) (SM-ML)	42.5				79.9		29	71			*		
K-2	SP7	Brown SILT, sm.f. sand (lower) (ML)	38.6	32	NP		84.1		15	85			*		

Note: Asterisk indicates that test results are shown on separate sheets.

Table D4

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massacusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	SIEVE ANALYSIS % Sand	SIEVE ANALYSIS % Fines	Hydrometer Analysis	Consolidation	Static Triaxial	Dynamic Triaxial	Resonant Column
K-2	SP8	Brown Sandy SILT (upper) (SM-ML)	37.7				84.7		35	65			*		
K-2	SP8	Brown SILT, sm.f. sand (lower) (ML)	35.7				87.5		25	75			*		
K-2	SP9	Brown Silty f. SAND (upper middle) (SM)	29.3				90.9		32	68				*	
K-2	SP9	Brown SILT, tr.f. sand (lower middle) (ML)	----				----		7	93	*				
K-2	SP9	Brown SILT, sm.f. sand (ML)	34.6				87.4		12	88	*			*	
K-2	SP10	Brown SILT, tr.f. sand (upper) (ML)	39.0				82.3		13	87	*			*	
K-2	SP10	Brown Sandy SILT (lower) (SM-ML)	35.9				84.9		25	75	*			*	
K-2	SP11	Brown SILT, sm.f. sand (upper) (ML)	38.3	33	NP	2.78	84.1		18	82			*		
K-2	SP11	Brown SILT, sm.f. sand (middle) (ML)	41.3				79.4					*			
K-2	SP11	Brown SILT, tr.f. sand (lower) (ML)	37.7				84.2		9	91			*		
K-2	SP12	Brown SILT, sm.f. sand (upper) (ML)	37.7			2.81	84.7		9	91			*		
K-2	SP12	Brown SILT, tr. sand (lower) (ML)	38.9				83.3		9	91			*		
K-2	SP13	Brown SILT, tr. f. sand (middle) (ML)	40.7				81.0		3	97	*			*	
K-2	SP13	Brown SILT, tr.f. sand (lower) (ML)	38.0				83.7		9	91	*				*
K-2	SP14	Brown SILT, tr.f. sand (middle) (ML)	32.7			2.76	84.6		8	92	*			*	
K-2	SP16	Brown SILT, sm.f. sand, tr.clay (upper) (ML)	32.2	35	5	2.77	89.8		9	91		*			
K-2	SP16	Brown SILT, sm.f. sand, tr.clay (middle) (ML)	31.4				92.4		9	91			*		
K-2	SP16	Brown Sandy SILT (lower) (SM-ML)	34.4				88.4		30	70			*		
K-2	SP17	Brown SILT, sm. sand (ML)	32.0	28	5		92.0		15	85			*		

Note: Asterisk indicates that test results are shown on separate sheets.

Table D5

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	SIEVE ANALYSIS % Sand	SIEVE ANALYSIS % Fines	Hydrometer Analysis	Compaction	Unconfined Compression	Consolidation	Triaxial
K-3	S-1	Gray SAND and CLAY, trace of gravel (SC)													
K-3	S-2	Gray SAND and CLAY, trace of gravel (SC)													
K-3	S-3	Brown SILTY SNAD, some gravel (SM)	14.7												
K-3	S-4	Brown SILT, some fine sand (ML)	36.5												
K-3	S-5	Brown SILT, some fine sand (ML)													
K-3	S-6	Brown SILT, some fine sand (ML)													
K-3	S-7	Brown Silty fine SAND (SM)													
K-3	S-8	Brown SILT and fine SAND (ML,SM)													
K-3	S-9	Brown SILT, some fine sand (ML)													
K-3	S-10	Brown SILT, some fine sand (ML)													
K-3	S-11	Brown SILT, some fine sand (ML)													
K-3	S-12	Brown SILT, some fine sand (ML)	39.4												
K-3	S-13	Brown SILT, some fine sand (ML)	37.7												

Note: Asterisk indicates that test results are shown on separate sheets.

Table D6

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	% Sand	% Fines	Hydrometer Analysis	Compaction	Unconfined Compression	Consolidation	Triaxial
K-4	S-1	Gray Brown GRAVEL, some sand, trace silt (GW)													
K-4	S-2	Gray Brown Gravelly SAND, some silt (SW)	8.4					23	65	12					
K-4	S-3	Gray Brown Gravelly SAND, some silt (SW)													
K-4	S-4	Gray GRAVEL, some sand (GW)													
K-4	S-5	Gray Brown Sandy GRAVEL, trace silt (GW)	4.2					75	21	4					
K-4	S-6	Gray Sandy GRAVEL, trace silt (GW)													
K-4	S-7	Gray Brown GRAVEL, some sand, trace silt (GW)													
K-4	S-8	Gray Brown Gravelly SAND, some silt (SW-SM)	10.7					28	60	12					
K-4	S-9	Gray Brown SAND and GRAVEL, trace silt (SW)													
K-4	S-10	Gray Brown SAND and GRAVEL, trace silt (SW)													
K-4	S-11	Gray Brown SAND and GRAVEL, some silt (SW-SM)	7.4					41	49	10					
K-4	S-12	Gray Brown Gravelly SAND, trace silt (SW)													
K-4	S-13	Gray Brown GRAVEL and SAND, trace silt (GW)													
K-4	S-14	Gray Brown Gravelly SAND, some silt (SW-SM)	7.8					31	59	11					
K-4	S-15	Gray Brown GRAVEL, some sand (GW)													
K-4	S-16	Gray Brown GRAVEL, some sand (GW)													
K-4	S-17	Gray Brown GRAVEL, some sand, trace silt (GW)													

Note: Asterisk indicates that test results are shown on separate sheets.

Table D7

SUMMARY OF LABORATORY TEST RESULTS
prepared for:

NEW ENGLAND DIVISION
CORPS OF ENGINEERS

Project: Dynamic Stability Analysis of Knightville Dam, Huntington, Massachusetts

Boring No.	Sample No.	CLASSIFICATION	Natural Water Content – %	Liquid Limit	Plasticity Index	Specific Gravity	Natural Dry Density	SIEVE ANALYSIS % Gravel	% Sand	% Fines	Hydrometer Analysis	Consolidation	Static Triaxial	Dynamic Triaxial	Resonant Column
K5	S 1	Brown Gravelly Sandy SILT (SM-ML)													
K5	S 2	Brown Gravelly Silty SAND (SM)													
K5	S 3	Brown Gravelly Silty SAND (SM)													
K5	S 5	Brown Gravelly SAND, some silt (GW-GM)						30	60	10					
K5	S12	Brown SAND and GRAVEL, trace silt (SW,GW)						45	46	9					
K5	S17	Brown Gravelly SAND, some silt (SW-SM)						30	58	12					
K5	P 2	Black-Brown GRAVEL and SAND, trace silt (GW,SW)						42	55	3					
K5	P 9	Silty SAND (3-in. size gravel) (SM)	8.4				143.8		50	50					*
K5	--	Split Spoon Samples S-4 through S-17	8.7												
		were screened of plus ½-inch particle	8.9												
		sizes. The minus ½-inch fraction was	8.7			2.92		15	70	15					
		mixed together and representative	8.1					11	74	15			**		
		batch samples were compacted for	9.0					15	70	15					
		static strength testing. The results	9.0												
		on the right are for the combined	8.4												
		sample.													
K6	--	Split spoon samples from boreholes													
		K-6 are not representative samples													
		since a 2" O.D. sampler was used.													
K6	P 2	----------					138.4								

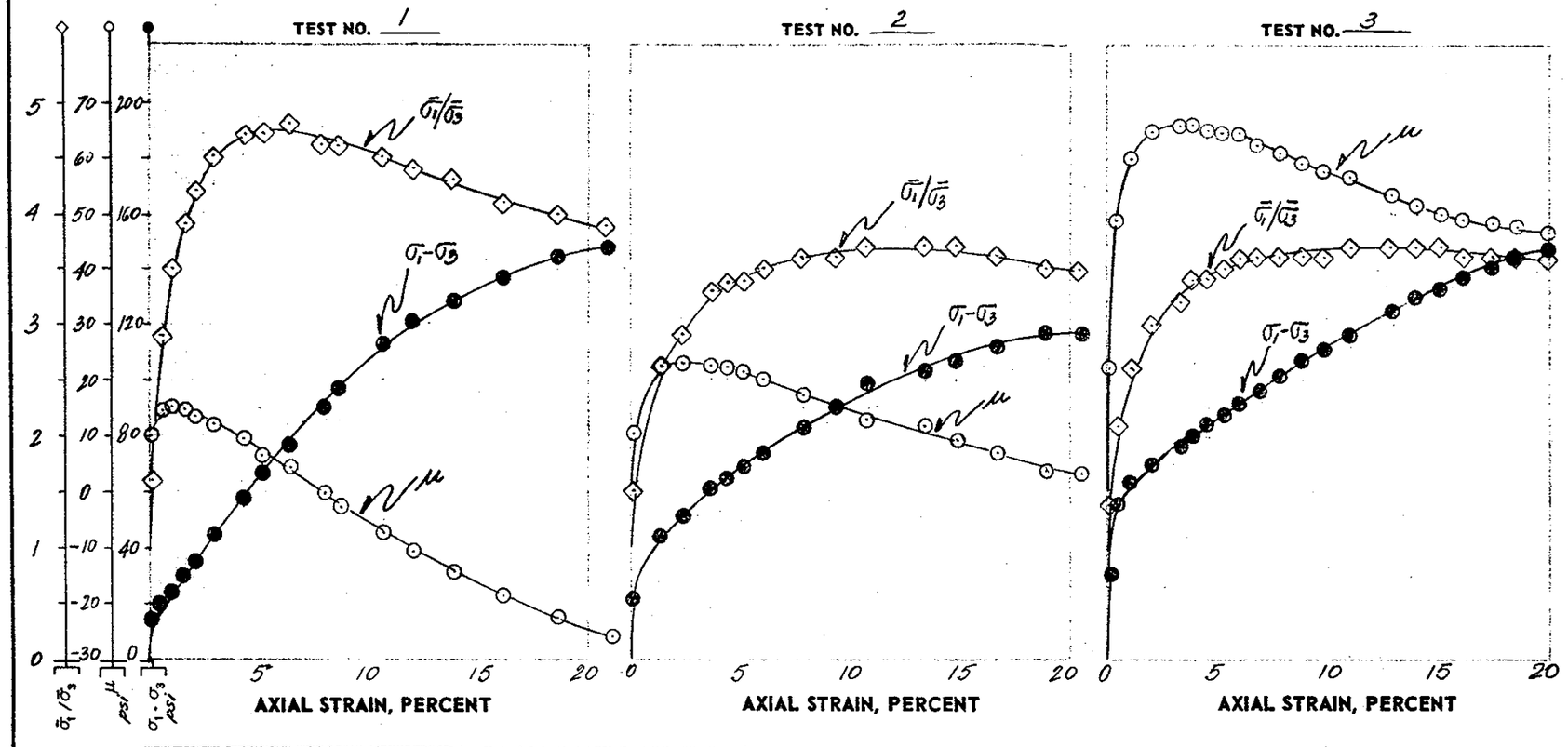
Note: Asterisk indicates that test results are shown on separate sheets.

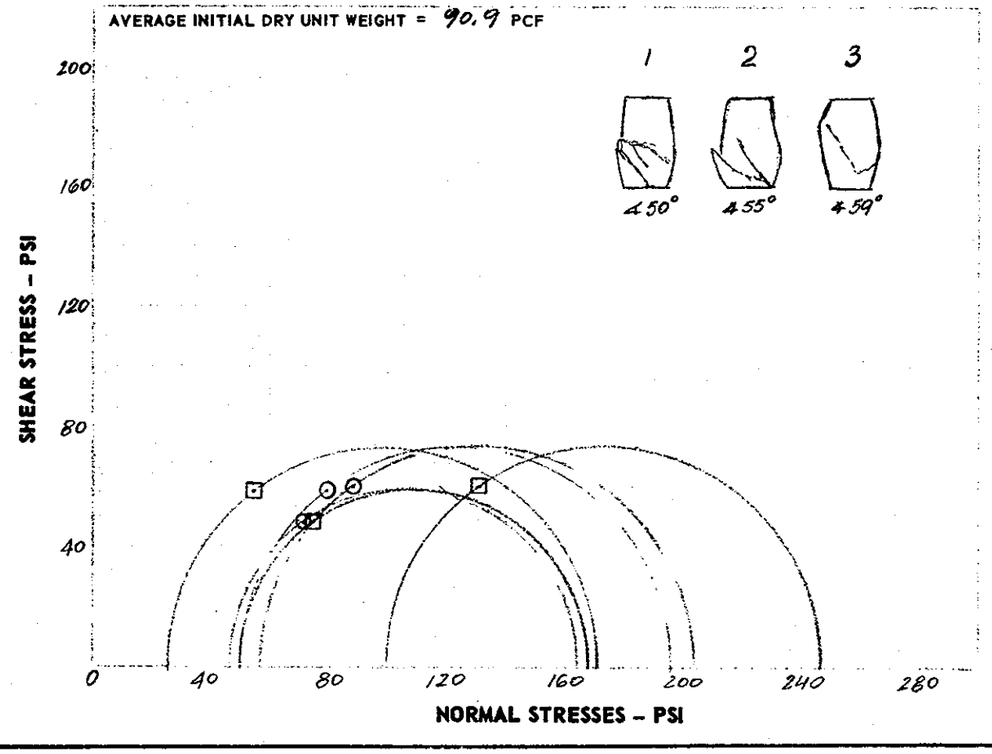
** SEVEN STATIC TRIAXIAL TEST PERFORMED ON COMPACTED SAMPLES

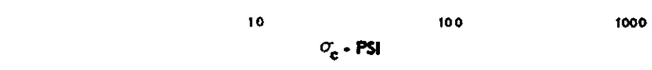
Section D2

Triaxial Compression Test Results

Figure	Test Type	Embankment Material	PAGE
D2-1	$\bar{R}$ (Stress-Controlled)	Core	D2-1
D2-2	$\bar{R}$ (Stress-Controlled)	Core	D2-2
D2-3	S (Stress-Controlled)	Core	D2-3
D2-4	$\bar{R}$ (Stress-Controlled)	Core	D2-4
D2-5	S (Stress-Controlled)	Core	D2-5
D2-6	$\bar{R}$ (Stress-Controlled)	Shell	D2-6
D2-7	S (Stress-Controlled)	Shell	D2-7
D2-8	S (Stress-Controlled)	Shell	D2-8
D2-9	$\bar{R}$ (Strain-Controlled)	Shell	D2-9
D2-10	$\bar{R}$ (Strain-Controlled)	Shell	D2-10







	TEST NO.	1	2	3
INITIAL	WATER CONTENT, %	31.4	34.4	32.0
	DRY UNIT WEIGHT, Lbs/cu.ft.	92.4	88.4	92.0
	VOID RATIO	0.870	0.956	0.879
	SATURATION, %	100	100	100
AFTER SATUR.	WATER CONTENT, %	29.5	31.1	27.2
	VOID RATIO	0.816	0.861	0.754
	SATURATION, %	100	100	100
AFTER CONSOL.	CONSOLIDATION PRESSURE, psi	25	50	100
	WATER CONTENT, %	28.4	29.9	27.2
	VOID RATIO	0.816	0.861	0.754
	SATURATION, %	96.3	96.2	100.0
TEST COND.	TOTAL LATERAL PRESSURE, psi	125	150	200
	BACK PRESSURE, psi	100	100	100
	NET LATERAL PRESSURE, psi	25	50	100
	EFFECTIVE STRESS RATIO $\bar{\sigma}_{1_0}/\bar{\sigma}_{3_0}$	1	1	1
FAILURE CRITERION		$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS, $\bar{\sigma}_1$ psi	197.0	164.4	203.7
	MINOR EFF. PRIN. STRESS, $\bar{\sigma}_3$ psi	50.0	46.8	57.0
	MAJOR PRIN. STRESS, σ_1 psi	172.0	167.6	246.7
	MINOR PRIN. STRESS, σ_3 psi	25.0	50.0	100.0
	WATER CONTENT, %	29.5	31.1	27.2
	DRY UNIT WEIGHT, Lbs/cu.ft.	95.2	92.9	98.5
	VOID RATIO	0.816	0.861	0.754
	SATURATION, %	100	100	100
SPECIMEN INITIAL DIAMETER in		2.858	2.858	2.858
SPECIMEN INITIAL HEIGHT in		6.814	6.850	6.910
TEST TIME TO FAILURE, min.		385	250	345

TEST NO.	BORING NO.	SAMPLE NO.	ELEV. OR DEPTH
1,2	K-2	SP-16	135.5-137.5
3	K-2	SP-17	145.5-147.5

TYPE OF SPECIMEN Undisturbed

CLASSIFICATION Bn. silt, sm. f sand, tr clay (ML)

	φ		TAN φ		C	
	ECU	CU	ECU	CU	ECU	CU
16-	LL = 34.5		PI = 5.3		G = 2.77	
17-	28.0		4.6			

REMARKS Samples consolidated before application of back pressure.

U.S. ARMY CORPS of ENGINEERS
NEW ENGLAND DIVISION

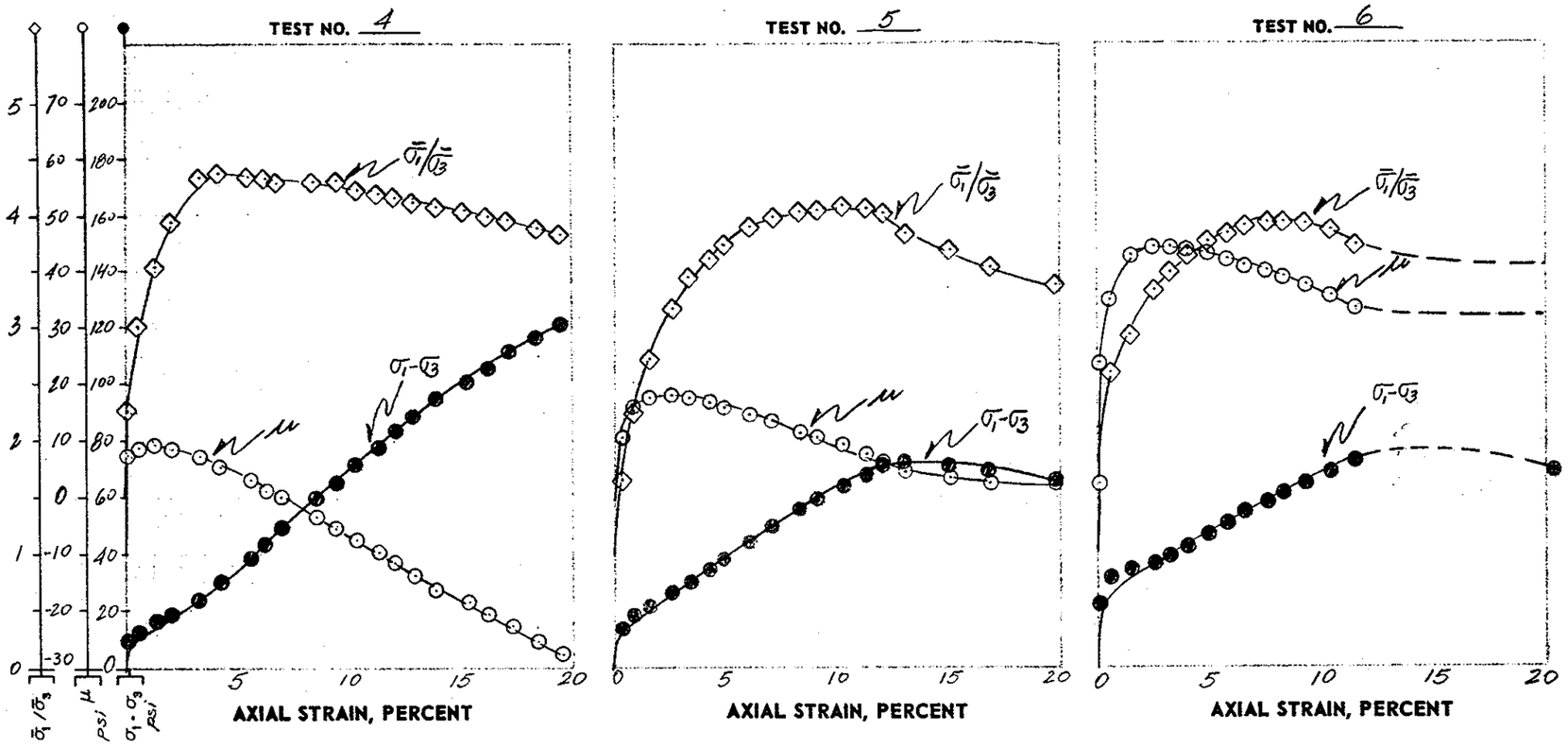
TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

DYNAMIC STABILITY ANALYSIS
KNIGHTVILLE DAM, MA.

TRIAXIAL COMPRESSION TEST
BORING NO. K-2 SAMPLE SP-16,17
CONSOLIDATED – UNDRAINED

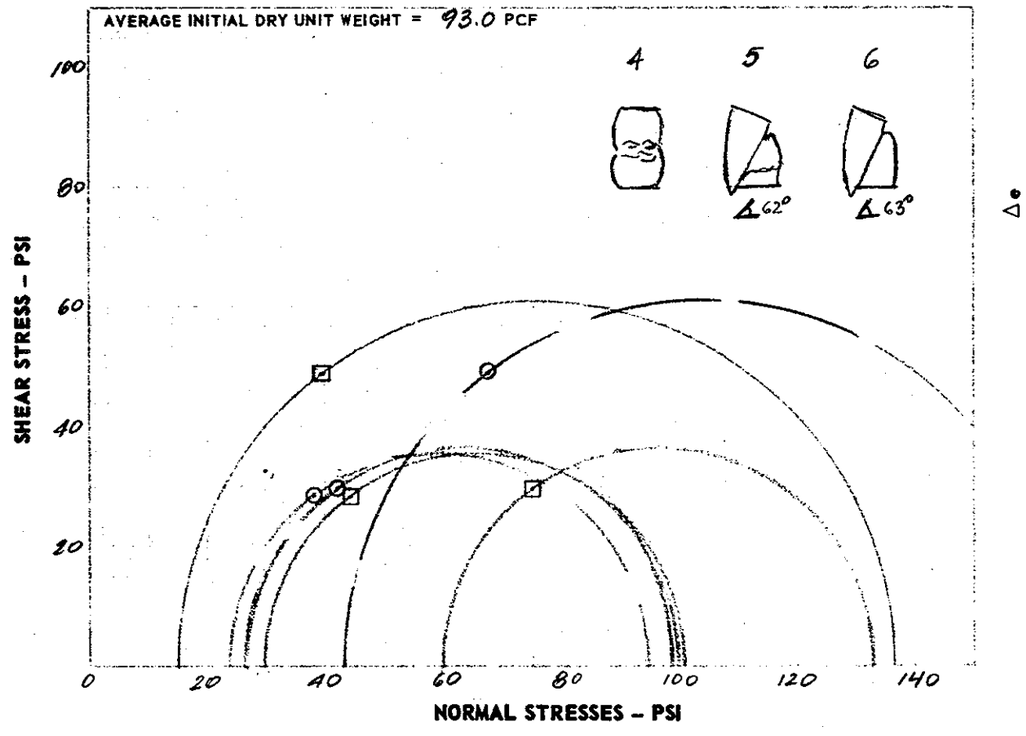
BY: ZB DATE: 1-82

DWG. NO. FIGURE D2-1



	TEST NO.	4	5	6
INITIAL	WATER CONTENT, %	30.3	24.5	28.1
	DRY UNIT WEIGHT, Lbs/cu.ft.	95.0	93.0	91.0
	VOID RATIO	0.845	0.885	0.926
	SATURATION, %	100	77.8	85.3
AFTER SATUR.	WATER CONTENT, %	26.9	28.6	29.3
	VOID RATIO	0.738	0.804	0.826
	SATURATION, %	100	100	100
AFTER CONSOL.	CONSOLIDATION PRESSURE, psi	15	30	60
	WATER CONTENT, %	26.4	24.5	28.1
	VOID RATIO	0.738	0.804	0.826
	SATURATION, %	100	85.6	95.6
TEST COND.	TOTAL LATERAL PRESSURE, psi	115	130	160
	BACK PRESSURE, psi	100	100	100
	NET LATERAL PRESSURE, psi	15	30	60
	EFFECTIVE STRESS RATIO $\bar{\sigma}_{1_0}/\bar{\sigma}_{3_0}$	1.0	1.0	1.0
FAILURE CRITERION		$\sigma_1-\sigma_3$ @ 20%		
AT FAILURE	MAJOR EFF. PRIN. STRESS, $\bar{\sigma}_1$ psi	165.0	99.0	104.0
	MINOR EFF. PRIN. STRESS, $\bar{\sigma}_3$ psi	43.0	27.0	28.0
	MAJOR PRIN. STRESS, σ_1 psi	137.0	102.0	133.0
	MINOR PRIN. STRESS, σ_3 psi	15.0	30.0	60.0
	WATER CONTENT, %	26.9	28.6	29.3
	DRY UNIT WEIGHT, Lbs/cu.ft.	100.9	97.2	96.0
	VOID RATIO	0.738	0.804	0.826
	SATURATION, %	100	100	100
SPECIMEN INITIAL DIAMETER in		2.870	2.846	2.846
SPECIMEN INITIAL HEIGHT in		7.097	7.063	6.815
TEST TIME TO FAILURE, min.		325	216	210

AVERAGE INITIAL DRY UNIT WEIGHT = 93.0 PCF



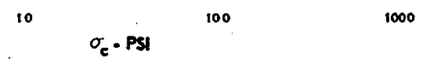
TEST NO.	BORING NO.	SAMPLE NO.	[redacted] DEPTH
4	K-2	SP-3	52.5-54.5
5,6	K-2	SP-4	57.5-59.5

TYPE OF SPECIMEN: Undisturbed

CLASSIFICATION: SP-3 : Bn. sandy silt; SP-4 : Bn. silt and sand

ϕ		TAN ϕ		C	
ECU	CU	ECU	CU	ECU	CU

SP-3 LL = 23.5 PI = 2.1 G = 2.81



REMARKS

Samples consolidated prior to application of back pressure

D2-2

U.S. ARMY CORPS of ENGINEERS
NEW ENGLAND DIVISION

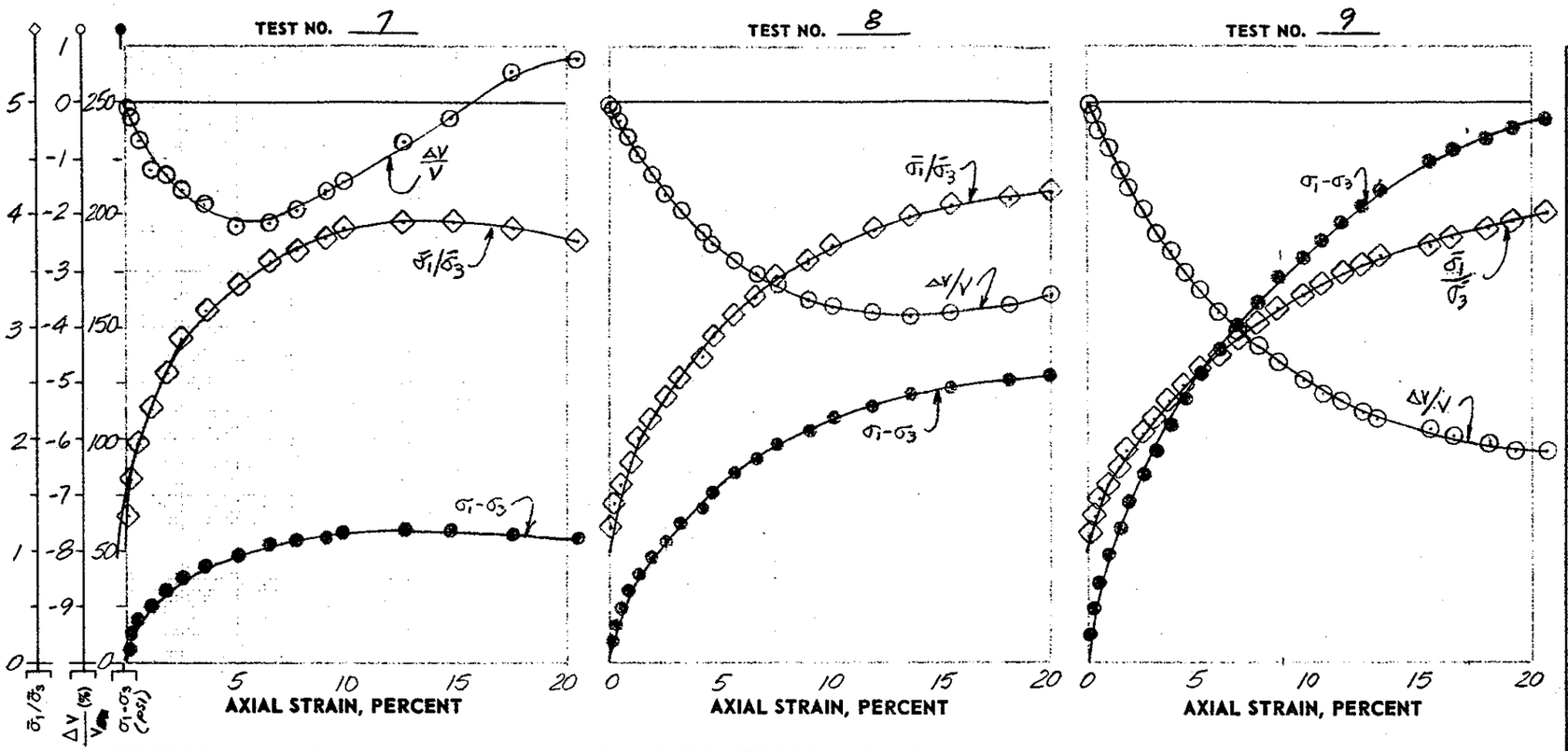
TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

DYNAMIC STABILITY ANALYSIS
KNIGHTVILLE DAM, MA.

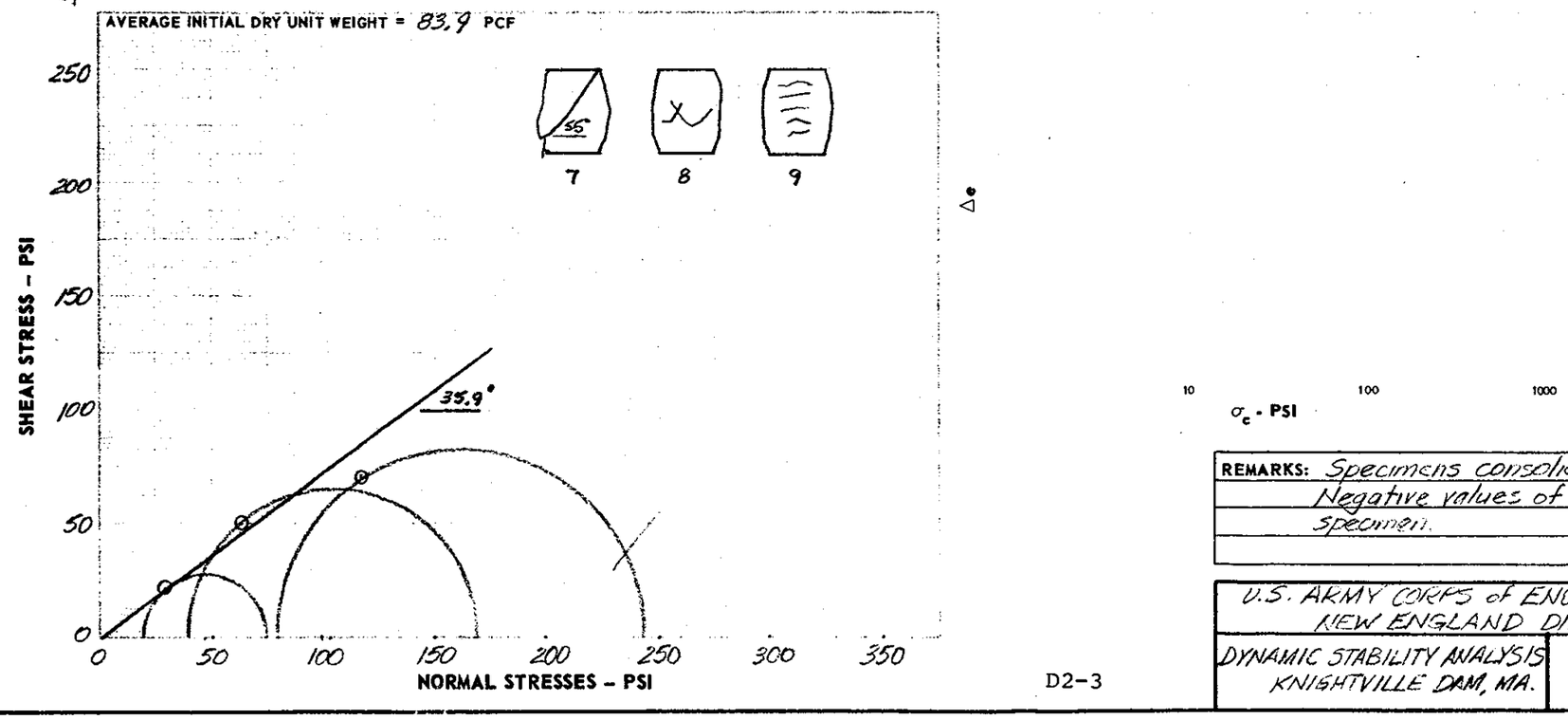
TRIAXIAL COMPRESSION TEST
BORING NO. K-2 SAMPLE SP-3,4
CONSOLIDATED – UNDRAINED

BY: ZB DATE: 2-82

DWG. NO. FIGURE D2-2



	TEST NO.	7	8	9
INITIAL	WATER CONTENT, %	37.7	38.3	38.9
	DRY UNIT WEIGHT, Lbs/cu.ft.	84.2	84.1	83.3
	VOID RATIO	1.061	1.063	1.081
	SATURATION, %	98.8	100	100
	BACK PRESSURE, psi	100	100	100
AFTER SATUR.	WATER CONTENT, %	34.4	34.9	33.7
	VOID RATIO	0.955	0.972	0.937
	SATURATION, %	100	100	100
	CONSOLIDATION PRESSURE, psi	20	40	80
AFTER CONSOL.	WATER CONTENT, %	33.9	34.2	33.3
	VOID RATIO	0.955	0.950	0.924
	SATURATION, %	98.6	100	100
	FAILURE CRITERION	$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS $\bar{\sigma}_1$ psi	75.4	167.9	322.3
	MINOR EFF. PRIN. STRESS $\bar{\sigma}_3$ psi	20.0	40.0	80.0
	WATER CONTENT, %	34.7	32.5	29.4
	DRY UNIT WEIGHT, Lbs/cu.ft.	88.4	91.1	95.5
	VOID RATIO	0.962	0.903	0.816
	SATURATION, %	100	100	100
SPECIMEN INITIAL DIAMETER in		2.85	2.86	2.86
SPECIMEN INITIAL HEIGHT in		6.85	7.09	6.84
TEST TIME TO FAILURE, min.		160	210	280



TEST NO.	BORING NO.	SAMPLE NO.	ELEV. OR DEPTH
7	K-2	SP-11	93.5'-94.5'
8	K-2	SP-11	92.5'-93.5'
9	K-2	SP-12	98.5'-99.5'

TYPE OF SPECIMEN: Undisturbed

CLASSIFICATION:

φ = 35.9°	TAN φ = 0.73	C = 0
LL =	PI =	G = 2.78 (SP-11)

REMARKS: Specimens consolidated prior to application of back pressure. Negative values of volumetric strain denote compression of specimen.

U.S. ARMY CORPS of ENGINEERS
NEW ENGLAND DIVISION

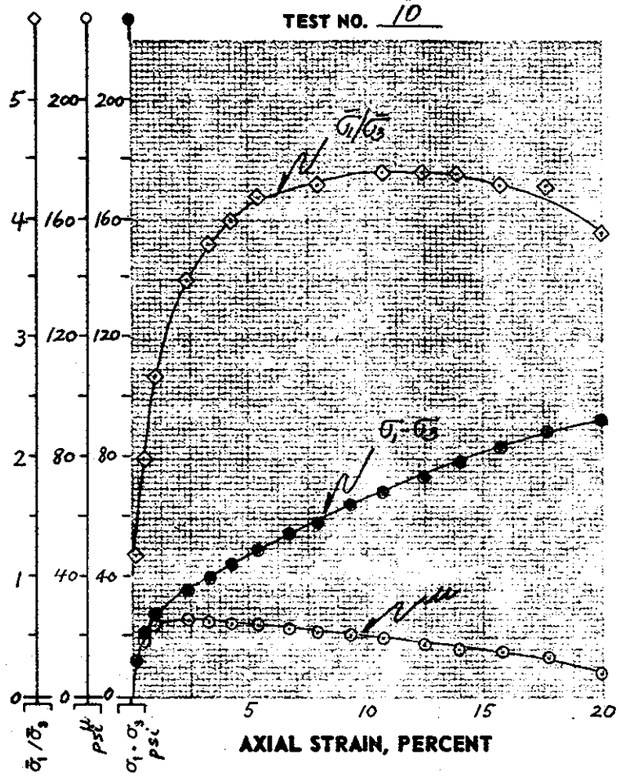
TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

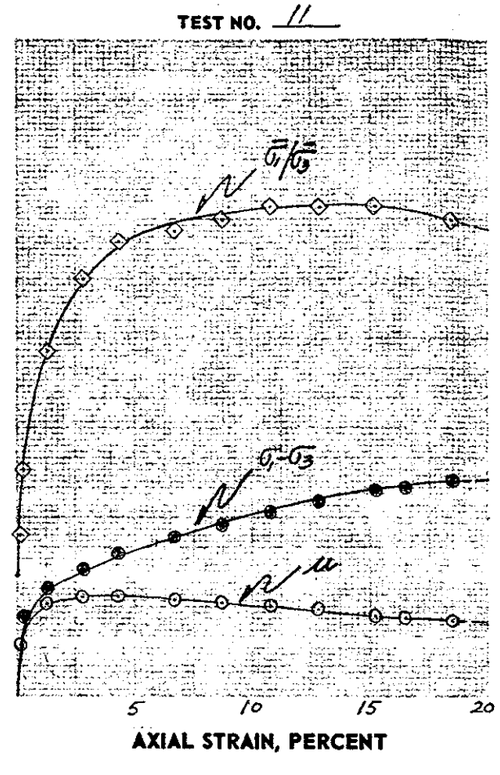
DYNAMIC STABILITY ANALYSIS
KNIGHTVILLE DAM, MA.

