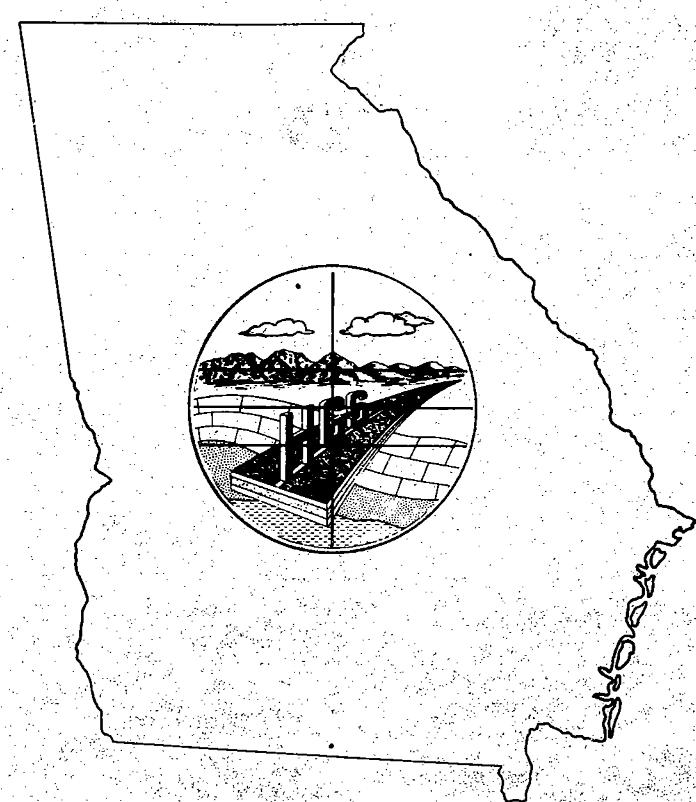
34th Annual
Highway Geology Symposium
Final Proceedings



May 2, 3 & 4 1983
Atlanta, Georgia

Sponsors
Georgia Department of Transportation
Georgia Geological Survey
Federal Highway Administration

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PAPERS

PREFACE

The 34th Annual Highway Geology Symposium was held in Atlanta, Georgia, at the Stone Mountain Inn at the base of Georgia's most famous mountain. One of the meeting's highlights was the field trip which concentrated on the multi million dollar downtown highway reconstruction and rapid transit construction. Dinner break was held at the historic Kennesaw Mountain National Park.

Probably, the most significant geotechnical techniques observed was the utilization of various types of precast wall systems. All of these walls owe much of their successful design to the crushed granite backfill which is being furnished by several giant quarries located around the city. One large quarry, Vulcan at Norcross, was visited and studied by the group.

Technology of the Symposium was a milestone for meetings of this sort. Georgia Department of Transportation's Geotechnical Engineering Bureau, the Georgia Department of Natural Resources (State Geologist's Office), and the Federal Highway Administration's Geotechnical Engineer compiled the team that planned the Symposium.

Input by Industry was excellent; several papers were given on new techniques outlining their usefulness in solving complex geotechnical problems safely and more economically. Vibration control in a city that requires blasting on practically every project without damaging adjacent structures was stressed.

Input by Georgia Tech was significant with Professor George Sowers giving the traditional banquet speech calling on us all to try harder for more economic well engineered jobs. Purdue University was also well represented with two excellent papers on soil mechanics.

Georgia's outstanding transportation program is steered by Commissioner Tom Moreland and his Georgia Tech colleague, D. J. Altobelli, Division Administrator for Federal Highway. It was Mr. Moreland himself that organized the first competent geotechnical section of Georgia Department of Transportation in 1960.

Dedication of this Symposium was to George Meadows of Virginia Department of Transportation who spent his career in geotechnical work to have it ended abruptly by his death just prior to this Symposium. George will be sadly missed.

A total of 125 people attended the Symposium, and our thanks goes out for their support. Apologies are also in order for the delay in publishing these proceedings. No excuses are offered other than Georgia Department of Transportation's work comes first.

David A. Mitchell

*HIGHWAY GEOLOGY SYMPOSIUM

Medallion Winners

Hugh Chase	-	1970
Tom Parrott	-	1970
Paul Price	-	1970
K. B. Woods	-	1971
R. J. Edmonson	-	1972
C. S. Mullin	-	1974
A. C. Dodson	-	1975
Burrell Whitlow	-	1978
Bill Sherman	-	1980
Virgil Burgat	-	1981
David L. Royster	-	1982
Henry Mathis	-	1982
Terry R. West	-	1983

*In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana, designed a plaque to be presented periodically to individuals who have made significant contributions to the HGS over a period of years. The award, a 3½" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

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INVESTIGATION, EVALUATION, AND QUALITY CONTROL

OF

AGGREGATE SOURCES IN GEORGIA

BY

ROBERT T. DICKERSON

PRESENTED AT THE
34TH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM
STONE MOUNTAIN, GEORGIA
MAY 2 - 3, 1983

Investigation, Evaluation, and Quality Control
of
Aggregate Sources in Georgia

by
Robert T. Dickerson

Have you ever wondered what this country would be like if there were no rock quarries? Millions of tons of crushed stone each year are produced and used in the construction of transportation systems, dams, and in the building industry. Without this import, necessary resource, just where would be be today?

Here in Georgia the crushed stone industry is the second largest mining industry in the State in production value. Kaolin mining continues to be the number one industry.

Of all the major cities in the United States, Atlanta enjoys the largest concentration of granite quarries.

It was 100 years ago, in 1883, that the first granite crushing plant was put into operation in Georgia. The material being crushed consisted of scrapped slabs and blocks of rock stockpiled at a dimension stone quarry. Today the rock crushing and processing business has evolved into a very big and highly sophisticated industry.

I have passed out copies of our Standard Operating Procedure One which concerns sampling and testing of aggregates and quality assurance programs. Our Quality Control Program operates according to these procedures. I will refer to portions of the SOP-l during this presentation.

All currently State approved aggregate sources are shown on the Physiographic Map of Georgia, Figure No. 1, Page No. 2 . The granite and quartzitic rock quarries are represented by dots, limestone and marble quarries by crosses, sand plants (mostly alluvial) by x's, rock quarry sand plants by triangles. The dashed circle represents a 20 miles radii from the capital.

Our present DOT Specifications concerning LA Abrasion and Magnesium Sulfate Soundness for Group I and Group II Aggregates are shown in Figure 2, Page 3. Crushed stone not meeting Class "A" Specifications is precluded for use in surface treatment and asphaltic concrete "D" type mix.

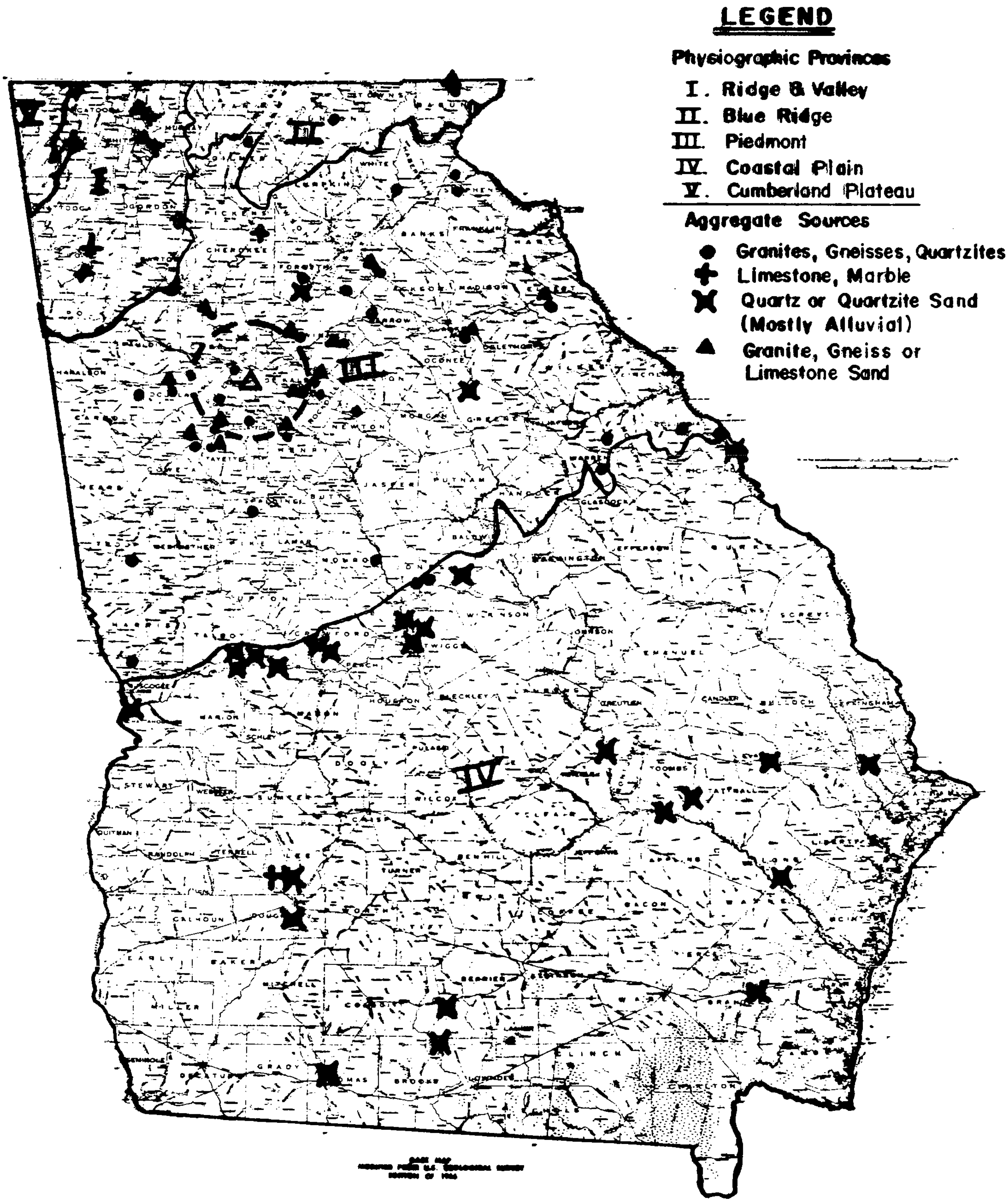


Figure 1
Physiographic Map of Georgia

PERCENT WEAR
(AASHTO:T-96)

	CLASS A	CLASS B
GROUP I AGGREGATE (CARBONATE)	0 - 40	41 - 60
GROUP II AGGREGATE (SILICEOUS)	0 - 50	51 - 65

SOUNDNESS – MAGNESIUM SULFATE
(AASHTO: T-104)

GROUP I AND II	15% (MAXIMUM LOSS ALLOWED)

Figure 2

LA ABRASION AND $MgSO_4$ SOUNDNESS SPECIFICATIONS

In Figures 3a and 3b on Page 4 are shown the number of Georgia DOT approved sources and the general types of rock and sand occurring. Approximately 80% of the coarse aggregate sources are silicious, and 20% carbonate. The natural alluvial sand sources outnumber manufactured (quarry) sand sources by a 2 to 1 ratio. Manufactured carbonate sand sources comprise only 7% to 8% of all sand sources.

A number of aggregate sources were visited and photographed by the author. Most of these sources are located in the general area surrounding Atlanta. The Aggregate Source Map, Figure 4, Page 5, shows the approximate location of each quarry as indicated by the Map Key Numbers on each arrow. Pages 6 through 14 show 35 photographs with captions of the various sites that were visited.

COARSE AGGREGATE SOURCES ON QUALIFIED PRODUCTS LIST	
TOTAL NUMBER – 57	
39 – Granite, Gneiss	
2 – Quartzite	(1) Conglomeritic
11 – Carbonate	(9) Crystalline limestone (1) Impure marble (1) Coastal limestone
5 – River Gravel	
GEORGIA SOURCES – 47	
37 – Granite, Gneiss	
2 – Quartzite	(1) Conglomeritic
8 – Carbonate	(6) Crystalline limestone (1) Impure marble (1) Coastal limestone
0 – River Gravel	

Figure 3a

NUMBER OF COARSE AGGREGATE SOURCES AND GEOLOGIC TYPES

FINE AGGREGATE SOURCES ON QUALIFIED PRODUCTS LIST	
TOTAL NUMBER – 49	
16 – Manufactured Sand	(12) Granite, Gneiss (4) Crystalline limestone
32 – Alluvial Sand	(3) Piedmont & Ridge & Valley
1 – Residual Sand	Piedmont
GEORGIA SOURCES – 42	
14 – Manufactured Sand	(12) Granite, Gneiss (2) Crystalline limestone
28 – Alluvial Sand	(3) Piedmont (25) Coastal Plain