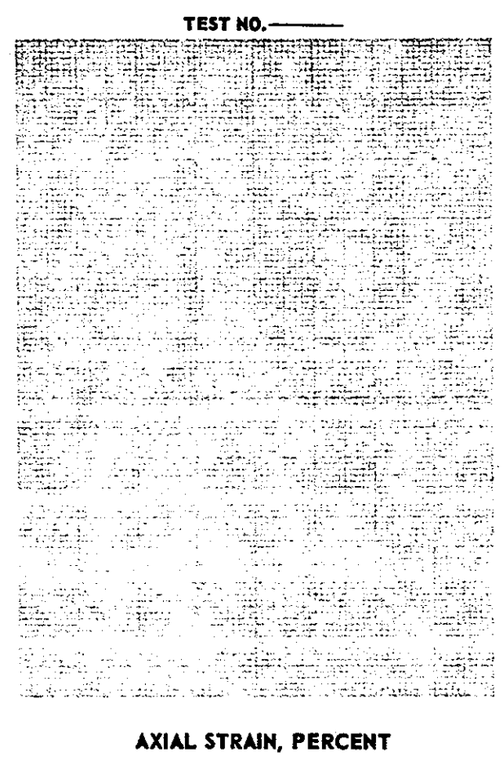
TRIAXIAL COMPRESSION TEST
CORE MATERIAL
CONSOLIDATED – DRAINED

BY: DATE:

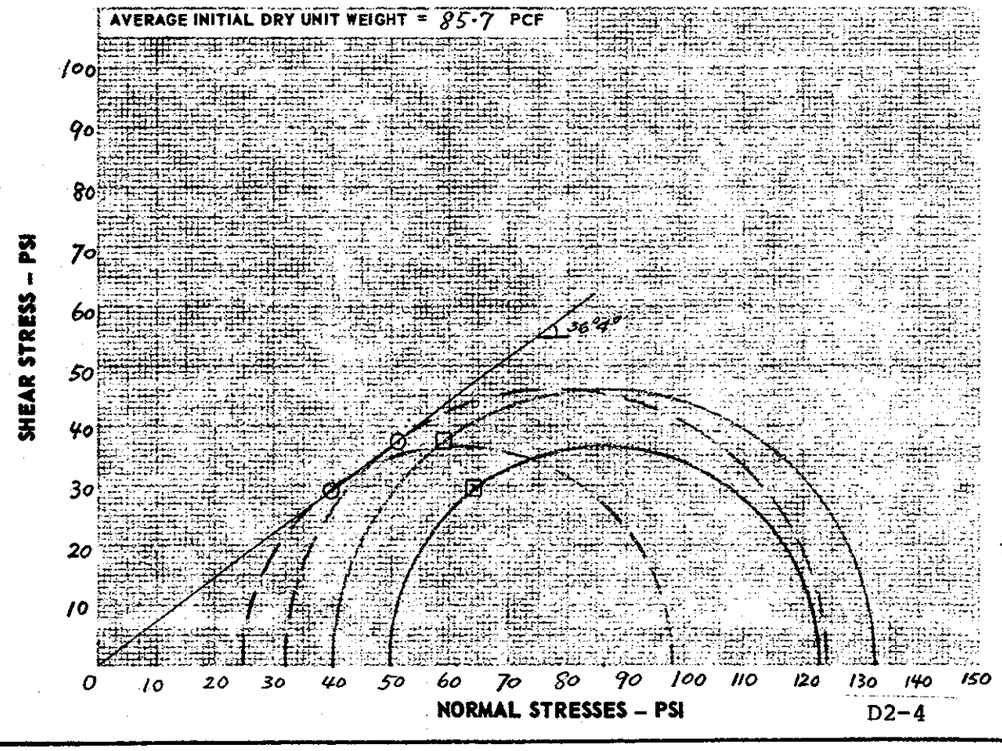
DWG. NO. FIGURE D2-3



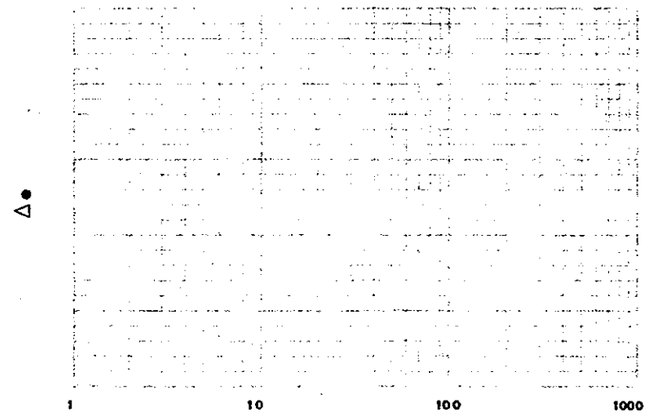


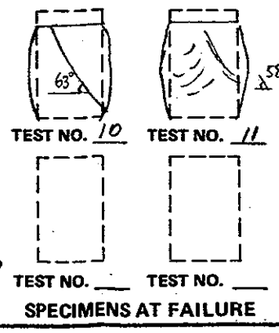


	TEST NO.	10	11	
INITIAL	WATER CONTENT, %	35.3	37.7	
	DRY UNIT WEIGHT, Lbs/cu.ft.	86.7	84.7	
	VOID RATIO	1.02	1.05	
	SATURATION, %	96.8	100	
AFTER SATUR.	WATER CONTENT, %	33.4	34.1	
	VOID RATIO	0.94	0.95	
	SATURATION, %	100.0	100	
AFTER CONSUL.	CONSOLIDATION PRESSURE, psi	40.0	50	
	WATER CONTENT, %	32.4	34.1	
	VOID RATIO	0.94	0.95	
	SATURATION, %	97.0	100	
TEST COND.	TOTAL LATERAL PRESSURE, psi	140	150	
	BACK PRESSURE, psi	100	100	
	NET LATERAL PRESSURE, psi	40	50	
	EFFECTIVE STRESS RATIO $\bar{\sigma}_{1_c}/\bar{\sigma}_{3_c}$	1	1	
	FAILURE CRITERION	$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS, $\bar{\sigma}_1$ psi	124.8	98.0	
	MINOR EFF. PRIN. STRESS, $\bar{\sigma}_3$ psi	32.1	25.0	
	MAJOR PRIN. STRESS, σ_1 psi	132.7	123.0	
	MINOR PRIN. STRESS, σ_3 psi	40.0	50.0	
	WATER CONTENT, %	33.6	33.9	
	DRY UNIT WEIGHT, Lbs/cu.ft.	90.4	89.1	
	VOID RATIO	0.94	0.95	
	SATURATION, %	100	100	
SPECIMEN INITIAL DIAMETER, in		2.85	2.86	
SPECIMEN INITIAL HEIGHT, in		6.88	7.04	
TEST TIME TO FAILURE, min.		405	260	
"A" PARAMETER		—	—	
"B" PARAMETER		1.0	1.0	



D2-4





SPECIMENS AT FAILURE

TEST NO.	BORING NO.	SAMPLE NO.	[redacted] DEPTH (ft)
10	K-2	SP-8	77.5 ~78.5
11	K-2	SP-12	97.5~ 98.5

TYPE OF SPECIMEN: UNDISTURBED

CLASSIFICATION: SP-8 - Bn. Sandy silt
SP-12 Bn. silt, sm-f. sand

φ		TAN φ		C	
ECU	CU	ECU	CU	ECU	CU
36.4		0.74			

LL = NP | PI = NP | G = 2.81

REMARKS: Sample consolidated before application of back pressure

U. S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

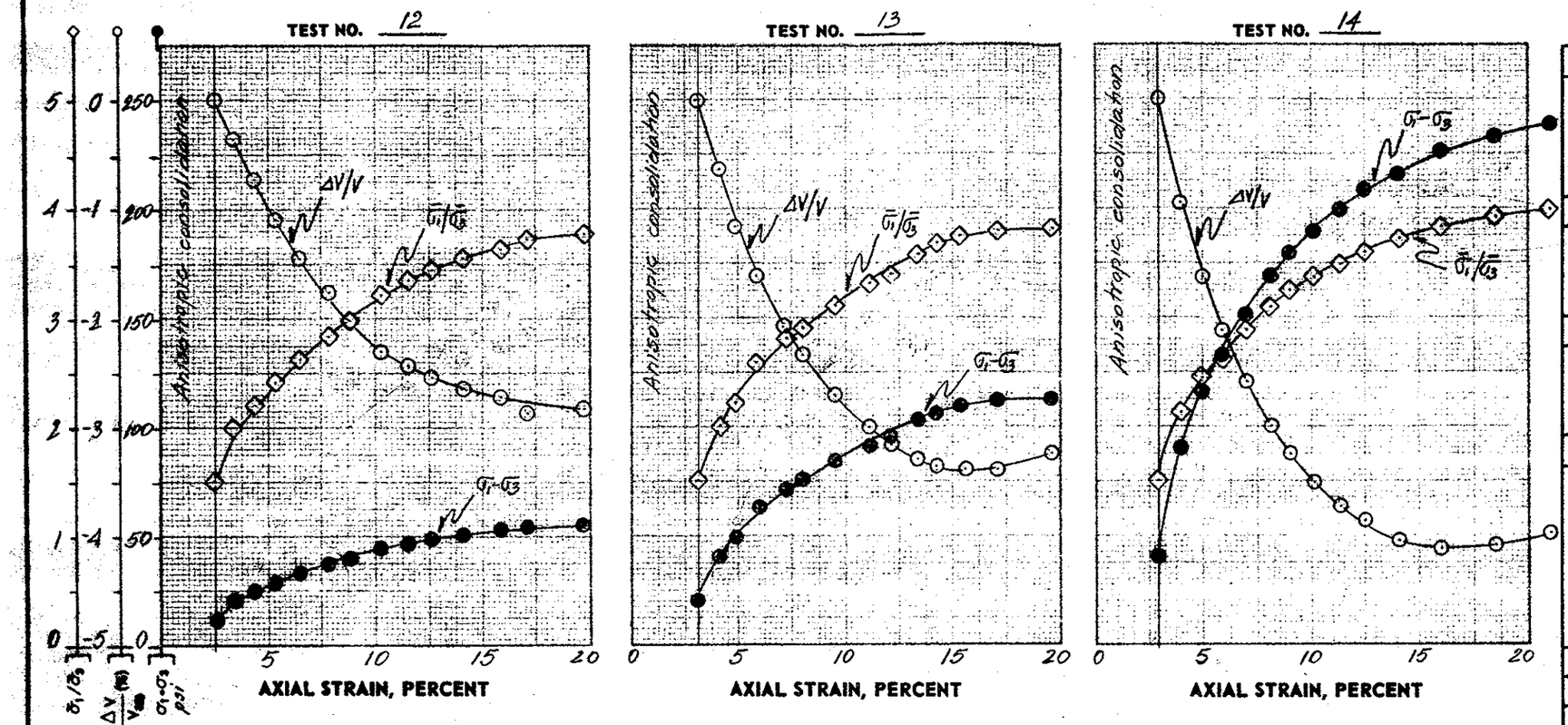
DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM

TRIAXIAL COMPRESSION TEST
BORING NO. K-2 SAMPLE 8 & 12
CONSOLIDATED – UNDRAINED

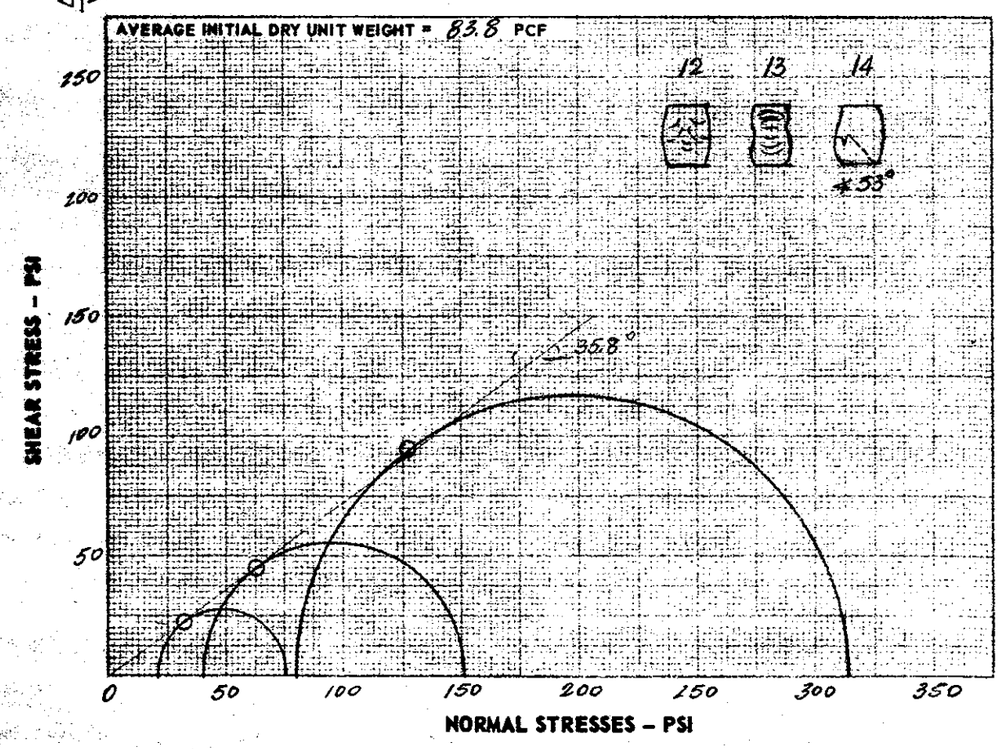
BY: W. Z. DATE: 4-12-82

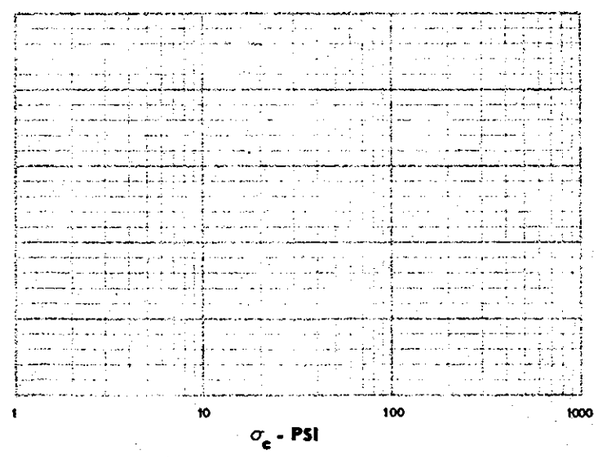
DWG. NO. FIGURE D2-4

FORM NO. 167-A1 1-77



TEST NO.		12	13	14
INITIAL	WATER CONTENT, %	42.5	38.6	35.7
	DRY UNIT WEIGHT, Lbs/cu.ft.	79.9	84.1	87.5
	VOID RATIO	1.194	1.085	1.003
	SATURATION, %	100	100	100
BACK PRESSURE, psi		100	100	100
AFTER SATUR.	WATER CONTENT, %	39.1	33.8	32.2
	VOID RATIO	1.099	0.950	0.905
	SATURATION, %	100	100	100
CONSOLIDATION PRESSURE, psi		20	40	80
AFTER CONSOL.	WATER CONTENT, %	39.1	33.8	32.2
	VOID RATIO	1.099	0.950	0.905
	SATURATION, %	100	100	100
FAILURE CRITERION		$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS $\bar{\sigma}_1$ psi	76	153	314
	MINOR EFF. PRIN. STRESS $\bar{\sigma}_3$ psi	20	40	80
	WATER CONTENT, %	37.1	31.7	29.5
	DRY UNIT WEIGHT, Lbs/cu.ft.	86.0	92.9	95.9
	VOID RATIO	1.043	0.887	0.829
	SATURATION, %	100	100	100
SPECIMEN INITIAL DIAMETER in		2.845	2.862	2.852
SPECIMEN INITIAL HEIGHT in		6.987	6.946	6.781
TEST TIME TO FAILURE, min.		195	250	362





TEST NO.	BORING NO.	SAMPLE NO.	[redacted] DEPTH ft.
12,13	K-2	SP-7	72.5-74.5
14	K-2	SP-8	78.5-79.5

TYPE OF SPECIMEN: Undisturbed

CLASSIFICATION: Bn. silt, sm. f. sand

φ = 35.8	TAN φ = 0.72	C = 0
LL = NP	PI = NP	G = 2.81

REMARKS: Samples anisotropically consolidated in increments, to a constant $\bar{\sigma}_1/\bar{\sigma}_3$ = 1.5, prior to application of back pressure. Negative values of volumetric strain denote compression of specimen.

U.S. ARMY CORPS of ENGINEERS
NEW ENGLAND DIVISION

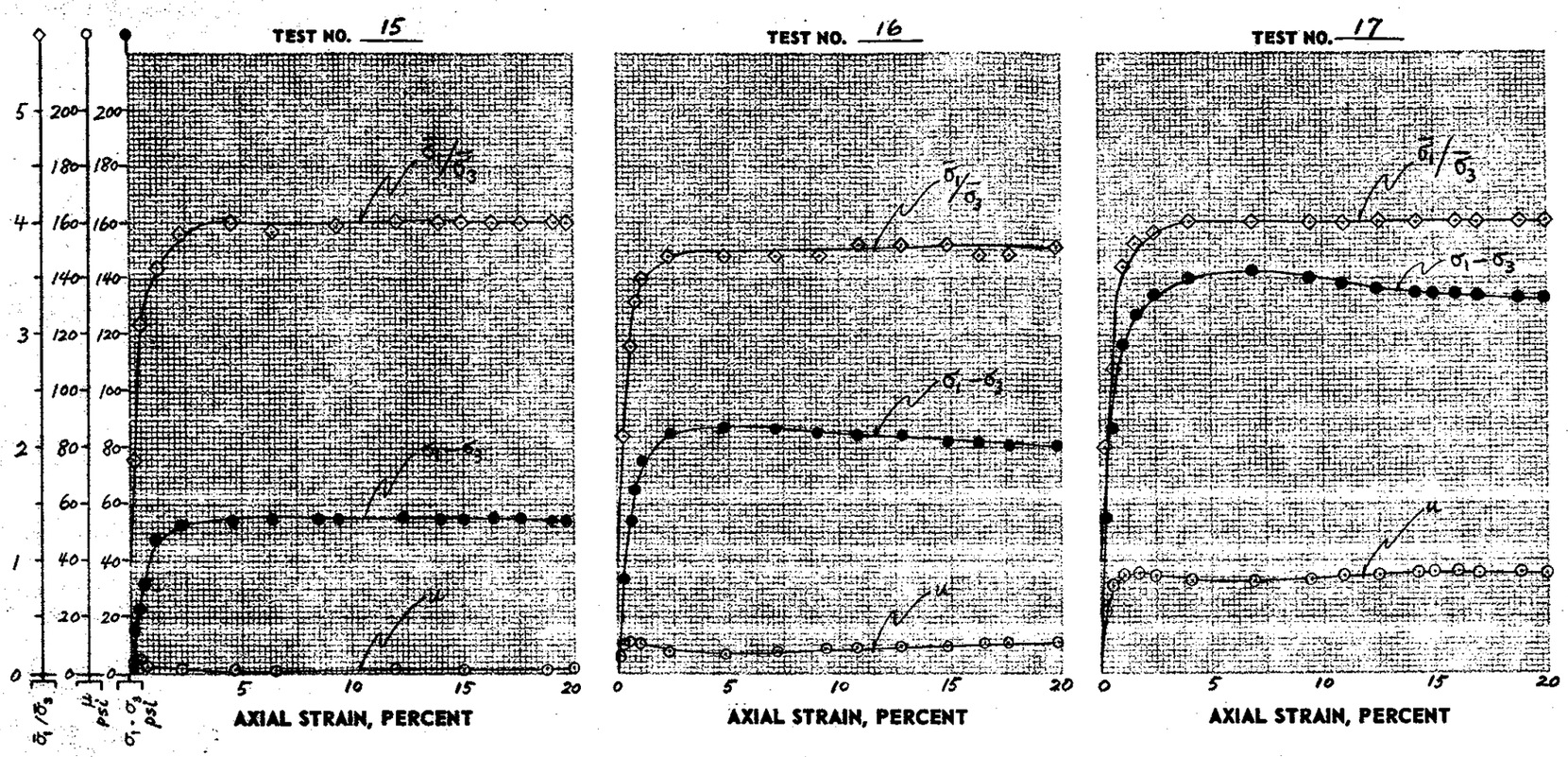
TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

DYNAMIC STABILITY ANALYSIS
KNIGHTVILLE DAM, MA.

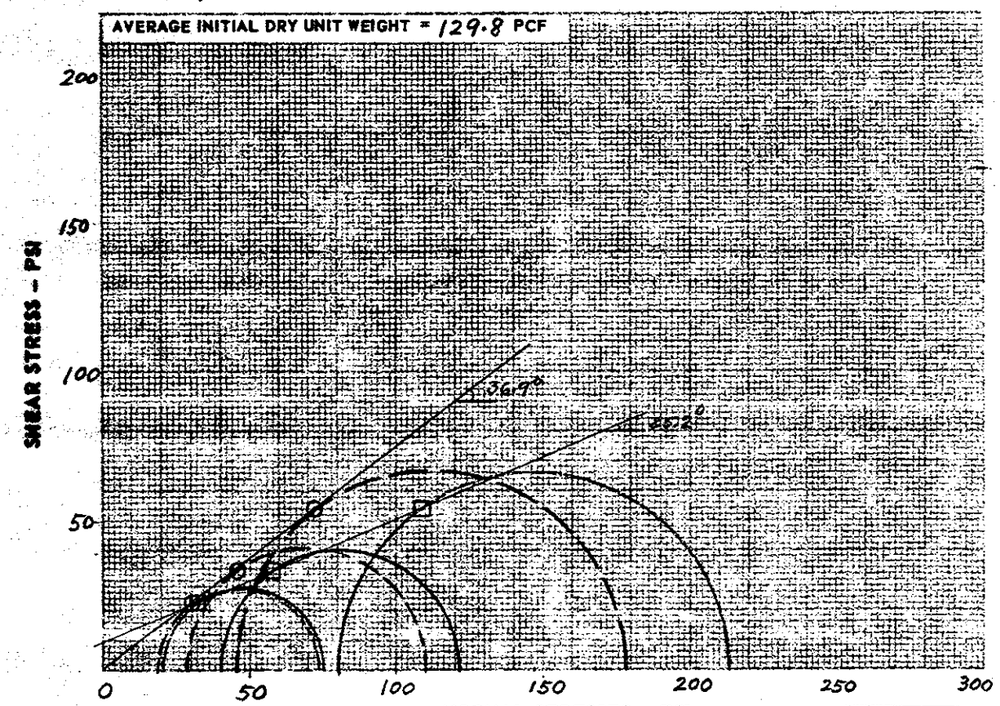
TRIAXIAL COMPRESSION TEST
ANISOTROPICALLY
CONSOLIDATED – DRAINED

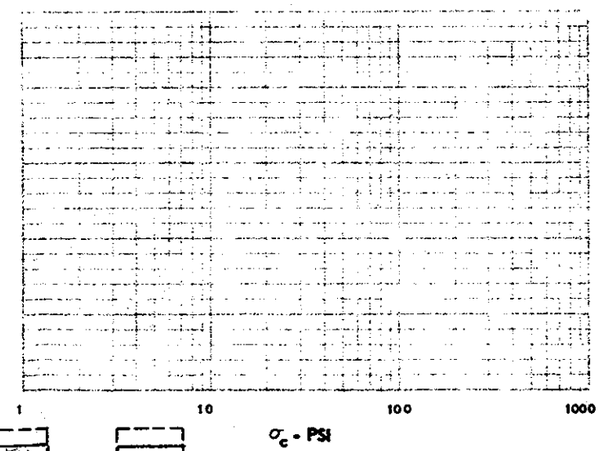
BY: ZB DATE: 4-82

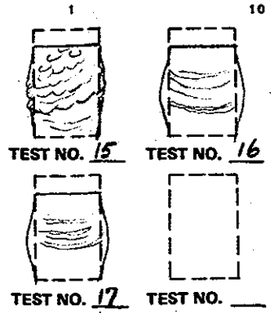
DWG. NO. FIGURE D2-5



	TEST NO.	15	16	17
INITIAL	WATER CONTENT, %	9.0	9.0	8.4
	DRY UNIT WEIGHT, Lbs/cu.ft.	129.0	129.7	130.8
	VOID RATIO	0.41	0.41	0.39
	SATURATION, %	63.8	64.5	62.5
AFTER SATUR.	WATER CONTENT, %	13.5	13.4	13.2
	VOID RATIO	0.39	0.39	0.38
	SATURATION, %	100	100	100
AFTER CONSUL.	CONSOLIDATION PRESSURE, psi	20	40	80
	WATER CONTENT, %	13.3	12.9	12.2
	VOID RATIO	0.39	0.38	0.36
	SATURATION, %	100	100	100
TEST COND.	TOTAL LATERAL PRESSURE, psi	120	140	180
	BACK PRESSURE, psi	100	100	100
	NET LATERAL PRESSURE, psi	20	40	80
	EFFECTIVE STRESS RATIO $\bar{\sigma}_{1c}/\bar{\sigma}_{3c}$	1.0	1.0	1.0
FAILURE CRITERION		$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS, $\bar{\sigma}_1$ psi	72.4	109.7	177.3
	MINOR EFF. PRIN. STRESS, $\bar{\sigma}_3$ psi	18.0	29.1	44.8
	MAJOR PRIN. STRESS, σ_1 psi	74.4	120.6	212.6
	MINOR PRIN. STRESS, σ_3 psi	20	40	80
	WATER CONTENT, %	13.3	12.9	12.2
	DRY UNIT WEIGHT, Lbs/cu.ft.	131.2	132.4	134.2
	VOID RATIO	0.39	0.38	0.36
	SATURATION, %	100	100	100
SPECIMEN INITIAL DIAMETER in		2.80	2.80	2.80
SPECIMEN INITIAL HEIGHT in		7.075	7.075	7.05
TEST TIME TO FAILURE, min.		235	225	235
"A" PARAMETER		—	—	—
"B" PARAMETER		1.0	1.0	1.0







SPECIMENS AT FAILURE

TEST NO.	BORING NO.	SAMPLE NO.	ELEV. OR DEPTH
15	K-5	Combined	
16	K-5	Combined	
17	K-5	Combined	

TYPE OF SPECIMEN: Compacted

CLASSIFICATION: Bn. sand, sm. gravel, sm. silt

φ		TAN φ		C	
ECU	CU	ECU	CU	ECU	CU
36.9	25.2	0.75	0.47	0	8.8

LL = NP PI = NP G = 2.92

REMARKS: Samples compacted statically to approximately 130 pcf at natural water content

Samples consolidated after application of back pressure

U.S. ARMY CORPS OF ENGINEERS
DIVISION OF NEW ENGLAND

TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM

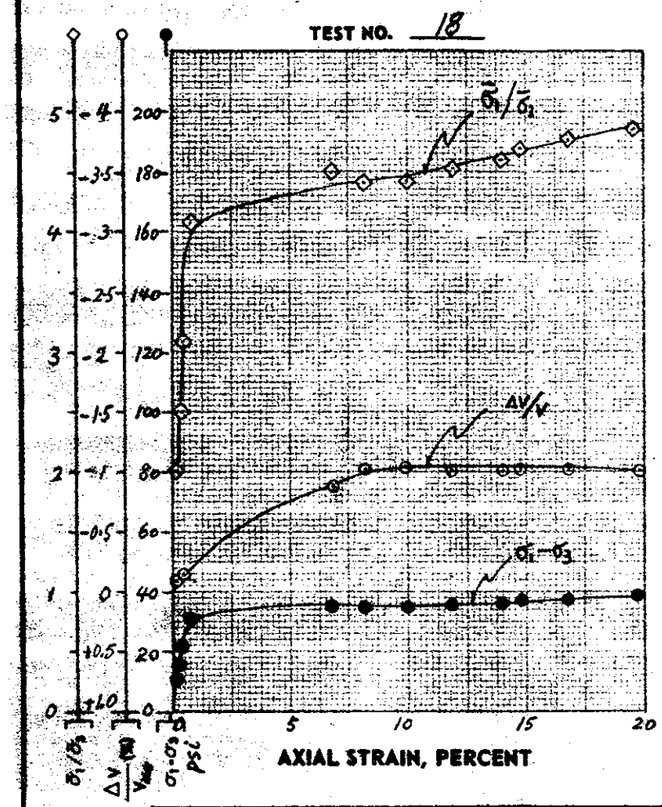
TRIAXIAL COMPRESSION TEST
BORING NO. K-5, SAMPLE COMBINED
CONSOLIDATED – UNDRAINED

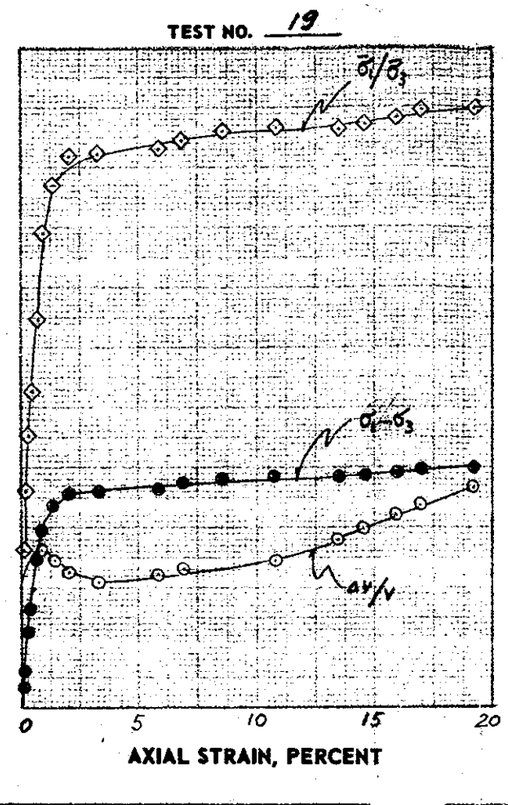
BY: W. Z. DATE: 4-12-82

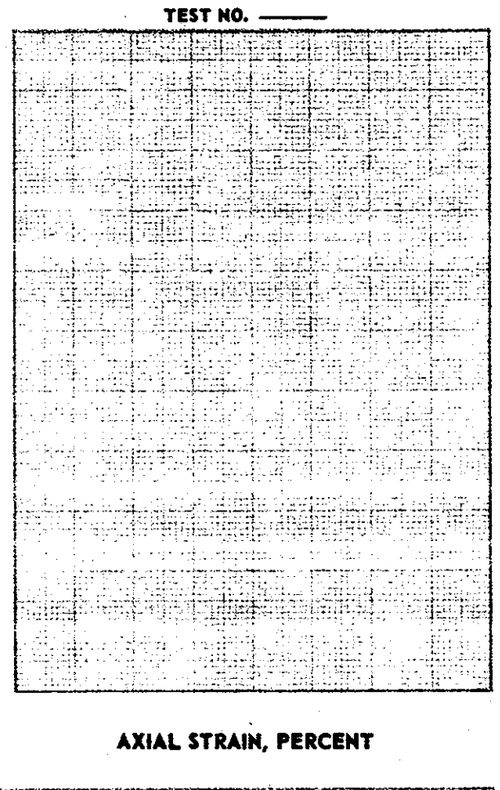
DWG. NO. FIGURE D2-6

D2-6

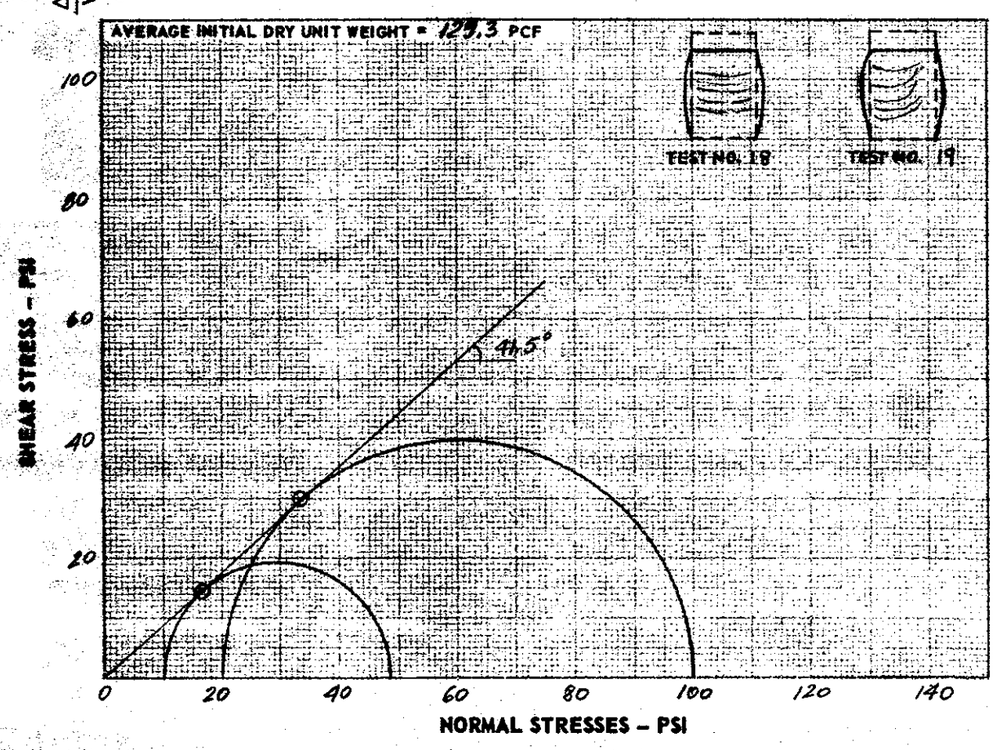
FORM NO. 167-A1 1-77

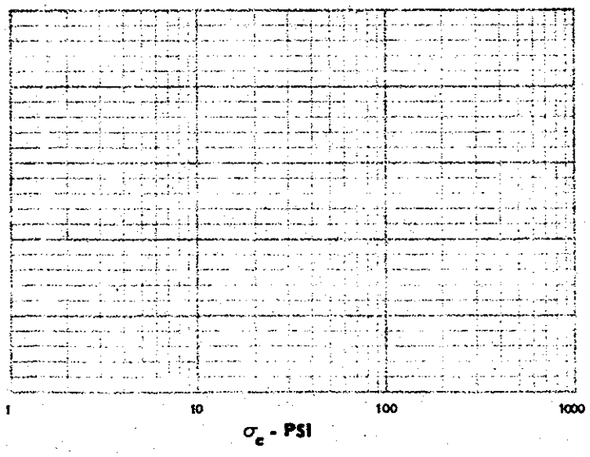






	TEST NO.	18	19	
INITIAL	WATER CONTENT, %	8.7	8.1	
	DRY UNIT WEIGHT, Lbs/cu.ft.	129.1	129.5	
	VOID RATIO	0.410	0.410	
	SATURATION, %	62	57.9	
	BACK PRESSURE, psi	100	100	
AFTER SATUR.	WATER CONTENT, %	13.8	13.9	
	VOID RATIO	0.402	0.406	
	SATURATION, %	100	100	
	CONSOLIDATION PRESSURE, psi	10	20	
AFTER CONSOL.	WATER CONTENT, %	[illegible]	13.7	
	VOID RATIO	[illegible]	0.400	
	SATURATION, %	100	100	
	FAILURE CRITERION	$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS $\bar{\sigma}_1$ psi	48.7	100.1	
	MINOR EFF. PRIN. STRESS $\bar{\sigma}_3$ psi	10	20	
	WATER CONTENT, %	13.6	13.3	
	DRY UNIT WEIGHT, Lbs/cu.ft.	130.5	131.2	
	VOID RATIO	0.397	0.388	
	SATURATION, %	100	100	
SPECIMEN INITIAL DIAMETER, in		2.80	2.80	
SPECIMEN INITIAL HEIGHT, in		7.08	7.08	
TEST TIME TO FAILURE, min.		155	220	





TEST NO.	BORING NO.	SAMPLE NO.	DEPTH
18,19	K-5	Combined	

TYPE OF SPECIMEN: Compacted

CLASSIFICATION: Bn. sand, sm gravel, sm. silt

ϕ = 41.5°	TAN ϕ = 0.88	C = 0
LL = NP	PI = NP	G = 2.92

REMARKS: Samples compacted statically to approximately 130 pcf at natural water content
Samples consolidated after application of back pressure

U. S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

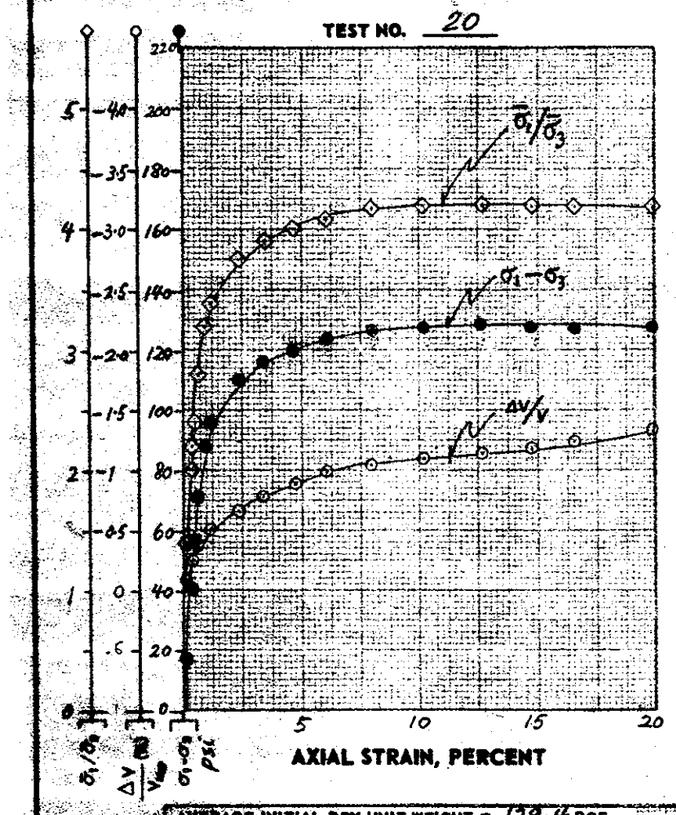
TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

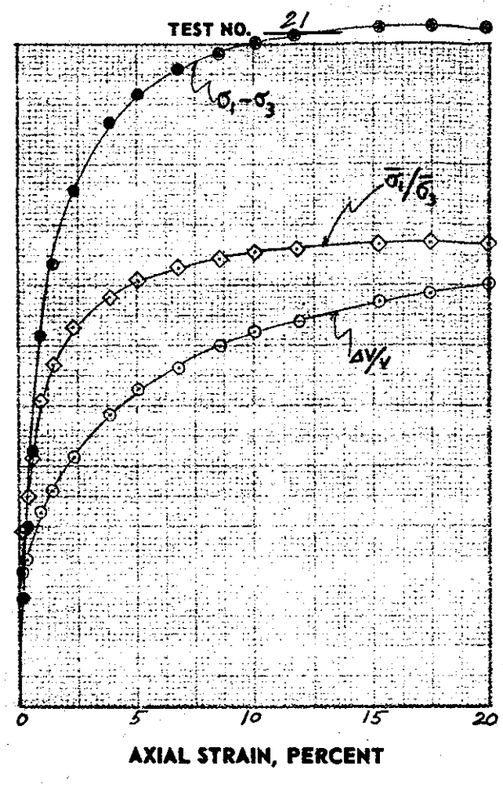
DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM

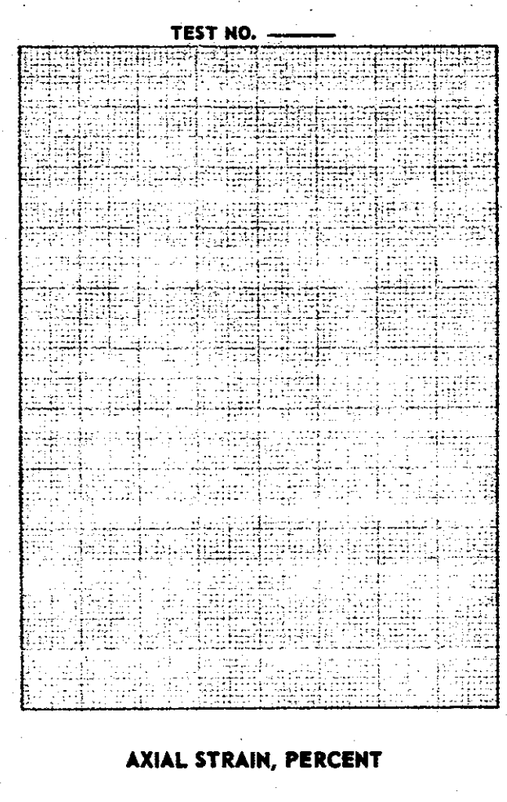
TRIAXIAL COMPRESSION TEST
BORING NO. K-5 ; SAMPLES COMBINED
CONSOLIDATED – DRAINED

BY: W. Z. DATE: 4-12-82

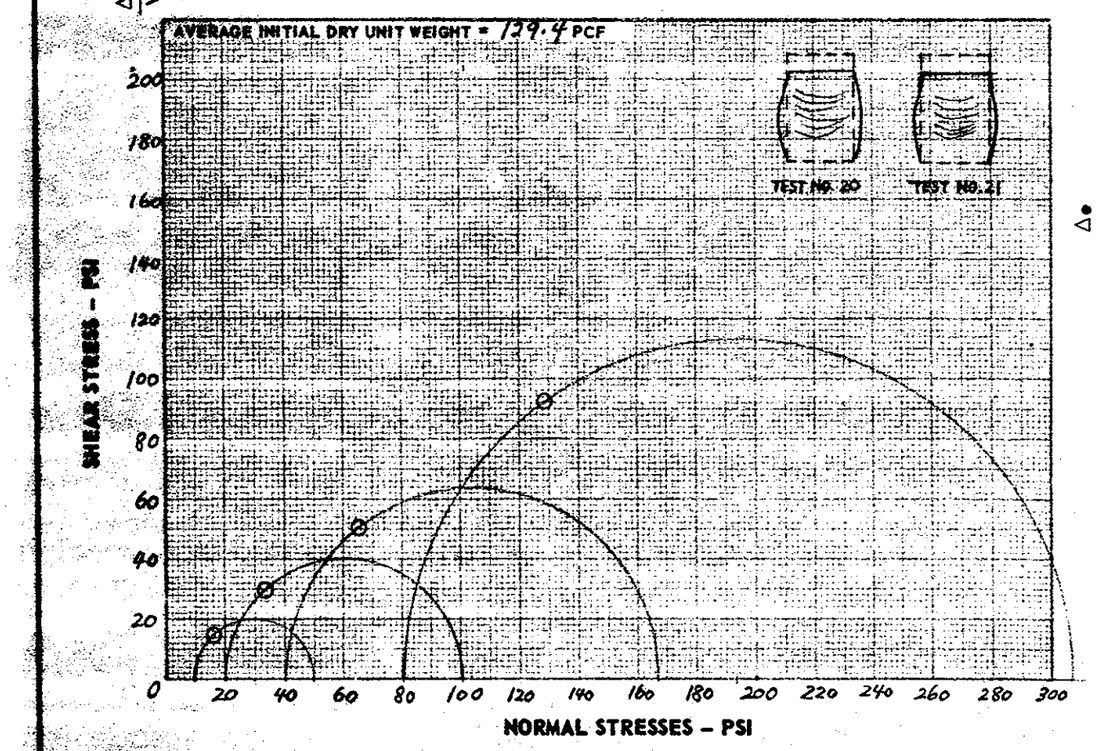
DWG. NO. FIGURE D2-7

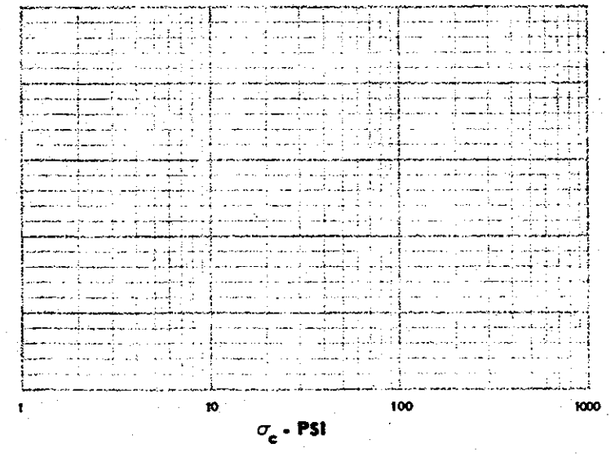






	TEST NO.	20	21	
INITIAL	WATER CONTENT, %	8.70	8.85	
	DRY UNIT WEIGHT, Lbs/cu.ft.	129.5	129.3	
	VOID RATIO	0.41	0.41	
	SATURATION, %	62	63	
BACK PRESSURE, psi		100	100	
AFTER SATUR.	WATER CONTENT, %	13.46	13.46	
	VOID RATIO	0.393	0.393	
	SATURATION, %	100	100	
CONSOLIDATION PRESSURE, psi		40	80	
AFTER CONSOL.	WATER CONTENT, %	12.88	12.76	
	VOID RATIO	0.375	0.373	
	SATURATION, %	100	100	
FAILURE CRITERION		$\sigma_1-\sigma_3$ @ 20% ε		
AT FAILURE	MAJOR EFF. PRIN. STRESS $\bar{\sigma}_1$ psi	167.7	306.2	
	MINOR EFF. PRIN. STRESS $\bar{\sigma}_3$ psi	40	80	
	WATER CONTENT, %	12.25	11.59	
	DRY UNIT WEIGHT, Lbs/cu.ft.	134.3	136.1	
	VOID RATIO	0.357	0.339	
	SATURATION, %	100	100	
SPECIMEN INITIAL DIAMETER in		2.80	2.80	
SPECIMEN INITIAL HEIGHT in		7.08	7.08	
TEST TIME TO FAILURE, min.		255	275	





TEST NO.	BORING NO.	SAMPLE NO.	ELEV. OR DEPTH
20, 21	K-5	Combined	

TYPE OF SPECIMEN: Compacted

CLASSIFICATION: Bn. sand, sm. gravel, sm. silt

ϕ =	TAN ϕ =	C =
LL = NP	PI = NP	G = 2.92

REMARKS: Samples compacted statically to approximately 130 pcf at natural water content
Samples consolidated after application of back pressure

U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

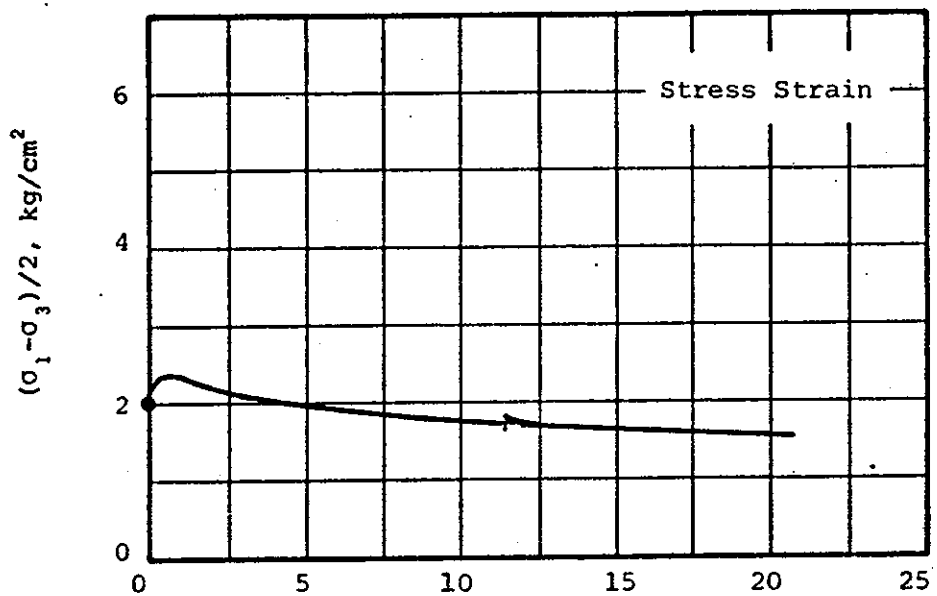
TIPPETTS–ABBETT–McCARTHY–STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

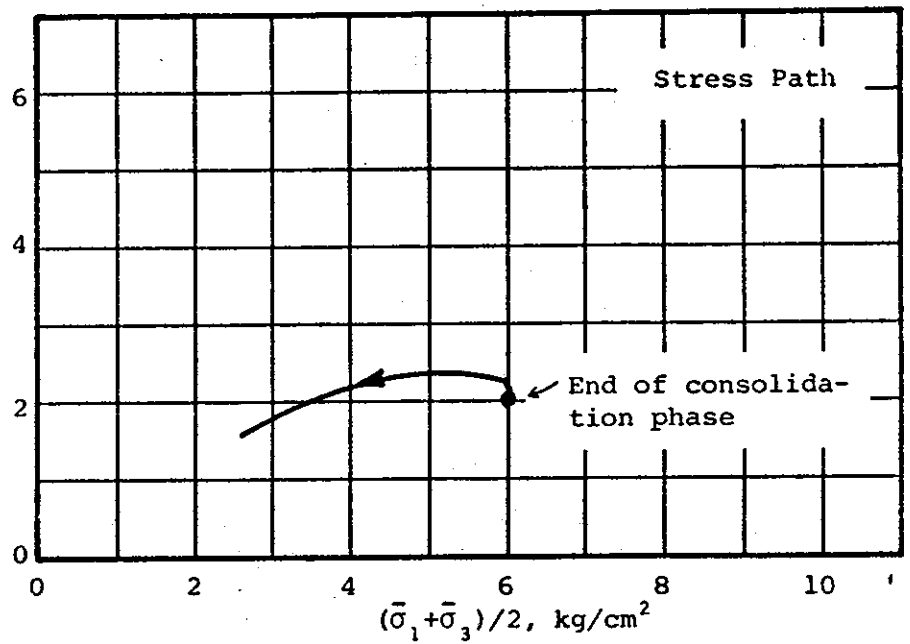
DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM

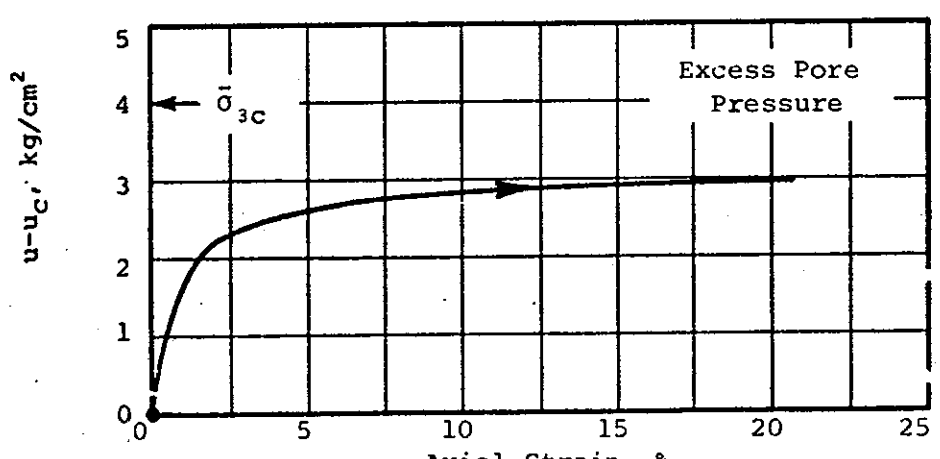
TRIAXIAL COMPRESSION TEST
BORING NO. K-5, SAMPLES COMBINED
CONSOLIDATED – DRAINED

BY: W. Z. DATE: 4-12-82

DWG. NO. FIGURE D2-8







Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

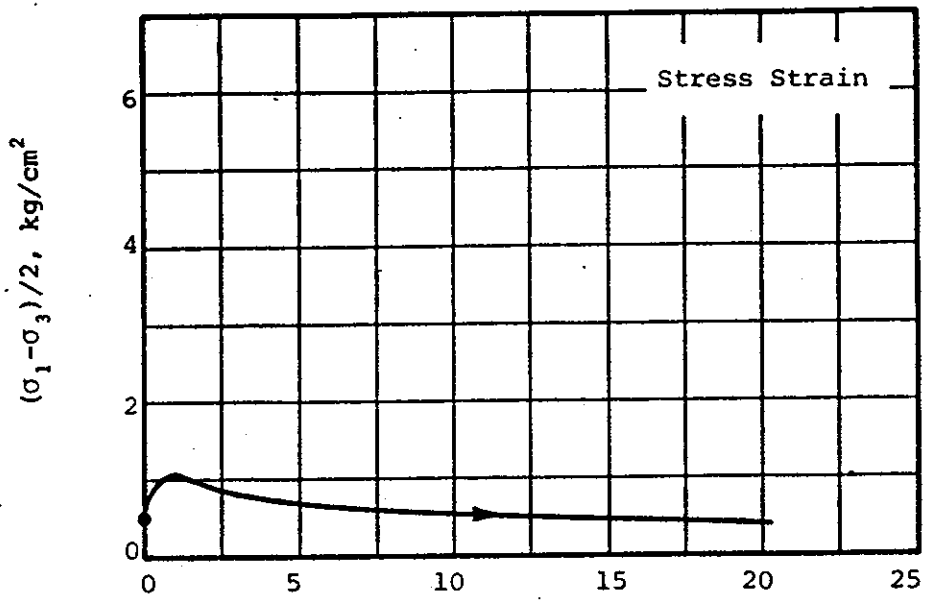
State: $\bar{\sigma}_{1c}$=8.00 kg/cm^2, $\bar{\sigma}_{3c}$=4.00 kg/cm^2, u_c=10.00 kg/cm^2, B=0.96. Dry Unit Weight in cell = initial 125.5 pcf, after consolidation 133.8 pcf.

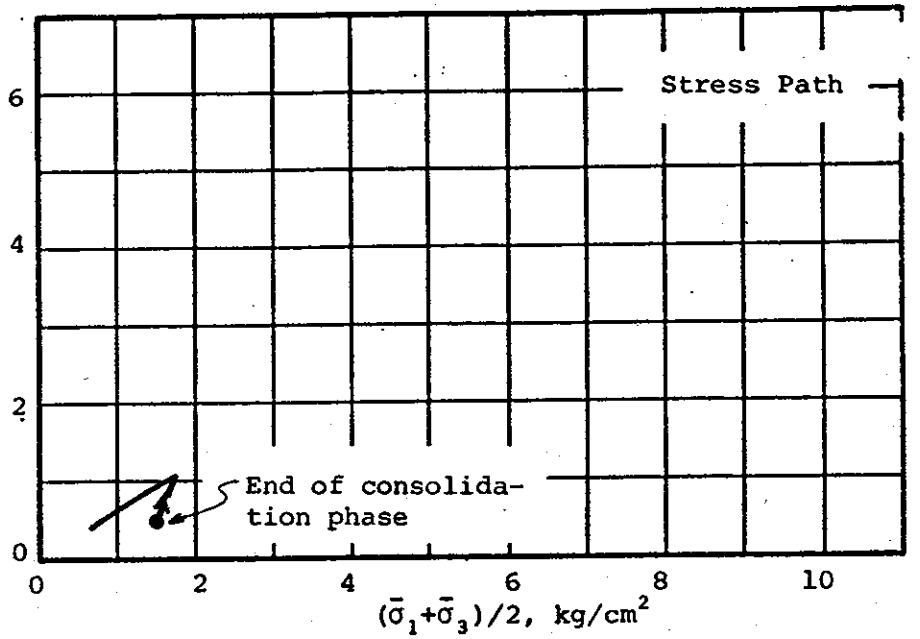
Loading: Undrained monotonic axial compression. Rate of strain = 0.20% per min.

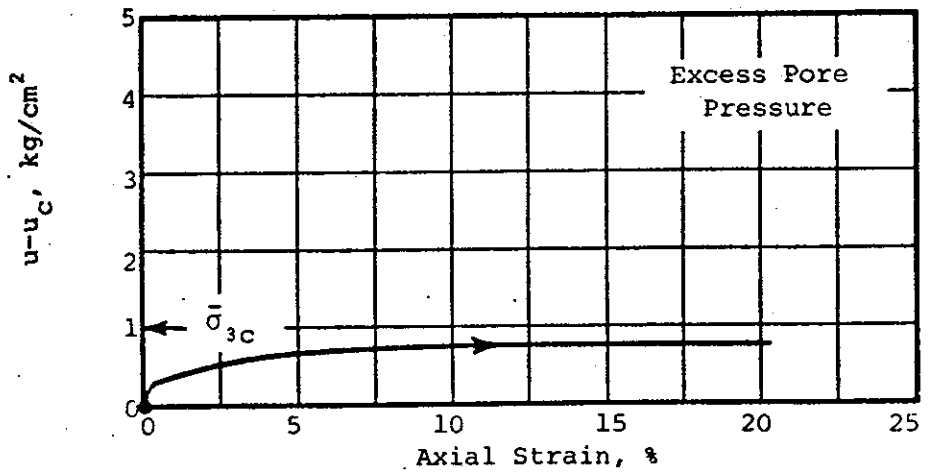
Size: Height 15.4 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST $\bar{R}$-101

FIGURE D2-8







Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

State: $\bar{\sigma}_{1c}$=2.00 kg/cm^2, $\bar{\sigma}_{3c}$=1.00 kg/cm^2, u_c=9.00 kg/cm^2, B=0.97. Dry Unit Weight in cell: initial 127.8 pcf, after consolidation 130.1 pcf.

Loading: Undrained monotonic axial compression. Rate of strain = 0.20% per min.

Size: Height 15.4 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST $\bar{R}$-102

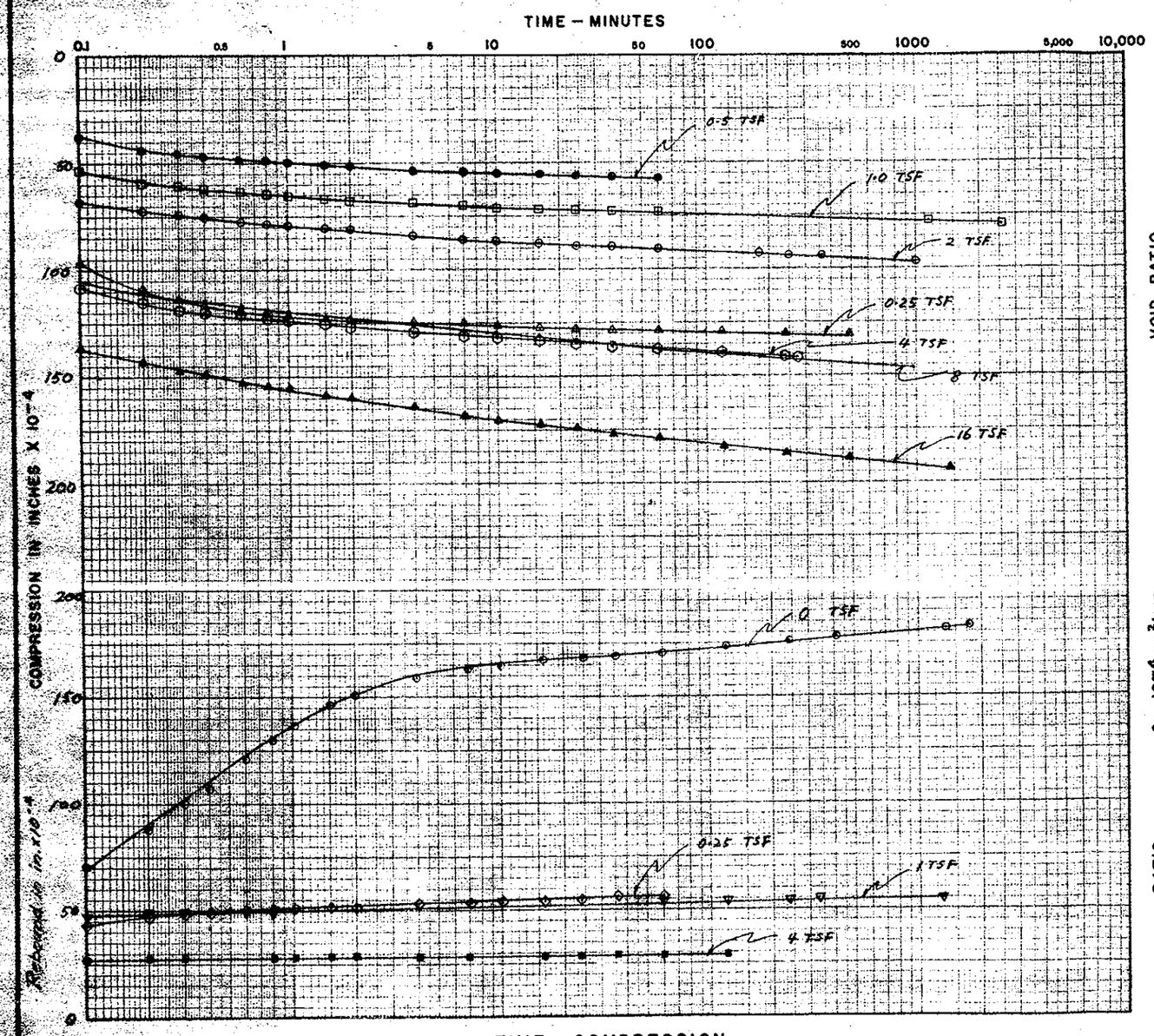
FIGURE D2-9

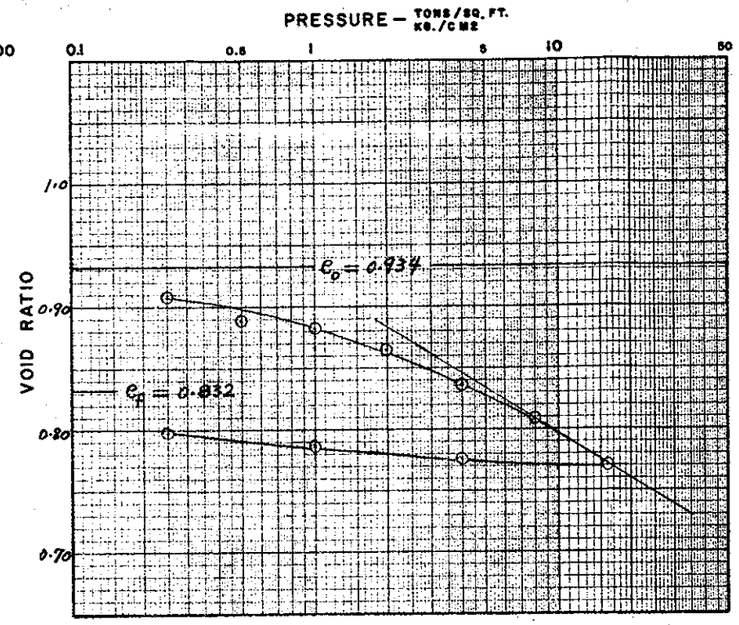
Section D3

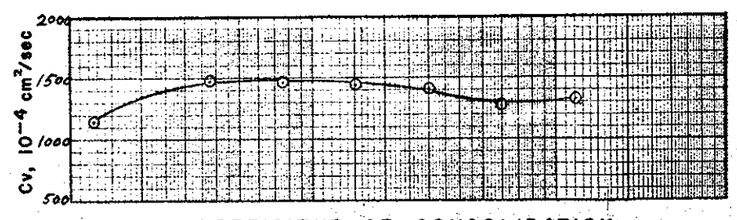
Consolidation Test Results

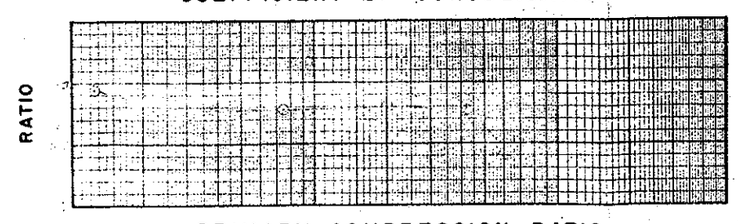
FIGURE	EMBANKMENT MATERIAL	PAGE
D3-1	Core	D3-1
D3-2	Core	D3-2
D3-3	Core	D3-3
D3-4	Core	D3-4

FORM NO. 144 4-61









PERCENT COMPRESSION

SAMPLE DATA

Sample No. SP-3
Boring No. K-2
Location KNIGHTVILLE DAM
Depth 53.5 ft
Description: Br. silt, sm. f. sand

PHYSICAL PROPERTIES

Natural Water Content - % 31.8
Liquid Limit NP
Plasticity Index NP
Specific Gravity 2.81
Dry Unit Weight - lbs./ft.³ 90.7

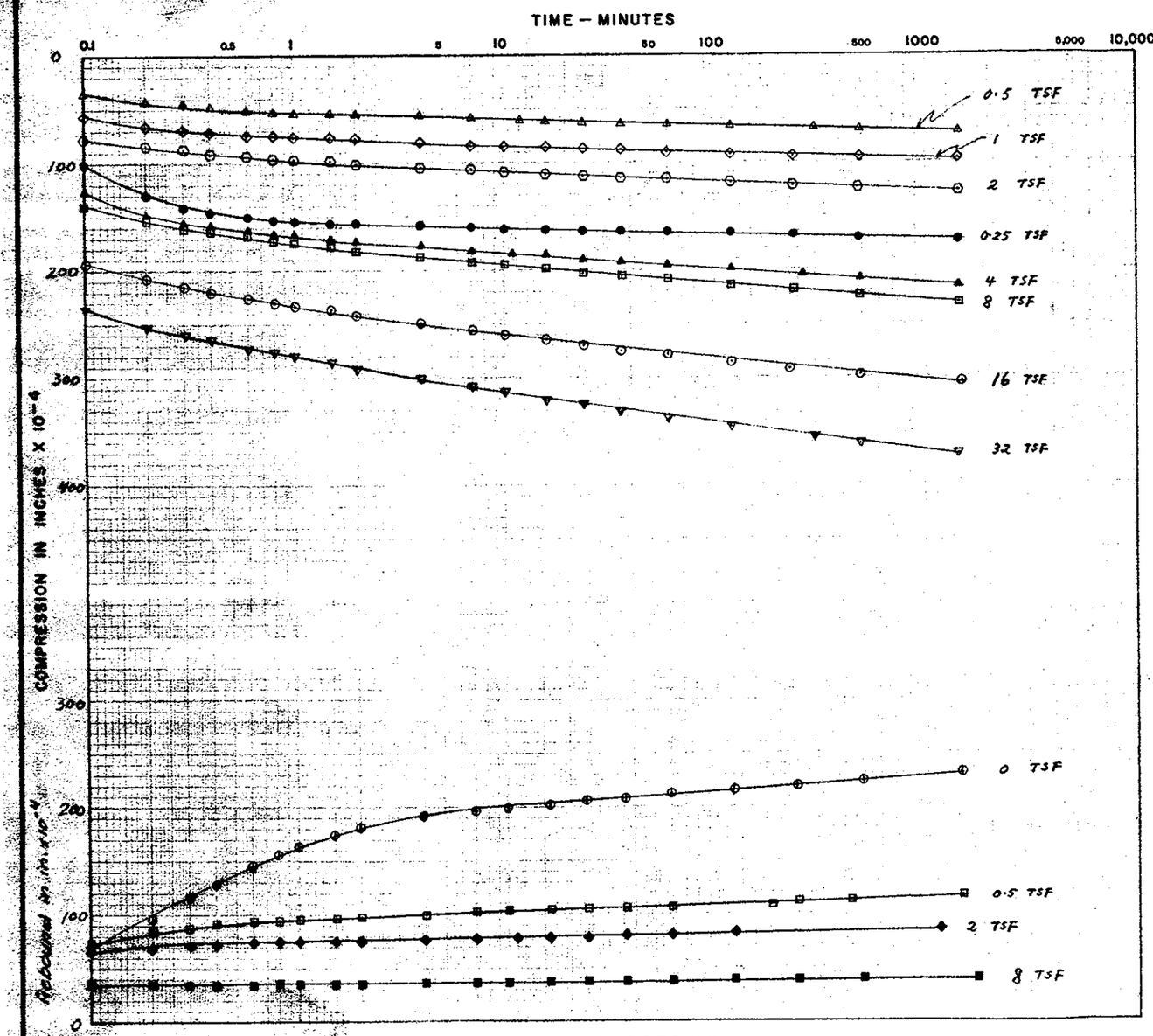
TEST DATA

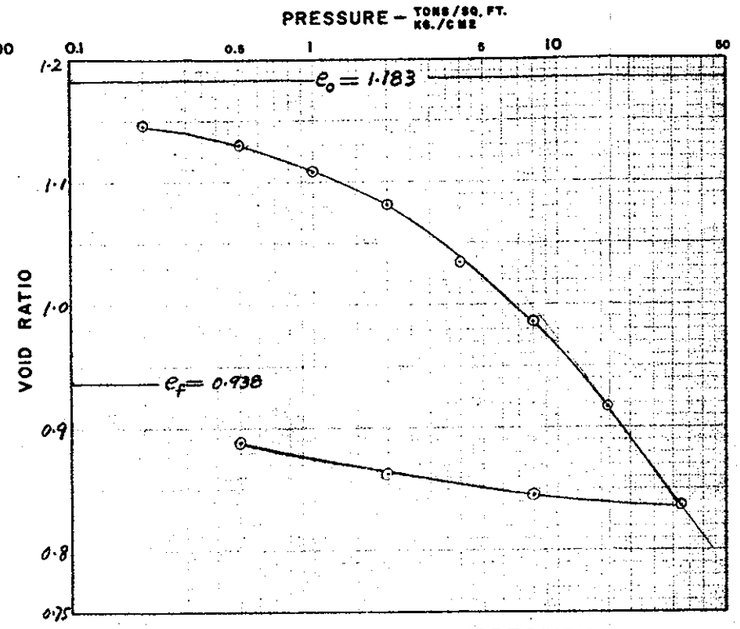
Initial Void Ratio 0.934
Final Void Ratio 0.832
Initial Degree of Saturation - % 95.6
Final Degree of Saturation - % 100
Initial Water Content - % 31.8
Final Water Content - % 27.9
Initial Height of Sample - ins. 1.005
Final Height of Sample - ins. 0.952
Area of Sample - sq. ins. 4.91
Compression Index - C_c 0.12
Preconsolidation Load
Remarks: Test performed in teflon-lined floating ring in a submerged condition with double drainage and back pressure equal to 100 psi

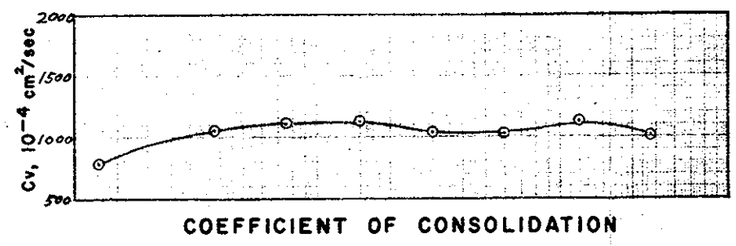
D3-1

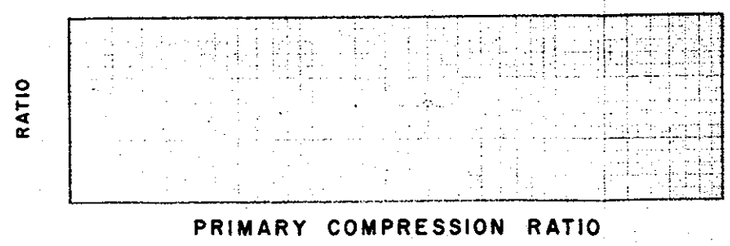
U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION	TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS NEW YORK	
DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM	CONSOLIDATION TEST RESULTS BORING NO. K-2, SAMPLE 3	BY: W. Z. DATE: 4-19-62 DWG: FIGURE D3-1

FORM NO. 144 4-61









SAMPLE DATA

Sample No. SP-7
Boring No. K-2
Location KNIGHTVILLE DAM
Depth 72.5 ft
Description: Br. silt sm. sand

PHYSICAL PROPERTIES

Natural Water Content - % 39.4
Liquid Limit NP
Plasticity Index NP
Specific Gravity 2.81
Dry Unit Weight - lbs./ft.³ 80.4

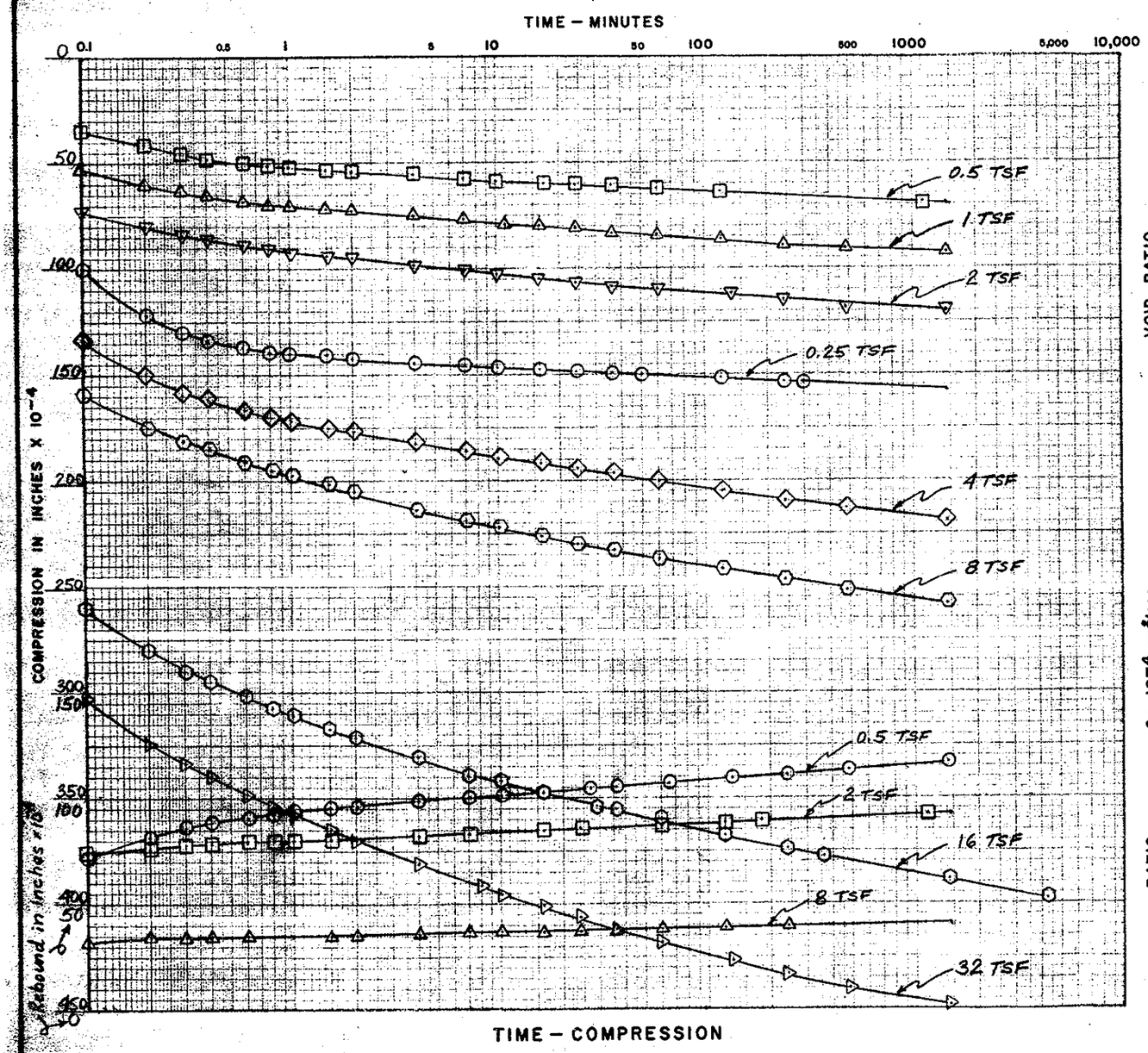
TEST DATA

Initial Void Ratio 1.183
Final Void Ratio 0.938
Initial Degree of Saturation - % 93.5
Final Degree of Saturation - % 100
Initial Water Content - % 39.4
Final Water Content - % 31.6
Initial Height of Sample - ins. 1.005
Final Height of Sample - ins. 0.892
Area of Sample - sq. ins. 4.91
Compression Index - C_c 0.25
Preconsolidation Load
Remarks: Test performed in teflon-lined floating ring in submerged condition with double drainage and back pressure equal to 100 psi

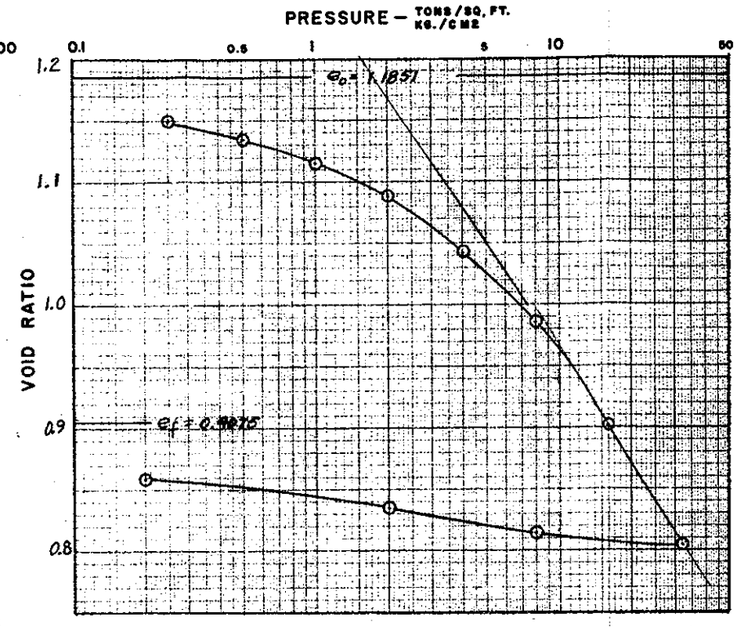
D3-2

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION	TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS NEW YORK	
DYNAMIC STABILITY ANALYSIS OF KNIGHTVILLE DAM	CONSOLIDATION TEST RESULTS BORING NO. K-2, SAMPLE 7	BY: W. Z. DATE: 4-19-82
		DWG: FIGURE D3-2

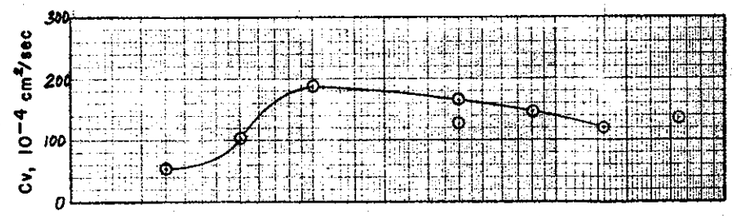
FORM NO. 144 4-61



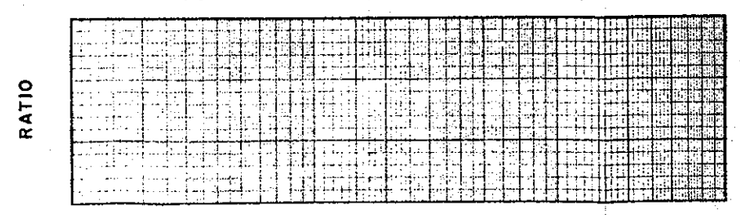
TIME – COMPRESSION



VOID RATIO – % COMPRESSION



COEFFICIENT OF CONSOLIDATION



PRIMARY COMPRESSION RATIO

PERCENT COMPRESSION

SAMPLE DATA

Sample No. SP-11
Boring No. K-2
Location Knightville Dam
Depth 93.5
Description: Bn. silt, sm. sand

PHYSICAL PROPERTIES

Natural Water Content - % 41.3
Liquid Limit NP
Plasticity Index NP
Specific Gravity 2.78
Dry Unit Weight - lbs./ft.³ 79.4

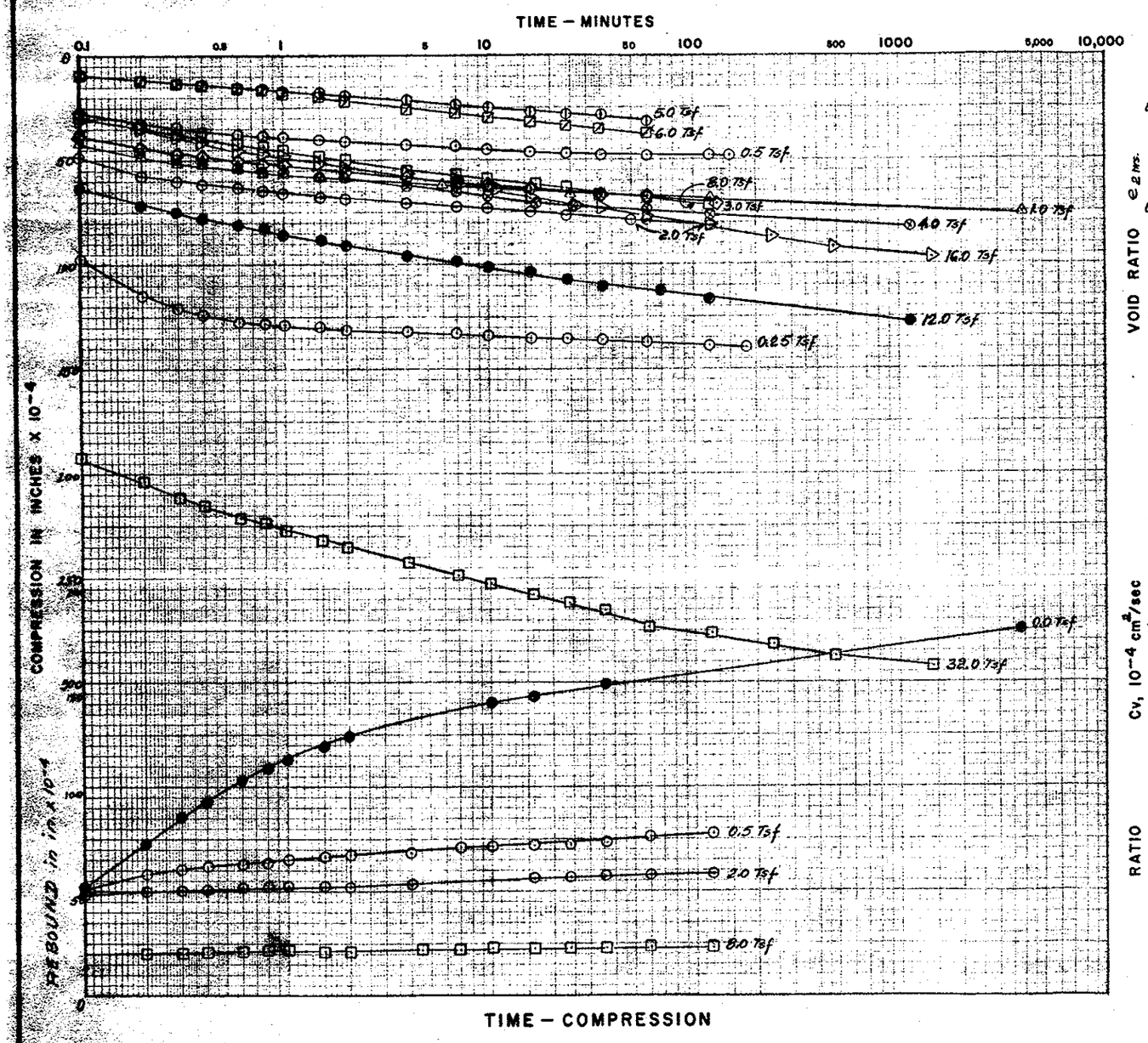
TEST DATA

Initial Void Ratio 1.1851
Final Void Ratio 0.9075
Initial Degree of Saturation - % 96.8
Final Degree of Saturation - % 100.0
Initial Water Content - % 41.3
Final Water Content - % 34.8
Initial Height of Sample - ins. 1.005
Final Height of Sample - ins. 0.8773
Area of Sample - sq. ins. 4.91
Compression Index - C_c 0.30
Preconsolidation Load
Remarks: Test performed in teflon-lined floating ring in a submerged condition with double drainage and back pressure equal to 100 psi

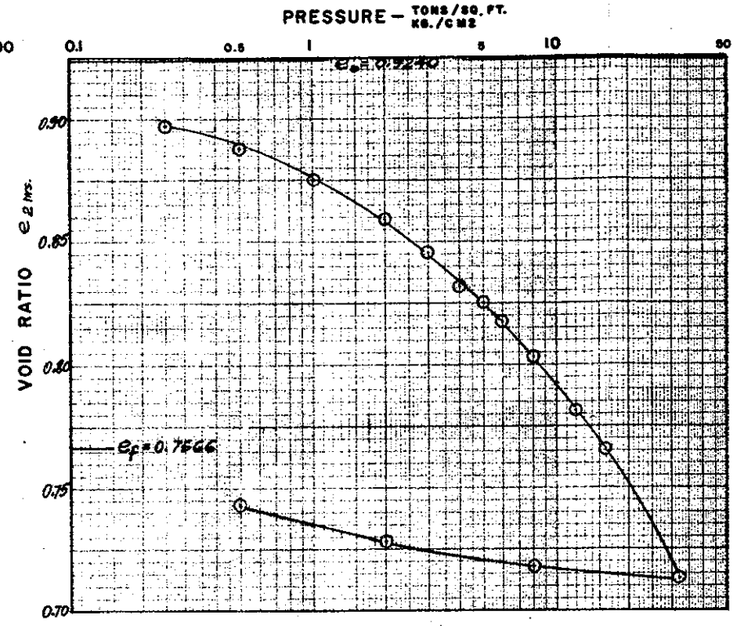
D3-3

U.S. ARMY CORPS of ENGINEERS NEW ENGLAND DIVISION	TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS NEW YORK	
DYNAMIC STABILITY ANALYSIS KNIGHTVILLE DAM, MA.	CONSOLIDATION TEST RESULTS BORING NO. K-2 SAMPLE NO. SP-11	BY: JB/JB DATE: 6-82 DWG: FIGURE D3-3

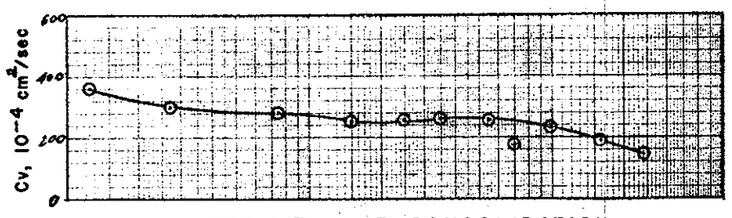
FORM NO. 144 4-61



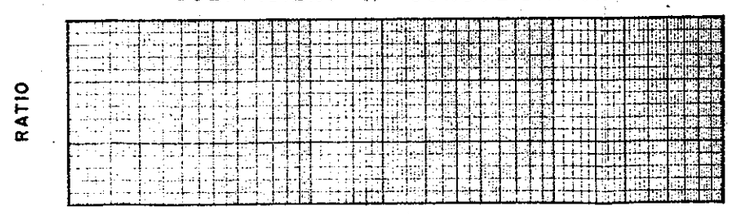
TIME – COMPRESSION



VOID RATIO – % COMPRESSION



COEFFICIENT OF CONSOLIDATION



PRIMARY COMPRESSION RATIO

PERCENT COMPRESSION

SAMPLE DATA

Sample No. SP-16
Boring No. K-2
Location Knightville Dam
Depth 135.5-137.5
Description: Br. silt, sm. f. sand, tr. clay.