Figure 3b

NUMBER OF FINE AGGREGATE SOURCES AND GEOLOGIC TYPES

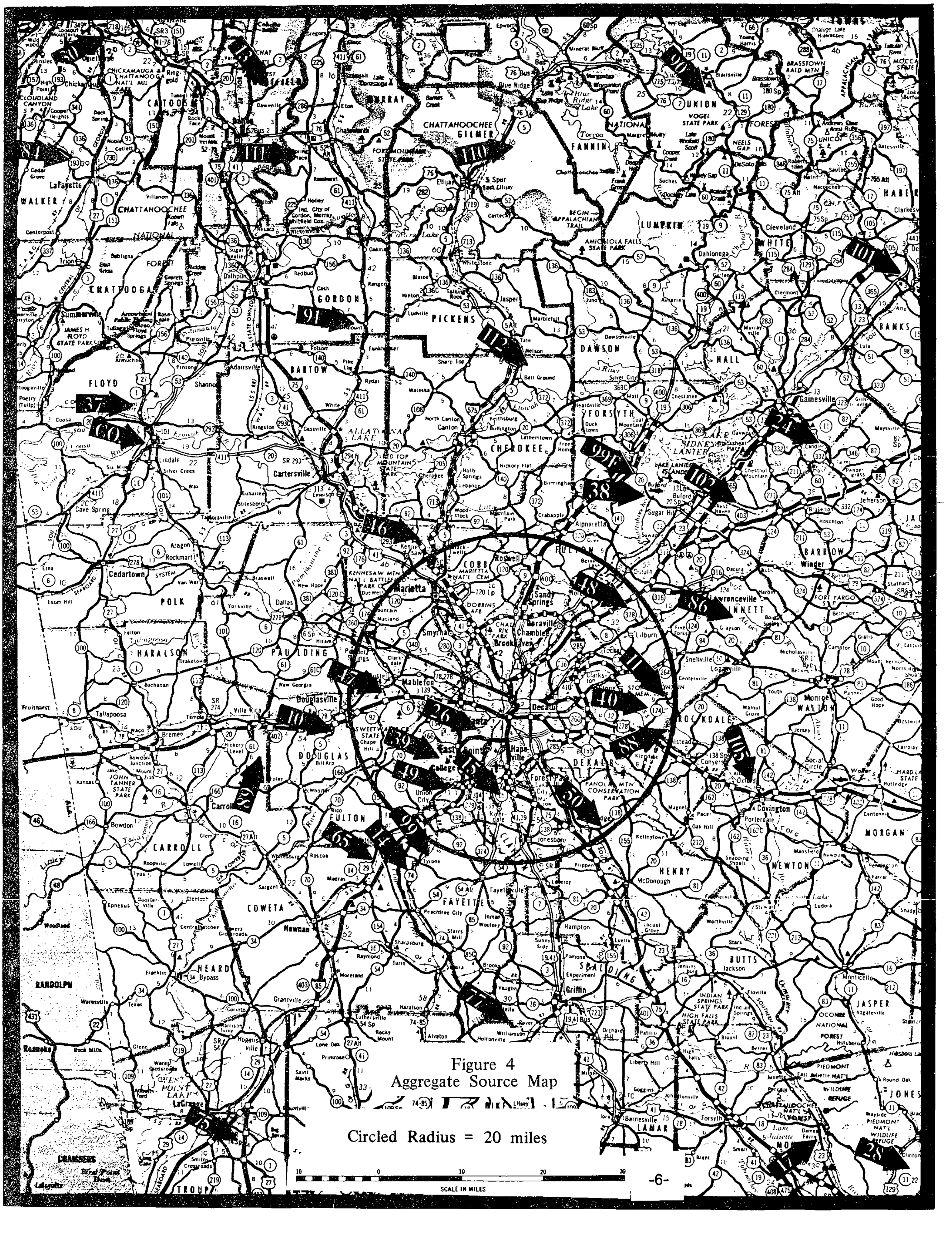


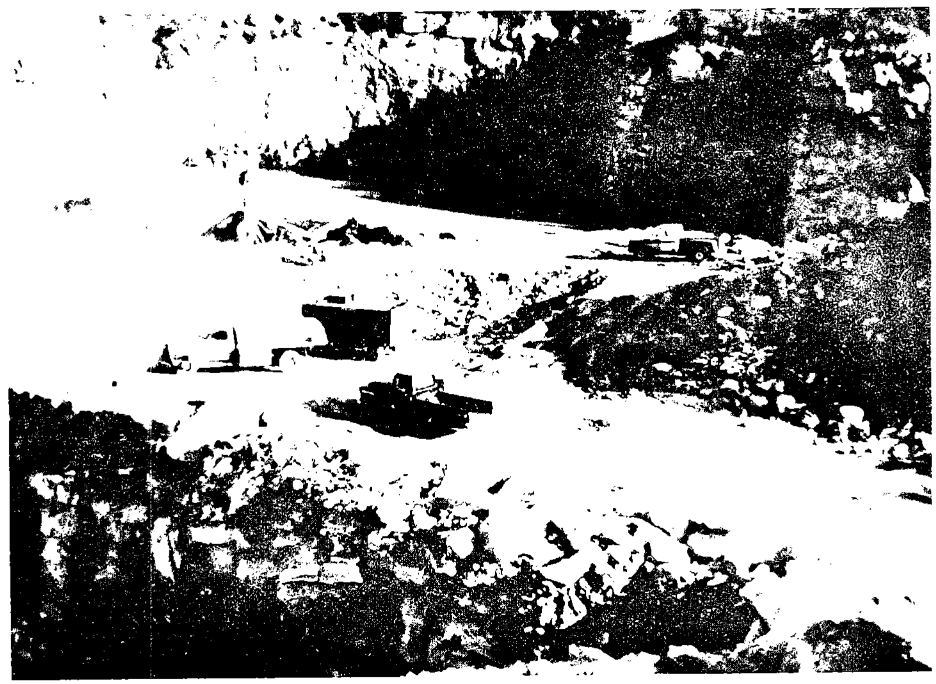
Figure 4
Aggregate Source Map

Circled Radius = 20 miles

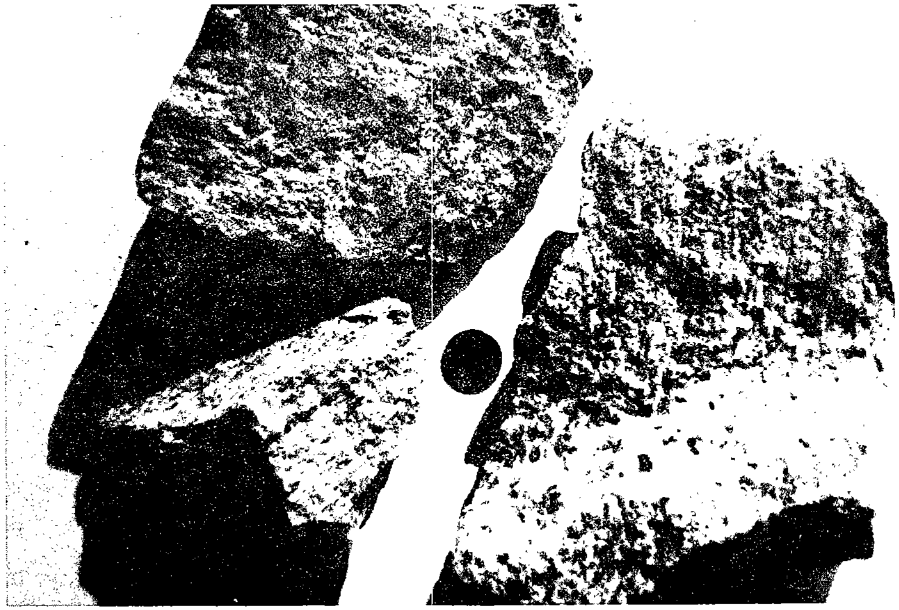
Source 11
Overview of Georgia Marble Quarry - Lithonia, Georgia

Source 11
Contorted Lithonia Granite Gneiss

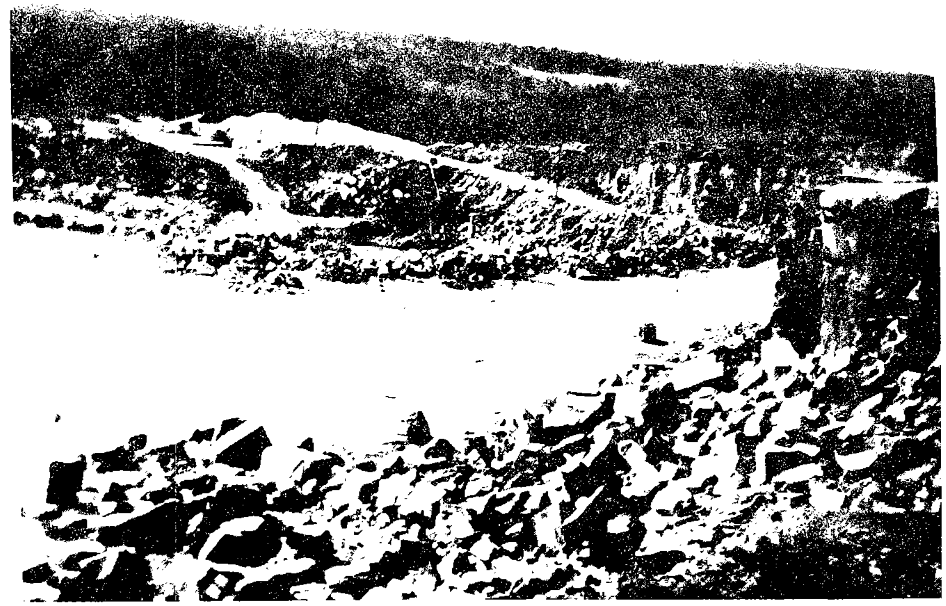
Source 11
Noncontorted Lithonia Granite



Source 109
Overview of Barrow Stone Co. - Auburn, Ga.



Source 109
Close-Up of Samples of Lithonia Gneiss

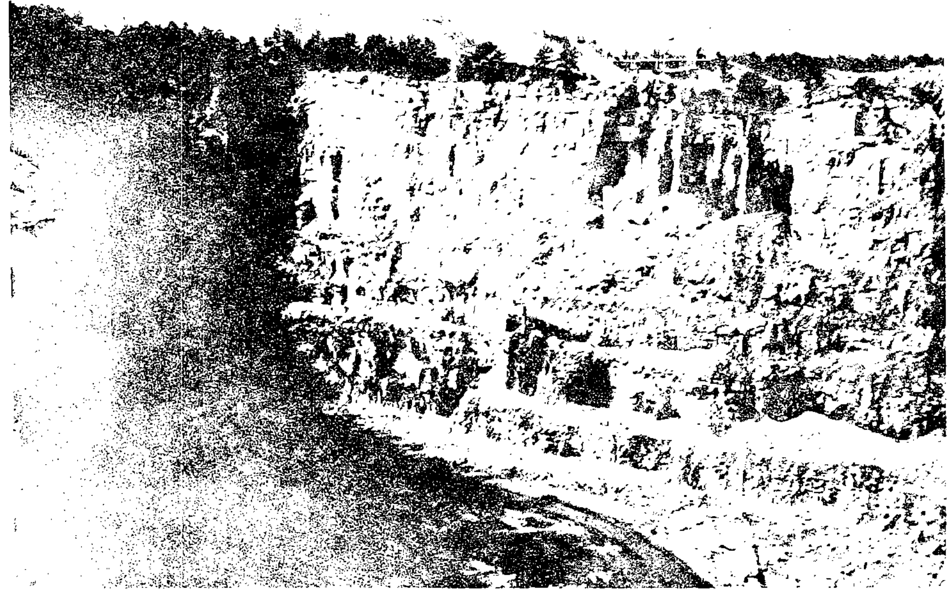


Source 89
Overview of Vulcan Materials Quarry, Villa Rica, Ga.

Source 89
Close Up of Samples of [illegible]

Source 65
View of Quarry Wall at Vulcan Materials "Madras"
[illegible] (Note staining on wall due to

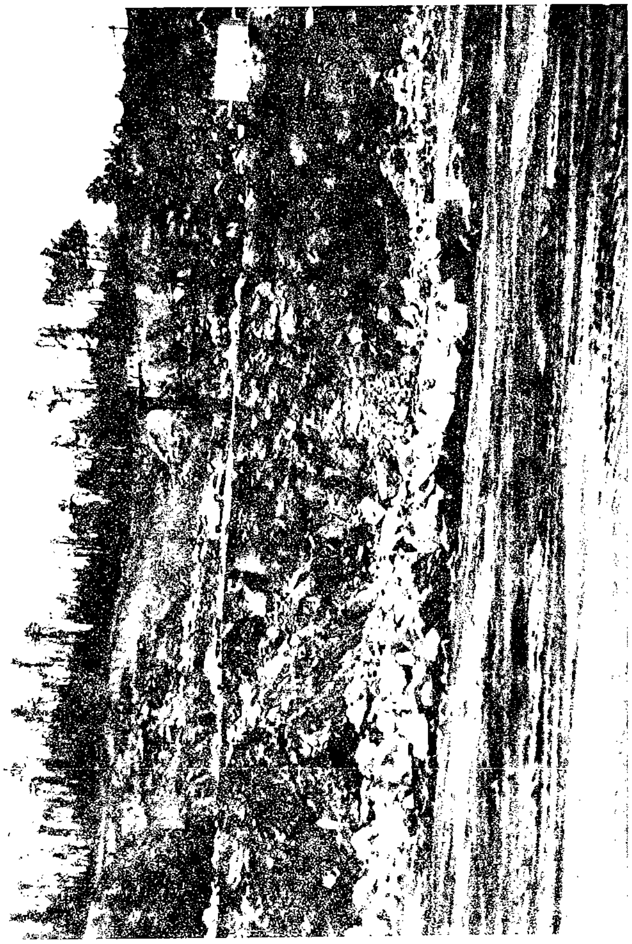
Source 65
Close-Up of Porphyritic Palmetto Granite



Source 14
Overview of Florida Rock, Inc. Quarry - Tyrone, Ga.

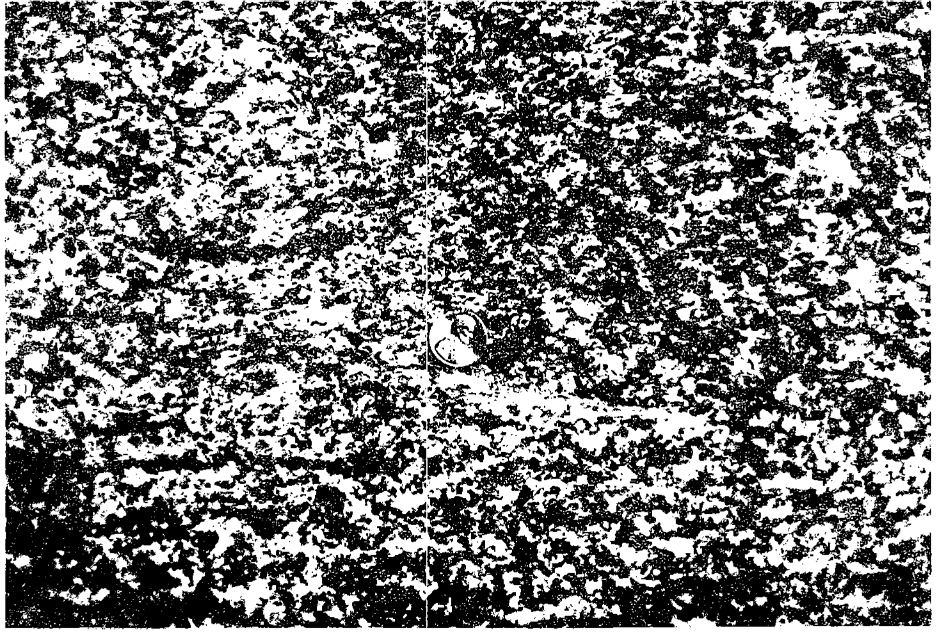
Source 14
Close-Up of Porphyritic Biotite Granite

Source 99
Overview of Davidson Mineral Properties Quarry
Tyrone, Ga.
(note new excavation in floor)

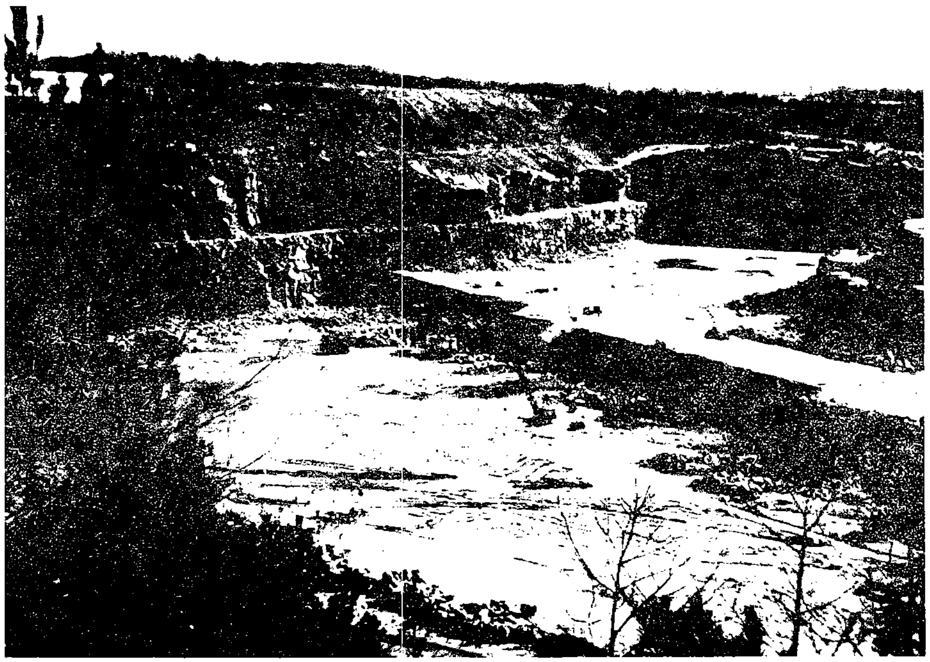


Source 46

Source 49



Source 46
Close-Up of Hornblend-Rich Kennesaw Granite



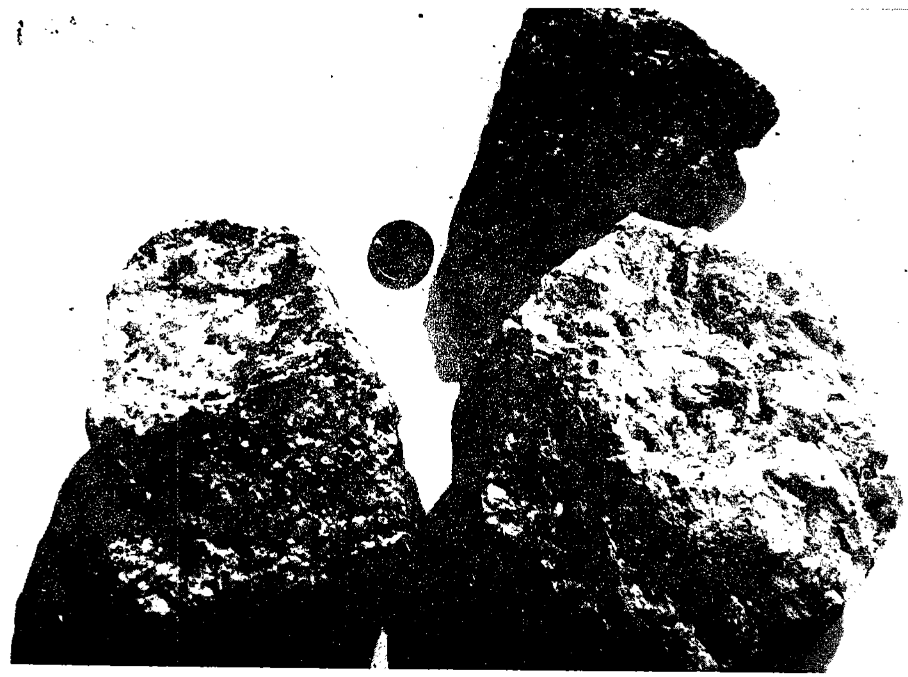
Source 48
Overview of Vulcan Materials Quarry - Norcross, Ga.

Source 48
Norcross Granite Gneiss Intruded by Diabase Dike
(Note: Dike about 18" to 24" wide)

Source 48
Amphibolite Zenolith

Source 48
Close-Up Of Norcross Granite Gneiss

Source 38



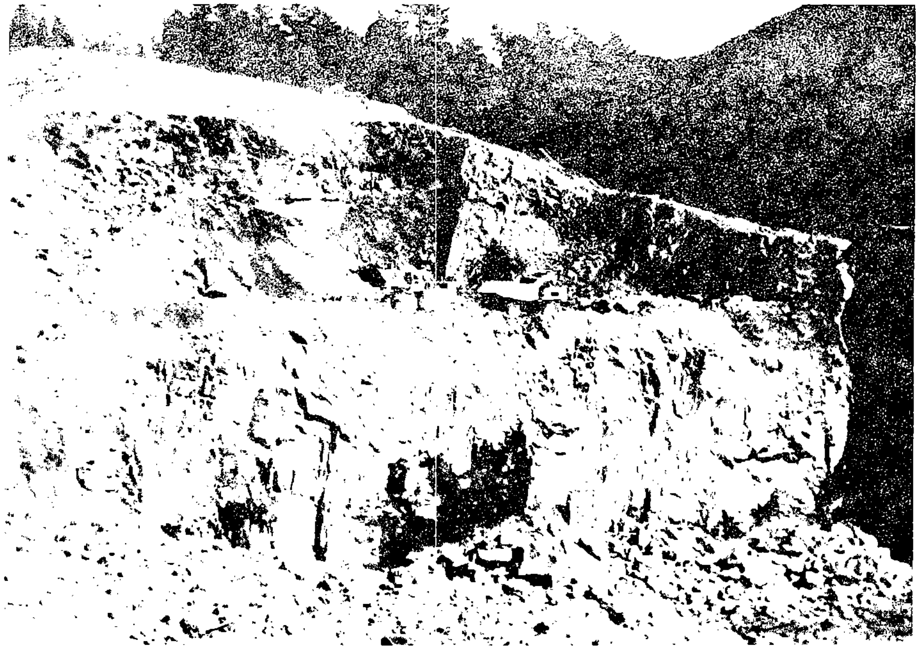
Source 38

Source 38
Close-Up of Granite, Schistose Pegmatitic Granite, Amphibolite, and Injection Hornblende Gneiss

Source 38
View of Georgia Marble Co., "Saprolytic" Sand Deposit, Cumming, Ga.

Source 38
Close Up of Saprolytic Specimens

Source 38
Close Up of Processed Sand



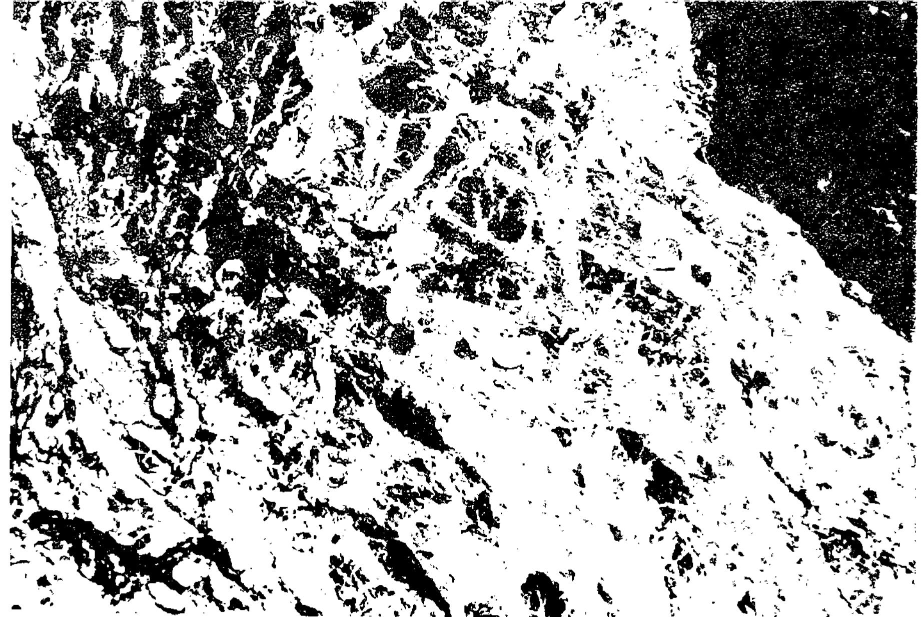
Source 110
View of Colwell Quarry, Ellijay

Source 110
Close-up of Prevalent Rock Types: Clockwise From Bottom Right. Conglomeritic Quartzite, Fine Grained Dark Quartzite, Pyritiferous Biotite Schist, and Arkosic Quartzite.

Source 91
[illegible] Rock Products (Limestone)

Source 91
Close-Up of Wall Showing Calcite-Filled Fractures

Source - County Owned
View of High Wall at Floyd County Quarry
Rome, Ga.



Source - County Owned
Close-Up of Typical Conasauga Limestone with Its Usual White Calcite-Filled Cracks (Note Quarter in middle of photo for size relationship)

Source 112
Overview of Georgia Marble Co., Ball Ground Quarry

Source 112
Close-Up - Large Block of Marble with Calcite Vein. Note: Penny in center of Photo for size Relationship

An aggregate producer seeking DOT approval should make a written request to the Office of Materials and Research to be placed on the Qualified Products List. An investigation of the aggregate source would then be made by the Aggregate Control Branch from a Geologicial and quality standpoint. This would be followed up by a meeting between the DOT and the Quarry's Quality Control Personnel to discuss and agree upon a viable Quality Assurance Program, including the type, number and frequency of tests to be performed. Both the test technician and the testing facilities must be DOT certified.

Figure 5, Page 16, shows an example of a typical quarry map.
Figure 6, Page 17, shows an example of a typical flow diagram
Figure 7, Page 18, Page taken from Approved Fine Aggregate List
Figure 8, Page 19, Page taken from Approved Coarse Aggregate List

Our Aggregate Control Branch Inspectors who live in various parts of the State have the responsibility of monitoring our Quality Control Program.

Under this program the burden of testing is placed on the Aggregate Producer. This has lowered our cost per ton in certifying aggregate, and is a savings to the taxpayers.

Our field inspectors will, from time to time, sample and test aggregates to insure that the quarry technician is doing his job right.

Quality assurance programs tend to make the producer more quality conscious and as a result the State is receiving high quality specification materials with minimal rejections. These are a few of the advantages we share with the aggregate producer.

This program has been in effect for several years now and the fact is it really works for all of us!