PHYSICAL PROPERTIES

Natural Water Content - % 32.2
Liquid Limit 34.5
Plasticity Index 5.3
Specific Gravity 2.77
Dry Unit Weight - lbs./ft.³ 89.8

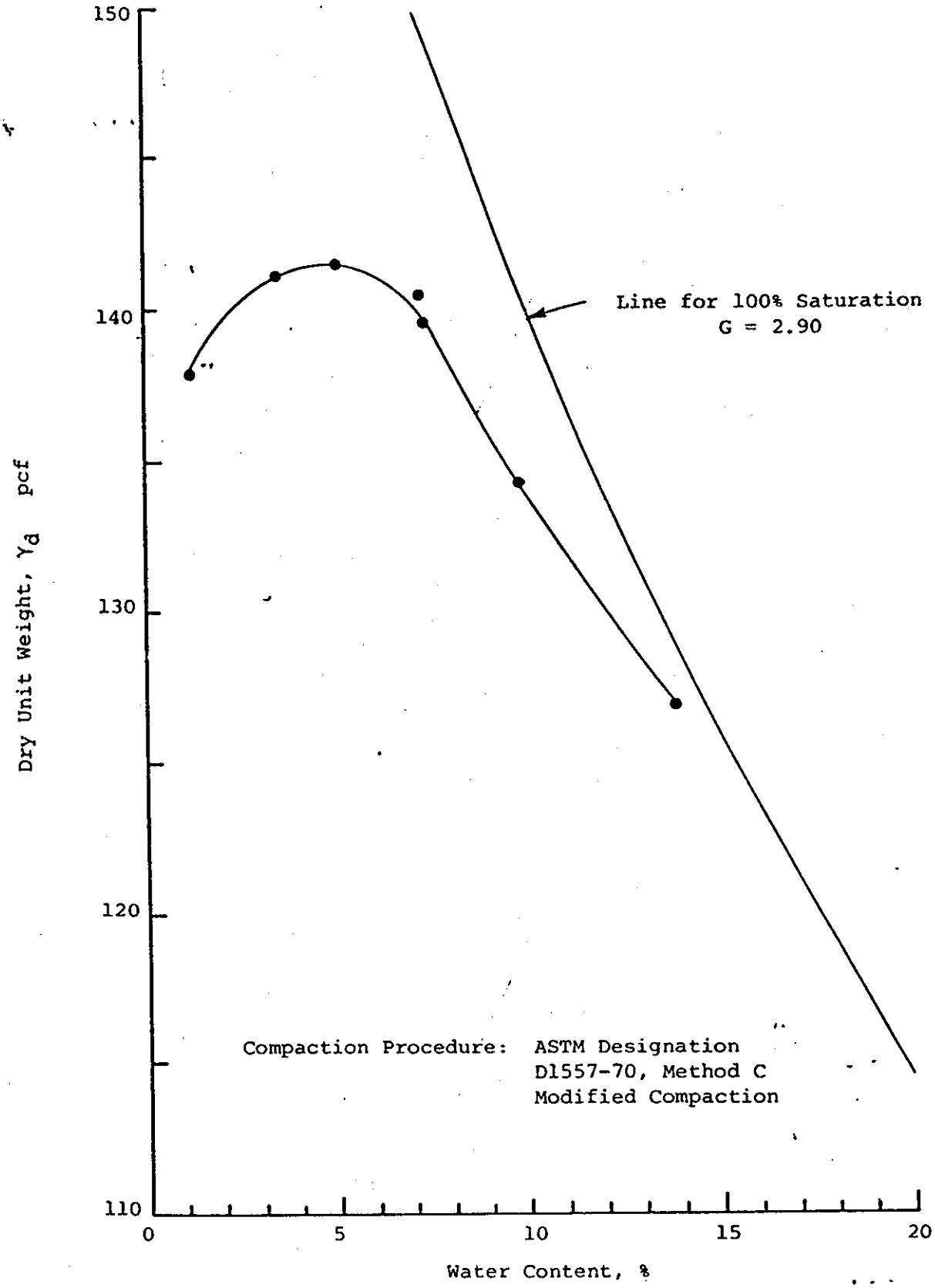
TEST DATA

Initial Void Ratio 0.9240
Final Void Ratio 0.7666
Initial Degree of Saturation - % 96.5
Final Degree of Saturation - % 100.0
Initial Water Content - % 32.2
Final Water Content - % 30.6
Initial Height of Sample - ins. 1.005
Final Height of Sample - ins. 0.9228
Area of Sample - sq. ins. 4.91
Compression Index - C_c 0.12
Preconsolidation Load
Remarks: Test performed in teflon-lined floating ring in a submerged condition with double drainage and a back pressure equal to 100 psi

U.S. ARMY CORPS of ENGINEERS NEW ENGLAND DIVISION	TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS NEW YORK	
DYNAMIC STABILITY ANALYSIS KNIGHTVILLE DAM, MA.	CONSOLIDATION TEST RESULTS BORING No. K-2 SAMPLE No. SP-16	BY: ZB DATE: 1-82 DWG: FIGURE D3-4

Section D4

Compaction Test Results



D4-1

COMPACTION CURVE
MIXTURE OF SHELL MATERIAL

FIGURE D4-1

APPENDIX E

DYNAMIC LABORATORY TESTING PROGRAM DATA

Sections

E1 - Cyclic Load Triaxial Test Data

E2 - Resonant Column and Small Strain Triaxial Test Data

E3 - Definitions of Dynamic Test Relationships

Section E1

Cyclic Load Triaxial Test Data

FIGURE NO.	TITLE	PAGE
E1-1	Consolidated Undrained Cyclic Triaxial Test $C\bar{R}$-1	E1-1
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Section E1
(continued)

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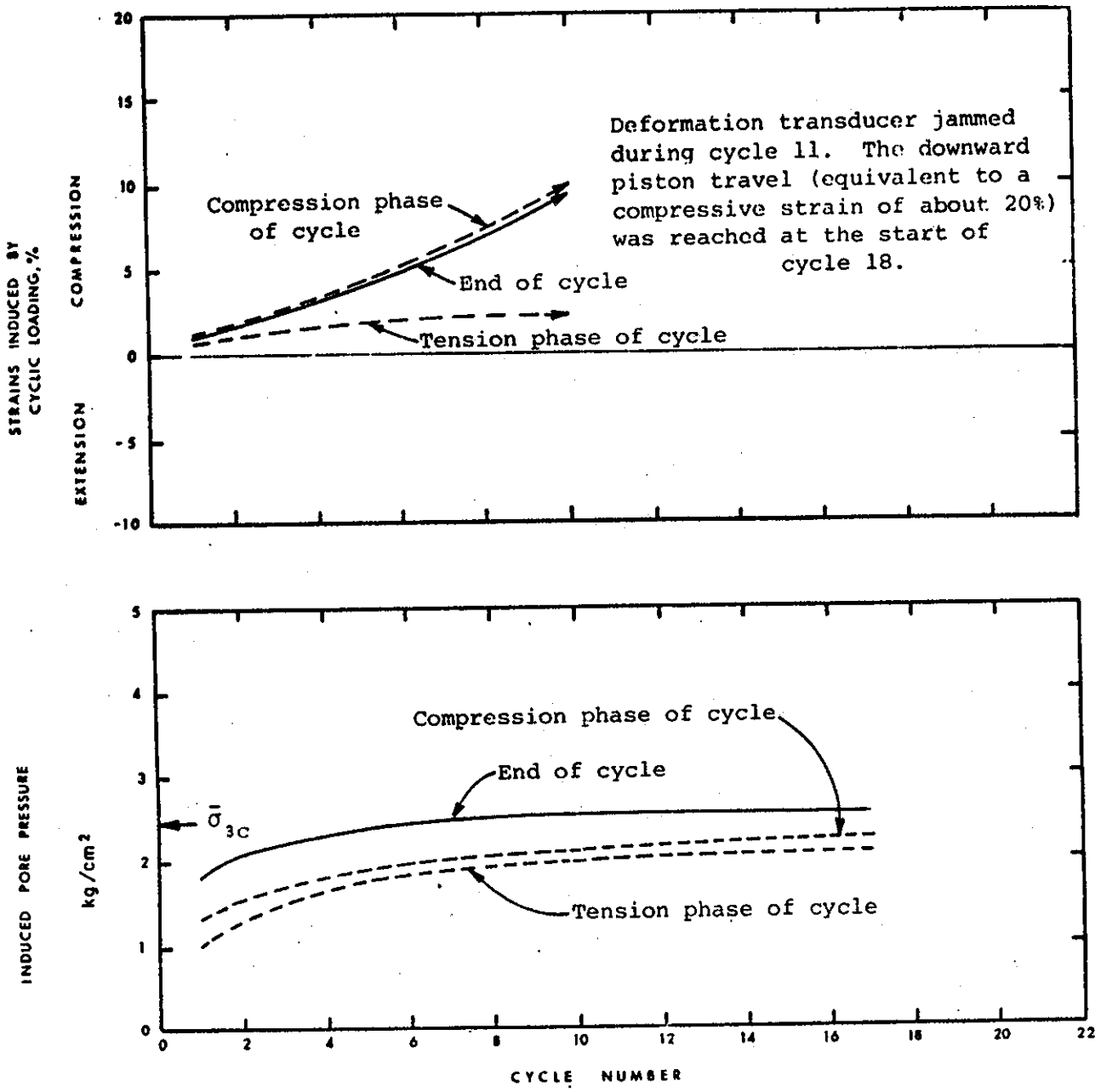


Soil: Boring K-2, Sample SP-2A, Depth 47.5 ft. Contorted layers and lenses of silt and fine sand. Top 2/3 of specimen is slightly coarser grained than the bottom 1/3. Mica flakes. Olive brown. Note: Grain size analysis performed on top 2/3 of specimen.

Structure: Undrained 3-in. tube sample of core material.

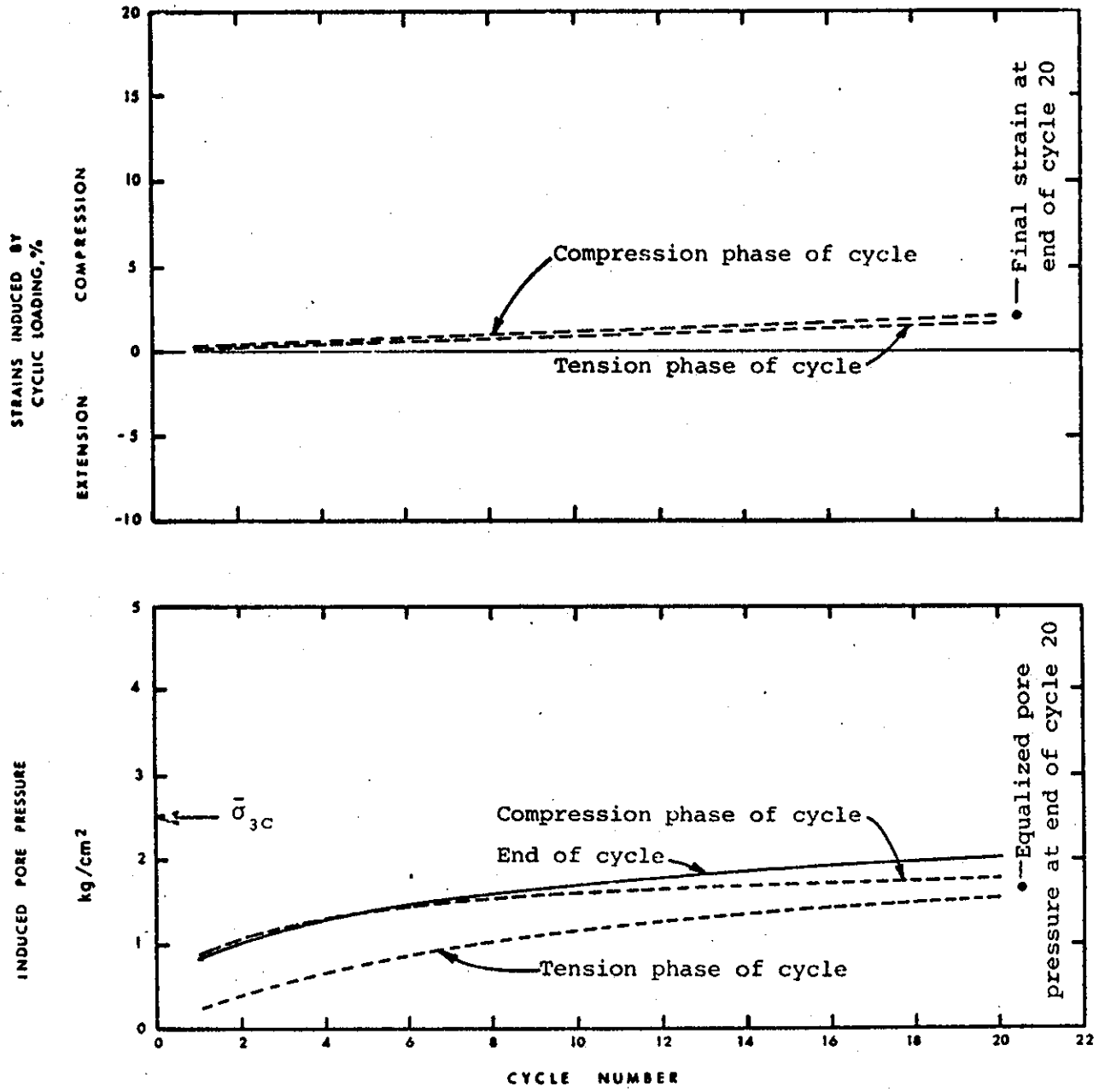
State: $\bar{\sigma}_{1c}$=3.75 kg/cm^2, $\bar{\sigma}_{3c}$=2.50 kg/cm^2, u_c=8.00 kg/cm^2, B =0.96. Dry Unit Weight - in tube 85.0 pcf, in cell after consolidation 89.5 pcf (See Table 4.1).

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.30. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 15.4 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST CR-1

FIGURE E1-1



Soil: Boring K-2, Sample SP-1B, Depth 42.5 ft. Horizontal layers and irregularly shaped lenses of silt and fine sand. Bottom 2/3 of specimen is slightly coarser grained than the top 1/3. Mica flakes. Olive brown. Note: Grain size analysis performed on bottom 2/3 of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1C}$=3.75 kg/cm^2, $\bar{\sigma}_{3C}$=2.50 kg/cm^2, u_C=8.00 kg/cm^2, B =0.92. Dry Unit Weight - in tube 84.7 pcf, in cell after consolidation 90.9 pcf (See Table 4.1).

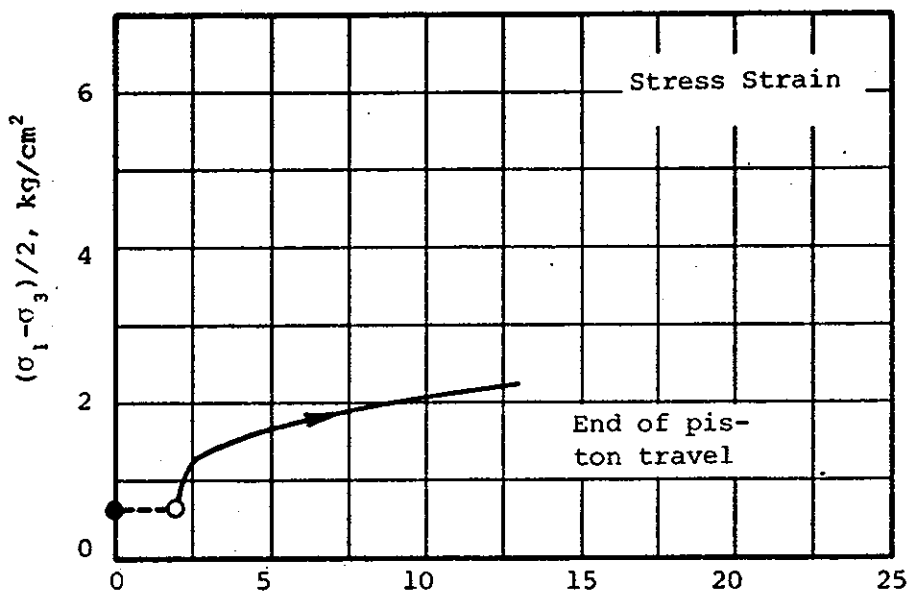
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3C}$= 0.24. No principal stress reversal. Frequency of loading 0.4Hz.

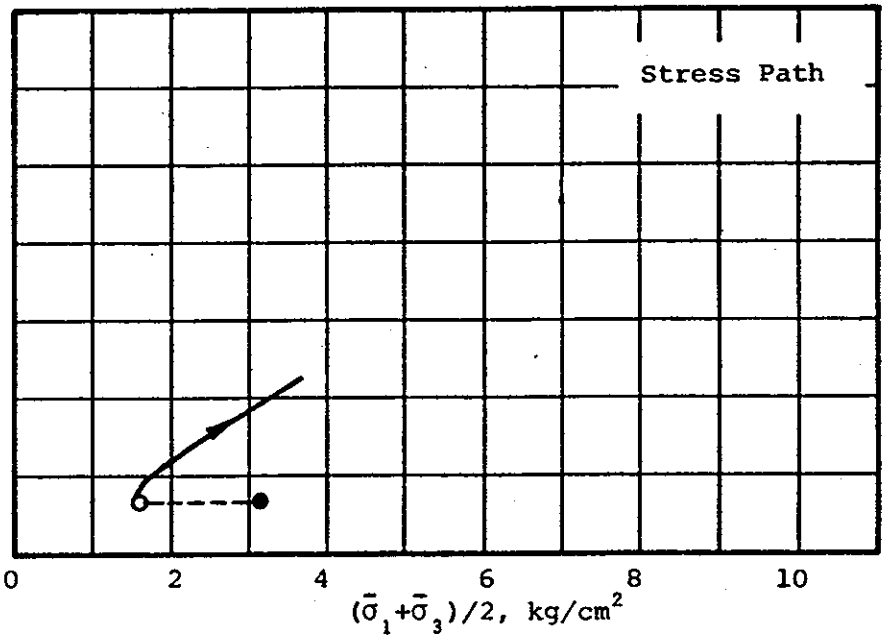
Size: Height 15.9 cm, Diameter 7.1 cm.

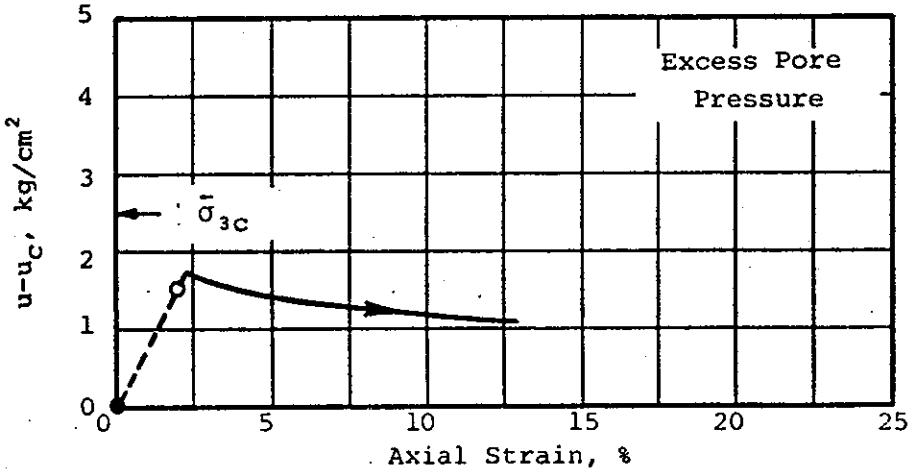
Comments: Test E-1 was performed on Sample SP-1B before this $C\bar{R}$ test. This $C\bar{R}$ test was followed by an $\bar{R}$ test. See Fig. 4.3 for results of $\bar{R}$ phase.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-$\bar{R}$-2

FIGURE E1-2







Soil: Boring K-2, Sample SP-1B, Depth 42.5 ft Horizontal layers and irregularly shaped lenses of silt and fine sand. Bottom 2/3 of specimen is slightly coarser grained than the top 1/3. Mica flakes. Olive brown.

Structure: Undisturbed 3-in. tube sample of core material.

State: Before cyclic loading phase - see Fig. 4.2
After cyclic loading - Axial strain = 2.07%, $u-u_c$=1.52 kg/cm^2

Loading: Undrained monotonic axial compression. Rate of strain = 0.19% per min.

Size: Height 15.9 cm, Diameter 7.1 cm.

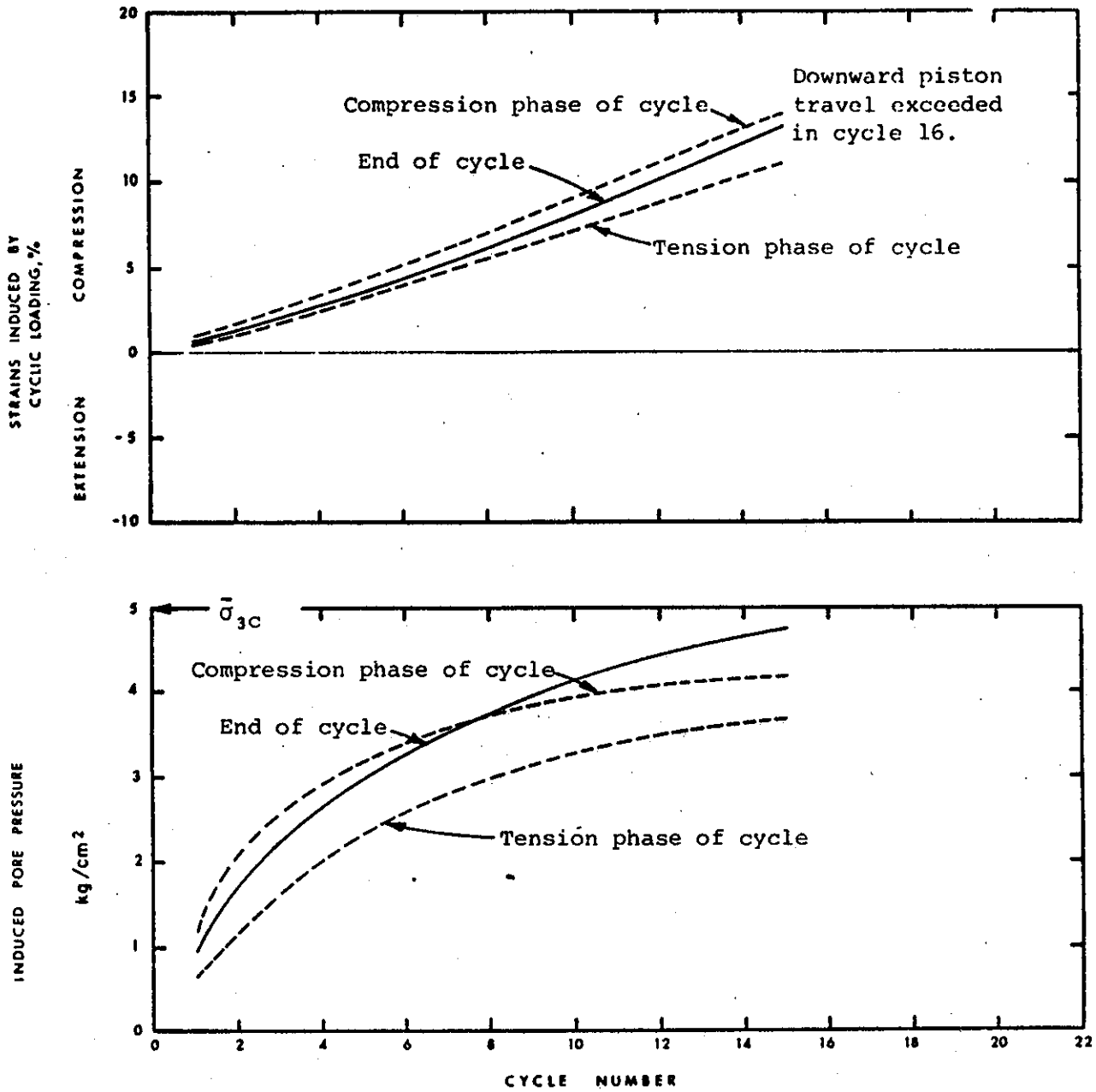
LEGEND:

● End of Consolidation Phase

○ End of Cyclic Loading Phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $C\bar{R}$-$\bar{R}$-2

FIGURE E1-3



Soil: Boring K-2, Sample SP-13B, Depth 106.5 ft.
Contorted layers and lenses of silt and fine sand. Mica flakes. Olive brown. Note: Grain size analysis performed on a mixture of the sample.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1c}$=7.50 kg/cm^2, $\bar{\sigma}_{3c}$=5.00 kg/cm^2, u_c=4.00 kg/cm^2, B=0.95. Dry Unit Weight - in tube 81.0 pcf, in cell after consolidation 88.6 pcf (See Table 4.1).

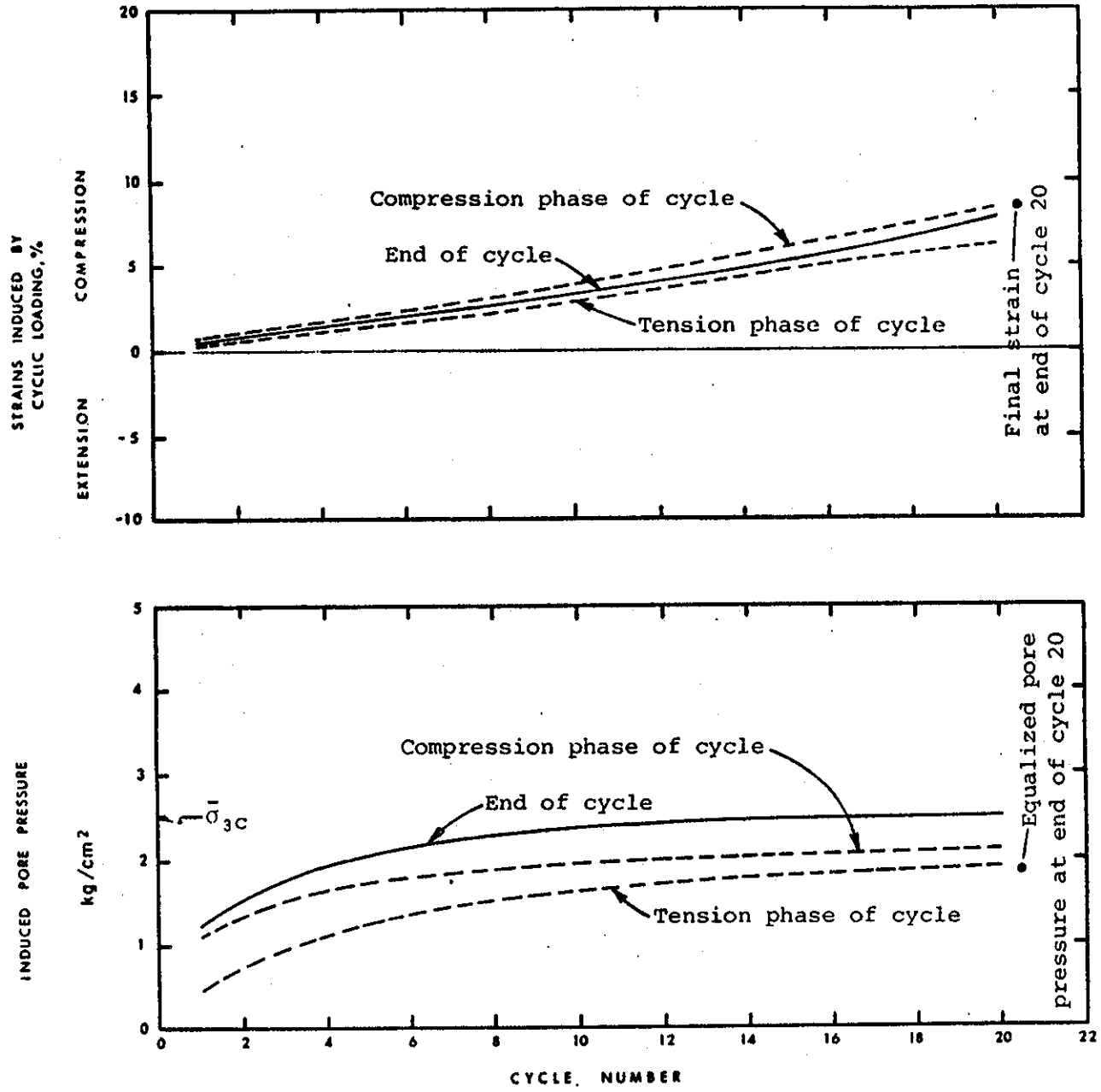
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.28. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 15.9 cm, Diameter 7.1 cm.

Comments: Test E-2 was performed on Sample SP-13B before this $C\bar{R}$ test.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-3

FIGURE E1-4



Soil: Boring K-2, Sample SP-5B, Depth 63.7 ft.
Sub-horizontal layers of silt and fine sand. Occasional silt lense. The center 2/3 of the specimen is predominantly fine sand. Several root fibers. Mica flakes. Olive brown. Note: Grain size analysis performed on center 2/3 of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

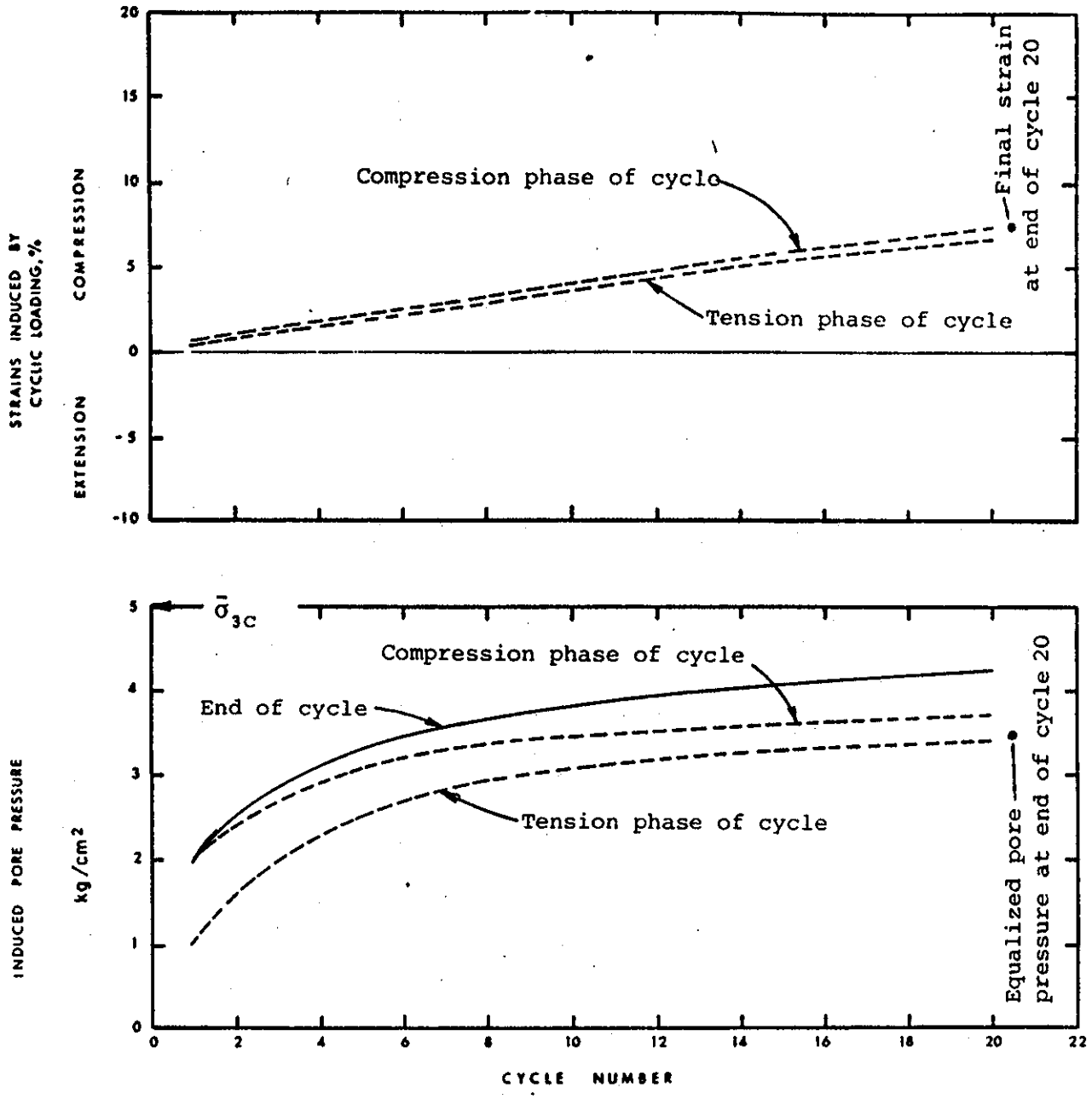
State: $\bar{\sigma}_{1c}$= 3.75 kg/cm^2, $\bar{\sigma}_{3c}$=2.50 kg/cm^2, u_c=8.00 kg/cm^2, B =0.97. Dry Unit Weight - in tube 90.6 pcf, in cell after consolidation 93.7 pcf. (See Table 4.1).

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.27. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 18.0 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED
CYCLIC TRIAXIAL TEST
CR-5

FIGURE E1-5



Soil: Boring K-2, Sample SP-9B, Depth 83.3 ft.
Top 1/3 - silty fine sand. Bottom 2/3 - silt with some fine sand. Specimen has mica flakes. Olive brown. Note: Separate grain size analyses performed on top and bottom portions of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

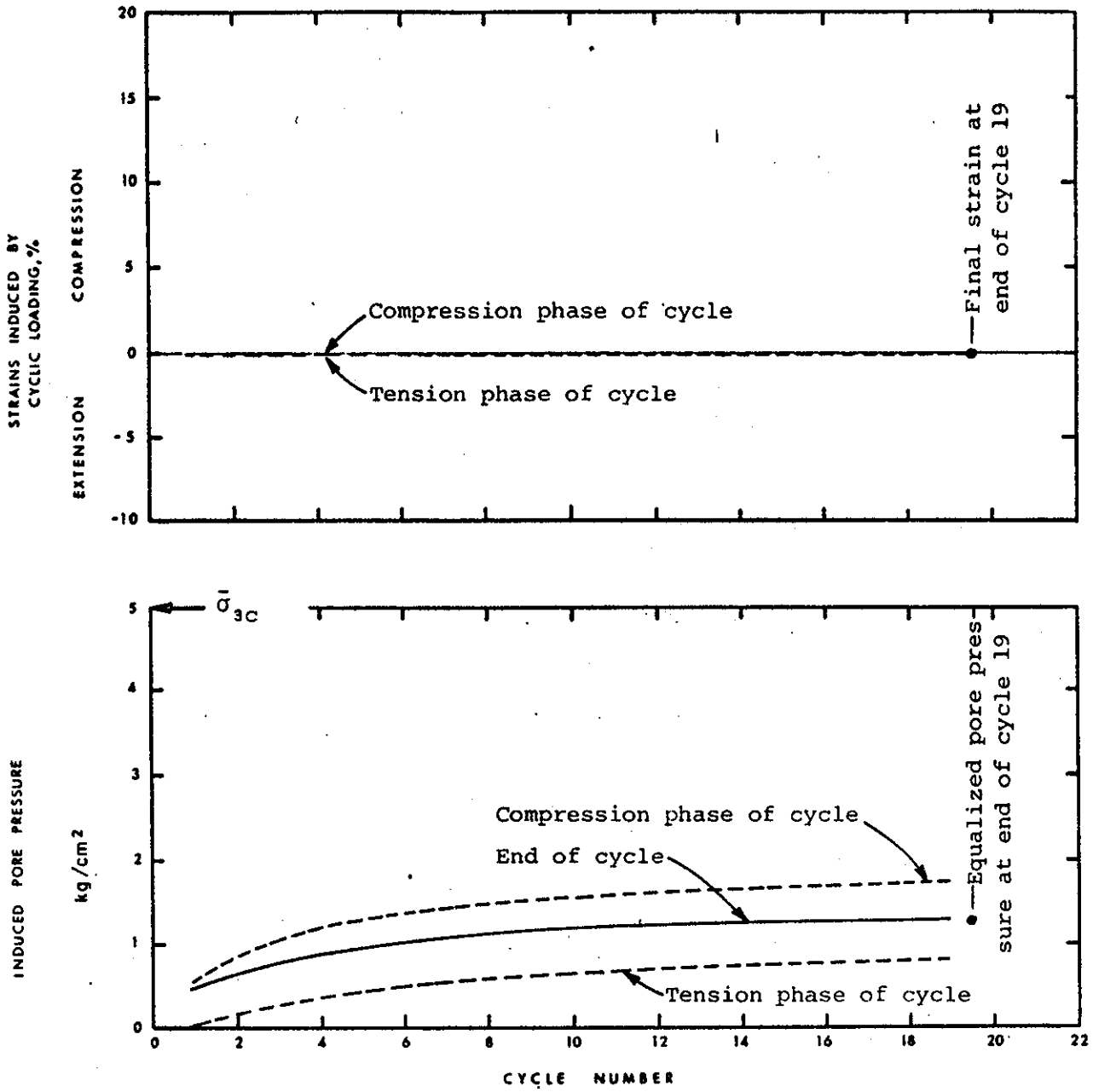
State: $\bar{\sigma}_{1c}$=7.50 kg/cm^2, $\bar{\sigma}_{3c}$=5.00 kg/cm^2, u_c=8.00 kg/cm^2, B = 0.95. Dry Unit Weight - in tube 90.9 pcf, in cell after consolidation 95.7 pcf (See Table 4.1).

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.21. No principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 18.1 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST CR-6

FIGURE E1-6



Soil: Boring K-2, Sample SP-9C, Depth 84.0 ft.
Contorted layers and lenses of silt and fine sand. Root fibers. Mica flakes. Olive brown. Note: Grain size analyses performed on a mixture of the sample.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1c}$=5.00 kg/cm^2, $\bar{\sigma}_{3c}$=5.00 kg/cm^2, u_c=8.00 kg/cm^2, B =0.97. Dry Unit Unit Weight - in tube 87.4 pcf, in cell after consolidation 91.7 pcf. (See Table 4.1).