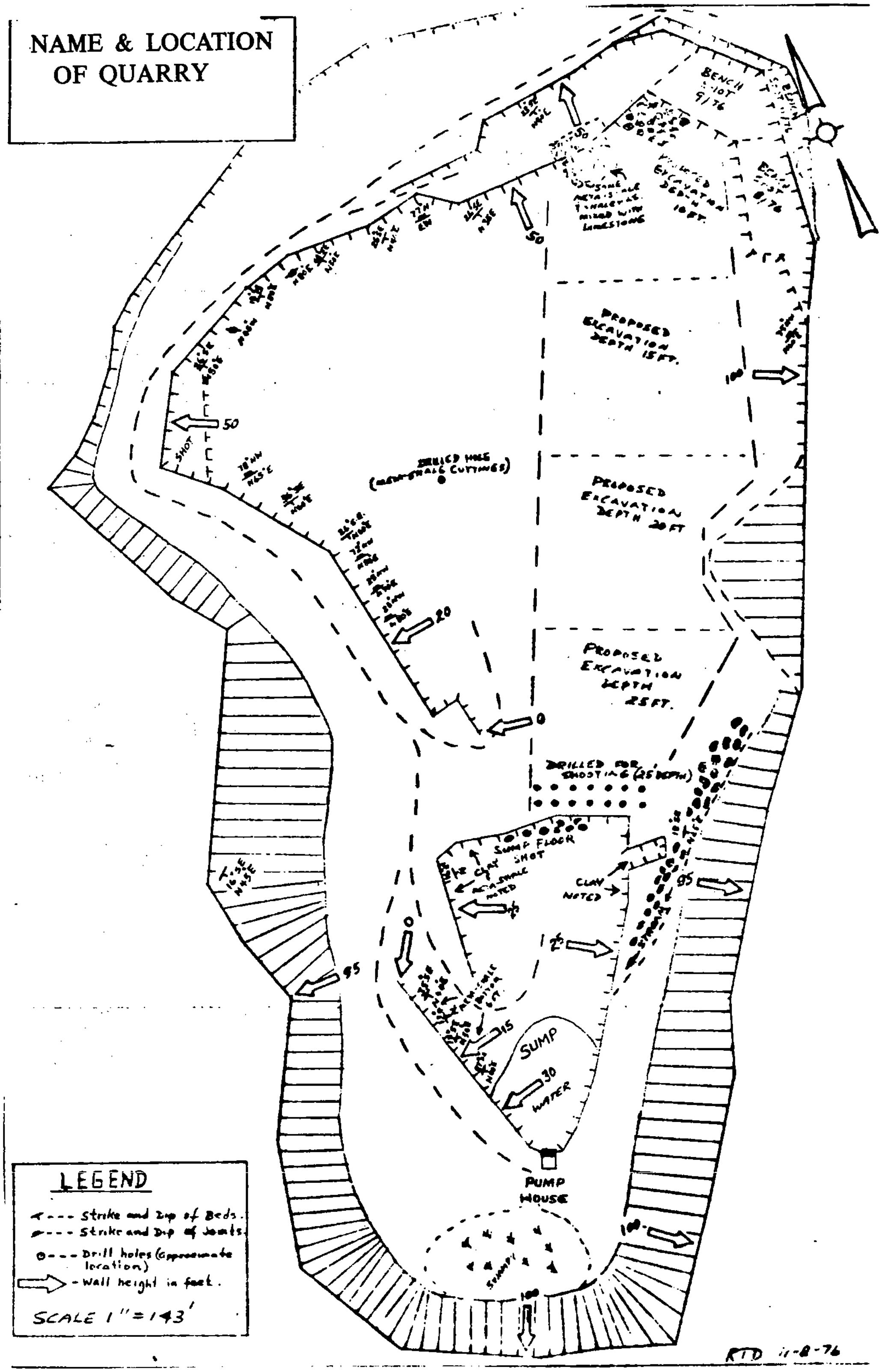


Figure 5
Example of A Typical Quarry Map

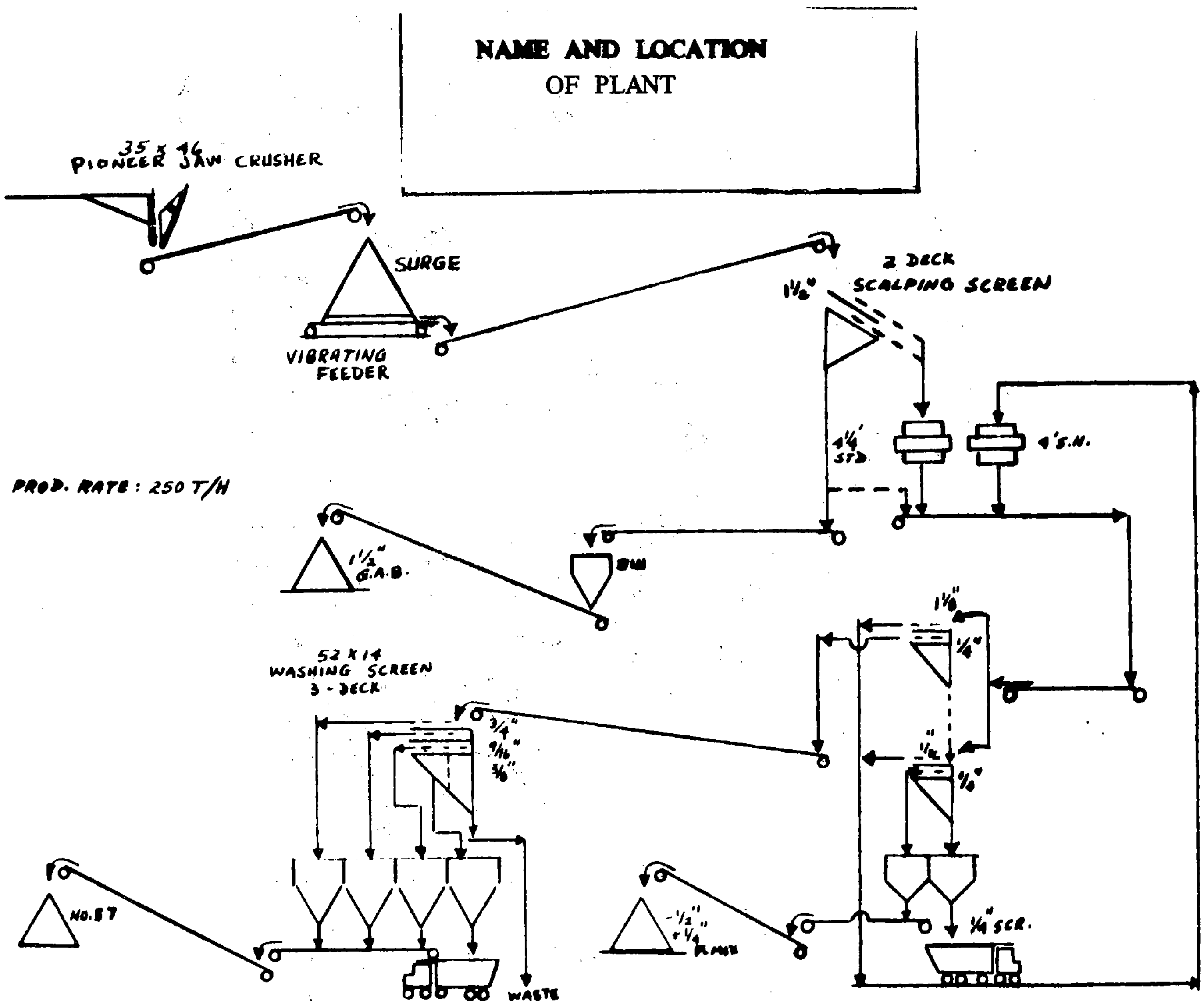


Figure 6
Example of a Typical Flow Diagram

FINE AGGREGATE

July 1, 19[illegible]

QUALIFIED PRODUCTS LIST 1

PRODUCER AND LOCATION OF SOURCE	DOT [illegible] CODE [illegible]	CHARACTER OF MATERIAL	DOT 31? CODE	AGGREGATE GROUP	SPECIFIC GRAVITIES DOT 310 CODE			% ABSORP.	SAND EQUIVALENT VALUE	DURABILITY FACTOR	REMARKS	APPROVED QUALITY CONTROL PROGRAM
					BULK	S.S.D.	APP.					
Consolidated Quarries Division of Georgia Marble Co. Douglasville, GA	016	Granite Gneiss Manufactured Sand	2	II	2.57	2.[illegible]	2.65	1.14	[illegible]	[illegible]	Note 2	Yes
Lithonia, GA	017	Granite Gneiss Manufactured Sand	2	II	2.5[illegible]	2.61	2.6[illegible]	.90	92	[illegible]	Note 2	Yes
Cornell-Young Elberta, GA	018	Alluvial Sand	1	II	2.5[illegible]	2.61	2.64	.82	79	94		Yes
Franklinton, GA	019	Alluvial Sand	1	II	2.57	2.60	2.65	1.11	[illegible]	93		Yes
Tobesofkee, GA	020	Alluvial Sand	1	II	2.61	2.63	2.65	.53	[illegible]	95		Yes
L. C. Curtis & Son Watkinsville, GA	022	Alluvial Sand Residual Sand	1	II	2.57	2.61	2.6[illegible]	1.17	93	[illegible]		Yes
Dalton Rock Products Co. Cleveland, TN	004	Limestone Manufactured Sand	3	I	2.64	2.67	2.73	1.23	[illegible]	78	Note 1	Yes
Dalton, GA	[illegible]	Limestone Manufactured Sand	3	I	2.65	2.68	2.74	1.14	94	93	Note 1	Yes
Davidson Mineral Properties, Inc. Lithonia, GA	06[illegible]	Biotite Granite Gneiss Manufactured Sand	2	II	2.60	2.62	2.65	.[illegible]	93	93	Note 2	Yes
Dawes Silica Mining Co. Eden, GA	[illegible]	Alluvial Sand	1	II	2.62	2.63	2.65	.[illegible]	[illegible]	[illegible]		Yes
Deerfield Sand & Mining Co. Tillman, SC	[illegible]	Alluvial Sand	1	II	2.61	2.62	2.65	.66	97	97		Yes
Dixie Sand & Gravel Co. Chattanooga, TN	031	Alluvial Sand	1	II	2.54	2.58	2.68	2.19	95	94		Yes
Evans Concrete Co. Daisy, GA	035	Alluvial Sand	1	II	2.63	2.64	2.65	.[illegible]	94	96		Yes
Florida Rock Industries, Inc. Tyrone, GA	[illegible]	Granite Gneiss Manufactured Sand	2	II	2.64	2.65	2.67	.[illegible]	93	92	Note 2	Yes
Gainesville Stone Co. Candler, GA	[illegible]	Granite Gneiss Manufactured Sand	2	II	2.5[illegible]	2.60	2.63	.[illegible]	[illegible]	90	Note 2	Yes
Great Southern Aggregates Adel, GA	1[illegible]	Alluvial Sand	1	II	2.63	2.64	2.6[illegible]	.[illegible]	96	95		Yes

NOTE 1: Limestone sand may be excluded from bridge deck concrete on high volume roadways because of aggregate polishing characteristics.
NOTE 2: Approximate mixes for air-entrained portland cement concrete pavement using this fine aggregate have been designed by the Central Laboratory.

Figure 7
Page Taken from Approved Fine Aggregate List

COARSE AGGREGATE

[illegible] LIST 2

[illegible] 1, [illegible]

PRODUCER AND LOCATION OF SOURCE	[illegible] CODE MAP NUM	CHARACTER OF MATERIAL	[illegible] 119 CODE	[illegible] GROUP	SPECIFIC GRAVITIES [illegible] CODE: BULK	S.S.D.	APP.	% ABSORP.	% WEAR	CLASS	[illegible] SULFATE SOUNDNESS LOSS	REMARKS	APPROVED QUALITY CONTROL PROGRAM
Patton [illegible] Co. LaFayette, GA	[illegible]	Limestone	3	I	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	A	[illegible]	Note 1	Yes
R & S Materials Mt. [illegible], AL	[illegible]7	Alluvial Gravel	1	[illegible]	[illegible]	[illegible]	[illegible]	.44	43	A	[illegible]	Note 1A	Yes
[illegible] Quarries, Inc. [illegible], GA	[illegible]	Granite Gneiss	2	II	[illegible]	[illegible]	[illegible]	.63	44	A	[illegible]		Yes
Radcliff Materials, Inc. Chattahoochee, FL	[illegible]	Alluvial Gravel	1	II	[illegible]	[illegible]	[illegible]	.47	[illegible]	A	[illegible]	Note 1A	Yes
Scott Brothers Construction Co. Toccoa, GA	[illegible]	Granite Gneiss	2	II	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	A	[illegible]	Note 1	Yes
Southern Aggregates Co. Appling, GA	[illegible]	Granite Gneiss	2	II	[illegible]	[illegible]	2.67	[illegible]	[illegible]	A	[illegible]	Note 2	Yes
Bostwell, GA	[illegible]	Granite Gneiss	2	II	2.71	2.72	[illegible]	[illegible]	[illegible]	A	[illegible]	Note 2	Yes
The Stone Man, Inc. [illegible]ville, GA	[illegible]	Limestone	3	I	2.71	2.78	2.73	[illegible]	[illegible]	A	[illegible]		Yes
[illegible], TN	[illegible]	Limestone	3	I	[illegible]	[illegible]	2.72	[illegible]	[illegible]	A	[illegible]	Note 8	Yes
Vulcan Materials Co. [illegible], GA	[illegible]	Granite Gneiss	2	II	[illegible]	[illegible]	[illegible]	.67	[illegible]	A	[illegible]	Note 6	Yes
Chattanooga, TN	[illegible]	Limestone	3	I	[illegible]	[illegible]	[illegible]	.61	[illegible]	A	3.100	Note 1	Yes
Grayson, GA	006	Granite Gneiss	2	II	[illegible]	2.64	[illegible]	[illegible]	[illegible]	B	[illegible]	Note 2	Yes
Kennesaw, GA	046	Granite Gneiss	2	II	2.77	2.79	2.81	[illegible]	[illegible]	A	[illegible]	Note 2	Yes
LaGrange, GA	075	Granite Gneiss	2	II	2.61	[illegible]	2.65	[illegible]	[illegible]	A	[illegible]	Note 2	Yes
Liberty, SC	[illegible]7	Granite Gneiss	2	II	[illegible]	2.66	[illegible]	[illegible]	[illegible]	B	[illegible]	Note 1	Yes
Lithia Springs, GA	047	Granite Gneiss	2	II	2.61	[illegible]	2.64	[illegible]	[illegible]	A	[illegible]65		Yes
McCollum, GA	065	Granite Gneiss	2	II	2.67	[illegible]	2.71	[illegible]	[illegible]	A	[illegible]	Note 2	Yes
Norcross, GA	[illegible]	Granite Gneiss	2	II	2.70	2.71	2.74	.47	[illegible]	B	[illegible]	Note 2	Yes
Red Oak, GA	[illegible]	Granite Gneiss	2	II	2.67	[illegible]	2.70	[illegible]	57	A	[illegible]		Yes
Villa Rica, GA	[illegible]	Muscovite Gneiss	2	II	[illegible]	2.67	[illegible]	[illegible]	31	A	3.167	Note 2	Yes
Stockbridge, GA	050	Granite Gneiss	2	II	[illegible]	[illegible]	[illegible]	.52	43	A	1.067	Note 2	Yes

NOTE 1: Asphaltic concrete design analyses covering every type of mix have not been made using this source of aggregate by the Central Laboratory.
NOTE 1A: No current asphaltic concrete design analysis has been made using this source of aggregate by the Central Laboratory.

NOTE 2: Approximate mixes for air-entrained portland cement concrete pavement using this source have been made by the Central Laboratory.

NOTE 3: Specific gravity and absorption are to be determined on samples taken from material actually delivered to the project.

NOTE 4: Contains excess sulfur computed as sulfide sulfur. Not to be used in bridge type structures.

NOTE 5: Contains excess local detrimental substances (reactive chert), or other constituents which have been found to produce pop-outs in exposed structural concrete. Not to be used in portland cement concrete.

NOTE 6: Material at times exceeds abrasion loss limits of Class A. Verification of class will be made when Class A material is [illegible]. Soundness average is approximate. Studies are in progress for evaluation based on [illegible].

NOTE 7: Material at times exceeds the abrasion loss limits of Class [illegible]. [illegible] of quality must be made prior to use.

NOTE 8: Mechanically graded rip-rap can be furnished from this source.

Figure 19

Page Taken From Approved Coarse Aggregate List

GEORGIA STABILIZED EMBANKMENT WALL CONSTRUCTION

By
Warren Bailey
Georgia Department of Transportation

Introduction

In the summer of 1982, consturction was begun on the first Georgia Stabilized Embankment (GASE) wall. Since that date thirteen GASE walls have been constructed. Twelve additional GASE walls are currently under construction and approximately sixty GASE walls have been let to contract. In this paper I would like to explain what a GASE wall is, how it works, why the Georgia Department of Transportation (Ga. DOT) developed GASE, and the instrumented GASE wall that has just been construction.

What is GASE?

GASE is an earth retaining system developed by the Ga. DOT. The main components of the system are face panels, wire mesh stabilizer mats and backfill material put together to form an earth retaining system as shown in Figure 1. There are other minor components in the wall but will not be discussed here. The method of construction is to set the first row of face panels in position on a leveling pad and to hold them in position temporarily with braces and clamps. The wire mats are then attached to the back of the face panels and a layer of granular backfill placed over the mats and compacted. Panels are set upon panels and the process repeated until the wall is finished.

The basic face panel for the GASE wall is shown in Figures 2 and 3. Panels can be modified in the precasting stage to various shapes to fit geometrical conditions of the wall. The panels have built-in lugs for lifting.

The stabilizing mat is a 64,000 psi welded wire stell mesh approximately 2' wide. There are four 3/8 inch diameter longitudinal bars. Transverse bares are 3/8 inch diameter, 2 feet long and generally located on two foot centers. There are four uniformly spaced mats per face panel. The mats are attached to the face panel as shown in Figures 3 and 4.

The backfill material for the wall consists of a free draining granular material. Clean sands and crushed stone with low corrosion potential are commonly used. In the Atlanta area it is common to see a No. 4 crushed granite stone used as backfill material.

How Does GASE Work?

Currently, the Ga. DOT has no means of analysis that will explain how a layered system of wire mesh stabilizes an embankment. We do know that it works. The Ga. DOT currently utilizes a modification of existing theory for the design of GASE. The basic principle behind the design is similar to a dead man system. Instead of a mass as an anchor, several stabilizer mats are used as anchors.

The method of design is to balance the force acting on the panels with the anchoring force of the mats. The first step in the design procedure is to assume a failure wedge exists behind the face panels. The pressure developed against the face panel is then determined by a Rankine analysis. The unit resistant force that can be developed by the stabilizer mat has been derived from large scale laboratory tests performed by California Department of Transportation (CALTRANS). By analysing the forces behind the wall, it can be shown that the number of cross bars in the mat required behind the theoretical failure wedge is independent of depth. The required length of mesh is then determined by the longest mat required which is generally at the top of the wall. The system is then analyzed for external stability.

Why GASE was Developed

The main reason for developing the GASE system was economics. Prior to 1982 there were a large number of walls constucted in Georgia. The majority of these walls were associated with Atlanta freeway reconstruction. With many more walls to design, the Georgia Department of Transportation decided in the latter part of 1981 to develop its own wall system in an attempt to reduce construction cost.

The stabilized earth system was selected due to its low cost and ease of construction. The wire mesh was selected for a stabilizer mat because of its good field anchoring characteristics as shown in studies made by CALTRANS. The shape of the face panel was developed for aesthetics and ease of installation.

Instrumentation of a GASE Wall

In March of 1982, the DOT requested Law Engineering and Testing Company (LETCO) to develop an instrumentation plan for analysing the performance of GASE walls. The objective of this research was to gain information for improvement of wall design.

The GASE wall chosen for instrumentation was Wall 12 located on I-75 near Northside Drive in Northwest Atlanta. The wall will be retaining the fill for the new location of the northbound exit ramp at Northside Drive. The wall will be approximately 650 feet long and 55 feet high at its highest point. The area for instrumentation was selected between Station 12+50 and 13+50. This area is the highest part of the wall and also has relatively uniform foundation conditions.

The instrumented section and typical instrumentation layout are shown in Figures 5 through 8. The instrumented section consists of two sections. One section will be backfilled with a large coarse stone while the other will be backfilled with a crusher run aggregate. The following legend gives the relationship between letter symbols in these figures and the instrumentation to be installed at location:

A = 38 foot fully gaged mat
B = 36 foot fully gaged mat
C = 24 foot fully gaged mat (pull-out test)
D = 12 foot fully gaged mat (pull-out test)
E = 38 foot tongue only gage mat
F = 36 foot tongue only gage mat
GH = horizontal earth pressure cell
GV = vertical earth pressure cell
EX = multiple point extensometer

The study of this wall will also include full scale pull-out tests on instrumental panels which are shown in the figures as Items C and D. Details are still being worked out for the pull-out tests. The mat to be pulled will be located in the middle of the panel and the surrounding mats instrumented for observation.

Several of these mats will be instrumented with strain gages as shown in mat types A and B. These mats should show us a relationship between overburden stress and stress developed in the mat. Some mats will only be instrumented with strain gages on the tongue only. These will be used generally as control mats for other instrumented mats.

Multiple Point Extensometers will be installed at six locations. The purpose of these devices is to help correlate internal move of the backfill material with the stress distribution within the stabilizer mat.

Earth pressure cells will be installed between the stabilized embankment and the earth fill to determine the lateral earth pressure against the wall. Earth pressure cells will also be installed beneath the stabilized embankment to determine the distribution of foundation pressures.

Instrumentation to be installed by the DOT is not shown. Slope inclinometers will be installed in the backfill and behind the backfill to help further correlate lateral movements. Survey points will be established to monitor settlement and movement.

The instrumentation will be monitored as the wall is constructed and after construction until the internal stresses have stabilized and internal and external movements have ceased.

Conclusion

The GASE system appears to have been well accepted by contractors. Current bid prices show that GASE wall is being bid significantly lower than similar systems.

The data obtained from the instrumented section will be used to make improvements in design and, if necessary, materials used in GASE construction. We expect the improvements in the design to also improve the economics of the system. The results of this research should also give us direction for further research.