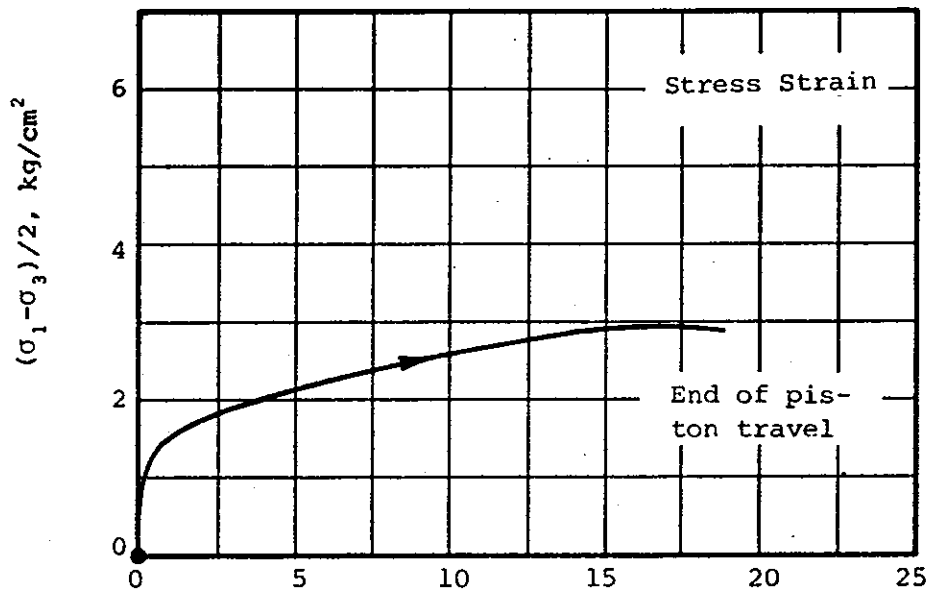
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.10. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

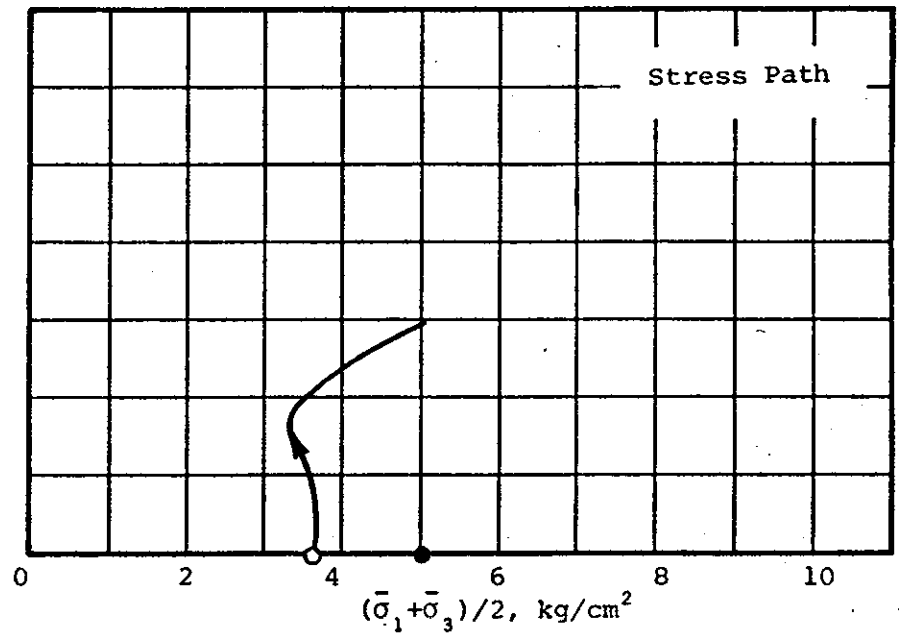
Size: Height 17.5 cm, Diameter 7.3 cm.

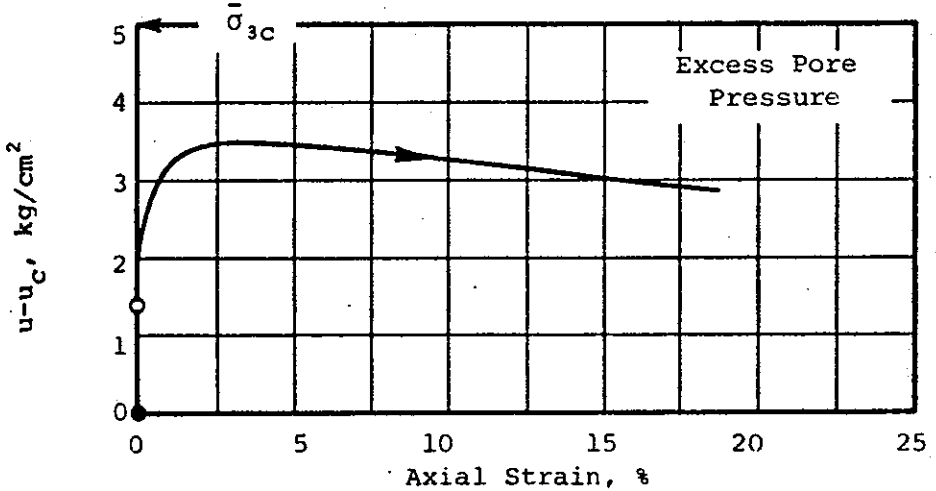
Comments: This $C\bar{R}$ test was followed by an $\bar{R}$ test.

CONSOLIDATED UNDRAINED
CYCLIC TRIAXIAL TEST
$C\bar{R}$-$\bar{R}$-7

FIGURE E1-7







Soil: Boring K-2, Sample SP-9C, Depth 84.0 ft
Contorted layers and lenses of silt and fine sand. Root fibers. Mica flakes. Olive brown.

Structure: Undisturbed 3-in. tube sample of core material.

State: Before cyclic loading phase - see Fig. 4.7
After cyclic loading - Axial strain = -0.02%, $u-u_c$ = 1.37 kg/cm^2

Loading: Undrained monotonic axial compression. Rate of strain = 0.18% per min.

Size: Height 17.5 cm, Diameter 7.3 cm.

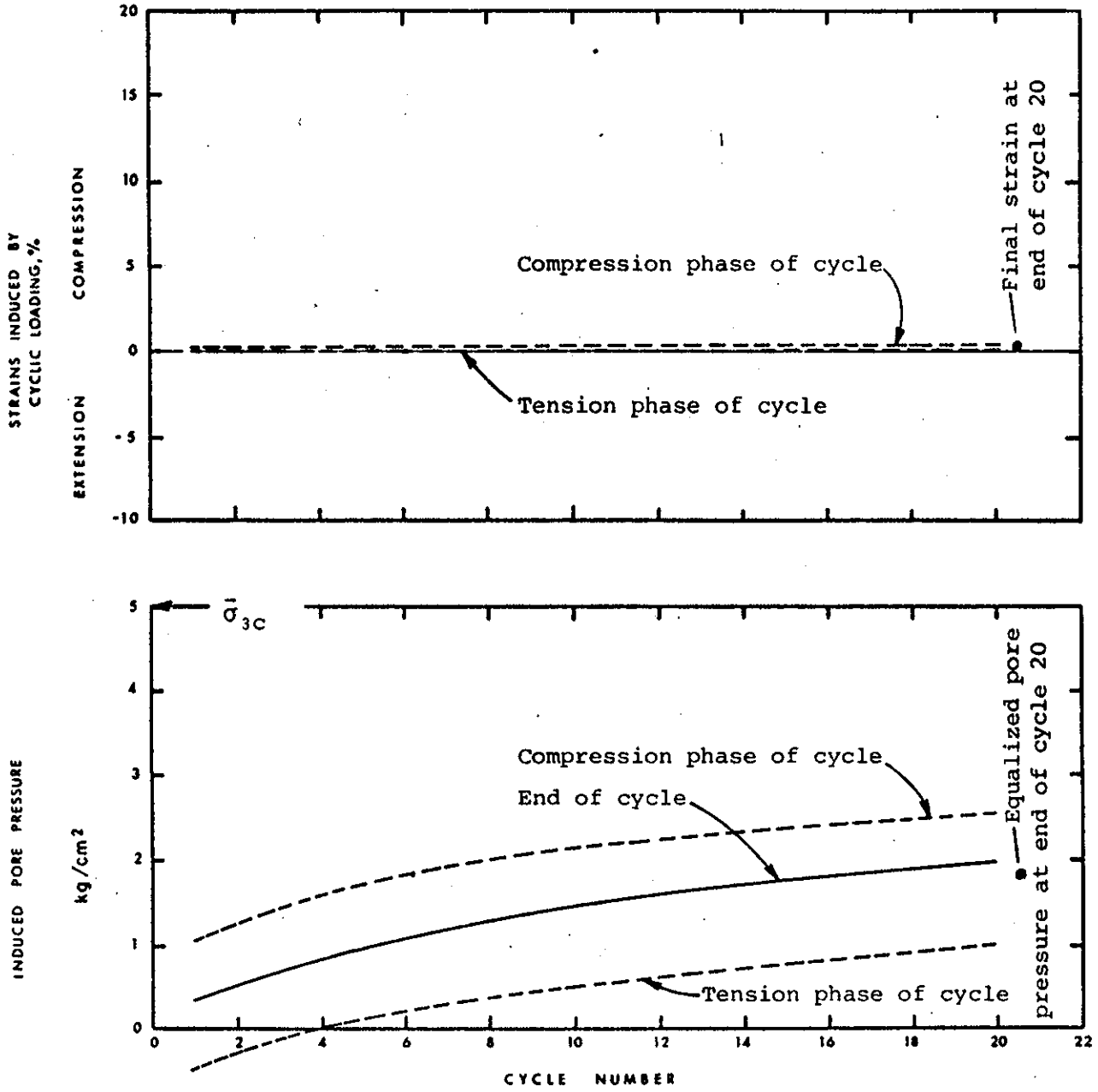
LEGEND:

● End of Consolidation Phase

○ End of Cyclic Loading Phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $\bar{CR}$-$\bar{R}$-7

FIGURE E1-8



Soil: Boring K-2, Sample SP-10A, Depth 87.8 ft.
Horizontal layers of silt and fine sand. Concentration of root fibers in a ¼-in. thick layer about 2-in. from bottom. Mica flakes. Olive brown. Note: Grain size analysis performed on a mixture of the sample.

Structure: Undisturbed 3-in. tube sample of core material.

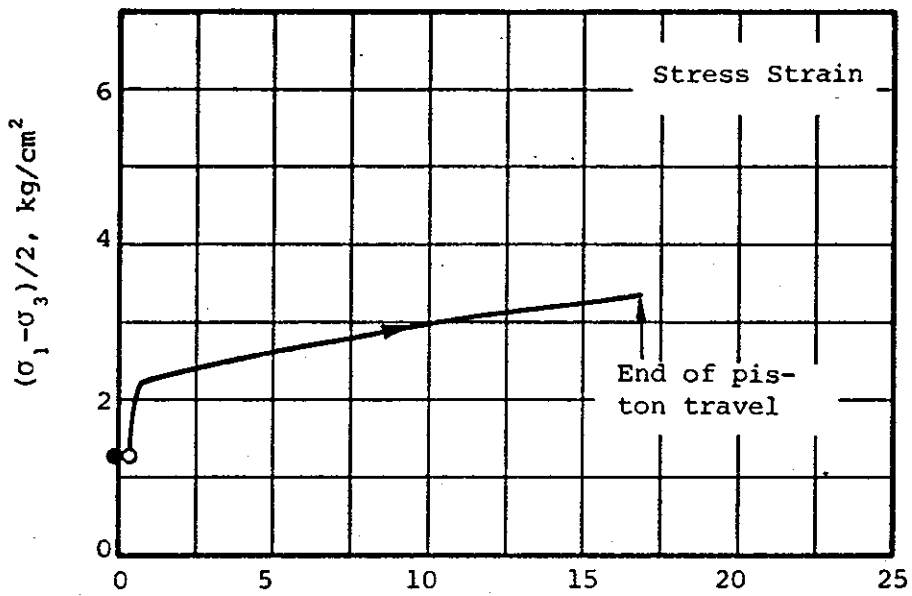
State: $\bar{\sigma}_{1C}$=7.50 kg/cm^2, $\bar{\sigma}_{3C}$=5.00 kg/cm^2, u_C=8.00 kg/cm^2, B =0.96. Dry Unit Weight - in tube 82.4 pcf, in cell after consolidation 87.9 pcf. (See Table 4.1).

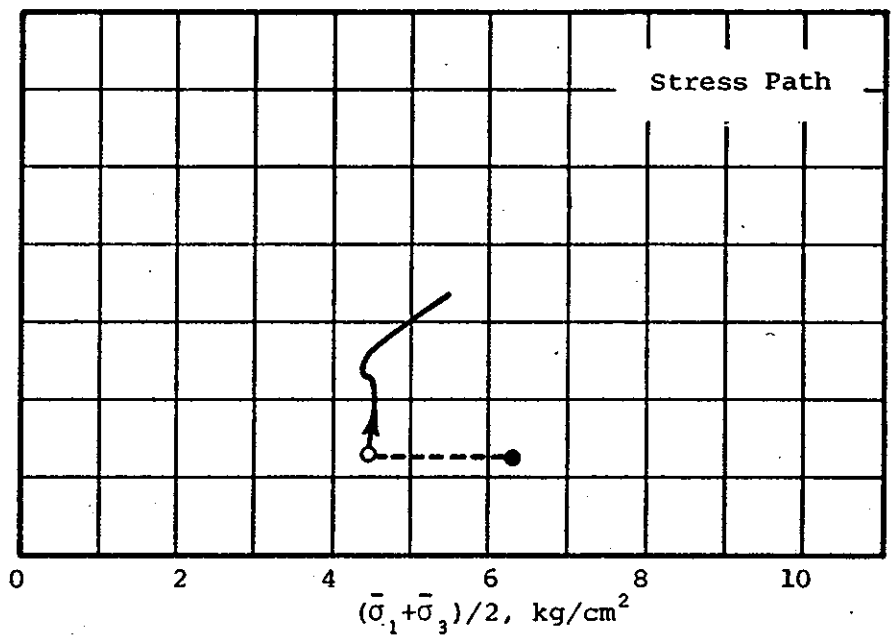
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3C}$= 0.16. No principal stress reversal. Frequency of loading 0.4Hz.

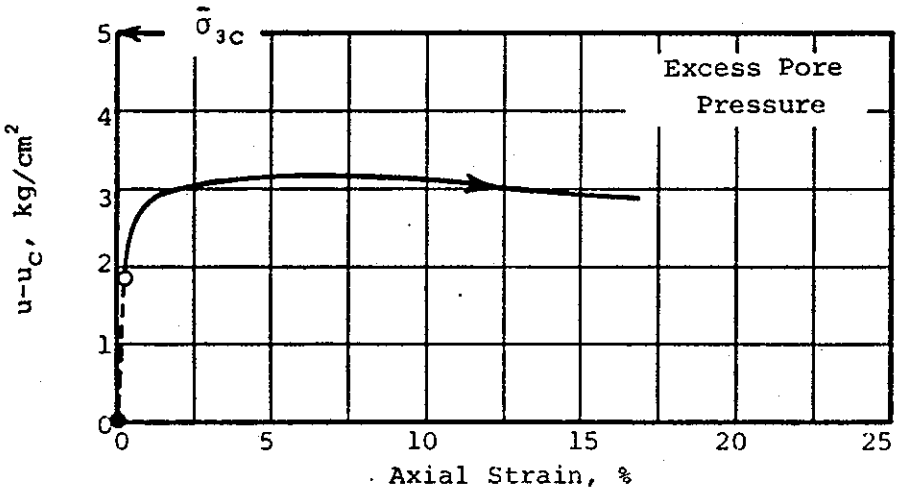
Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED
CYCLIC TRIAXIAL TEST
$C\bar{R}$-$\bar{R}$-8C

FIGURE E1-9







Soil: Boring K-2, Sample SP-10A, Depth 87.8 ft
Horizontal layers of silt and fine sand. Concentration of root fibers in a ¼-in. thick layer about 2-in. from bottom. Mica flakes. Olive brown.

Structure: Undisturbed 3-in. tube sample of core material.

State: Before cyclic loading phase - see Fig. 4.9
After cyclic loading - Axial strain = 0.38%, $u-u_c$ = 1.82 kg/cm^2

Loading: Undrained monotonic axial compression. Rate of strain = 0.18% per min.

Size: Height 17.6 cm, Diameter 7.3 cm.

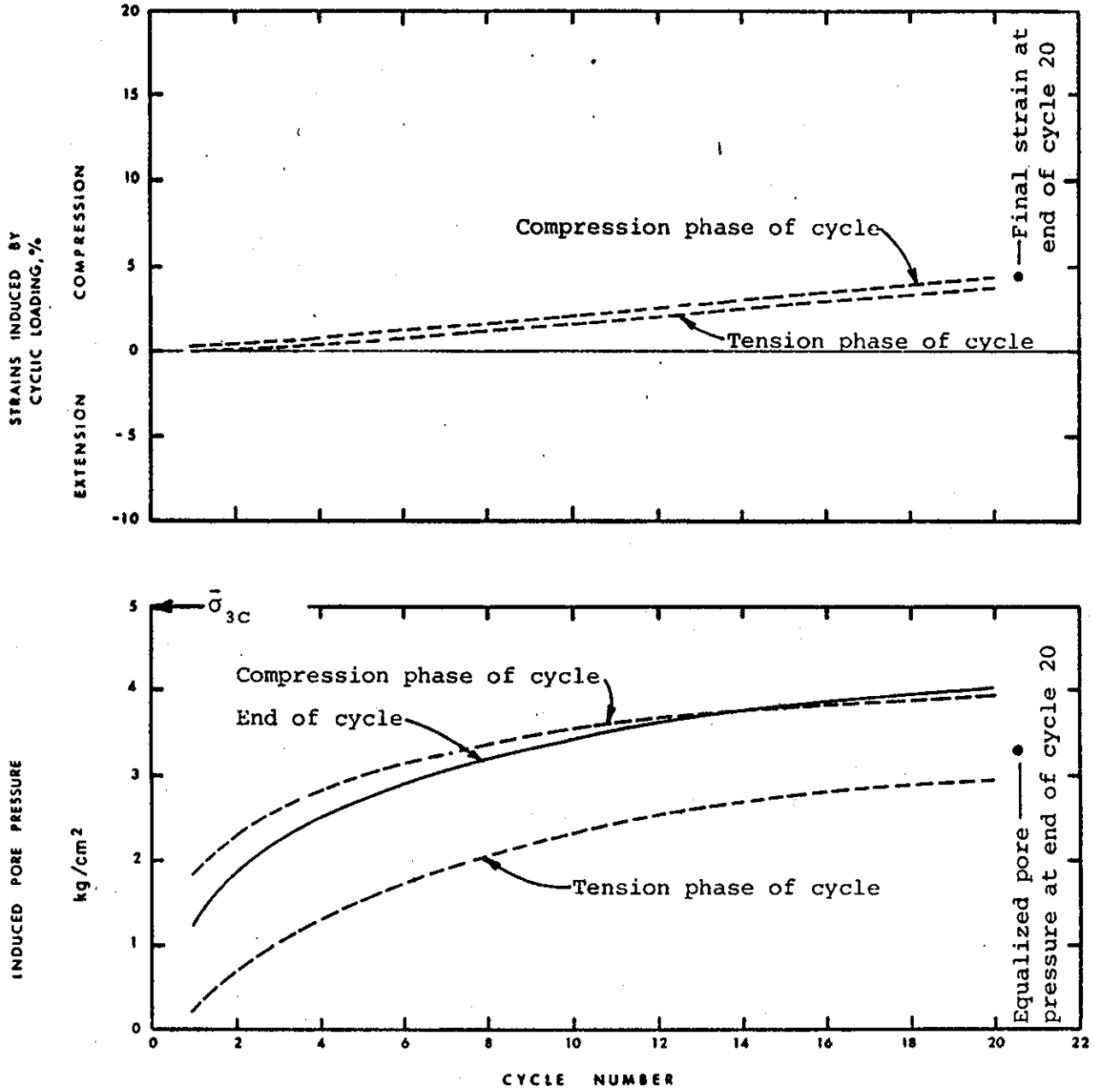
LEGEND:

● End of Consolidation Phase

○ End of Cyclic Loading Phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $\overline{CR}$-$\overline{R}$-8

FIGURE E1-10



Soil: Boring K-2, Sample SP-14A, Depth 116.5 ft.
Subhorizontal layers and lenses of silt and ifne sand. Top ¼ of specimen is slightly coarser grained than bottom 3/4. Mica flakes. Olive brown. Note: Grain size analysis performed on bottom 3/4 of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1c}$=7.50 kg/cm^2, $\bar{\sigma}_{3c}$=5.00 kg/cm^2, u_c=8.00 kg/cm^2, B =0.96. Dry Unit Weight - in tube 84.6 pcf, in cell after consolidation 89.9 pcf. (See Table 4.1).

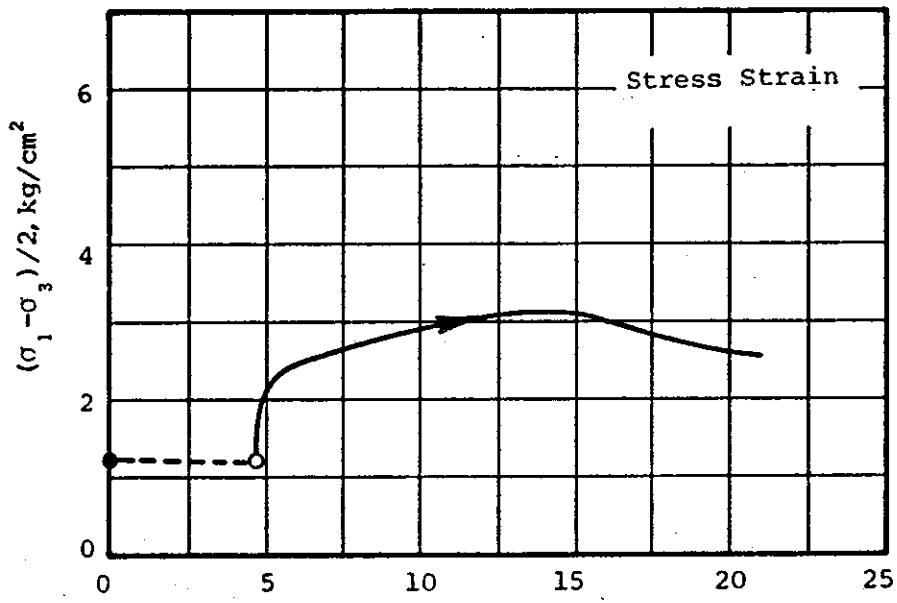
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.22. No principal stress reversal. Frequency of loading 0.4Hz.

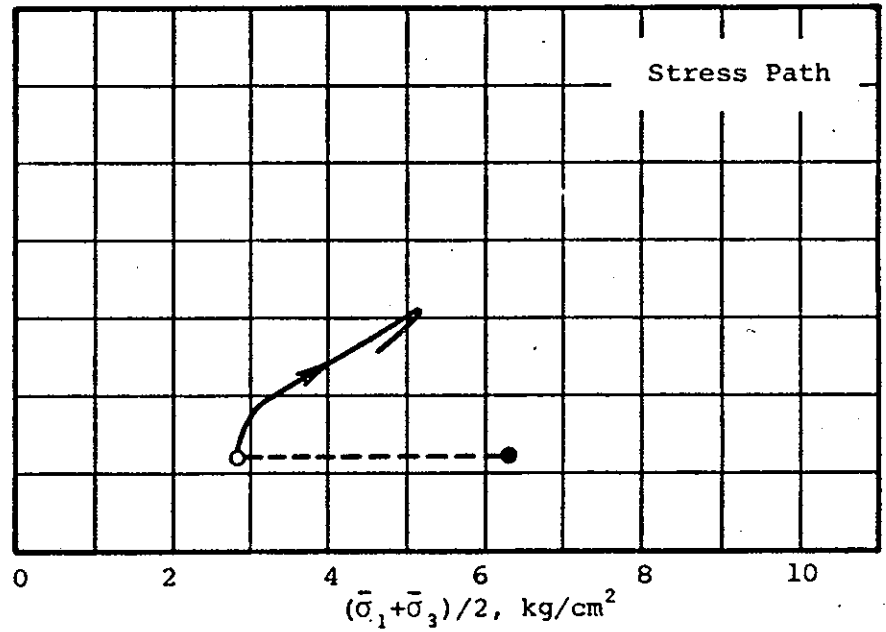
Size: Height 18.7 cm, Diameter 7.3 cm.

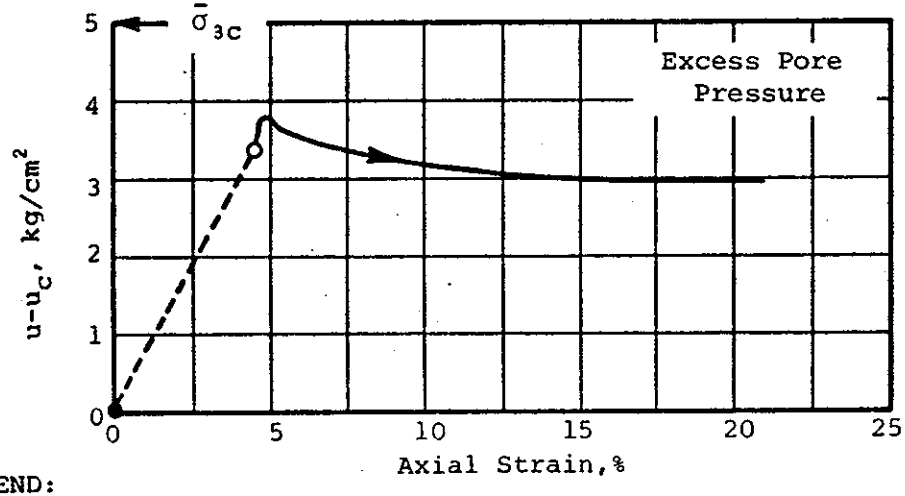
Comments: This $C\bar{R}$ test was followed by an $\bar{R}$ test. See Fig. 4.12 for results of the $\bar{R}$ phase.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-$\bar{R}$-9

FIGURE E1-11







Soil: Boring K-2, Sample SP-14A, Depth 89.1 ft Subhorizontal layers and lenses of silt and fine sand. Top ¼ of specimen is slightly coarser than bottom 3/4. Mica flakes. Olive brown.

Structure: Undisturbed 3-in. tube sample of core material.

State: Before cyclic loading phase - see Fig. 4.11.
After cyclic loading - Axial strain = 4.56%, $u-u_c$=3.37 kg/cm^2

Loading: Undrained monotonic axial compression. Rate of strain = 0.17% per min.

Size: Height 18.7 cm, Diameter 7.3 cm.

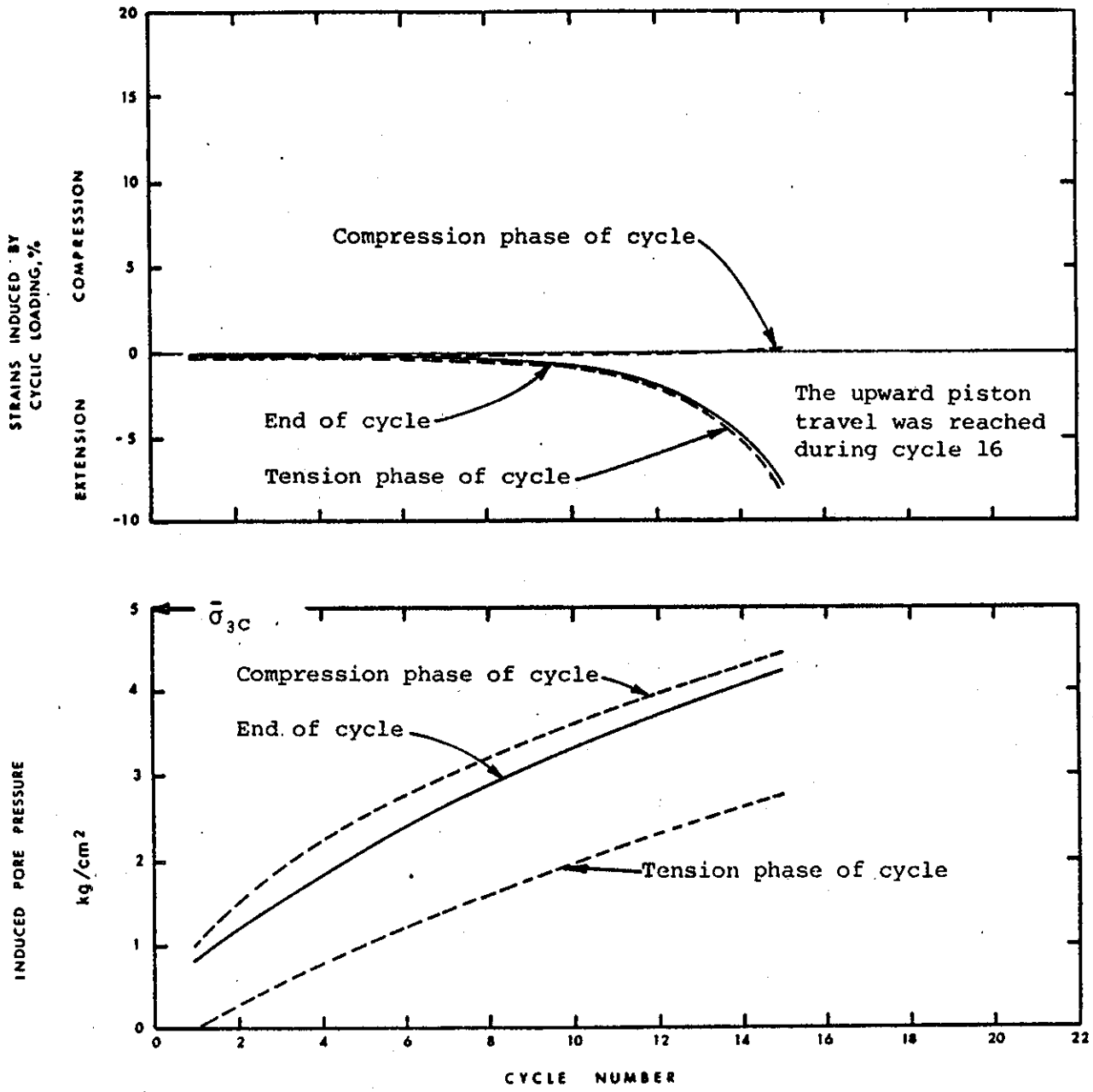
LEGEND:

● End of Consolidation Phase

○ End of Cyclic Loading Phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $\bar{CR}$-$\bar{R}$-9

FIGURE E1-12



Soil: Boring K-2, Sample SP-10C, Depth 89.1 ft.
Contorted layers and lenses of silt and fine sand. Mica flakes. Olive brown. Note: Grain size analysis performed on a mixture of the sample.

Structure: Undisturbed 3-in. tube sample of core material.

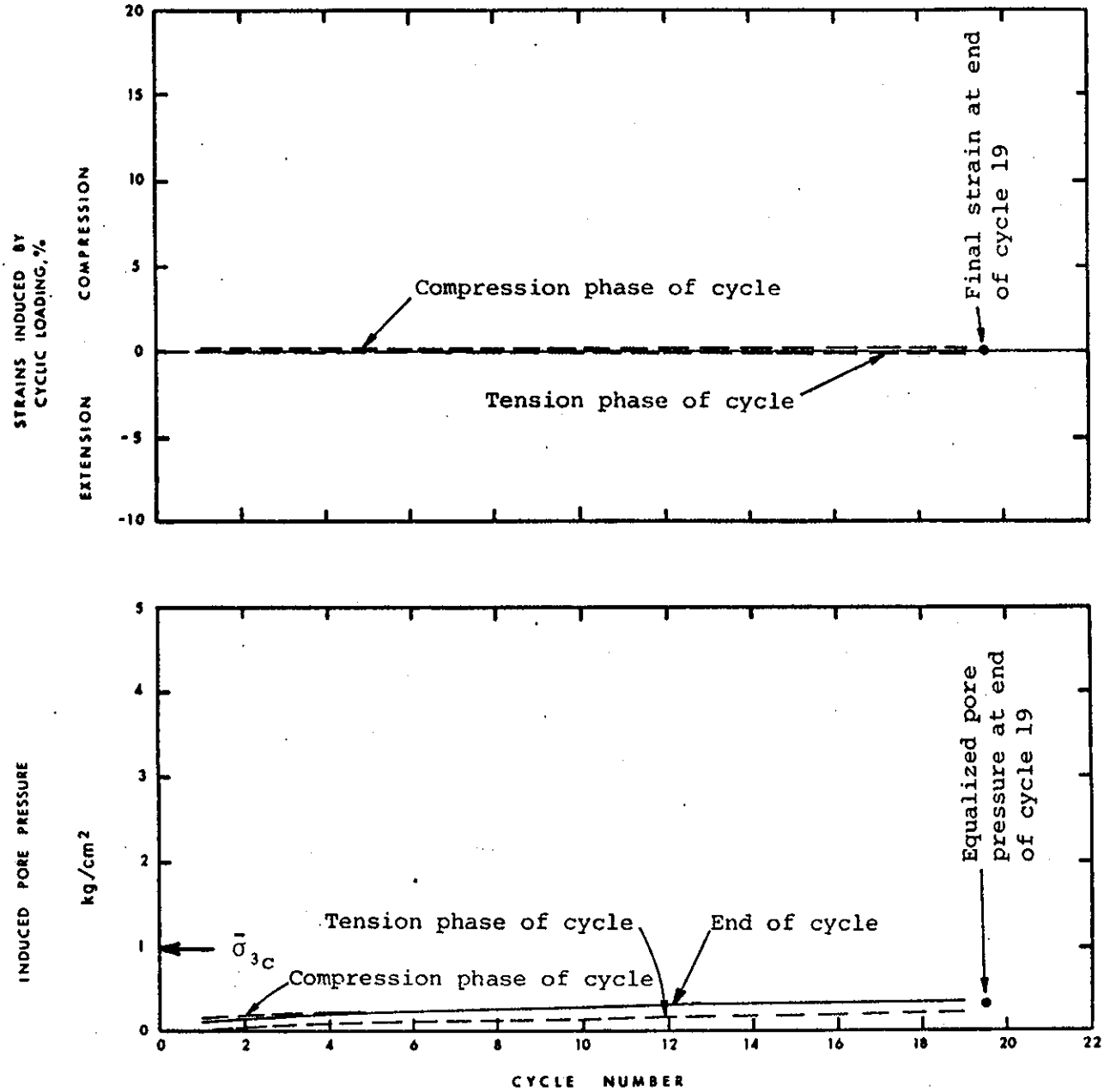
State: $\bar{\sigma}_{1c}$=5.00 kg/cm^2, $\bar{\sigma}_{3c}$=5.00 kg/cm^2, u_c=8.00 kg/cm^2, B =0.96. Dry Unit Weight - in tube 84.9 pcf, in cell after consolidation 91.0 pcf. (See Table 4.1).

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.15. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 18.0 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-10

FIGURE E1-13



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

State: $\bar{\sigma}_{1c}$=1.50 kg/cm^2, $\bar{\sigma}_{3c}$=1.00 kg/cm^2, u_c=9.00 kg/cm^2, B =0.87. Dry Unit Weight in cell: initial 131.0 pcf, after consolidation 132.4 pcf.

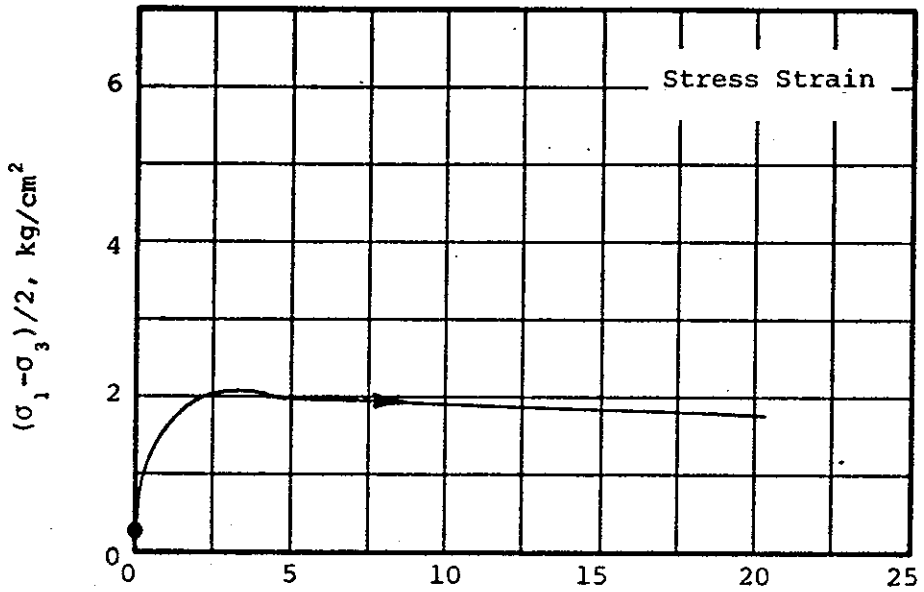
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.29. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

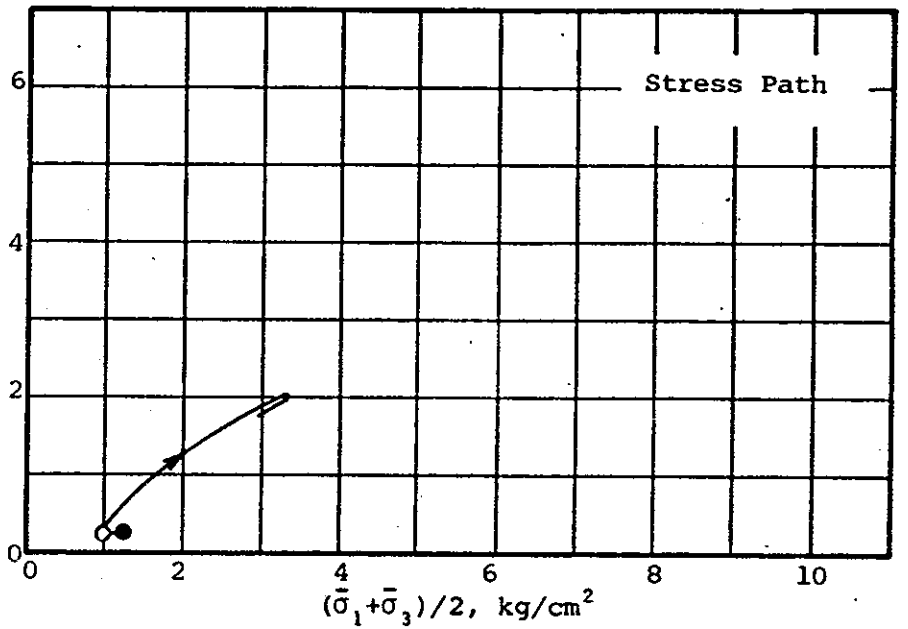
Size: Height 17.7 cm, Diameter 7.3 cm.

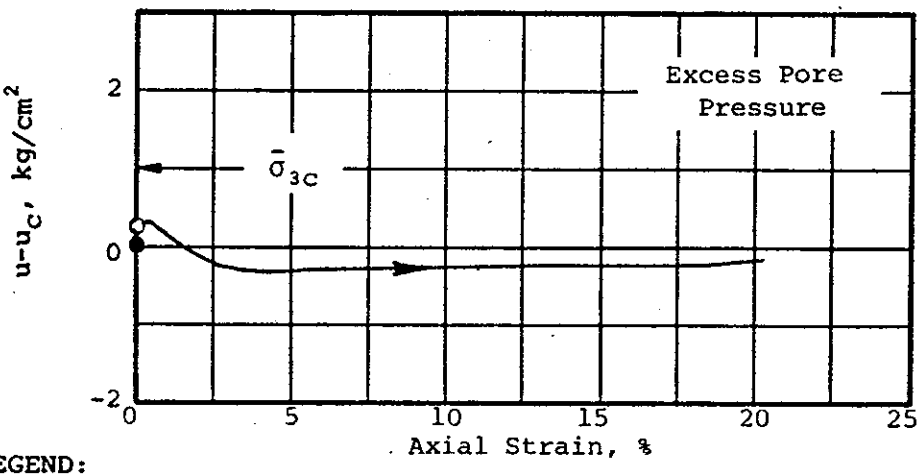
Comments: This $C\bar{R}$ test was followed by an $\bar{R}$ test. See Fig. 4.15 for results of the $\bar{R}$ phase.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-$\bar{R}$-11

FIGURE E1-14







Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Boring K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

State: Before cyclic loading phase see Fig. 4.14
After cyclic loading - Axial strain = 0.05%, $u-u_c$ = 0.26 kg/cm^2.

Loading: Undrained monotonic axial compression. Rate of strain = 0.17% per min.

Size: Height 17.7 cm, Diameter 7.4 cm.