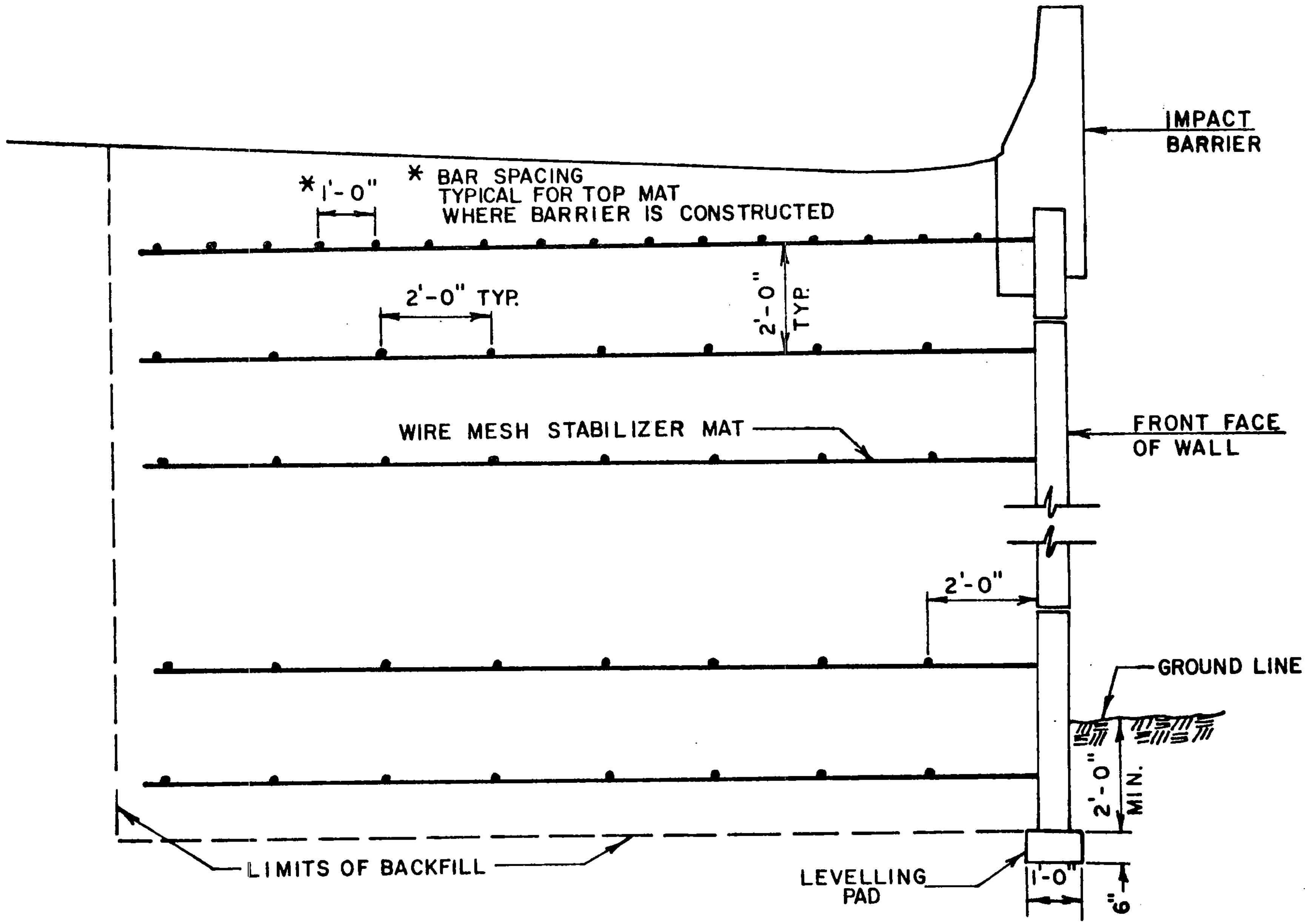


FIGURE I: TYPICAL CROSS SECTION OF GASE

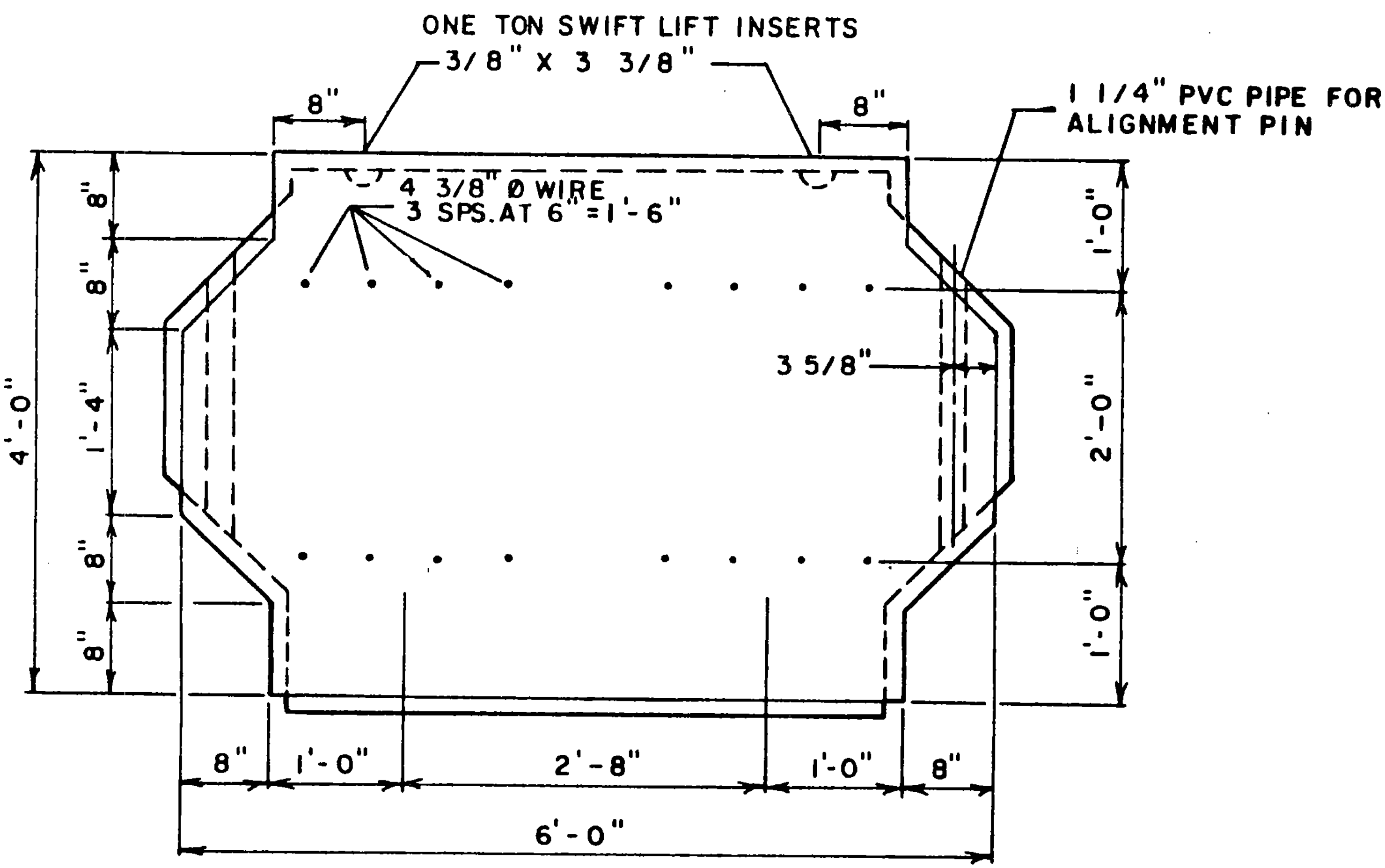


FIGURE 2: STANDARD TYPE A PANEL

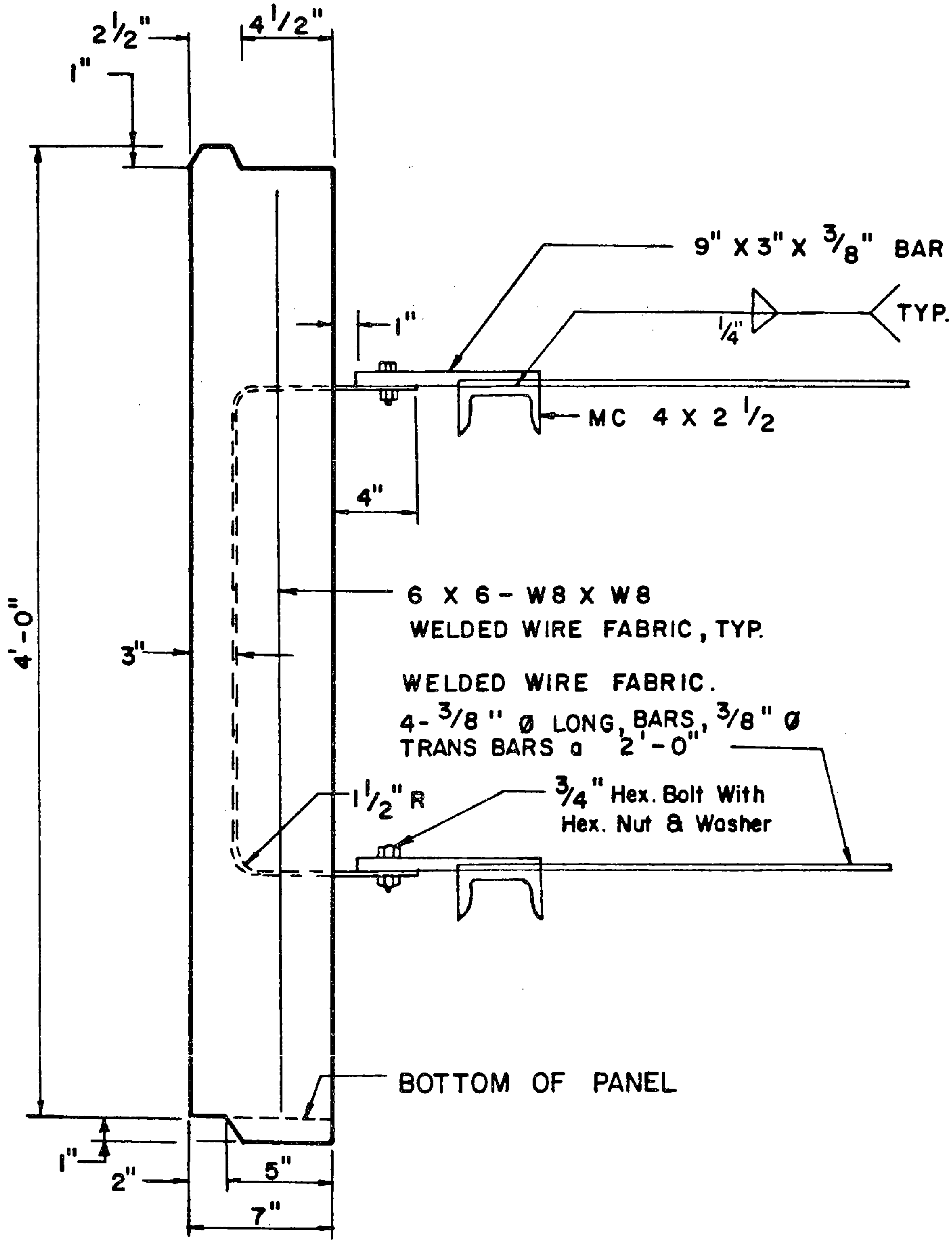


FIGURE 3: TYPICAL PANEL CROSS SECTION AND
AND MAT CONNECTION

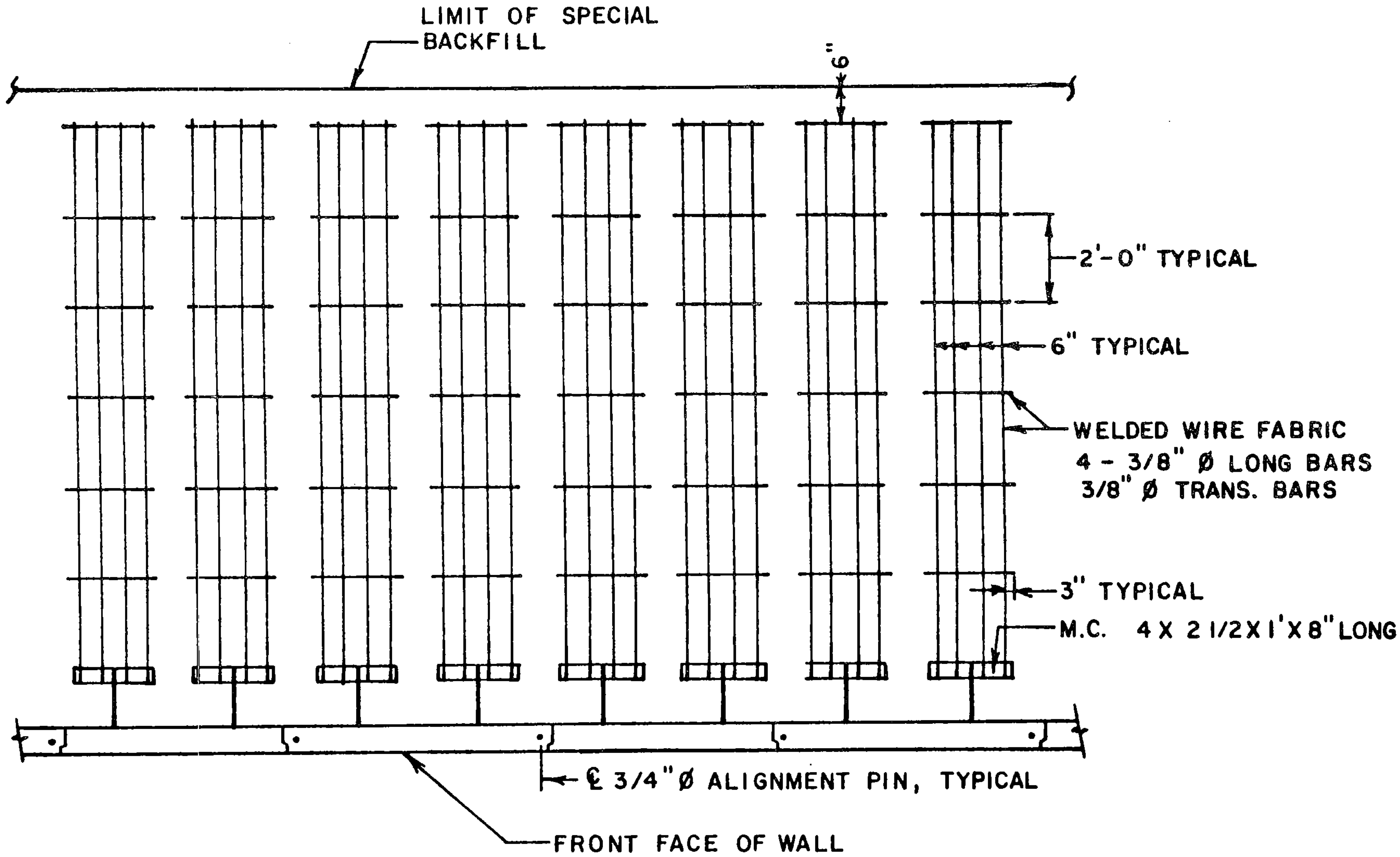


FIGURE 4: PLAN VIEW OF TYPICAL MAT ARRANGEMENT

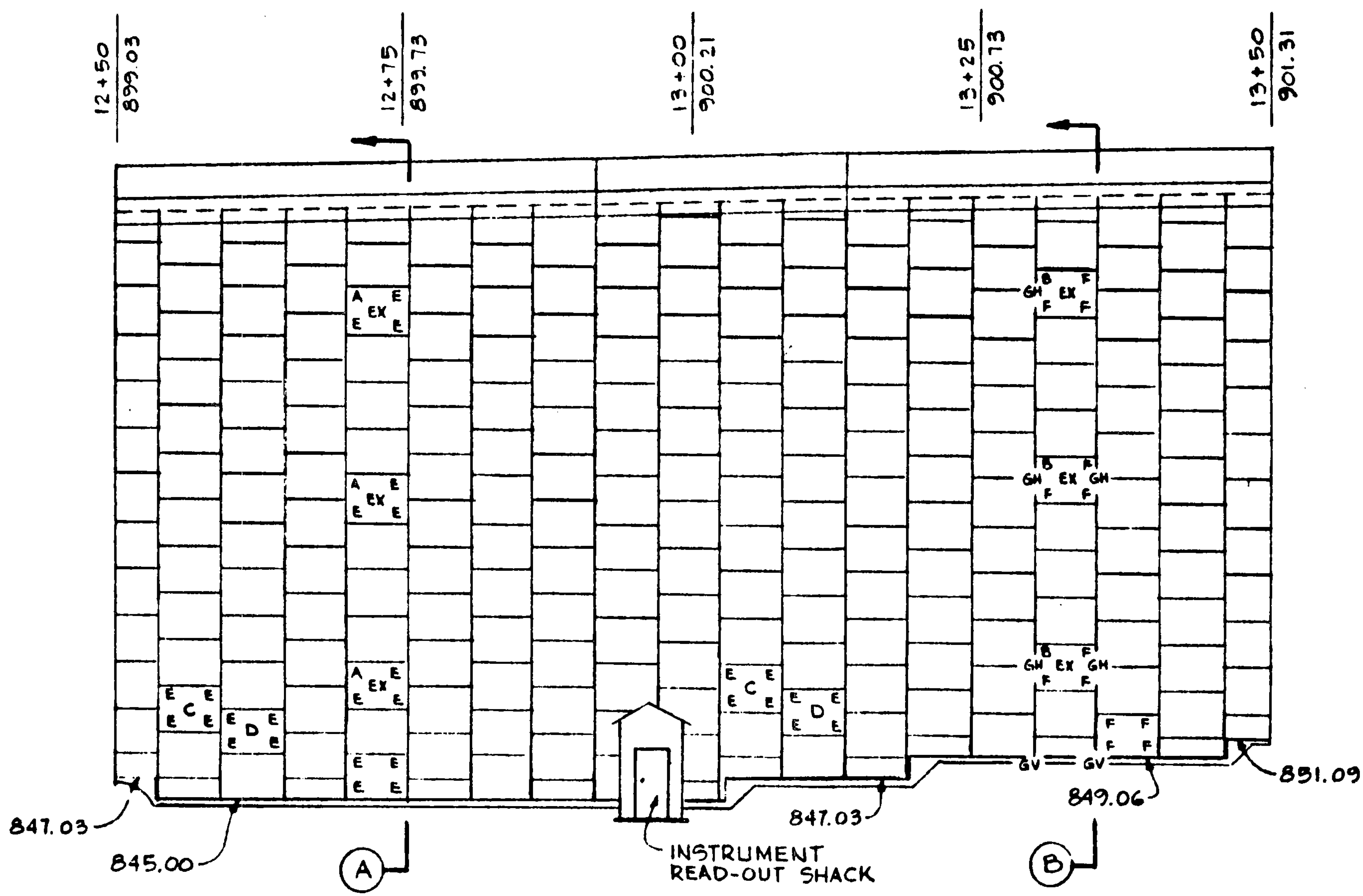


FIGURE 5: FRONT ELEVATION OF WALL SHOWING INSTRUMENTATION LOCATION

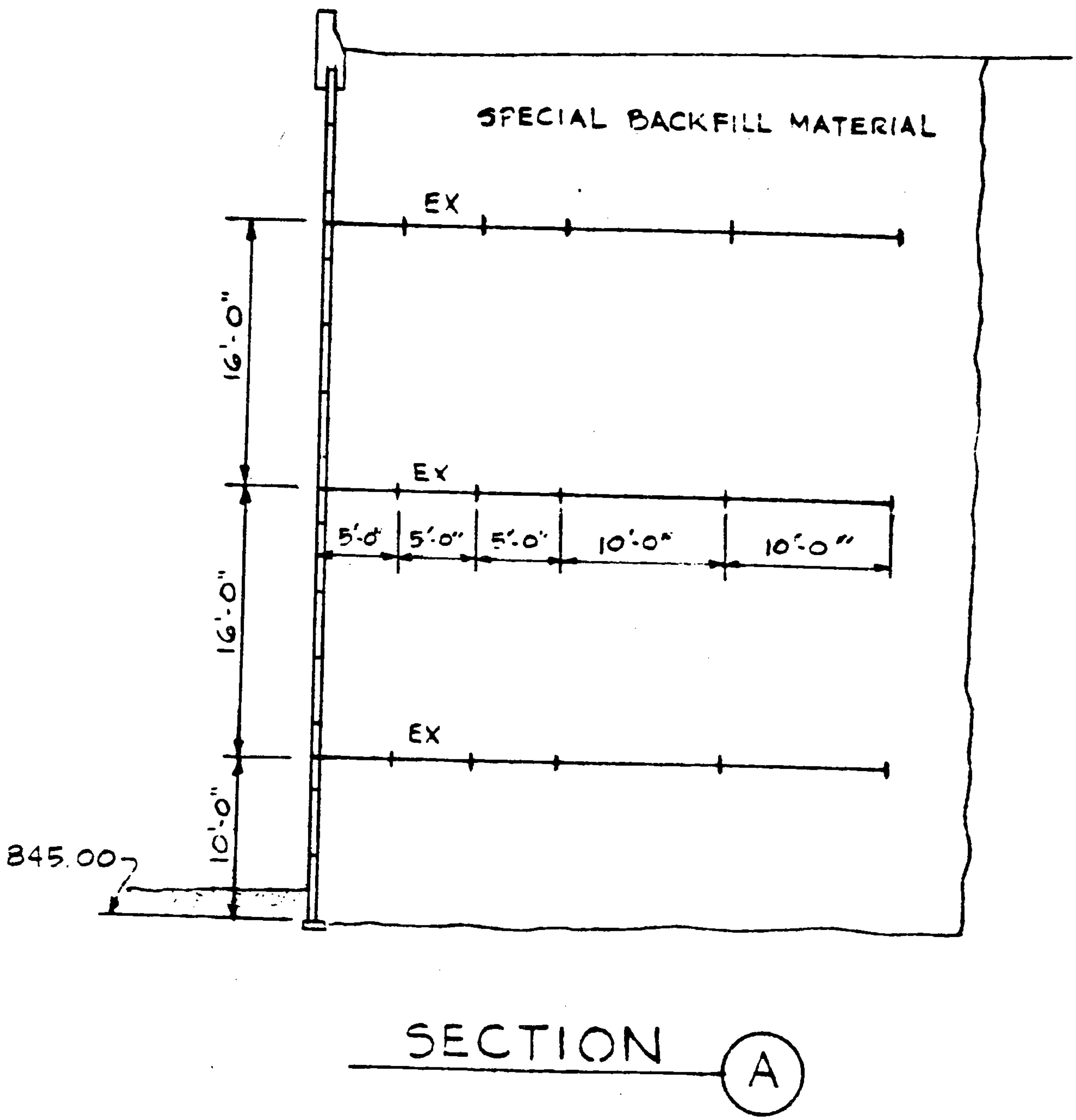


FIGURE 6: INSTRUMENTATION LOCATION

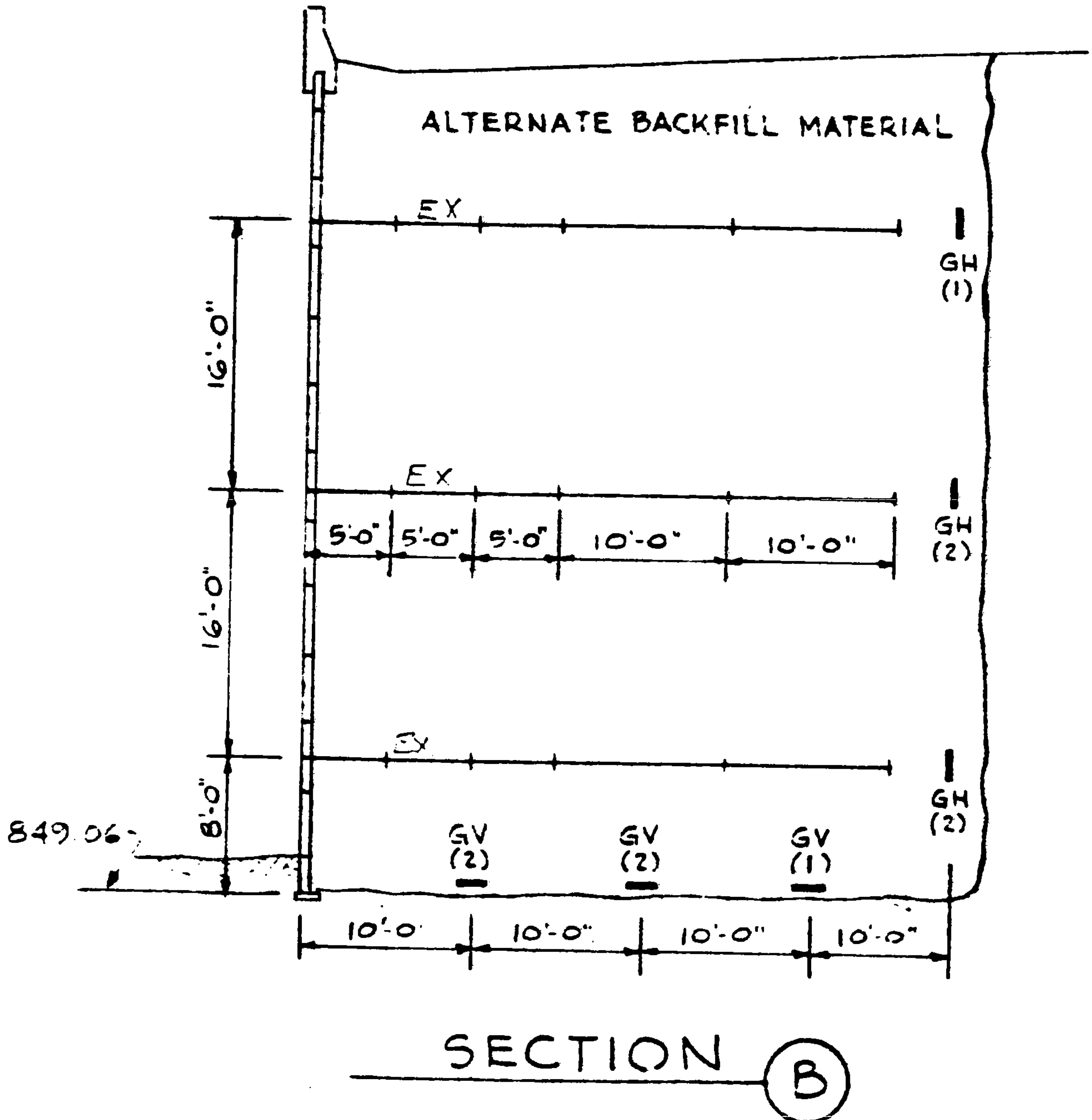


FIGURE 7: INSTRUMENTATION LOCATION

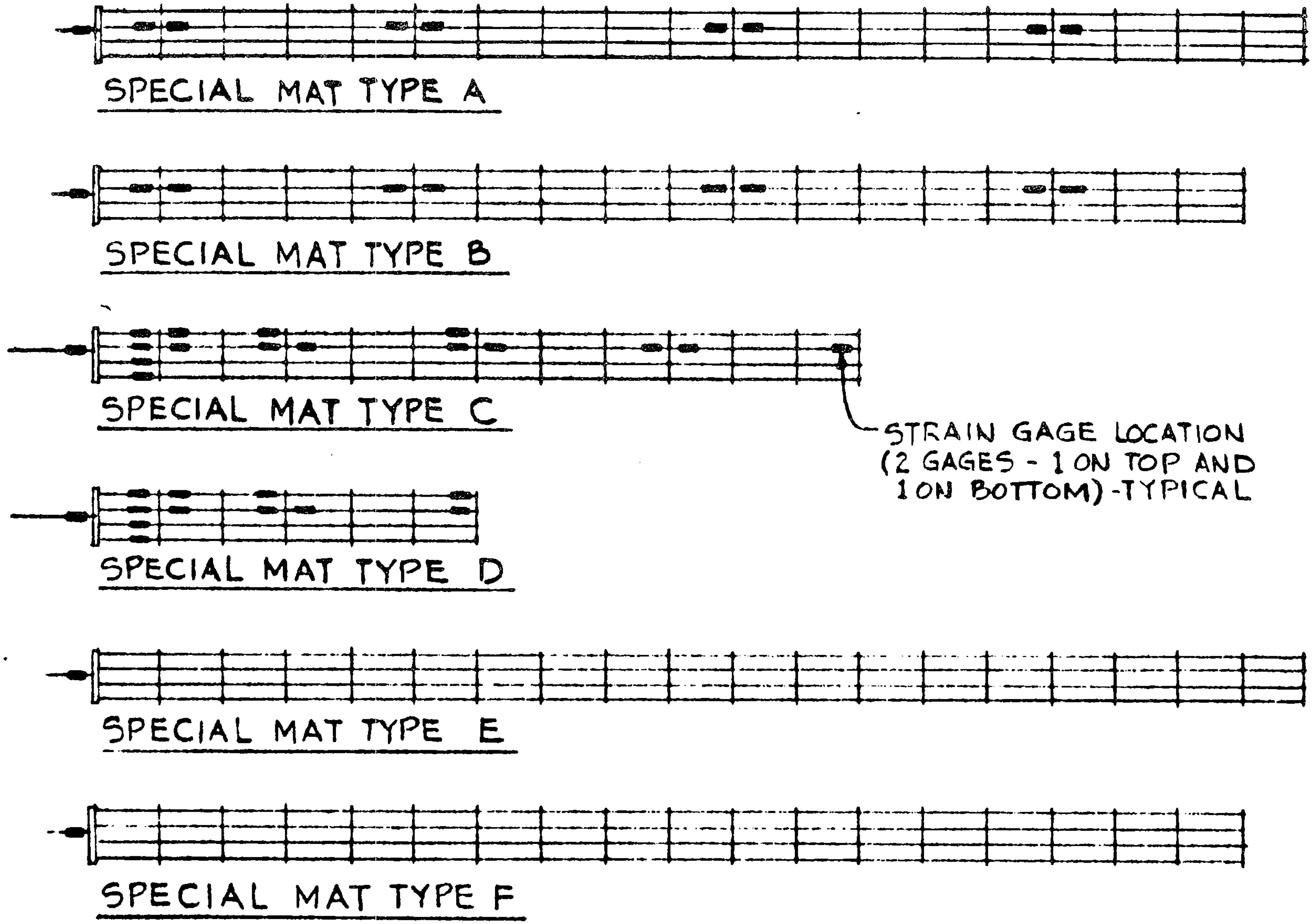


FIGURE 8: LOCATION OF STRAIN GAGES ON MATS

Geologic and Economic Aspects Regarding the Development of an Underground Limestone Mine, Indianapolis, Indiana

by T.R. West and M.R. Fein*
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ABSTRACT

An underground limestone mine owned by Martin Marietta Corporation has been in operation since November 1981 in Indianapolis, Indiana. Accomplished by room and pillar mining, this is the first underground mine in Indianapolis. A decline tunnel on a 25% slope extends a distance of 622 feet through alluvial and glacial deposits and the New Albany Shale to reach the limestone units that are mined. Two, 22 foot thick sections will eventually be mined from the North Vernon and Jeffersonville Limestones yielding a 75% extraction ratio.

Two limestone quarries provided the primary source of crushed stone for Indianapolis (Marion County) prior to the opening of this underground mine. One quarry is located just north of Marion County on the north side of the city and the other is in the southern part of the county one mile east of the new underground mine. Glacial cover is extensive for Marion County, averaging about 140 feet thick. These two limestone quarries were developed within glacial sluiceways which contained mineable gravels. After the unconsolidated materials were excavated, the quarry operation was begun in the bedrock immediately below.

Ground water inflow is extensive in both of these quarries because of the thick alluvial deposits associated with their location. In addition, the New Albany Shale must be stripped away in the southern quarry involving an increased cost, and some of the black shale may remain with the limestone yielding a weak component in the final crushed stone product.

By contrast, the underground limestone mine has relatively little water inflow and the mine roof is located 8.5 feet below the base of the New Albany Shale. The limestone is good quality, qualifying as Class A stone by Indiana State Highway standards.

The underground mine entrance is located in a compact area of an industrial complex. Adjacent to the decline tunnel is an extensive sanitary landfill which covers the area of the previous gravel extraction operation. Several gravel pits partially filled with water still remain. The mine site is on the west bank of the White River with the property bounding that of a large coal-fired, electric generating plant. The City stock yards and the major sewage treatment plant for Indianapolis are located immediately to the north and major manufacturing facilities lie northward from there. Locating the underground mine in this area of heavy industrial use provides a good example for optimizing land use in such industrial areas within a metropolitan complex. It also, of course, reduces the transportation distance for the crushed stone product.

---

* Current address - Shell Minerals Corporation, Houston, Texas.

INTRODUCTION

Since November, 1981, an underground limestone mine owned by Martin Marietta Corporation has been in operation in Indianapolis, Indiana. Located within three miles of downtown Indianapolis at 2605 Kentucky Avenue, it is the only limestone mine operating within the city limits. Indianapolis, located near the center of the state, makes up nearly all of Marion County.

More than fifteen years ago a study by French and Carr (1967) suggested that a local underground mine should prove economical because of the heavy demand for concrete aggregates in Indianapolis and the extensive unconsolidated overburden existing there. At that time much of the limestone aggregate was supplied by quarries located more than 30 miles away. Subsequently, two limestone quarries were opened much closer to the city to provide the needed crushed stone.

One of the quarries is located just north of Marion County near the northern boundary of Indianapolis in Hamilton County. Operated by American Aggregates Corporation, it is located in the White River floodplain and has 50 to 60 feet of sand and gravel above the bedrock surface (West and Warder, 1983). This overburden is processed for sand and gravel aggregates. The other quarry (also owned by American Aggregates) is located just one mile to the east of the Martin Marietta limestone mine, on the south side of Indianapolis. It has about 70 feet of sand, gravel and glacial till over bedrock, which at this location is the New Albany Shale. That 40-foot thick unit is stripped off in the mining operation to expose the North Vernon Limestone below. Water inflow problems are a major complication to surface mining at the site.

Regarding the underground limestone mine, a feasibility study by Martin Marietta revealed that the mine should prove more profitable than a quarry because of the great thickness of overburden and the excessive amounts of groundwater involved with surface mining in a floodplain area. A gravel extraction operation was already underway at the Kentucky Avenue site by Martin Marietta so that the land was available for exploitation. The crushed stone from the North Vernon Limestone and the Jeffersonville Limestone below it have proved to be Class A stone according to Indiana Highway Department standards. Available reserves for 70 years of production was also a significant factor.