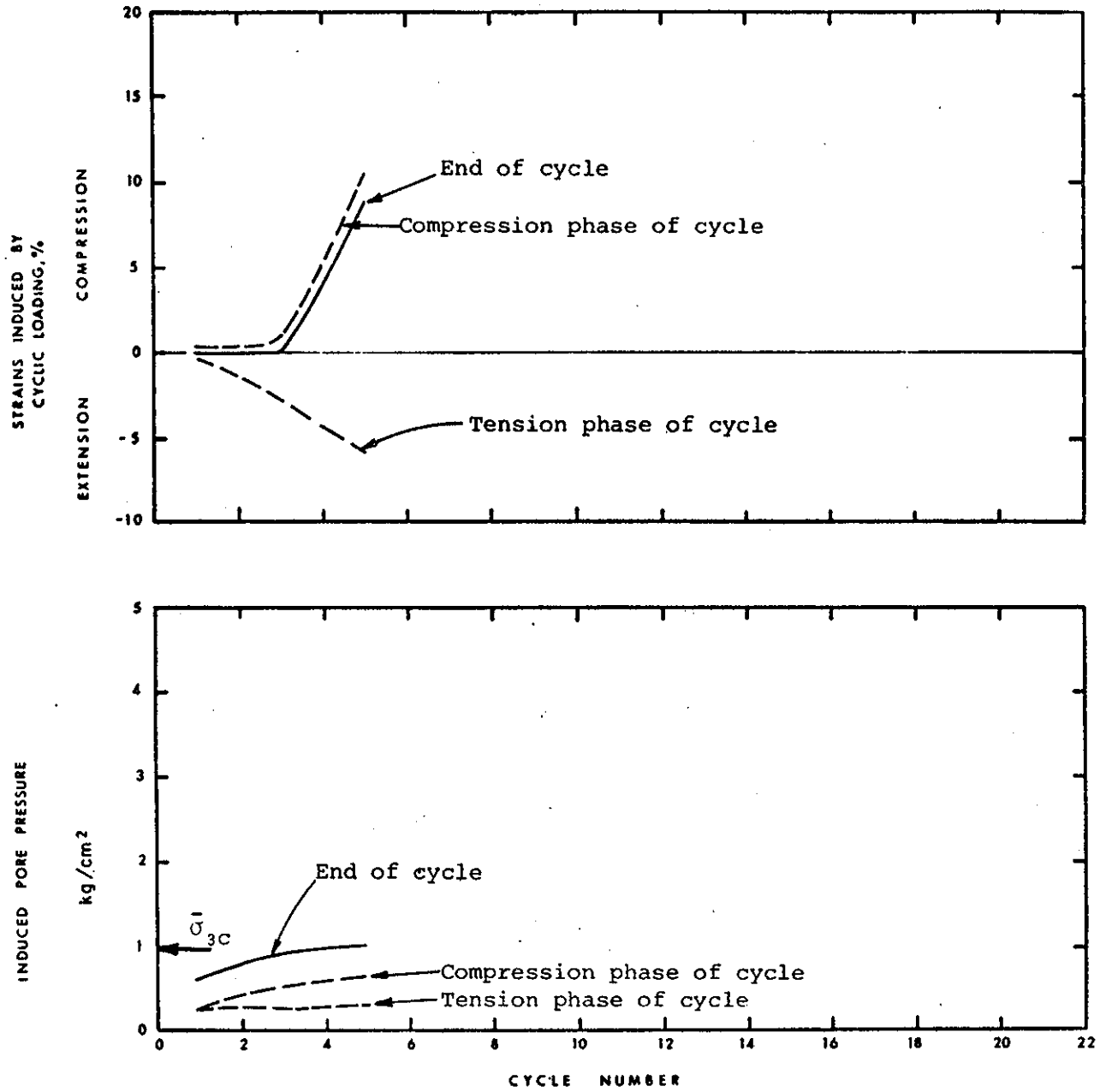
LEGEND:

● End of consolidation phase

○ End of cyclic loading phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $\overline{CR}$-$\bar{R}$-11

FIGURE E1-15



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

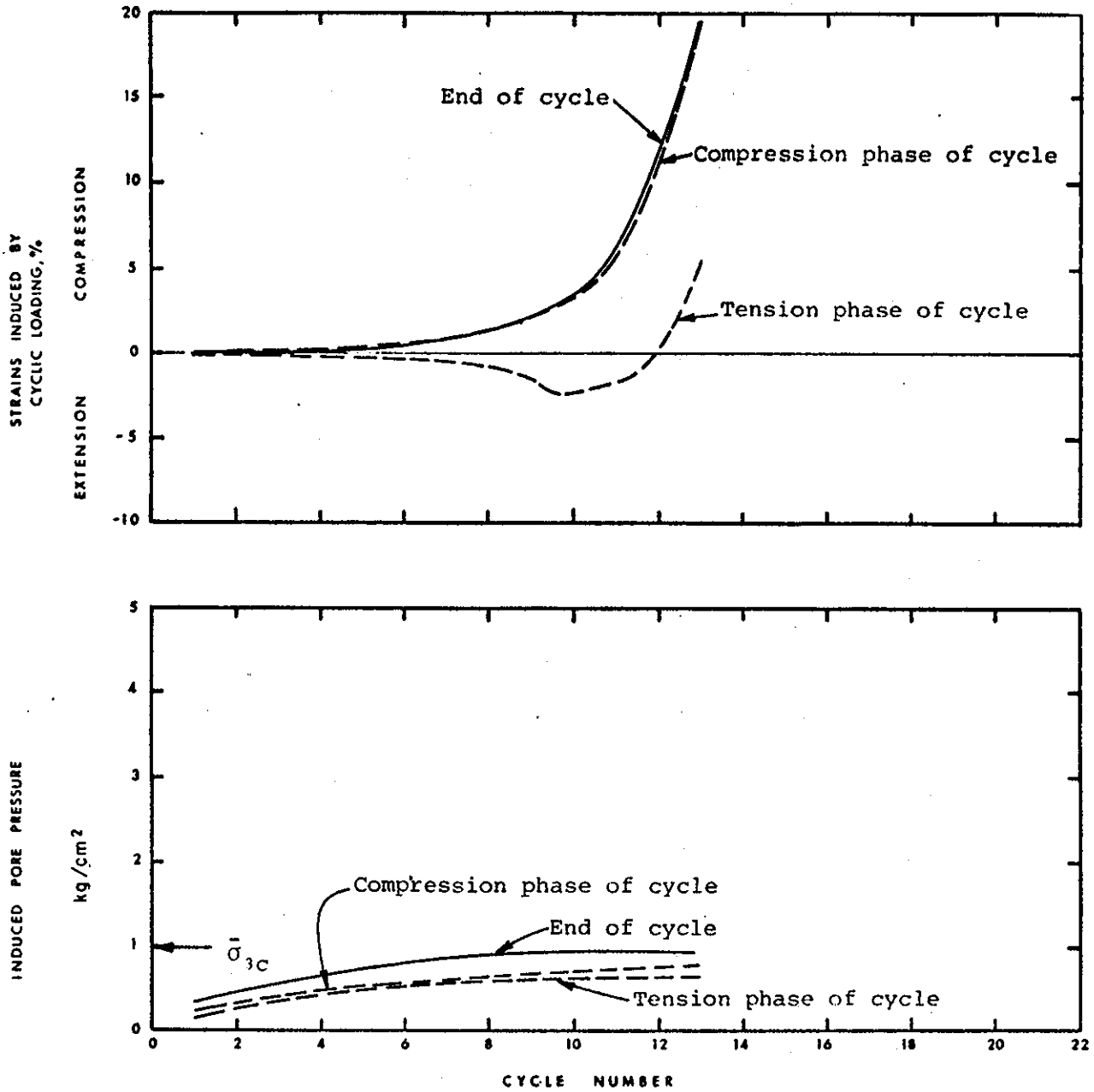
State: $\bar{\sigma}_{1c}$=1.49 kg/cm², $\bar{\sigma}_{3c}$=1.00 kg/cm², u_c=9.00 kg/cm², B =0.95. Dry Unit Weight in cell: initial 128.2 pcf, after consolidation 129.5 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.50. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-12

FIGURE E1-16



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

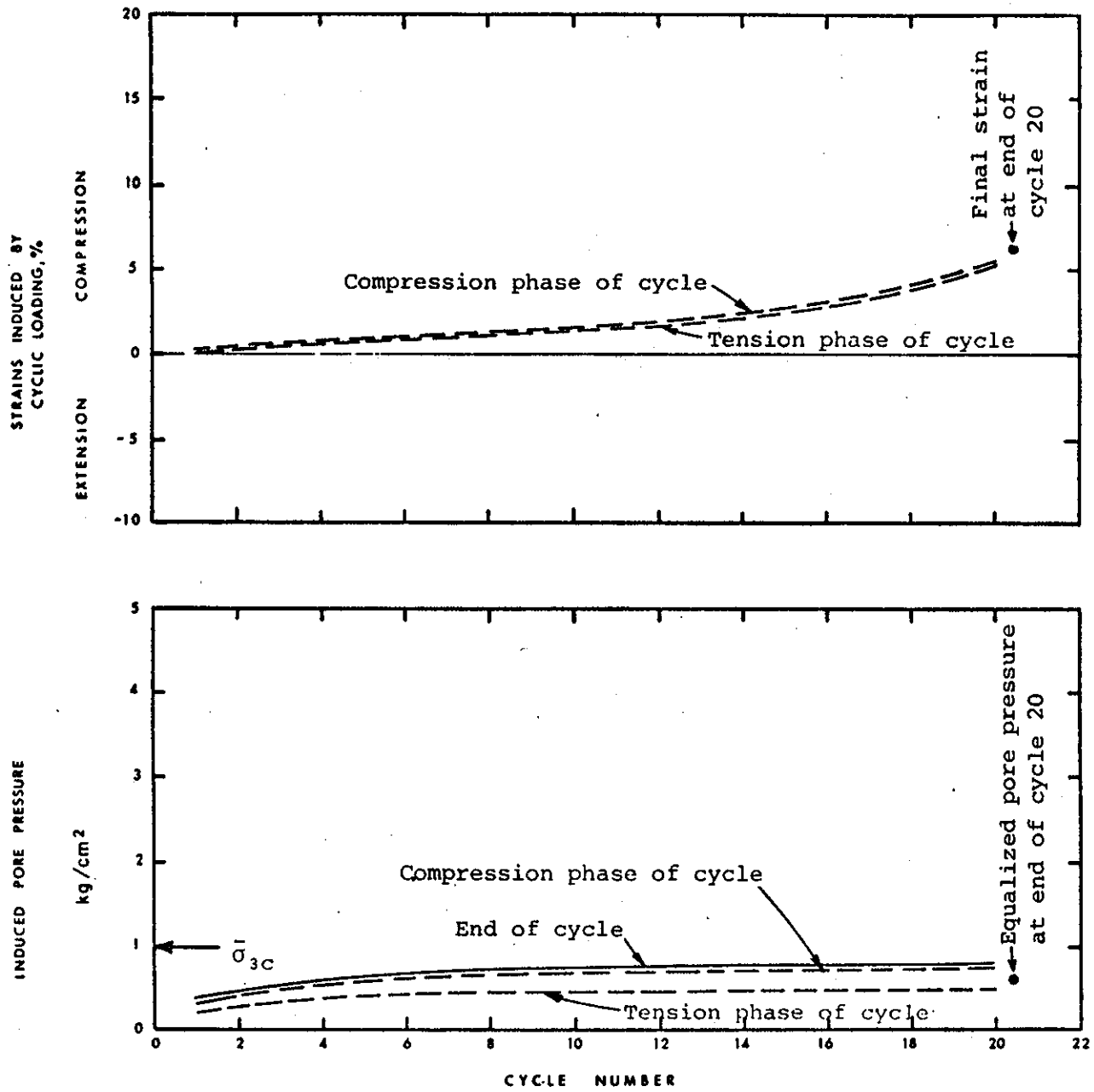
State: $\bar{\sigma}_{1c}$=1.50 kg/cm², $\bar{\sigma}_{3c}$=1.00 kg/cm², u_c=9.00 kg/cm², B =0.95. Dry Unit Weight in cell: initial 128.8 pcf, after consolidation 130.0 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.40. Cyclic loading produced principal stress reversal. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-13

FIGURE E1-17



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

State: $\bar{\sigma}_{1c}$=2.00 kg/cm², $\bar{\sigma}_{3c}$=1.00 kg/cm², u_c=9.00 kg/cm², B =0.94. Dry Unit Weight in cell: initial 128.1 pcf, after consolidation 130.3 pcf.

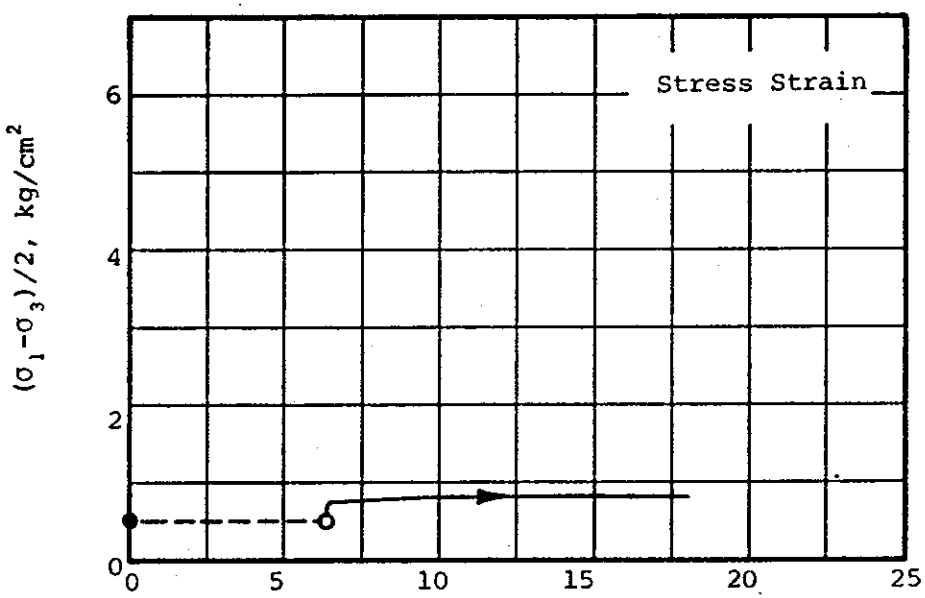
Loading: Undrained cyclic axial compression. Cyclic stress ration $\sigma_d/2\bar{\sigma}_{3c}$= 0.42. No reversal of principal stresses. Frequency of loading 0.4Hz.

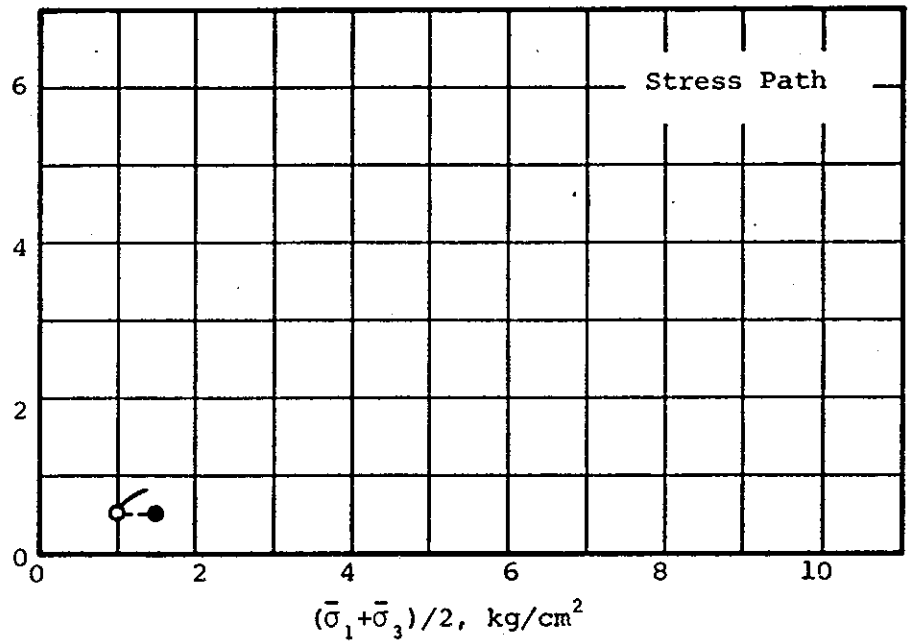
Size: Height 17.6 cm, Diameter 7.3 cm.

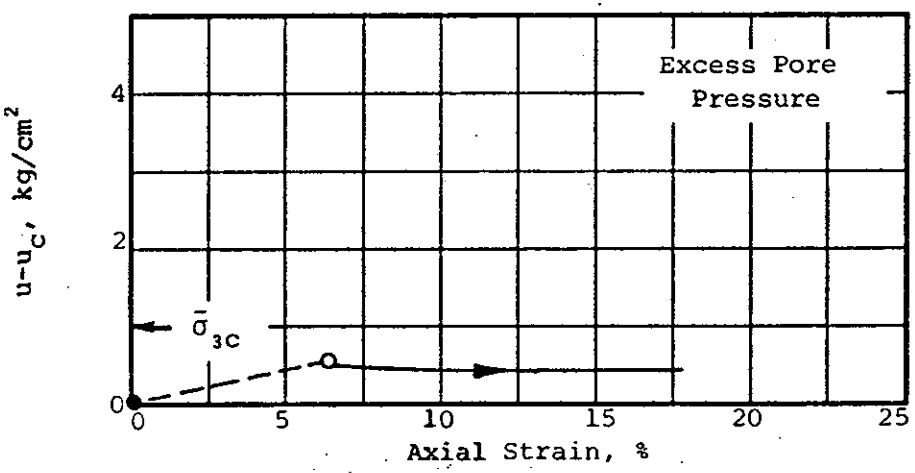
Comments: This $C\bar{R}$ test was followed by an $\bar{R}$ test. See Fig. 4.19 for results of the $\bar{R}$ phase.

CONSOLIDATED UNDRAINED CYCLIC TRAXIAL TEST $C\bar{R}$-$\bar{R}$-14

FIGURE E1-18







Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Boring K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

State: Before cyclic loading phase - see Fig. 4.18
After cyclic loading - Axial strain = 0.05%, $u-u_c$ = 0.56 kg/cm^2.

Loading: Undrained monotonic axial compression. Rate of strain = 0.17% per min.

Size: Height 17.6 cm, Diameter 7.4 cm.

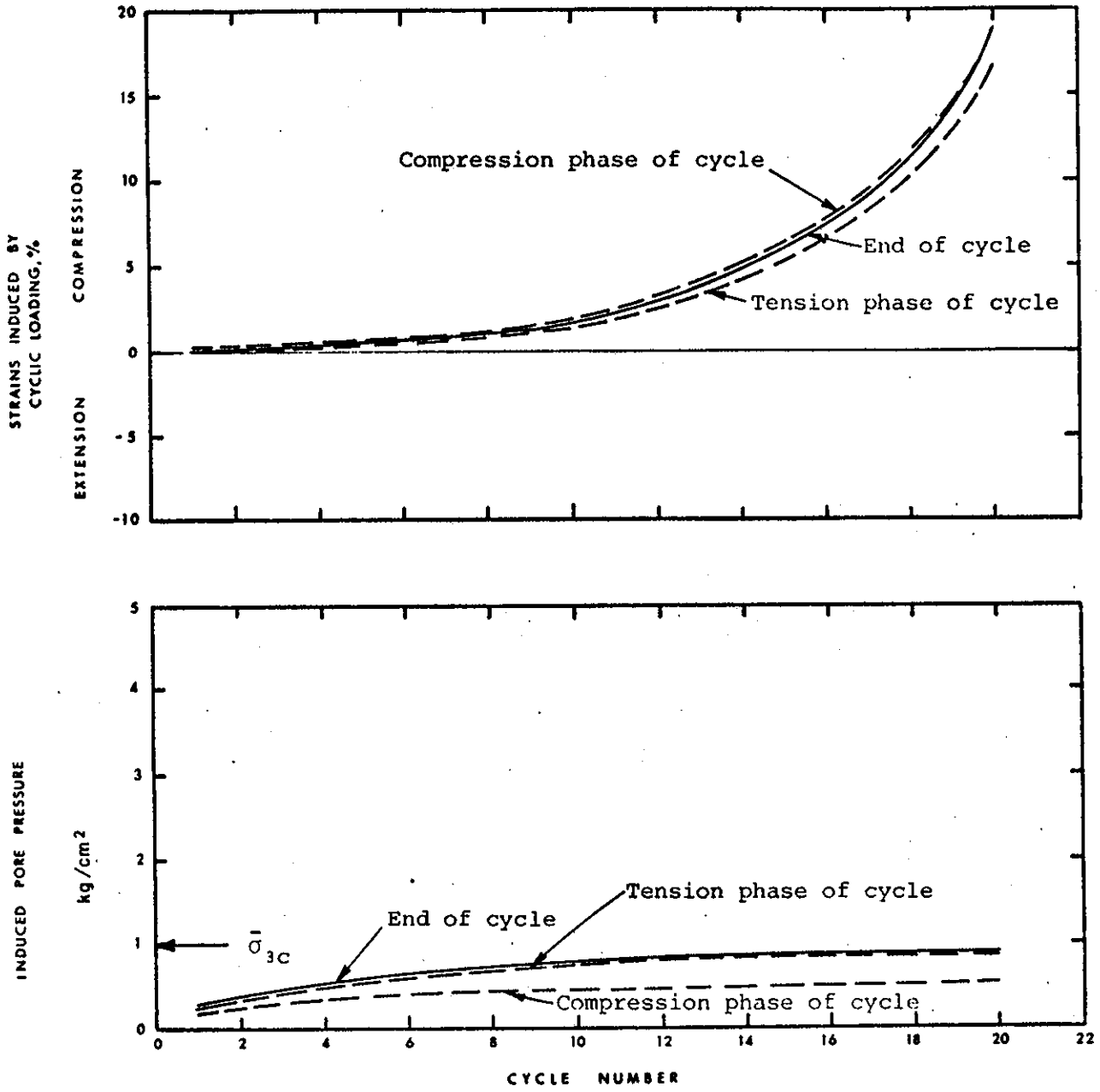
LEGEND:

● End of consolidation phase

○ End of cyclic loading phase

MONOTONIC LOADING PHASE OF CONSOLIDATED UNDRAINED TEST $C\bar{R}$-$\bar{R}$-14

FIGURE E1-19



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

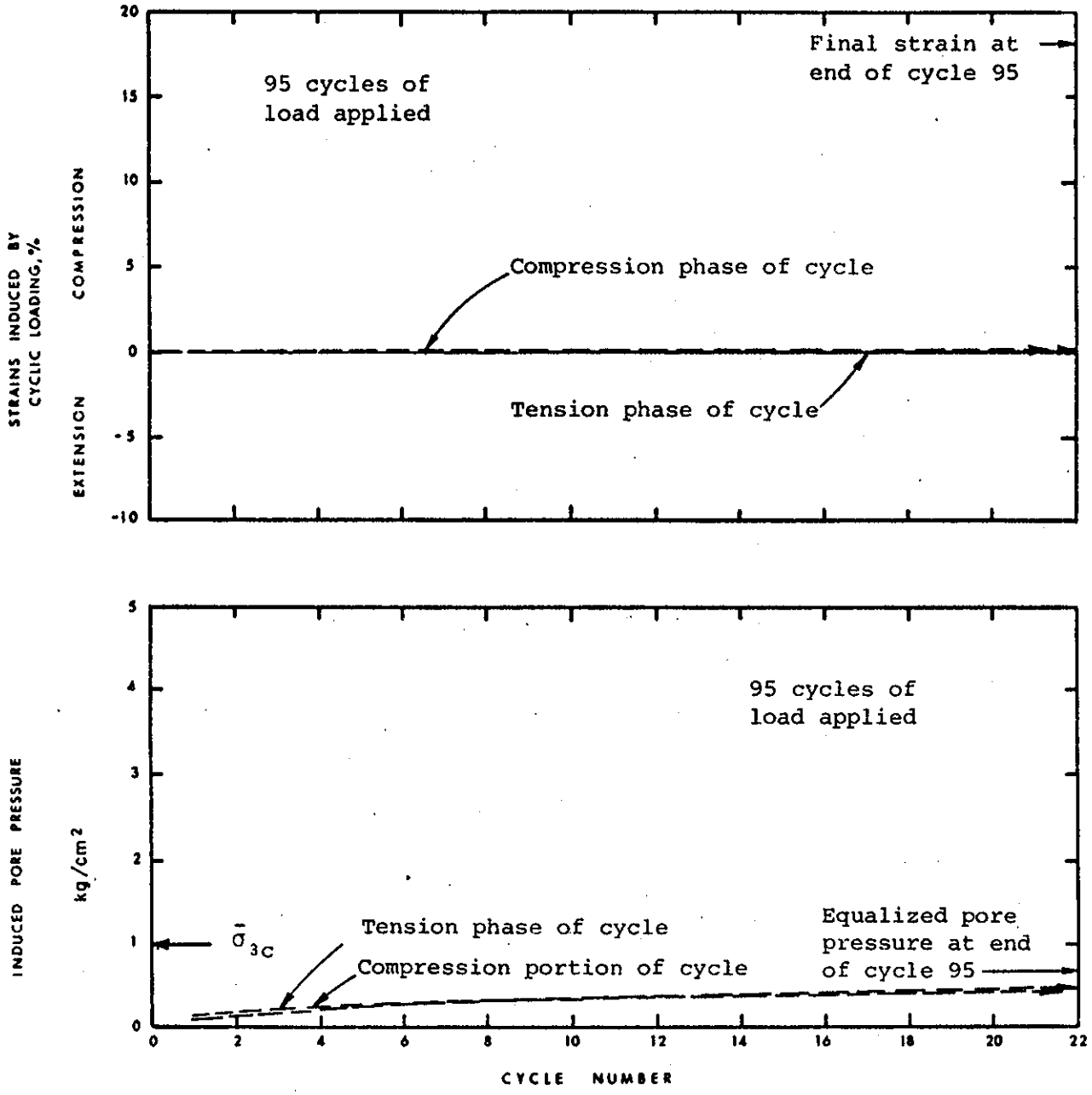
State: $\bar{\sigma}_{1c}$=2.00 kg/cm², $\bar{\sigma}_{3c}$=1.00 kg/cm², u_c=9.00 kg/cm², B =0.99. Dry Unit Weight in cell: initial 129.6 pcf, after consolidation 131.4 pcf

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.50. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.5 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-15

FIGURE E1-20



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

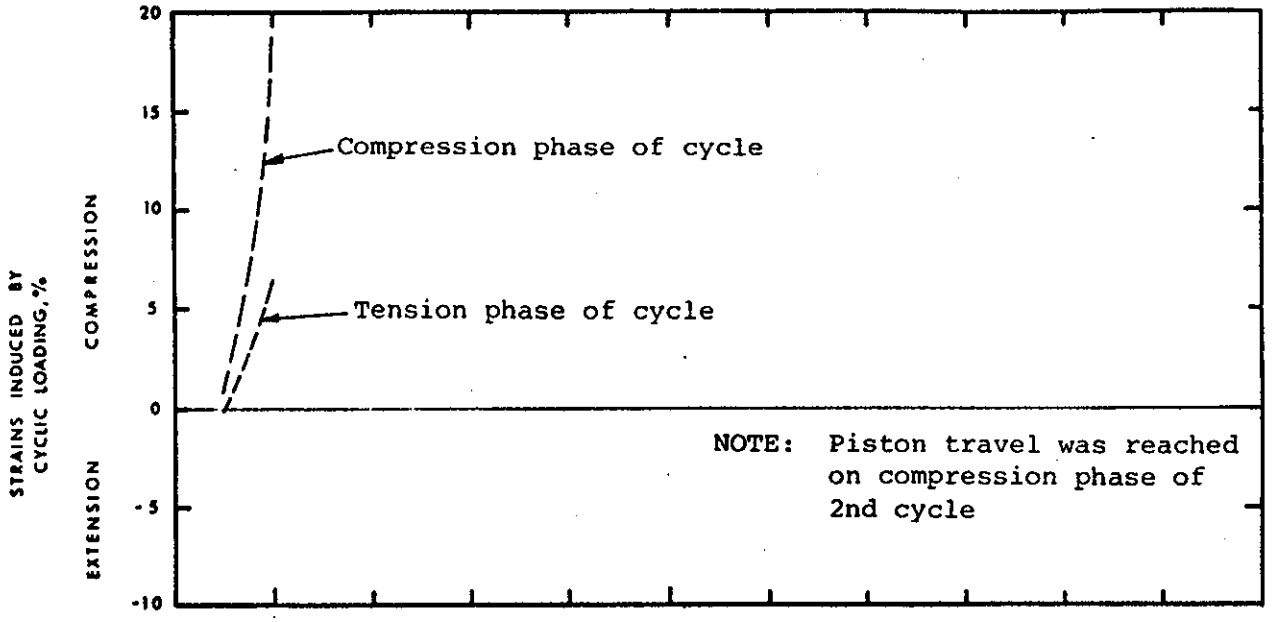
State: $\bar{\sigma}_{1c}$=2.00 kg/cm^2, $\bar{\sigma}_{3c}$=1.00 kg/cm^2, u_c=9.00 kg/cm^2, B =0.97. Dry Unit Weight in cell: initial 129.1 pcf, after consolidation 131.0 pcf

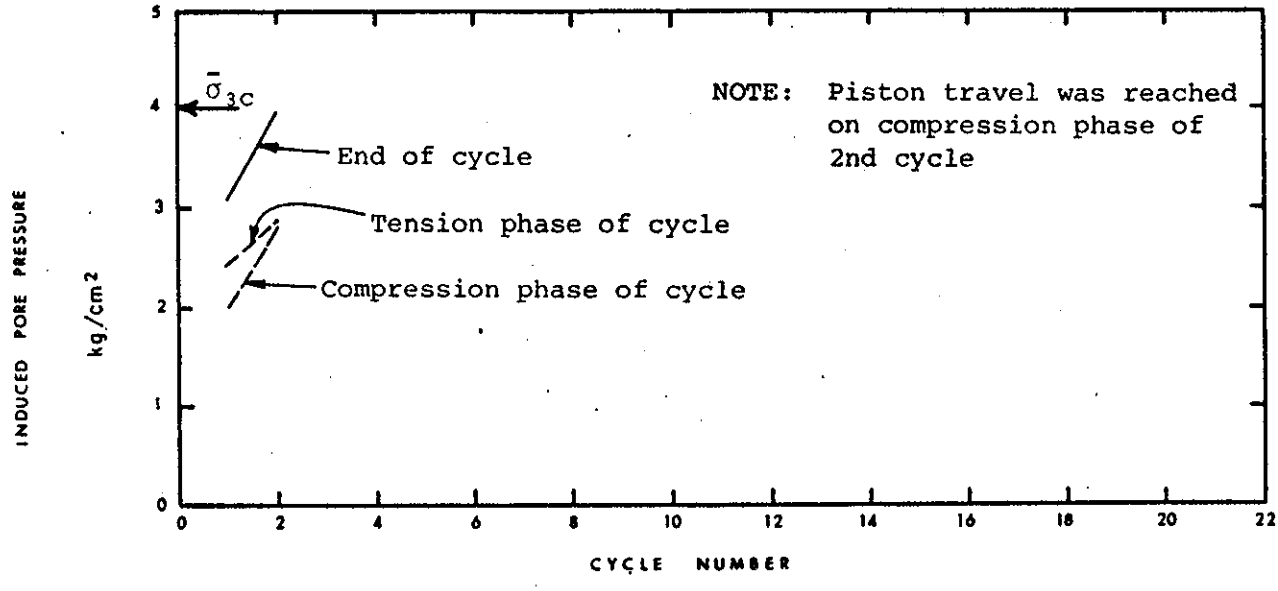
Loading: Undrained cyclic axial compression. Cyclic stress ration $\sigma_d/2\bar{\sigma}_{3c}$= 0.34. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST
CR-16

FIGURE E1-21





Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

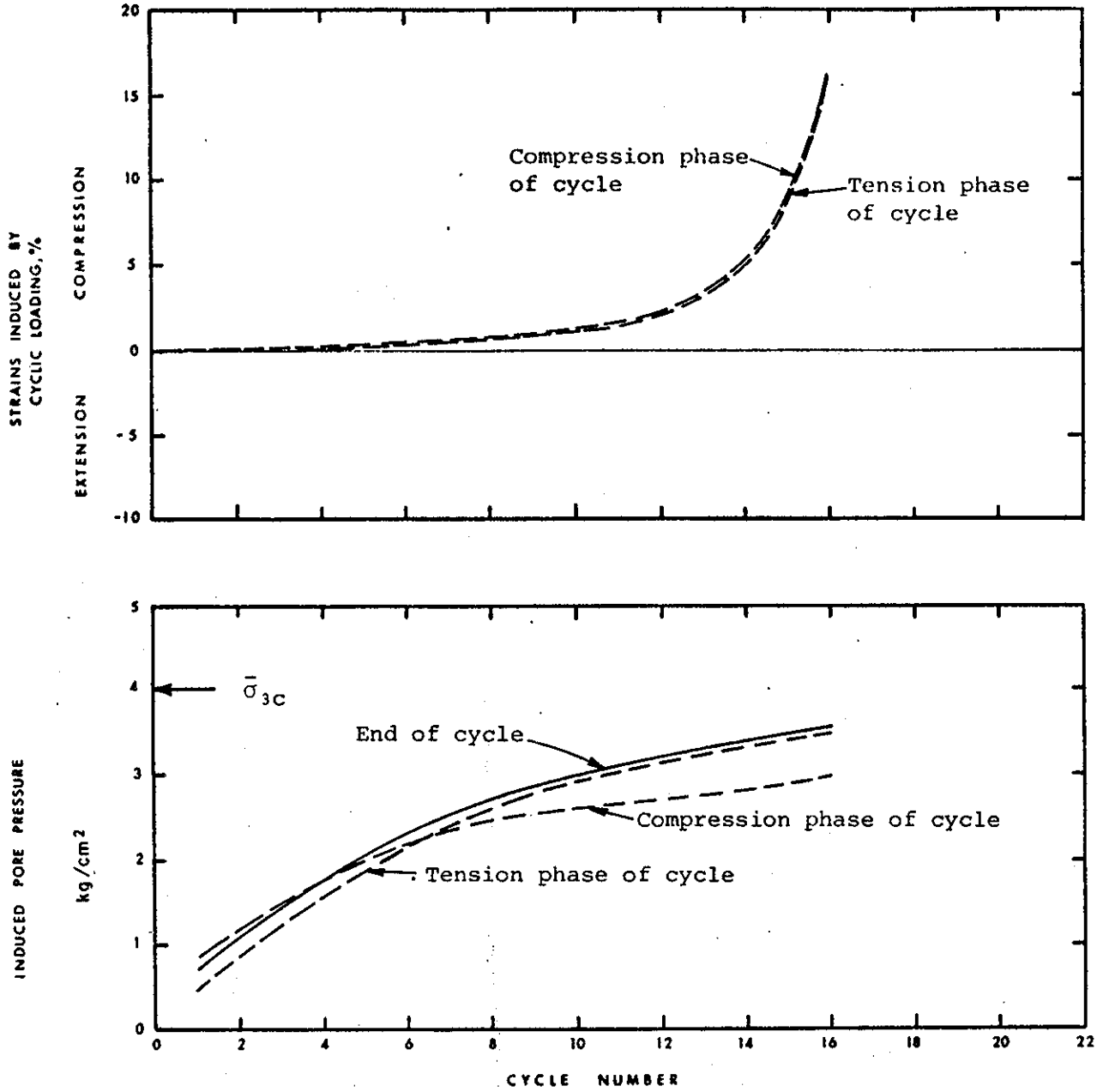
State: $\bar{\sigma}_{1c}$=6.03 kg/cm^2, $\bar{\sigma}_{3c}$=4.00 kg/cm^2, u_c=9.00 kg/cm^2, B =0.96. Dry Unit Weight in cell: initial 127.2 pcf, after consolidation 133.7 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.37. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.7 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST CR-17

FIGURE E1-22



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Boring K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

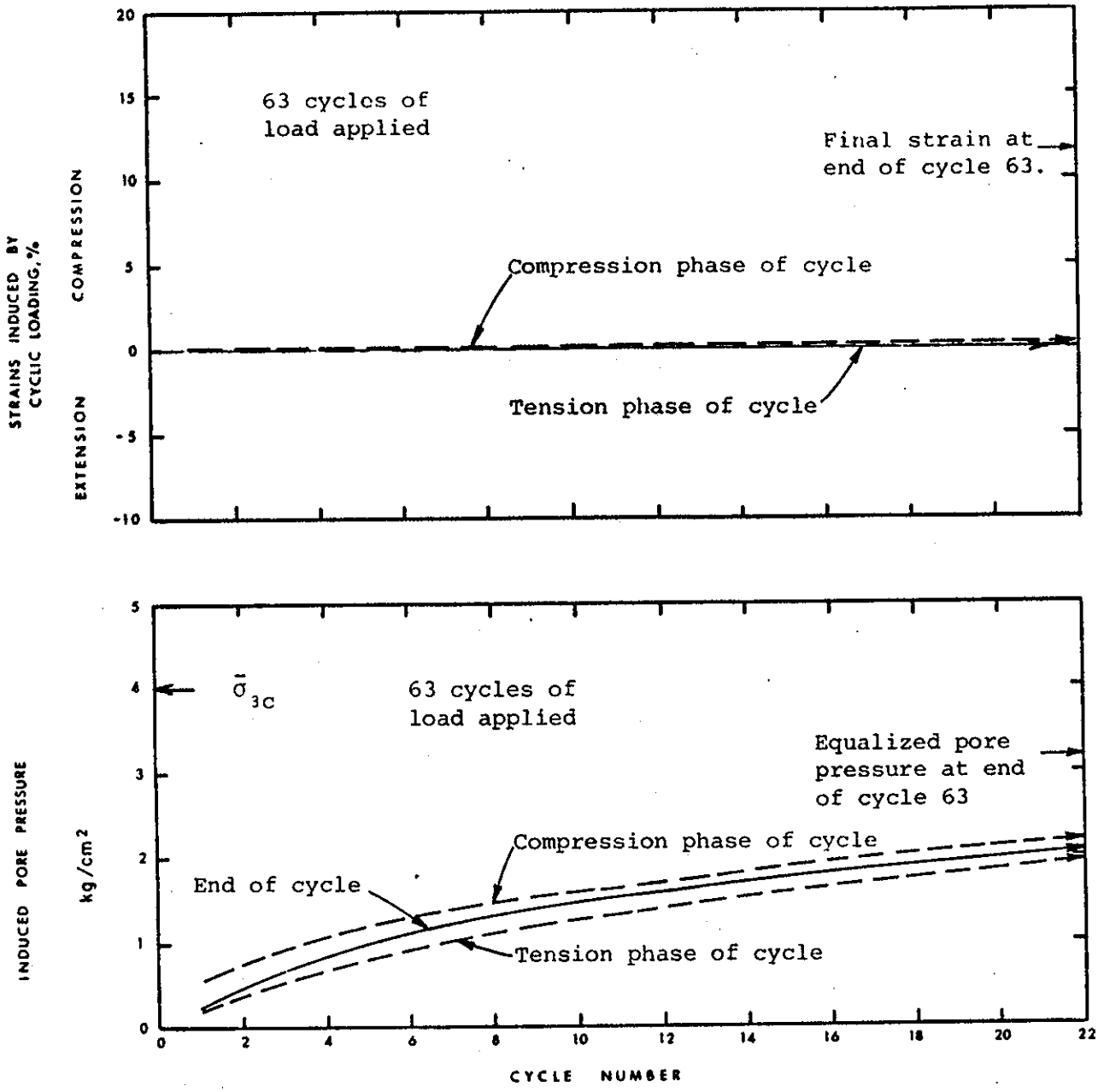
State: $\bar{\sigma}_{1c}$=6.03 kg/cm², $\bar{\sigma}_{3c}$=4.00 kg/cm², u_c=9.00 kg/cm², B =0.99. Dry Unit Weight in cell: initial 125.8 pcf, after consolidation 131.6 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.20. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-18

FIGURE E1-23



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

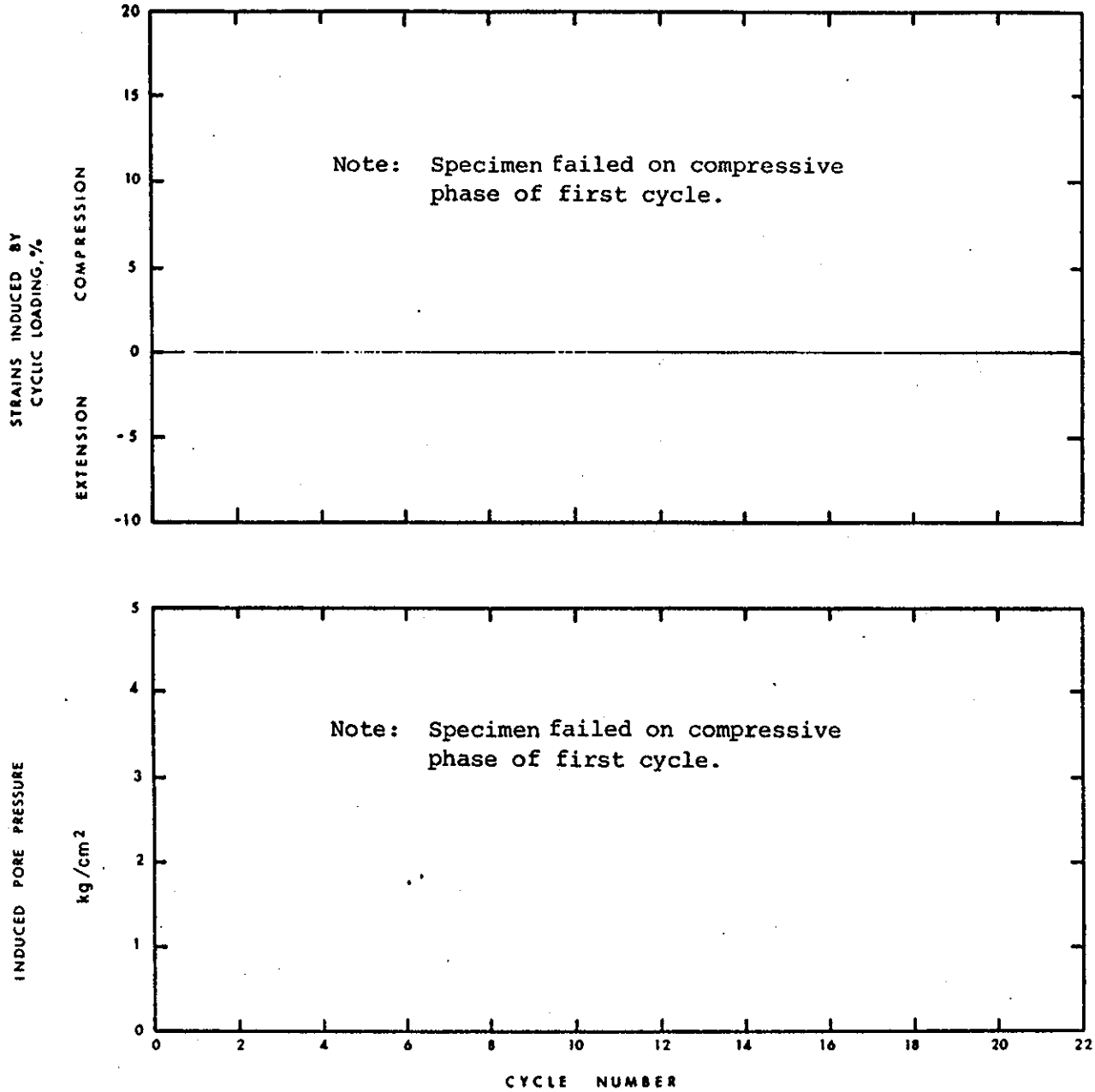
State: $\bar{\sigma}_{1c}$=6.03 kg/cm^2 , $\bar{\sigma}_{3c}$=4.00 kg/cm^2, u_c=9.00 kg/cm^2, B =0.96. Dry Unit Weight in cell: initial 125.4 pcf, after consolidation 132.5 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$= 0.14. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $\bar{CR}$-19

FIGURE E1-24



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

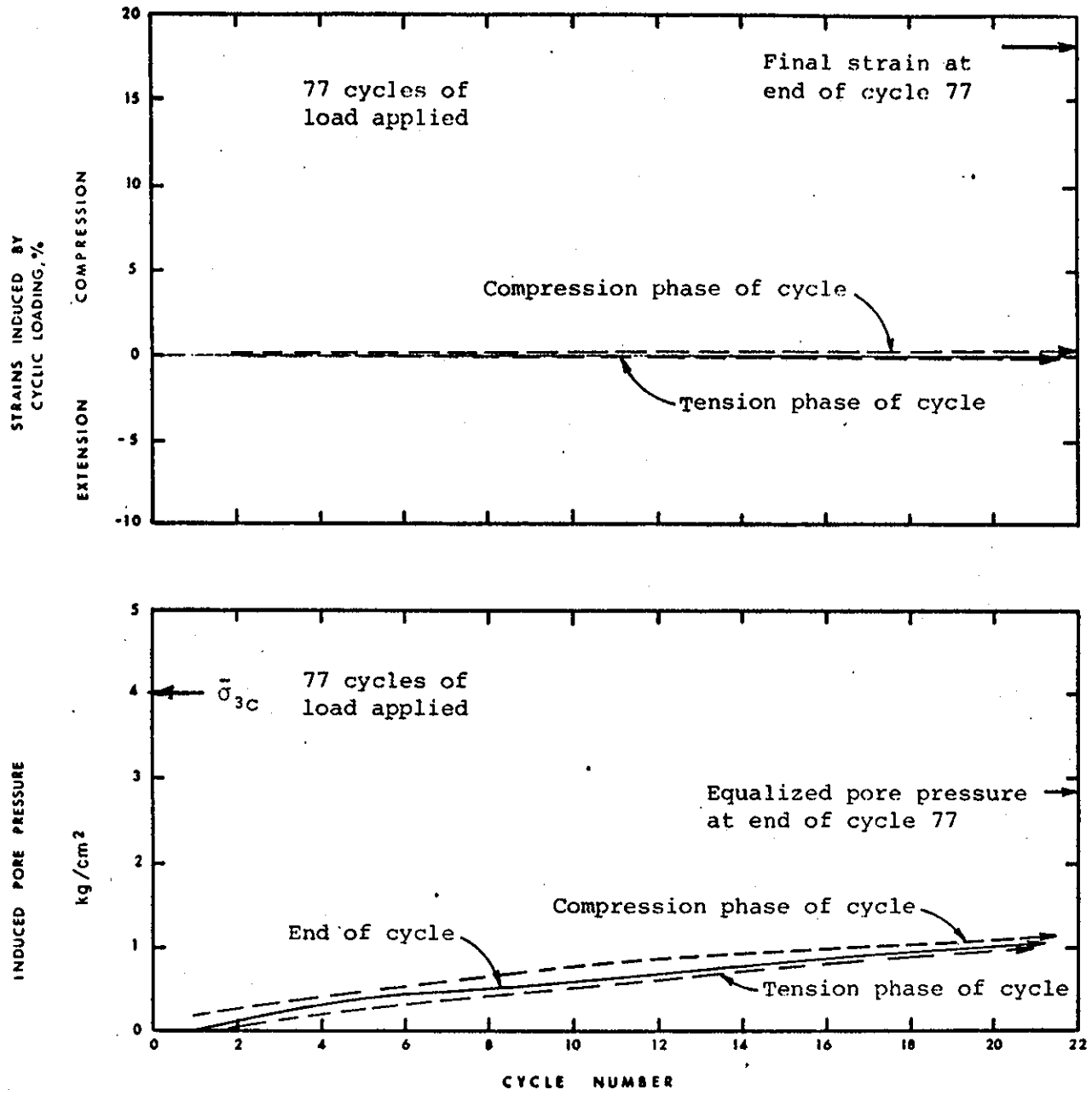
State: $\bar{\sigma}_{1c}$=8.00 kg/cm^2, $\bar{\sigma}_{3c}$=4.00 kg/cm^2, u_c=9.00 kg/cm^2, B =0.99. Dry Unit Weight in cell: initial 125.3 pcf, after consolidation 133.4 pcf.

Loading: Undrained cyclic axial compression. Maximum applied $\sigma_d/2\bar{\sigma}_{3c}$=0.25. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST $C\bar{R}$-20

FIGURE E1-25



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

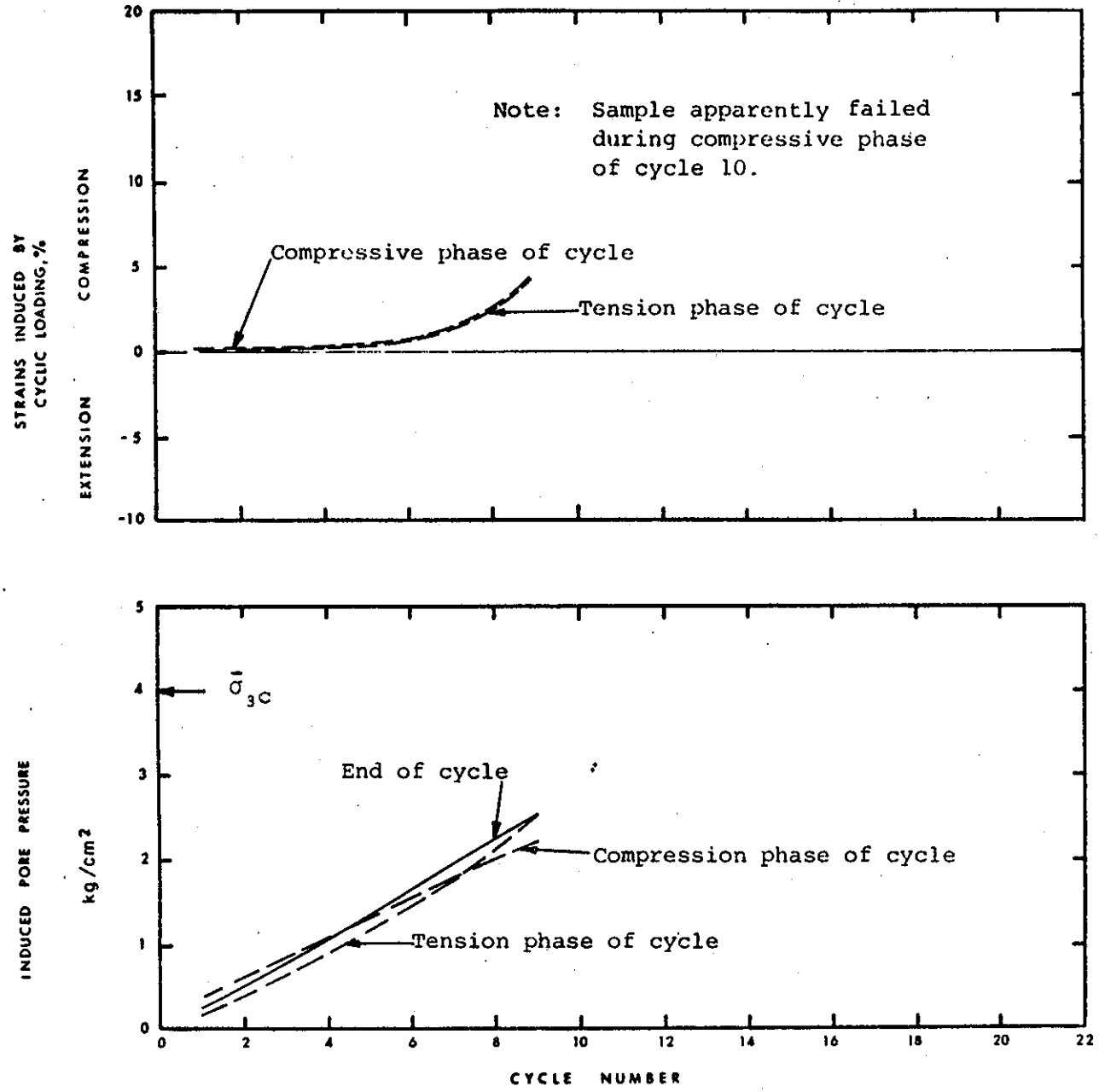
State: $\bar{\sigma}_{1c}$=8.00 kg/cm², $\bar{\sigma}_{3c}$=4.00 kg/cm², u_c=9.00 kg/cm², B=1.00. Dry Unit Weight in cell: initial 126.1 pcf, after consolidation 132.9 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$=0.10. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST
$C\bar{R}$-21

FIGURE E1-26



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

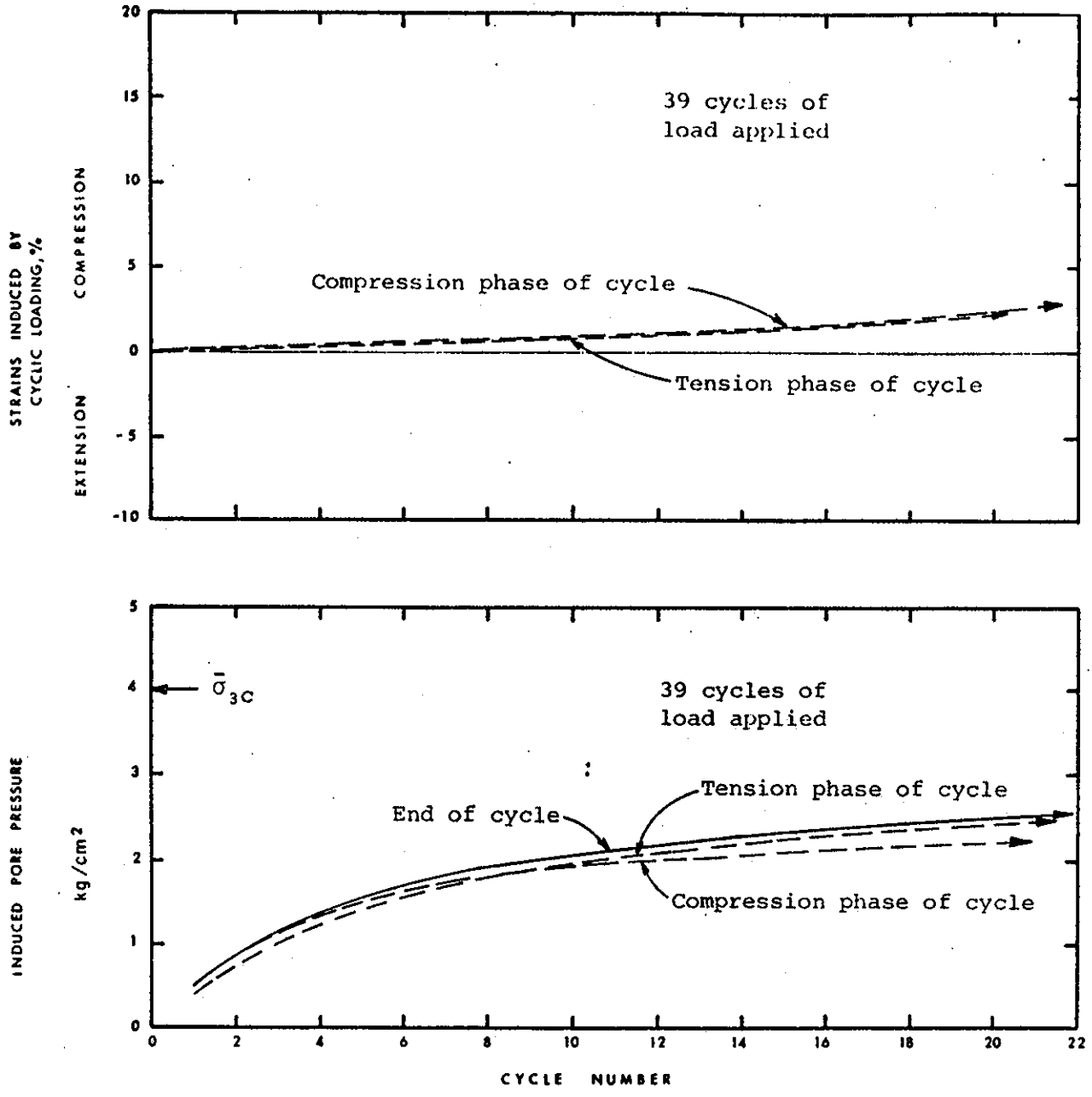
State: $\bar{\sigma}_{1c}$=8.01 kg/cm², $\bar{\sigma}_{3c}$=4.00 kg/cm², u_c=9.00 kg/cm², B =0.97. Dry Unit Weight in cell: initial 124.7 pcf, after consolidation 132.0 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$=0.15. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.3 cm.

CONSOLIDATED UNDRAINED
CYCLIC TRIAXIAL TEST
$C\bar{R}$-22

FIGURE E1-27



Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired unit weight.

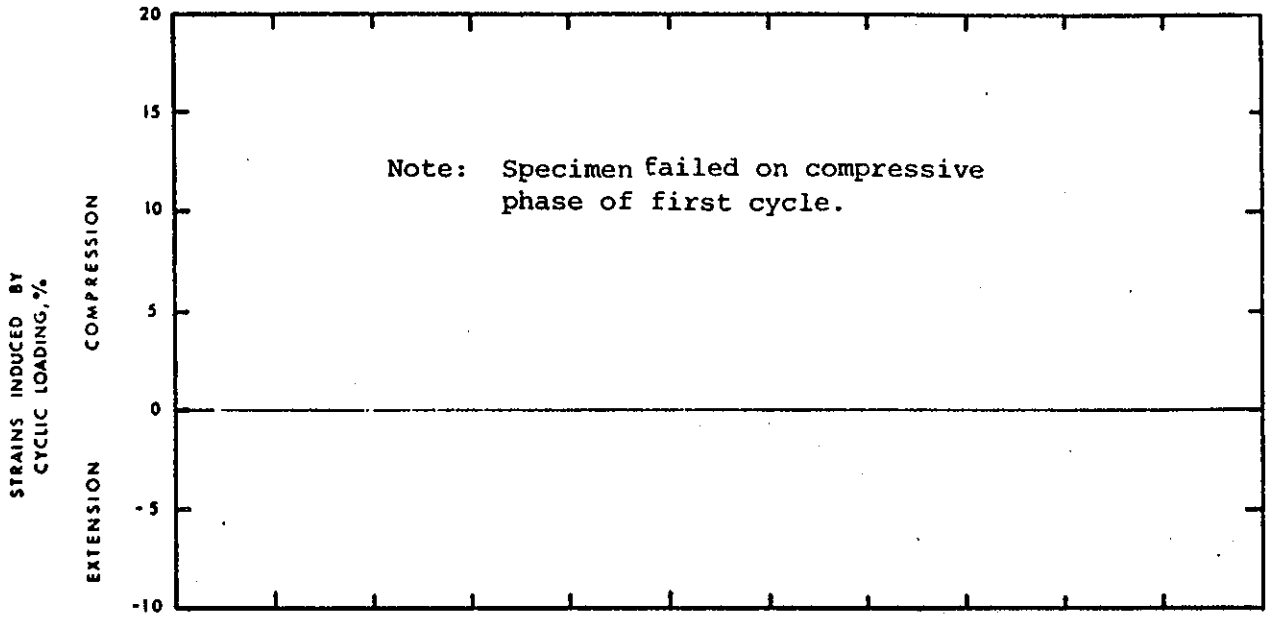
State: $\bar{\sigma}_{1c}$=8.03 kg/cm², $\bar{\sigma}_{3c}$=4.00 kg/cm², u_c=9.00 kg/cm², B=0.97. Dry Unit Weight in cell: initial 129.5 pcf, after consolidation 135.2 pcf.

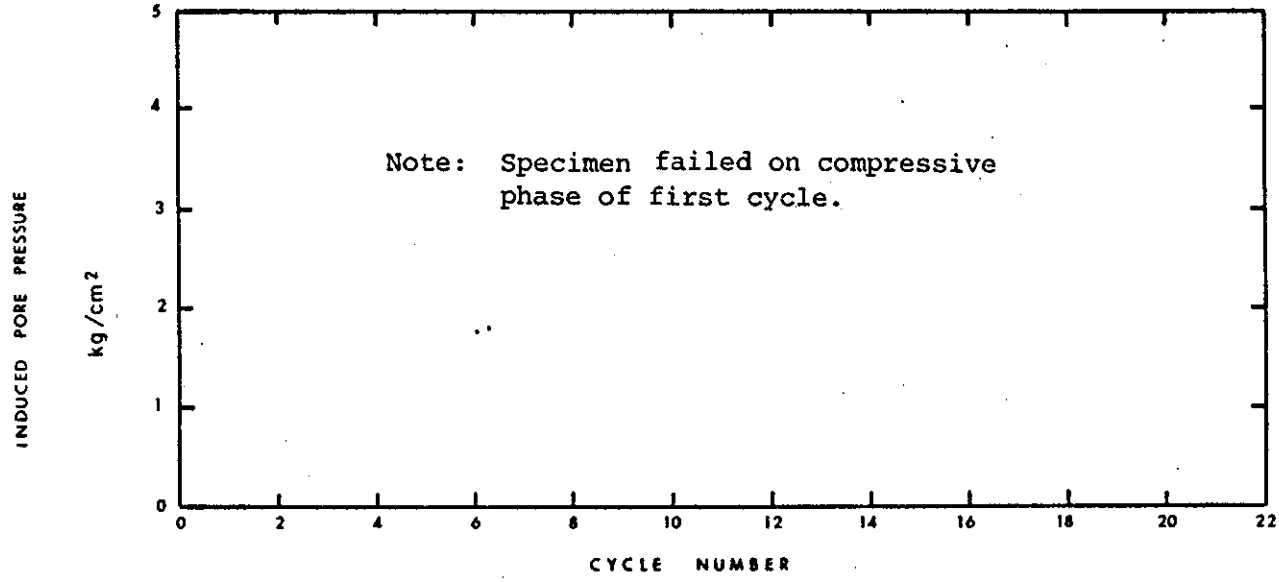
Loading: Undrained cyclic axial compression. Cyclic stress ratio $\sigma_d/2\bar{\sigma}_{3c}$=0.19. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.6 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST CR-23

FIGURE E1-28





Soil: Shell Mixture. Gravelly silty sand. Max. size ½-in. Mica flakes. Olive brown. Mixture consists of the minus ½-in. fraction of selected jar samples from Boring K-4 and K-5.

Structure: $\bar{\sigma}_{1c}$=2.00 kg/cm², $\bar{\sigma}_{3c}$=1.00 kg/cm², u_c=9.00 kg/cm², B =0.99. Dry Unit Weight in cell: initial 123.2 pcf, after consolidation 125.3 pcf.

Loading: Undrained cyclic axial compression. Cyclic stress ratio, $\sigma_d/2\bar{\sigma}_{3c}$= 0.34. No reversal of principal stresses. Frequency of loading 0.4Hz.

Size: Height 17.8 cm, Diameter 7.4 cm.

CONSOLIDATED UNDRAINED CYCLIC TRIAXIAL TEST
CR-24

FIGURE E1-29

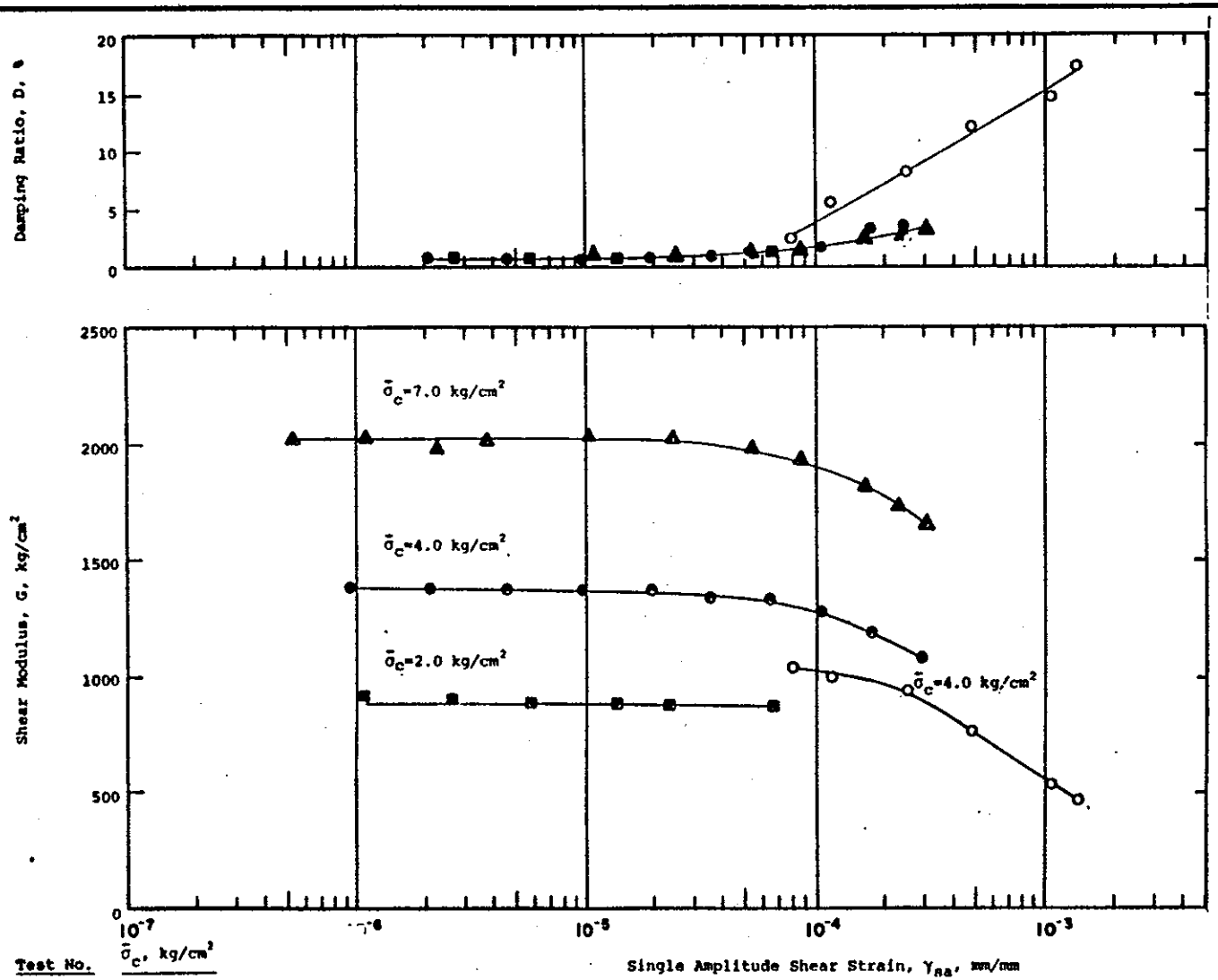
Section E2

Resonant Column and Small Strain Triaxial Test Data

FIGURE NO.	TITLE	PAGE
E2-1	Shear Modulus & Damping VS Shear Strain Tests RC-1 and E-2	E2-1
E2-2	Shear Modulus & Damping VS Shear Strain Tests RC-2 and E-1	E2-2
E2-3	Shear Modulus & Damping VS Shear Strain Test RC-3	E2-3
E2-4	Shear Modulus & Damping VS Shear Strain Test RC-4	E2-4
E2-5	Shear Modulus & Damping VS Shear Strain Test RC-5	E2-5
E2-6	Shear Modulus & Damping VS Shear Strain Tests RC-6 and E-3	E2-6
TABLE NO.		
E2-1	Test RC-1, Boring K-2 Sample SP-13C	E2-7
E2-2	Test RC-2, Boring K-2 Sample SP-1C	E2-8
E2-3	Test RC-3, Boring K-5 Sample SP-9	E2-9
E2-4	Test RC-4, Borings K-4 and K-5 Shell Mixture	E2-10
E2-5	Test RC-5, Borings K-4 and K-5 Shell Mixture	E2-11
E2-6	Test RC-6, Borings K-4 and K-5 Shell Mixture	E2-12

Section E2
(continued)

TABLE No.	TITLE	PAGE
E2-7	Test E-1, Boring K-2 Sample SP-1B	E2-13
E2-8	Test E-2, Boring K-2 Sample SP-13B	E2-14
E2-9	Test E-3, Borings K-4 and K-5 Shell Mixture	E2-15



Symbol	Test No.	$\bar{\sigma}_c$, kg/cm²
▼	RC-1	2.0
●	RC-1	4.0
▲	RC-1	7.0
○	E-2	4.0

RESONANT COLUMN TEST RC-1

Soil: Boring K-2, Sample SP-13C, Depth 107.0 ft. Top ⅓ and bottom ⅓: Predominately silt with some fine sand and fine sand lenses. Central ⅓: Silty fine sand. Mica flakes. Olive brown. Note: Grain size analysis performed on bottom ⅓ of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 2.0, 4.0 and 7.0 kg/cm², u_c = 6.00 kg/cm², B_c = 0.97

Dry Unit Weights:
In Tube = 83.7 pcf
In Cell, initial = 84.2 pcf
at $\bar{\sigma}_c$=2.0 kg/cm² = 85.6 pcf
at $\bar{\sigma}_c$=4.0 kg/cm² = 87.1 pcf
at $\bar{\sigma}_c$=7.0 kg/cm² = 89.3 pcf

Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 16.6 cm, Diameter 7.3 cm.

SMALL STRAIN CYCLIC TRIAXIAL TEST E-2

Soil: Boring K-2, Sample SP-13B, Depth 106.5 ft. Contorted layers and lenses of silt and fine sand. Mica flakes. Olive brown. Note: Grain size analysis performed on a mixture of the sample.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 4.0 kg/cm², u_c = 4.00 kg/cm², B_c = 0.95.

Dry Unit Weight:
In Tube = 81.0 pcf
In Cell after consolidation = 86.0 pcf

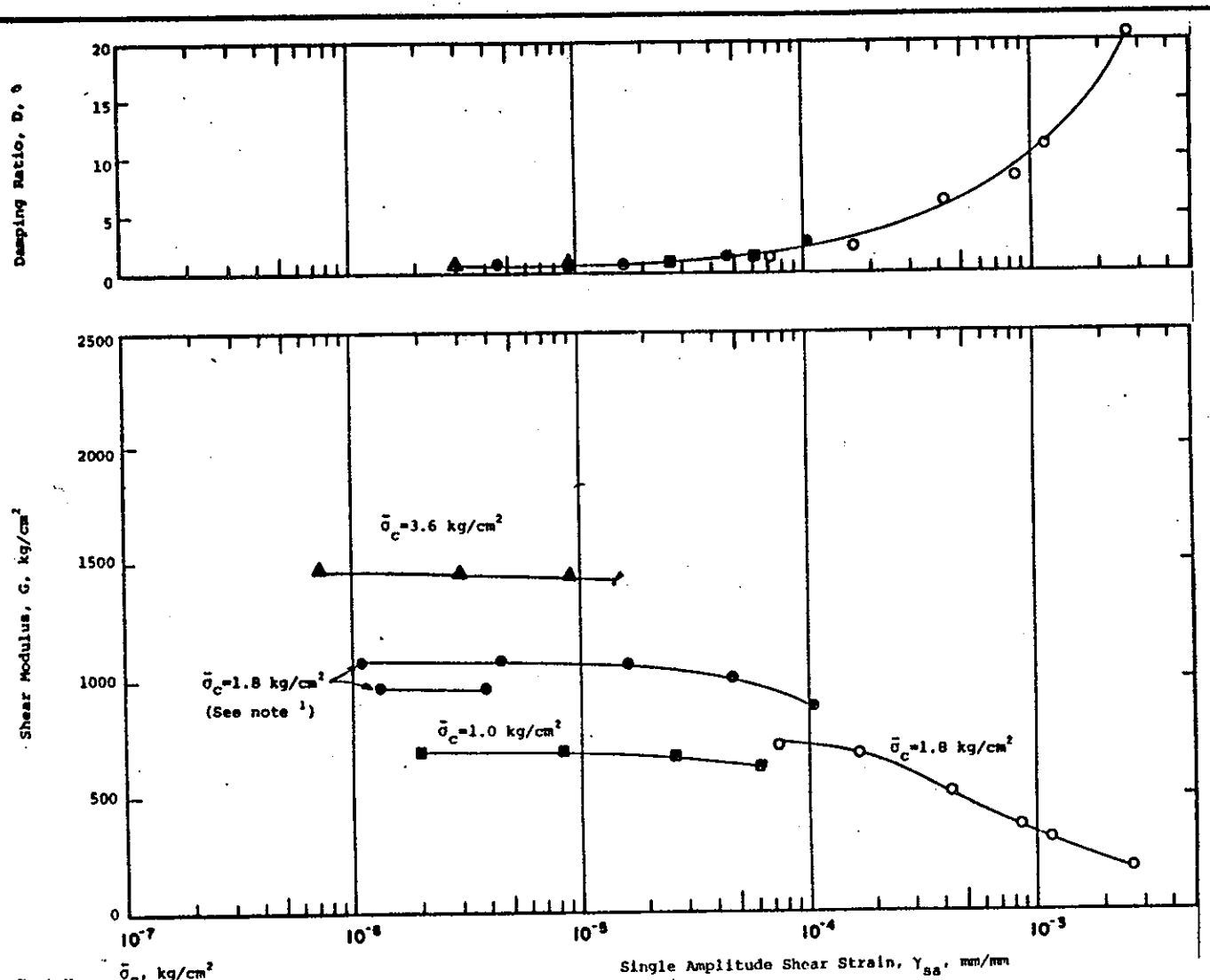
Loading: Undrained axial cyclic loading at approximately 40 seconds per cycle. Stress levels were varied. Excess pore pressure generated during cycling at a particular stress level were dissipated before cyclic loading at the next stress level.

Size: Height 16.4 cm, Diameter 7.3 cm.

Comments: Test CR-3 was performed on Sample 13-B after completion of Test E-2.

SHEAR MODULUS & DAMPING VS SHEAR STRAIN TESTS RC-1 AND E-2

FIGURE E2-1



Symbol	Test No.	$\bar{\sigma}_c$, kg/cm²
■	RC-2	1.0
●	RC-2	1.8[1]
▲	RC-2	3.6
○	E-1	1.8

NOTES: [1] Resonant column determinations were performed at two specimen dry unit weights of 94.0 and 94.3 pcf. The increase in dry unit weight resulted from rebounding and reconsolidation of the specimen. This occurred because the cell was dismantled to reposition the magnets in the electric coils.

RESONANT COLUMN TEST RC-2

Soil: Boring K-2, Sample SP-1C, Depth 43.0 ft Top 1/3 - silty find sand. Bottom 2/3 - predominately silt with some fine sand. Specimen contains mica flakes. Olive brown. Note: Separate grain size analyses performed on top and bottom portions of sample.

Structure: Undisturbed 3-in. tube sample of core material.

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 1.0, 1.8 & 3.6 kg/cm², u_c = 6.00 kg/cm², B_c = 0.48
Dry Unit Weights:
In tube = 90.3 pcf
In cell, initial = 90.6 pcf
at $\bar{\sigma}_c$=1.0 kg/cm² = 92.8 pcf
at $\bar{\sigma}_c$=1.8 kg/cm² = 94.0 & 94.3 pcf
at $\bar{\sigma}_c$=3.6 kg/cm² = 94.8 pcf

Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 16.9 cm, Diameter 7.3 cm.

SMALL STRAIN CYCLIC TRIAXIAL TEST E-1

Soil: Boring K-2, Sample SP-1B, Depth 42.5 ft Horizontal layers and irregularly shaped lenses of silt and fine sand. Bottom 2/3 of specimen is slightly coarser grained than top 1/3. Mica flakes. Olive brown. Note: Grain size analysis performed on bottom 2/3 of specimen.

Structure: Undisturbed 3-in. tube sample of core material.

State: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 1.8 kg/cm², u_c = 8.00 kg/cm²
B_c = 0.92
Dry Unit Weight:
In tube = 84.7 pcf
In cell after consolidation = 90.1 pcf

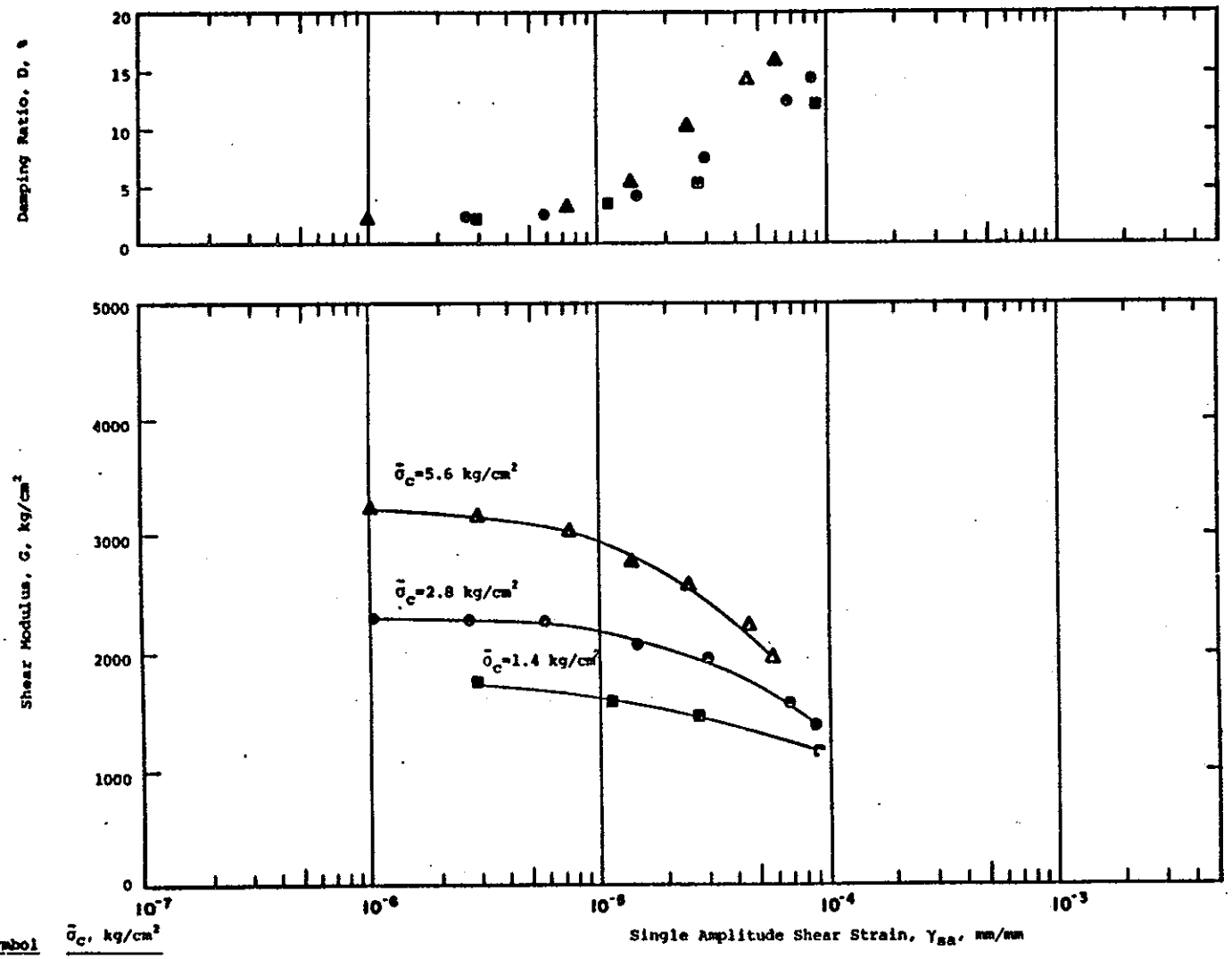
Loading: Undrained axial loading at approximately 40 seconds per cycle. Stress levels were varied. Excess pore pressure generated during cycling at a particular stress level were dissipated before cyclic loading at the next stress level.

Size: Height 16.2 cm, Diameter 7.3 cm.

Comments: Test CR-2 was performed on SampleSP-1B after completion of Test E-1.

SHEAR MODULUS & DAMPING VS SHEAR STRAIN TESTS RC-2 and E-1

FIGURE E2-2



Symbol	$\bar{\sigma}_c$, kg/cm²
■	1.4
●	2.8
▲	5.6

Soil: Boring K-5, Sample P-9, Depth 64.0 ft
One coarse gravel piece (~3-in. size) and several fine gravel pieces embedded in a matrix of widely graded silty sand. About 57% of the dry weight of the sample is +¼-in. size. The gravel pieces contact each other. A ¼-in. thick vertical lense of uniform medium sand (probably wash from the boring operation) was adjacent to one side of the gravel pieces.

Structure: "Undisturbed" 3-in. pitcher sample of shell material. Examination of the specimen after the test revealed that it was disturbed.

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 1.4, 2.8 and 5.6 kg/cm², u_c = 2.0 kg/cm²

Dry Unit Weights:

In Tube	= 143.8 pcf
In Cell, initial	= 139.2 pcf
at $\bar{\sigma}_c$=1.4 kg/cm²	= 139.4 pcf
at $\bar{\sigma}_c$=2.8 kg/cm²	= 139.8 pcf
at $\bar{\sigma}_c$=5.6 kg/cm²	= 140.3 pcf

Note: The dry unit weights in the cell are slightly lower than the dry unit weight in the tube because a ¼-in. thick sand layer was placed on top of the specimen to level the top cap.

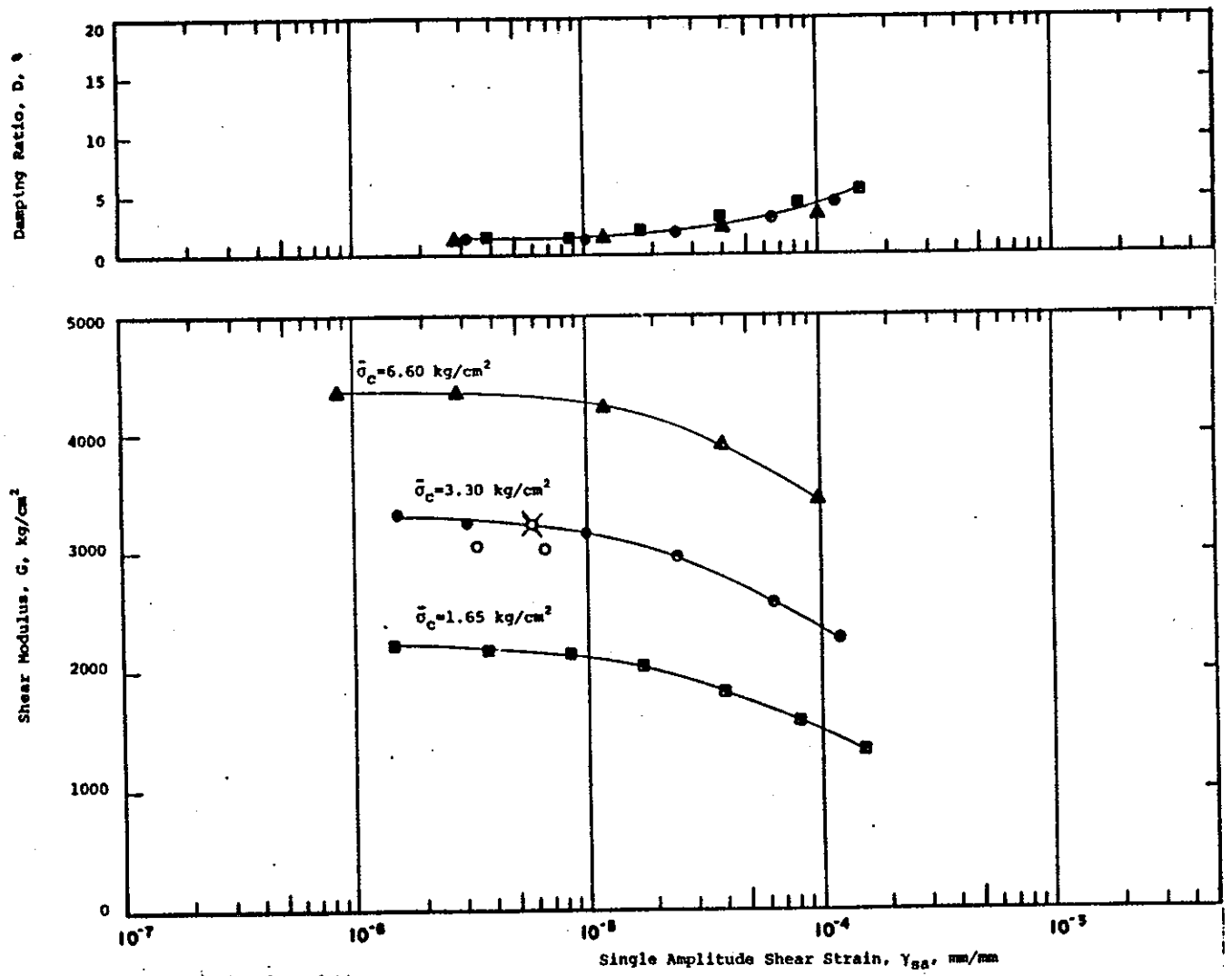
Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 10.7 cm, Diameter 7.3 cm.

Comments: Note that high damping was measured between strains of 10^{-5} and 10^{-4} mm/mm. The high damping may be caused by slipping of gravel pieces in the specimen. The data from this test should be used with caution because of the presence of large gravel and "wash" in the specimen.

SHEAR MODULUS & DAMPING
VS SHEAR STRAIN
TEST RC-3

FIGURE E2-3



Soil: Shell Mixture.
Gravelly silty sand. Max. size ¼-in. Mica flakes. Olive brown. Mixture consists of the minus ¼-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired initial unit weight.

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 1.65, 3.30 & 6.60 kg/cm², u_c = 5.00 kg/cm², B_c = 0.50

Dry Unit Weights:
In cell, initial = 137.1 pcf
at $\bar{\sigma}_c$=1.65 kg/cm² = 138.4 pcf
at $\bar{\sigma}_c$=3.30 kg/cm² = 138.7 & 138.8 pcf
at $\bar{\sigma}_c$=6.60 kg/cm² = 139.0 pcf

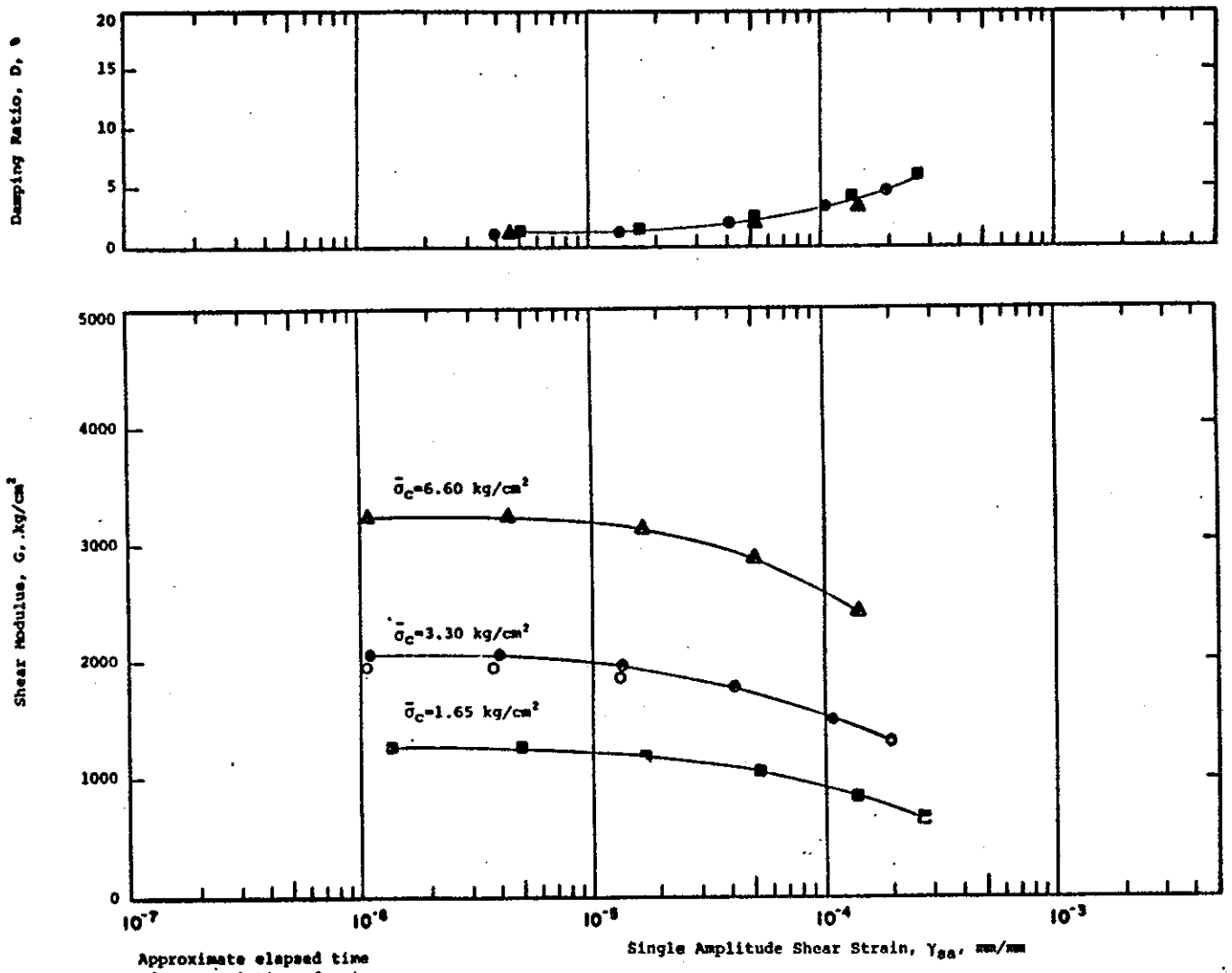
Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 16.9 cm, Diameter 7.4 cm.

Symbol	$\bar{\sigma}_c$, kg/cm²	Approximate elapsed time after completion of primary consolidation, min.
■	1.65	300
○	3.30	25
⊗	3.30	210
●	3.30	950
▲	6.60	45

SHEAR MODULUS & DAMPING VS SHEAR STRAIN
TEST RC-4

FIGURE E2-4



Symbol	$\bar{\sigma}_c$, kg/cm²	Approximate elapsed time after completion of primary consolidation, min.
■	1.65	60
○	3.30	60
●	3.30	1,240
▲	6.60	170

Soil: Shell Mixture
Gravelly silty sand. Max. size ¼-in. Mica flakes. Olive brown. Mixture consists of the minus ¼-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired initial unit weight.

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c} = \bar{\sigma}_{3c}$ = 1.65, 3.30 & 6.60 kg/cm², u_c = 5.00 kg/cm², B_c = 0.59.

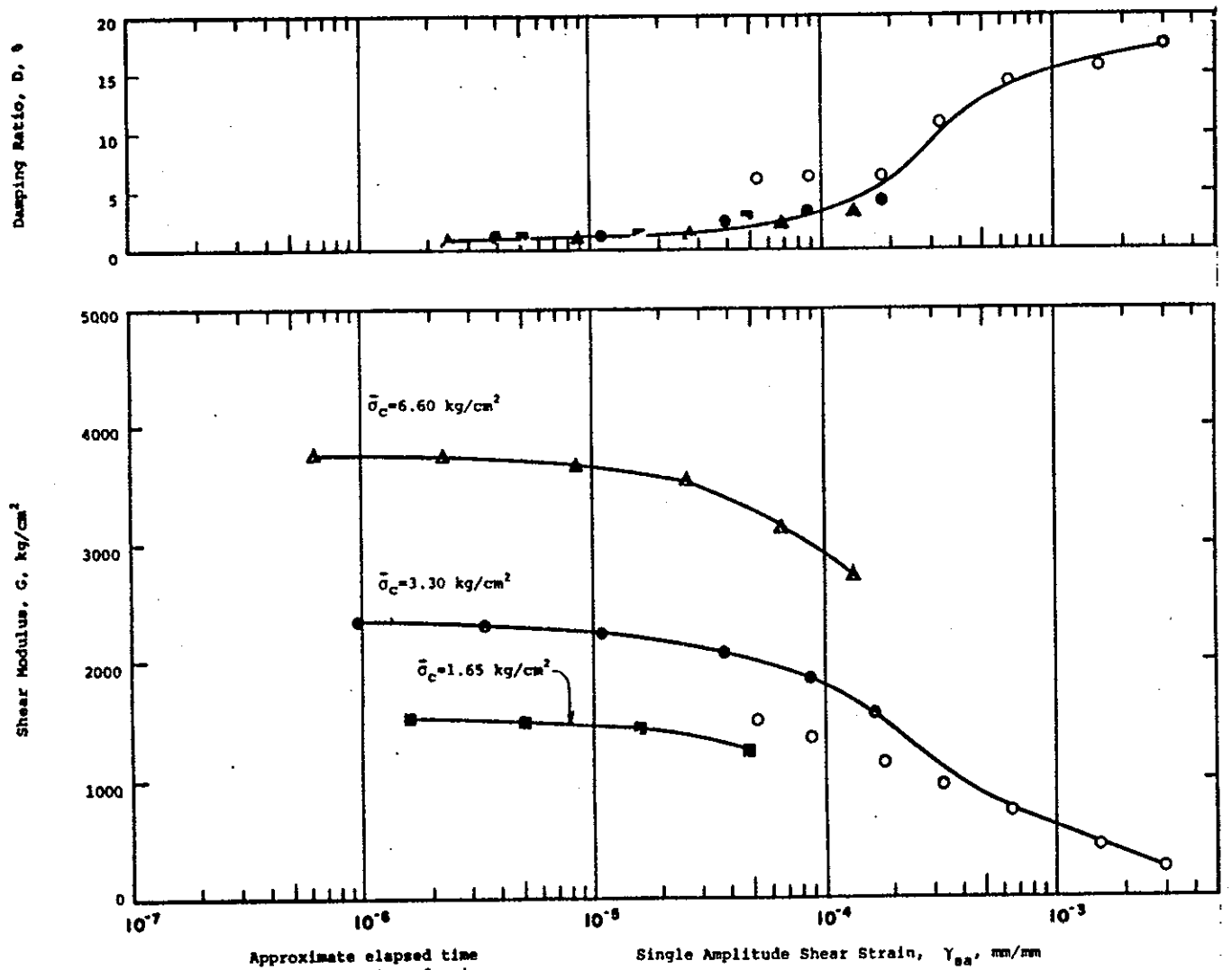
Dry Unit Weights:
In cell, initial = 123.4 pcf
at $\bar{\sigma}_c$=1.65 kg/cm² = 125.8 pcf
at $\bar{\sigma}_c$=3.30 kg/cm² = 127.8 & 128.1 pcf
at $\bar{\sigma}_c$=6.60 kg/cm² = 129.7 pcf

Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 16.1 cm, Diameter 7.4 cm

SHEAR MODULUS & DAMPING VS SHEAR STRAIN
TEST RC-5

FIGURE E2-5



Symbol	Test No.	$\bar{\sigma}_c$, kg/cm²	Approximate elapsed time after completion of primary consolidation, min.
▼	RC-6	1.65	3,850
●	RC-6	3.30	300
▲	RC-6	6.60	920
○	E-3	3.30	120

Soil: Shell Mixture
Gravelly silty sand. Max. size ¼-in. Mica flakes. Olive brown. Mixture consists of the minus ¼-in. fraction of selected jar samples from Borings K-4 and K-5.

Structure: Compacted specimen of shell material. Vibratory air hammer used to achieve desired initial unit weight.

RESONANT COLUMN TEST RC-6

State: Test performed at three consolidation stresses: $\bar{\sigma}_{1c}$= $\bar{\sigma}_{3c}$= 1.65, 3.30 and 6.60 kg/cm², u_c = 6.00 kg/cm², B_c = 0.69
Dry Unit Weights:
In cell, initial = 127.2 pcf
at $\bar{\sigma}_c$=1.65 kg/cm² = 129.0 pcf
at $\bar{\sigma}_c$=3.30 kg/cm² = 130.6 pcf
at $\bar{\sigma}_c$=6.60 kg/cm² = 132.1 pcf

Loading: Torsional cyclic loading at first resonant frequency. Stress levels were varied. Drainage of specimen was allowed during loading.

Size: Height 15.9 cm, Diameter 7.4 cm.

SMALL STRAIN CYCLIC TRIAXIAL TEST E-3

State: $\bar{\sigma}_{1c}$ = $\bar{\sigma}_{3c}$ = 3.30 kg/cm², u_c = 3.50 kg/cm², B_c = 0.95
Dry unit weight in cell:
Initial = 129.2 pcf
After consolidation = 131.7 pcf

Loading: Undrained axial loading at approximately 40 seconds per cycle. Stress levels were varied. Excess pore pressure generated during cycling at a particular stress level was dissipated before cyclic loading at the next stress level.

Size: Height 14.2 cm, Diameter 7.3 cm.

SHEAR MODULUS & DAMPING VS SHEAR STRAIN TESTS RC-6 AND E-3

FIGURE E2-6

Table E2-1

TEST RC-1, BORING K-2
SAMPLE SP-13C

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm²	Dry Unit Weight: In Tube: in[2] field pcf	Dry Unit Weight: In Tube: as delivered to lab pcf	Dry Unit Weight: In Cell: initial pcf	Dry Unit Weight: In Cell: after consolidation pcf	Water Content: In Tube: as measured in lab %	Water Content: In Cell: after consolidation %	Determination No.	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm²	Damping Ratio D %
2.0	83.7	83.7	84.2	85.6	38.0	36.5	1a	0.108	21,810	910	-
							1b	0.263	21,710	900	1.0
							1c	0.570	21,590	890	1.0
							1d	1.38	21,590	890	0.9
							1e	2.53	21,480	880	1.0
							1f	6.71	21,360	870	1.5
4.0				87.1		35.3	2a	0.094	26,740	1,380	-
							2b	0.205	26,790	1,380	1.0
							2c	0.454	26,790	1,380	0.9
							2d	0.994	26,790	1,380	0.9
							2e	1.93	26,620	1,360	0.9
							2f	3.59	26,370	1,340	1.1
							2g	6.54	26,200	1,320	1.3
							2h	10.6	25,710	1,270	1.8
							2i	17.2	24,810	1,180	3.3
							2j	29.3	23,570	1,070	3.6
7.0				89.3		33.5	3a	0.052	32,120	2,010	-
							3b	0.110	32,190	2,020	-
							3c	0.226	31,940	1,990	1.3
							3d	0.383	32,120	2,010	2.2
							3e	1.05	32,300	2,030	1.2
							3f	2.44	32,300	2,030	1.1
							3g	5.23	31,940	1,990	1.2
							3h	8.82	31,610	1.950	1.5
							3i	16.6	30,450	1,810	2.2
							3j	23.5	29,710	1,720	2.8
							3k	30.8	29,070	1,650	3.6

Notes: 1) The specimen was back pressure saturated to u_c = 6.0 kg/cm².
2) The dry unit weight in the field was determined by multiplying the dry unit weight, as received in the laboratory, by a factor to account for settlement of soil in the tube during transportation, see text.

Table E2-2

TEST RC-2, BORING K-2
SAMPLE SP-1C

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm²	Dry Unit Weight: In Tube: in[2] field pcf	Dry Unit Weight: In Tube: as delivered to lab pcf	Dry Unit Weight: In Cell: initial pcf	Dry Unit Weight: In Cell: after consolidation pcf	Water Content: In Tube: as measured in lab %	Water Content: In Cell: after consolidation %	Determination No.	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm²	Damping Ratio D %
1.0	90.3	90.3	90.6	92.8	26.6	30.9	1a	0.198	18,730	690	-
							1b	0.830	18,790	700	1.0
							1c	2.65	18,470	670	1.2
							1d	6.11	17,800	620	1.7
1.8[3]				94.0		30.0	2a	0.129	22,070	970	-
							2b	0.382	22,070	970	-
				94.3		29.8	3a	0.109	23,290	1,080	-
							3b	0.445	23,420	1,090	1.0
							3c	1.61	23,100	1,060	1.0
							3d	4.58	22,470	1,010	1.5
							3e	10.4	21,190	890	2.9
3.6				94.8		29.5	4a	0.072	27,190	1,480	-
							4b	0.308	27,040	1,460	1.1
							4c	0.914	26,940	1,450	1.3
							4d	1.53	26,750	1,430	-

Notes:
1) The specimen was back pressure saturated to u_c = 6.00 kg/cm².
2) The dry unit weight in the field was determined by multiplying the dry unit weight, as received in the laboratory, by a factor to account for settlement of soil in the tube during transportation
3) At $\bar{\sigma}_c$ = 1.8 kg/cm², resonant column determinations were performed at two specimen dry unit weights. The magnets were touching the electrical coils during Determination No. 2 at shear strains greater than about 4 x 10^{-6} mm/mm. The cell was dismantled and the magnets repositioned inside the electrical coils. Even though the specimen drainage line was closed during the dismantling process, the specimen densified slightly when $\bar{\sigma}_c$ = 1.8 kg/cm² was reapplied.

Table E2-3

TEST RC-3, BORING K-5
SAMPLE SP-9

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm²	Dry Unit Weight: In Tube, as delivered to lab[2] pcf	Dry Unit Weight: In Cell, initial pcf	Dry Unit Weight: In Cell, after consolidation pcf	Water Content: In Tube, as measured in lab %	Water Content: In Cell, after consolidation %	Determination No.	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm²	Damping Ratio D %
1.4	143.8	139.2	139.4	8.4	10.3	1a	0.291	26,370	1,710	2.5
						1b	1.14	25,420	1,590	3.8
						1c	2.73	24,220	1,440	5.3
						1d	8.97	21,730	1,160	12.1
2.8			139.8		10.1	2a	0.102	30,540	2,300	-
						2b	0.262	30,450	2,280	2.7
						2c	0.578	30,370	2,270	2.9
						2d	1.50	28,970	2,070	4.4
						2e	2.93	27,990	1,930	7.8
						2f	6.61	25,320	1,580	12.4
						2g	8.63	23,740	1,390	14.7
5.6			140.3		10.0	3a	0.101	36,120	3,220	2.1
						3b	0.294	35,750	3,150	2.4
						3c	0.746	35,010	3,020	3.3
						3d	1.44	33,610	2,790	5.5
						3e	2.51	32,430	2,590	10.4
						3f	4.54	29,910	2,200	14.3
						3g	5.76	28,040	1,940	16.0

Notes:
1) The specimen was back pressure saturated to u_c = 2.0 kg/cm².
2) The dry unit weight in the tube includes coarse and fine gravel pieces. for description of soil. The dry unit weight in the tube corrected for + 1/2-in. in size particles is 112.6 pcf.
3) The dry unit weights in the cell are less than the dry unit weight in the tube because a 1/4-in.-thick layer of widely graded sand was placed on top of the specimen. This sand layer was placed so that the top cap could be installed in a level position.
4) This sample was obtained with a Pitcher sampler. The sample contained about 57% by weight of + 1/2-in.-size particles. One coarse gravel piece was 3-in. size. It is likely that the sample was significantly disturbed by the sampling process.
5) The damping ratios measured at strains between 10^{-5} and 10^{-4} mm/mm are unusually high. The high damping ratios may be caused by slipping of interconnected gravel pieces in the sample.

Table E2-4

TEST RC-4, BORINGS K-4 AND K-5
SHELL MIXTURE

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm²	Dry Unit Weight In cell initial pcf	Dry Unit Weight In cell after consolidation pcf	Water Content In cell after consolidation %	Determination No.	Elapsed Time After Completion of Primary Consolidation (min)	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm²	Damping Ratio D %
1.65	137.1	138.4	10.6	1a	∿300	0.146	29,820	2,210	-
				1b		0.373	29,510	2,160	1.7
				1c		0.830	29,230	2,120	1.8
				1d		1.72	28,500	2,020	2.2
				1e		3.89	26,910	1,800	3.1
				1f		8.04	25,110	1,570	4.4
				1g		15.2	22,950	1,310	5.7
3.30		138.7	10.5	2a	∿25	0.338	35,000	3,040	-
				2b		0.653	34,800	3,010	-
				2c		1.45	34,200	2,910	-
		138.8	10.5	3a	∿210	0.584	36,120	3,240	-
		138.8	10.5	4a	∿950	0.105	36,620	3,320	-
				4b		0.306	36,260	3,250	1.4
				4c		0.995	35,790	3,170	1.5
				4d		2.51	34,470	2,940	2.0
				4e		6.41	32,260	2,570	3.1
				4f		12.0	30,170	2,250	4.4
6.60		139.0	10.4	5a	∿45	0.083	42,010	4,360	-
				5b		0.281	42,050	4,370	1.4
				5c		1.19	41,430	4,240	1.5
				5d		4.00	39,740	3,900	2.2
				5e		10.1	37,430	3,460	3.4

Notes: 1) The specimen was back pressure saturated to u_c = 2.0 kg/cm².

Table E2-5

TEST RC-5, BORINGS K-4 AND K-5
SHELL MIXTURE

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm^2	Dry Unit Weight In cell initial pcf	Dry Unit Weight In cell after consolidation pcf	Water Content In cell after consolidation %	Determination No.	Elapsed Time After Completion of Primary Consolidation (min)	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm^2	Damping Ratio D %
1.65	123.4	125.8	15.1	1a	∿60	0.134	23,220	1,260	-
				1b		0.482	23,040	1,240	1.6
				1c		1.68	22,550	1,190	1.9
				1d		5.07	21,120	1,040	2.8
				1e		13.7	18,800	830	4.3
				1f		26.9	16,660	650	6.0
3.30		127.8	14.3	2a	∿60	0.103	28,670	1,940	-
				2b		0.367	28,540	1,930	-
				2c		1.33	28,035	1,860	-
		128.1	14.2	3a	∿1,240	0.109	29,420	2,050	-
				3b		0.385	29,420	2,050	1.2
				3c		1.38	28,790	1,960	1.4
				3d		4.07	27,470	1,790	2.1
				3e		10.5	25,100	1,490	3.3
				3f		19.4	23,480	1,310	4.7
6.60		129.7	13.6	4a	∿170	0.106	36,790	3,230	-
				4b		0.443	36,740	3,220	1.4
				4c		1.70	36,120	3,110	1.6
				4d		5.07	34,600	2,860	2.2
				4e		14.2	31,700	2,400	3.5

Notes: 1) The specimen was back pressure saturated to u_c = 5.00 kg/cm^2.

Table E2-6

TEST RC-6, BORINGS K-4 AND K-5
SHELL MIXTURE

Effective Consolidation Stress[1] $\bar{\sigma}_c$ kg/cm²	Dry Unit Weight In cell initial pcf	Dry Unit Weight In cell after consolidation pcf	Water Content In cell after consolidation %	Determination No.	Elapsed Time After Completion of Primary Consolidation (min)	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-5}	Shear Wave Velocity cm/sec	Shear Modulus G kg/cm²	Damping Ratio D %
1.65	127.2	129.0	13.9	1a	∿3,850	0.160	25,240	1,520	-
				1b		0.491	25,020	1,500	1.3
				1c		1.58	24,450	1,430	1.8
				1d		4.83	22,900	1,250	3.0
3.30		130.6	13.3	2a	∿300	0.097	31,250	2,350	-
				2b		0.337	31,020	2,320	1.5
				2c		1.08	30,750	2,280	1.5
				2d		3.68	29,450	2,090	2.3
				2e		8.56	27,830	1,870	3.2
				2f		16.4	25,600	1,580	4.2
6.60		132.1	12.7	3a	∿920	0.061	39,420	3,770	-
				3b		0.224	39,340	3,760	1.1
				3c		0.842	38,980	3,690	1.2
				3d		2.56	38,250	3,550	1.7
				3e		6.82	36,010	3,150	2.5
				3f		13.4	33,650	2,750	3.3

Notes: 1) The specimen was back pressure saturated to 6.00 kg/cm².

Table E2-7

TEST E-1, BORING K-2
SAMPLE SP-1B

Effective Consolidation Stress, $\bar{\sigma}_{1c} = \bar{\sigma}_{3c} = 1.80$ kg/cm^2
Dry Unit Weights: In tube = 84.7 pcf
In cell, initial = 85.4 pcf
At $\bar{\sigma}_c = 1.80$ kg/cm^2 = 90.1 pcf

Determination No.	Number of Load Cycles	Cyclic Deviator Stress[3)] kg/cm^2	Pore Pressure Increase Δu kg/cm^2	Single Amplitude Axial Strain ϵ_{sa} mm/mm x 10^{-4}	Secant Elastic Modulus, E_s kg/cm^2	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-4}	Secant Shear Modulus, G_s kg/cm^2	Damping Ratio D %
1	3	0.10	0.02	0.49	2,140	0.74	715	1.6
2	5	0.23	0.01	1.12	2,030	1.68	680	1.9
3	3	0.44	0.07	2.82	1,560	4.23	520	6.2
4	7	0.65	0.32	5.78	1,130	8.67	375	8.4
5	5	0.71	0.42	7.45	950	11.2	315	11.1
6	5	0.93	0.77	17.5	535	26.2	180	21.0

Notes:

1) The specimen was back pressure saturated to 8.00 kg/cm^2.
2) The cyclic deviator stress is one half of the difference between the maximum and minimum deviator stress in the cycle used for computations.
3) Pore pressure increases were released before beginning the next determination.
4) Single amplitude axial strain is one half of the difference between the maximum and minimum axial strain in the cycle used for computations.

Table E2-8

TEST E-2, BORING K-2
SAMPLE SP-13B

Effective Consolidation Stress, $\bar{\sigma}_{1c} = \bar{\sigma}_{3c} = 4.00$ kg/cm^2
Dry Unit Weights: In tube = 81.0 pcf
In cell, initial = 81.2 pcf
At $\bar{\sigma}_c = 4.00$ kg/cm^2 = 86.0 pcf

Determination No.	Number of Load Cycles	Cyclic Deviator Stress[3] kg/cm^2	Pore Pressure Increase, Δu kg/cm^2	Single Amplitude Axial Strain ε_{sa} mm/mm x 10^{-4}	Secant Elastic Modulus, E_s kg/cm^2	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-4}	Secant Shear Modulus, G_s kg/cm^2	Damping Ratio D %
1	6	0.17	6)	0.53	3,120	0.80	1,040	-
2	4	0.23	6)	0.78	2,980	1.17	995	5.8
3	5	0.47	6)	1.67	2,810	2.50	935	8.2
4	4	0.74	0.41	3.25	2,270	4.87	755	12.2
5	5	1.10	0.77	6.96	1,580	10.4	525	14.6
6	6	1.26	1.21	9.22	1,370	13.8	455	17.4

Notes:

1) The specimen was back pressure saturated to 4.00 kg/cm^2.
2) The cyclic deviator stress is one half of the difference between the maximum and minimum deviator stress in the cycle used for computations.
3) Pore pressure increases were released before beginning the next determination.
4) Single amplitude axial strain is one half of the difference between the maximum and minimum axial strain in the cycle used for computations.

5) Pore pressure measurements were not made for this determination.

Table E2-9

TEST E-3, BORINGS K-4 AND K-5
SHELL MIXTURE

Effective Consolidation Stress, $\bar{\sigma}_{1c} = \bar{\sigma}_{3c} = 3.30$ kg/cm^2
Dry Unit Weights: In cell, initial = 129.2 pcf
At $\bar{\sigma}_c = 3.30$ kg/cm^2 = 131.7 pcf

Determination No.	Number of Load Cycles	Cyclic Deviator Stress kg/cm^2	Pore Pressure Increase Δu kg/cm^2	Single Amplitude Axial Strain[5] ε_{sa} mm/mm x 10^{-4}	Secant Elastic Modulus, E_s kg/cm^2	Single Amplitude Shear Strain γ_{sa} mm/mm x 10^{-4}	Secant Shear Modulus, G_s kg/cm^2	Damping Ratio D %
1	7	0.15	0.08	0.34	4,530	0.507	1,510	6.3
2	6	0.24	0.12	0.59	4,110	0.883	1,370	6.4
3	6	0.42	0.27	1.21	3,510	1.81	1,170	6.4
4	5	0.64	0.44	2.17	2,950	3.25	985	11.0
5	5	0.98	0.79	4.36	2,230	6.55	745	14.3
6	6	1.33	1.55	10.2	1,300	15.3	435	15.5
7	6	1.37	2.30	19.9	690	29.9	230	17.3
8[7]	6	1.31	2.76	35.2	370	52.8	125	14.1
9[7]	2	2.14	2.76	66.9	320	100.0	105	19.1

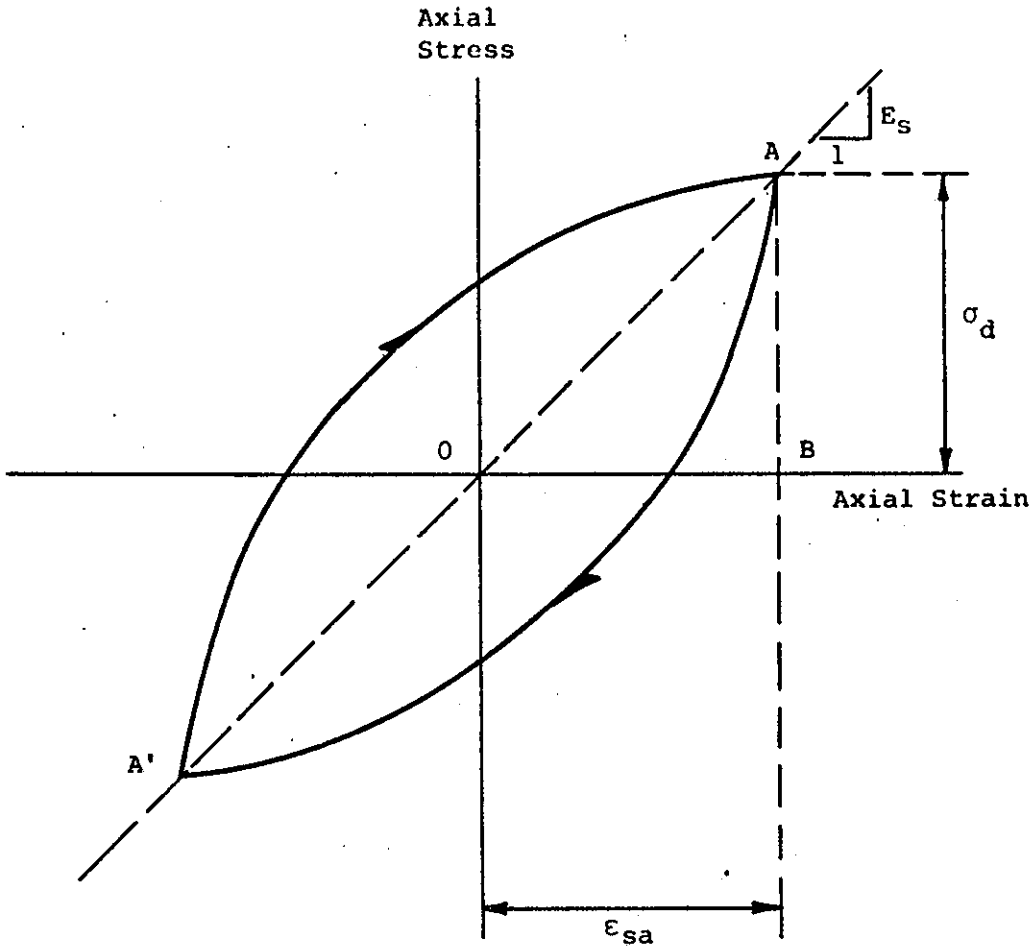
Notes:

1) The specimen was back pressure saturated to 3.50 kg/cm^2.
2) The cyclic deviator stress is one half of the difference between the maximum and minimum deviator stress in the cycle used for computations.
3) Pore pressure increases were released before beginning the next determination.
4) Single amplitude axial stress is one half of the difference between the maximum and minimum axial strain in the cycle used for computations.
5) The dry unit weight increased from about 131.7 pcf to 133.7 pcf during Determinations 5 through 9.
6) A repetitive stress strain behavior was not reached for this determination

Section E3

Definitions of Dynamic Test Relationships

FIGURE NO.	TITLE	PAGE
E3-1	Definition of Terms with Respect to the Stress-Strain Hysteresis Loop Developed in E Tests	E3-1
E3-2	Damping, Resonant Column Test	E3-2



E_s = Secant elastic modulus

σ_d = Cyclic deviator stress

ε_{sa} = Single amplitude axial strain

Damping ratio = $D = \frac{1}{4\pi} \frac{A_L}{A_T}$

A_L = area of hysteresis loop

A_T = area of triangle AOB

G_s = Secant shear modulus = $\frac{E_s}{2(1+\nu)}$

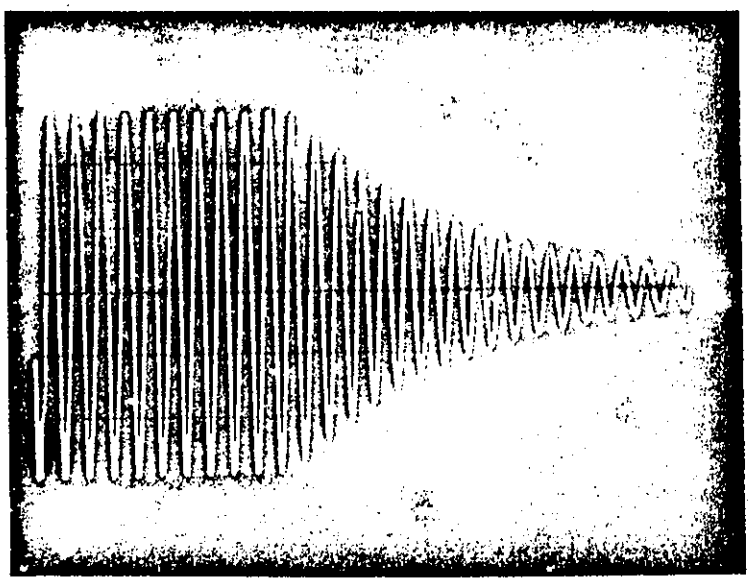
γ_{sa} = Single amplitude shear strain = $\varepsilon_{sa}(1+\nu)$

ν = Poissons ratio; ν=0.5 for undrained conditions

NOTE: Point 0 is not defined by absolute zero values of axial stress and axial strain, but rather by the midpoint on the secant modulus line A-A'.

DEFINITION OF TERMS WITH RESPECT TO THE STRESS-STRAIN HYSTERESIS LOOP DEVELOPED IN E TESTS

FIGURE E3-1



10 ms/div.

Damping Vibrations from Resonant Column Test RC-4

Determination No. 4d

Logarithmic Decrement $\delta = \frac{1}{n} \ln \frac{Z_1}{Z_{1+n}} = 0.128$

Z_1 = peak-peak amplitude for cycle 1

Z_{1+n} = peak-peak amplitude for cycle 1+n

n = number of damped cycles examined

Damping Ratio $D = \frac{\delta}{(4\pi^2+\delta^2)^{1/2}} \times 100\% = 2.0\%$

DAMPING

RESONANT COLUMN TEST

FIGURE E3-2

APPENDIX F

GENERAL DESCRIPTION OF COMPUTER ANALYSIS OUTPUT

Programs	Analysis
FEADAM	Static State of Stress
SHAKE	One-Dimensional Dynamic
FLUSH	Two-Dimensional Dynamic
STRAIN	Post-processor, 2-D Dynamic
SAM4	Sarma Stability
YDBH9	Permanent Deformation

APPENDIX F

GENERAL DESCRIPTION OF OUTPUT FROM COMPUTER ANALYSES

The following describes in general terms the output obtained from several computer analyses performed for the Dynamic Stability Analysis of Knightville Dam and which was submitted to NED as an Attachment to the draft Dynamic Response Analysis milestone report. The analyses performed included:

1) FEADAM - a two-dimensional finite element static state of stress analysis,

2) SHAKE - a one-dimensional dynamic analysis,

3) FLUSH - a two-dimensional finite element dynamic analysis,

4) STRAIN - a post processing program to FLUSH,

5) SAM4 - a seismic stability analysis using Sarma's procedures,

6) YDBH9 - a permanent deformation analysis.

Each of these programs have been described in Volume 1 of this Report.

A. FEADAM

The first several pages of the output are the information read from the control cards, and include the input control cards, geometry of the finite element model, soil properties, and numbering of nodal points and their relationships to the grid elements.

Following this information is a description of the construction layering system to be used in the model; then a

listing of all the elements, the initial values of soil properties and initial computation of stresses. Once this portion of the analysis is complete, the modelled construction layering sequence is begun. The output defining which elements and nodel points are to be added for the layer is printed, and the final stresses and strains are printed for every element as a result of the addition of the layer. The analysis proceeds with the addition of the next construction layer, this procedure is continued until the last construction layer is analyzed. The stresses and strains computed after placement of the final layer are the final values of the analysis. The final stresses used in this analysis are presented in Plates 43 to 46 of Volume 1 of the Report.

B. SHAKE

The SHAKE program allows the user several alternative input and output options. Each of the options however must be input in a logical order (not necessarily in a numerical order) in order that the necessary data are available for use in subsequent options. In general, the first item input is the shear modulus (G) and damping (λ) curves versus shear strain (γ) for the materials used in the vertical soil column selected for analysis. The program permits the input of only two G vs γ curves. If more than two soils exist within the profile then a multiplier or scaling factor can be applied to the input curves.

The next input item is generally the design earthquake. The input includes the time increment between acceleration values, the required scaling value and the values of acceleration at each time increment. There are eight values of acceleration on each card. If the input cards are not numbered or out of sequence a message is printed to indicate "out of sequence". (In this analysis the error message was printed on the output because the cards of the Puddingstone Record were not numbered.)

The next step is to input the vertical soil profile. The input data includes the thickness of the soil layers (or sublayers) and the initial soil and/or rock properties. This generally concludes the input data necessary for computation; additional input defines the output options. A significant output requirement is to obtain compatible soil properties using the iterative process; the number of iterations is limited by selecting a percent error (usually 5%) criteria calculated in the time domain. This option will provide values of maximum acceleration at the top of each layer and/or sublayer and the maximum dynamic shear stress imposed at the center of each layer and/or sublayer by the seismic event.

Additional output options include the tabulations and/or printer plots of response spectra of acceleration, velocity, and displacement for selected values of damping, and the computation and plotting of stress and/or strain time series for any two layers of the vertical column.

The important results are the values of maximum acceleration and imposed dynamic shear stresses within the layers and/or sublayers. These results are shown on Plates 49 to 52 of Volume 1 of this Report.

C. FLUSH

FLUSH permits the user three options for output: Option 1, Option 2 and Option 3. The sequence of operation consists of using Option 1 and/or Option 3 and then Option 2. Option 3 can be performed without using Option 1. Generally, however Option 1 is performed first when a new problem or different soil properties are to be used.

The FLUSH analysis uses an iterative process for computing the acceleration-time series at nodal points located in the finite element model. The number of iterations can

be controlled in two ways; by limiting the error difference between values of shear modulus and damping computed in successive iterations and/or by specifying that only a maximum number of iterations be performed. If the limiting error difference criteria is selected, then a 5 percent error is considered acceptable. If the number of iterations is selected as the control, the percent error differences must be examined to see if they are acceptable, i.e., less than 5 percent. If the error is greater than five percent, additional iterations are then required and can be performed using Option 3.

A general description indicates that the first few pages of the output contain a reorganization and definition of input control and requested output data, and shear modulus and damping curves vs shear strain for the materials used in the analysis. (As in the SHAKE program, only 2 curves may be used.) Next, the geometry of the elements and their boundary nodal points and the initial soil properties within each element are tabulated, followed by the coordinate geometry of all the nodal points. This is followed by the earthquake record (input the same way as in the SHAKE) with scaling factors, input properties of the free field layers and the controls for output. The computed results are then presented as output.

First to be printed are the values of maximum accelerations for the requested free field layers. This is followed by intermediate output of calculated soil properties for each element, for each iteration in the frequency and/or time domains. The iterative process continues until either of the previously mentioned conditions are met. All final values are stored on file (these data must be saved for later use) and the maximum accelerations at desired nodal points are presented. If the final iterative values are acceptable, then the next step is

to determine the shear stress-time series using the post processing program STRAIN. This program is used to compute the maximum values of dynamic shear stress and to compute the equivalent number of cycles for that nodal point.

D. STRAIN

The output from the STRAIN program consists of a reorganization and definition of the control input commands, basic information regarding the finite element model and earthquake record, and a description of requested frequency content. This is followed by a listing of the required output for the elements, and the strain and stress values computed at each of the requested elements. The data are printed out on two pages for each element. Two simple accessory programs need to be written to obtain additional output of shear stress-time history tabulations and/or plots. This is easily accomplished since the data are stored.

E. SAM4

The SAM4 program computes critical yield accelerations for a potential failure surface using a stability analysis proposed by Dr. Sarma. The output data initially restates the input data concerning the geometry of the potential failure surface and the material properties above and below the failure surface. The output presents the value of computed yield acceleration for a factor of safety of unity and the stresses which act on and within the sliding mass for that condition. To perform a complete analysis several potential failure surfaces have to be analyzed in order to determine the surface which results in the most critical acceleration. These values are shown on Plates 62 and 63 of Volume 1.

F. YDBH9

The initial output from this program consists of a reorganization and definition of the input data. It contains data on the pertinent characteristics of the design earthquake including the duration, time interval for integration, the time at which the integration is to start and end, the maximum accelerations (original and scaled values), and the increment of yield acceleration for which displacement computations are to be made.

The design earthquake record is then listed with eight values of accelerations shown on a line. This is followed for selected intervals of yield acceleration, by calculated values of time, acceleration, velocity, displacement, the magnitudes of average positive and negative accelerations. These calculations repeat until the value of yield acceleration equals the requested end time.

The total displacement is computed by summing the maximum displacement, which is taken as the larger of the positive or negative displacements, for each yield acceleration.