The underground mine entrance is located in a compact area of an industrial complex. Adjacent to the decline tunnel is an extensive sanitary landfill which covers the area of the previous gravel extraction operation. Several gravel pits partially filled with water still remain. The mine site is on the west bank of the White River with the property bounding that of a large coal-fired, electric generating plant. The City stock yards and the major sewage treatment plant for Indianapolis are located immediately to the north and major manufacturing facilities lie northward from there. Locating the underground mine in this area of heavy industrial use provides a good example for optimizing land use in such industrial areas within a metropolitan complex. It also, of course, reduces the transportation distance for the crushed stone product.

The study reported here is based on a portion of a master's thesis completed at Purdue University (Fein, 1983). The purpose of that research was to conduct an engineering geology investigation of the underground limestone mine. This included a geological evaluation of the mine along with an assessment of the mine of stability. The work was supported in part by the U.S. Department of Education, Domestic Mining and Mineral Conservation Fellowship Program.

OVERVIEW OF MINING OPERATIONS

The mine is developed in the middle and lower portions of the North Vernon Limestone plus the upper part of the Jeffersonville Limestone. Mining is accomplished by the room and pillar method with rooms and pillars 40 feet wide. Pillars are square in plan. This layout yields an extraction ratio of 75%. Presently a 22 foot high section is being mined. In the future, a second 22 foot high lift will be removed below the present one, to yield rooms 40 feet wide and 44 feet high.

Access is gained to the mine via a 622 foot long decline tunnel at a 4 to 1 slope (25%). The floor of the tunnel consists of 3.5 feet of reinforced concrete and the sides and back vary from 18 to 42 inches of reinforced concrete.

The invert of the tunnel ends at a depth of 133 feet below the ground surface and 8.5 feet into the North Vernon Limestone. The total overburden at this point consists of 10 feet of fill material, 15 feet of sand and gravel, 43 feet of glacial till and 56 feet of New Albany Shale (Hale, 1982).

The limestone is drilled, blasted and subsequently transported to a primary portable crusher located in the mine. This transportation is by front-end loader to the crusher, where the rock is crushed, and then transported to the surface via conveyor belt in the tunnel. At the ground surface, the stone is stored above the final crusher in the form of a surge pile. The surge pile allows the mine and surface plant to function independently so that interruptions in one does not affect progress in the other.

Figure 1 shows the generalized cross section of the mine. The fill material shown in Figure 1 is from the sanitary landfill operation on the site.

Ventilation is provided by a 96 inch Joy fan located at the top of a vertical shaft positioned 165 feet north of the tunnel entrance. The fan draws air from the mine and can obtain a maximum rate of 190,000 cubic feet per minute.

The mine layout was originally oriented to yield north-south and east-west streets. After only several rooms and pillars were developed it was decided to re-orient the mine at 45 degrees to the north-south direction. This was necessary because of the prominent east-west trending joints which caused spalling of the pillars parallel to the openings (east-west streets).

GENERAL GEOLOGY OF MARION COUNTY

Physiography

The state of Indiana is divided into two major physiographic divisions (Fenneman, 1938). The northern part of the state lies in the Till Plains Section of the Central Lowland Province whereas the southern portion of the state lies in the Highland Rim Section of the Interior Low Plateau Province. The dividing line between the two provinces in Indiana is the boundary marking the furthest advance of Pleistocene glaciers. The study site thus lies in the Till Plains Section of the Central Lowlands Province.

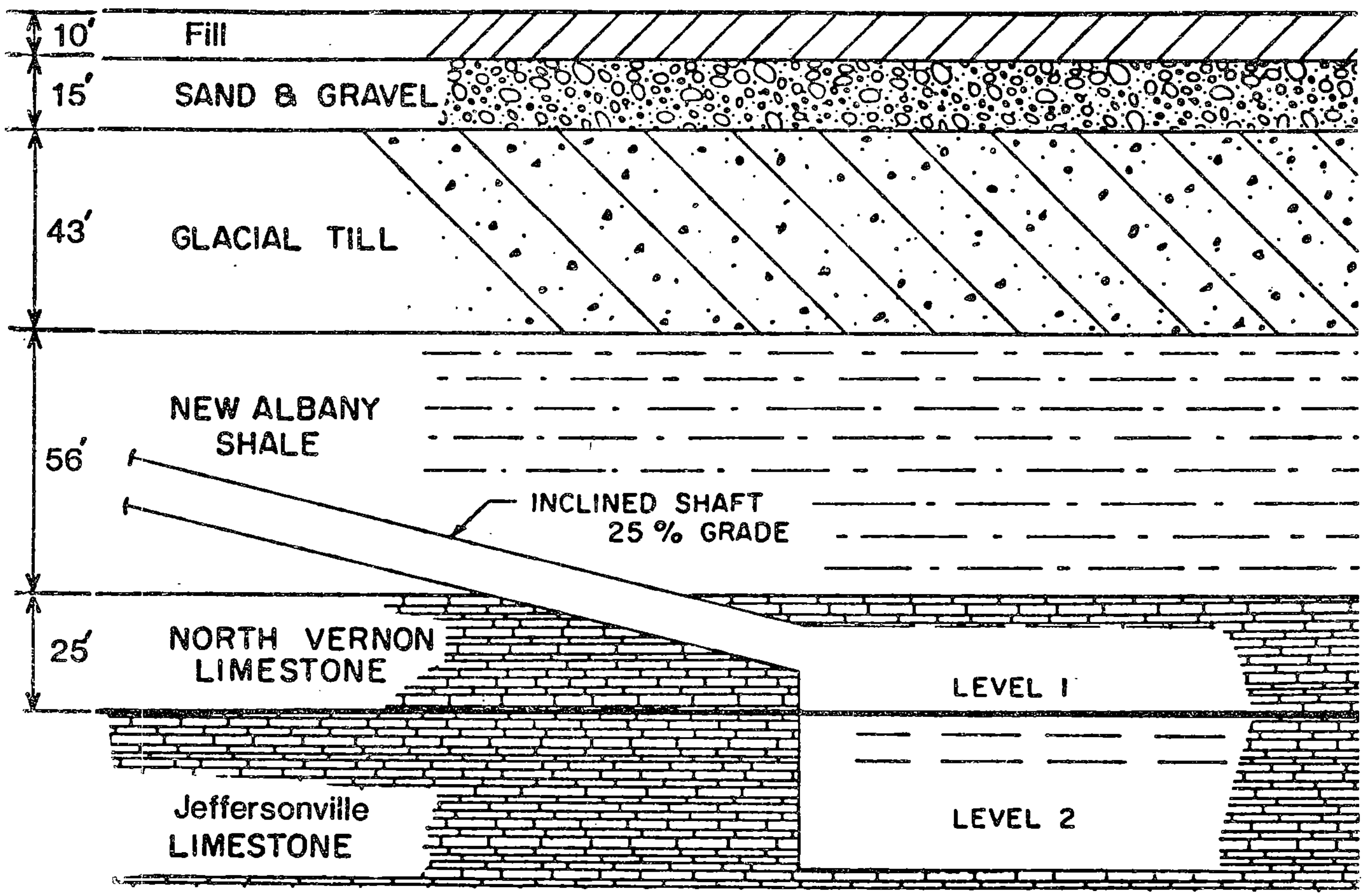


Figure 1 Generalized cross-section of the mine.

Indiana can be further subdivided into smaller physiographic units some of which have thick glacial deposits at the surface and others which are bedrock physiographic units. The Indianapolis area has Wisconsinan aged materials at the surface and it lies within the Tipton Till Plain physiographic unit.

Glacial Geology

Indiana was covered by ice at least three times during the Pleistocene Epoch. Kansan, Illinoian and the last advance Wisconsinan ice covered Marion County. Glacial drift ranges from 10 to 350 feet thick for the county, the average is about 140 feet (West and Warder, 1983).

The majority of the drift is glacial till which was deposited as a result of the melting of stagnated basal glacial ice (Harrison, 1963). Stratified drift was deposited by meltwater flowing in channels walled with ice and flowing in sluiceways beyond the margins of the ice. Harrison estimates the ratio of unstratified drift to stratified drift to be 4 or 5 to 1.

Most of the Tipton Till Plain can be classified as "hummocky disintegration till" (Harrison, 1963). The relief is usually less than 10 feet. Harrison believes that most of the surface features are a result of uncontrolled breakup and disintegration of the glacier, hence the name.

One of the major surface features of the County is the White River. It, like most other major streams in the area, originated as a stream occupying an ice walled channel which evolved into a major sluiceway carrying meltwater. Harrison (1963), Hartke et al. (1980) and West and Warder (1983) recognized that the White River is rimmed by outwash terraces formed during Wisconsinan time. It is from one of these terraces that Martin

Marietta produces its sand and gravel at the Kentucky Avenue Plant. Included in these terraces of course is post-glacial alluvium composed of sand, silt and clay sized materials overlying the outwash. Figure 2 shows a glacial geology map for Marion County.

Bedrock Geology

Owing to glaciation, no naturally exposed bedrock exists in the county. Late Silurian through Early Mississippian rocks form the immediate bedrock surface. No Lower Devonian rocks are present. The regional dip of the sedimentary rocks is about 23 feet per mile to the southwest toward the Illinois Basin. Figure 3 shows a geologic map of the county and Figure 4 is an east-west geologic cross section through the central portion of the county. A geologic column for the bedrock of central Indiana is provided in Figure 5.

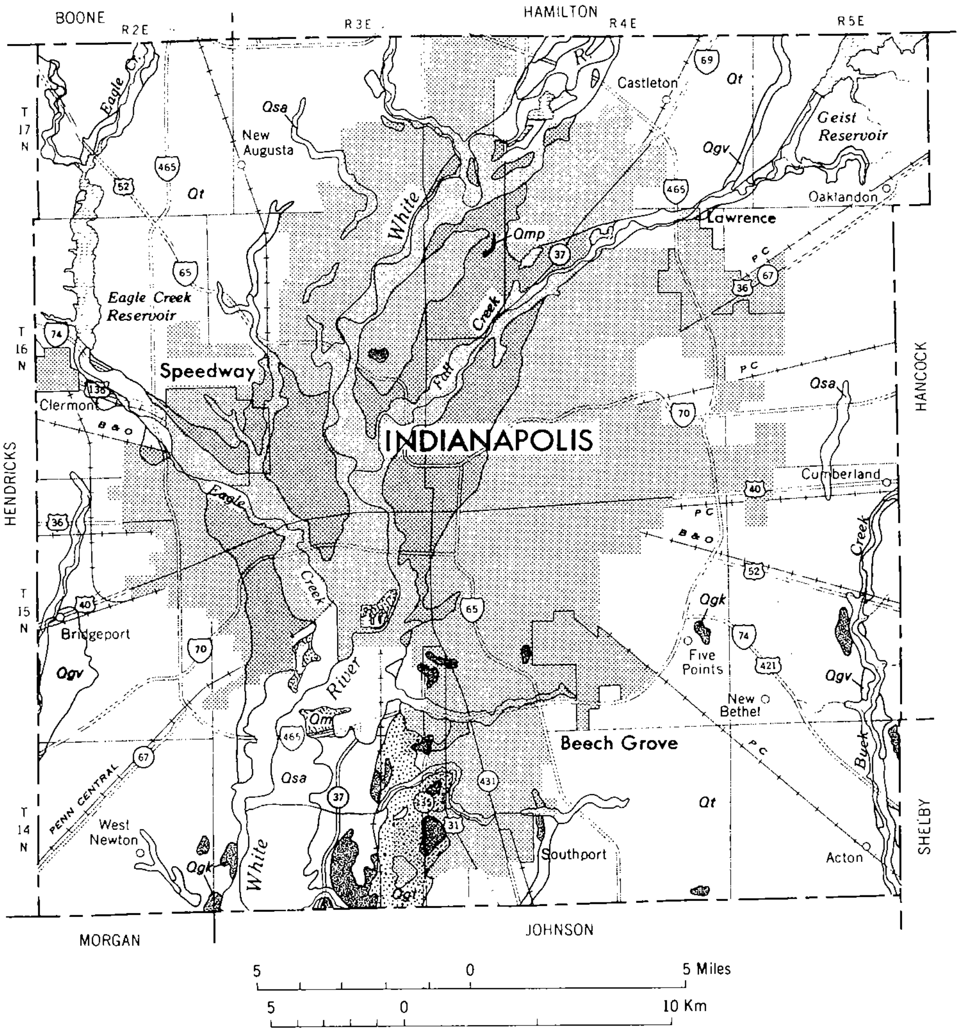
Geology of the Site

Location

The entrance to the inclined tunnel leading to the mine is located near the center of Section 28, T15N, R3E on the Maywood, Indiana 7-1/2 minute series topographic quadrangle. The surface works and property owned by Martin Marietta occur on a terrace of the White River as observed on the topographic map (Figure 6).

Unconsolidated Overburden

As the mine lies above a river terrace, till outwash, soil and alluvium are present. The unconsolidated material has been explored by use of many auger holes drilled prior to and during gravel pit operations. A cross section of the terrace material and underlying consolidated materials is shown in Figure 7. The materials were interpreted by the authors based



EXPLANATION

Qm — Modified land
Land extensively modified by excavation for limestone or gravel. Small areas not shown

Osa — Silt, sand, and gravel
Deposits of present streams; includes associated slopewash and swamp deposits

Omp — Muck and peat
Paludal and lacustrine deposits

Ogv — Gravel, sand, and silt
Deposits of meltwater streams. Valley-train deposits as partial valley fill

Ogk — Gravel, sand, and some silt
Hummocky patches of ice-contact stratified drift

Ogt — Sand, gravel, and till
Undifferentiated ice-contact stratified drift and till with hummocky topography

Qt — Till
Mostly loam till. Till sheets of low relief

Figure 2 Map showing surficial geology of Indianapolis, Indiana (after Hartke et al., 1980).

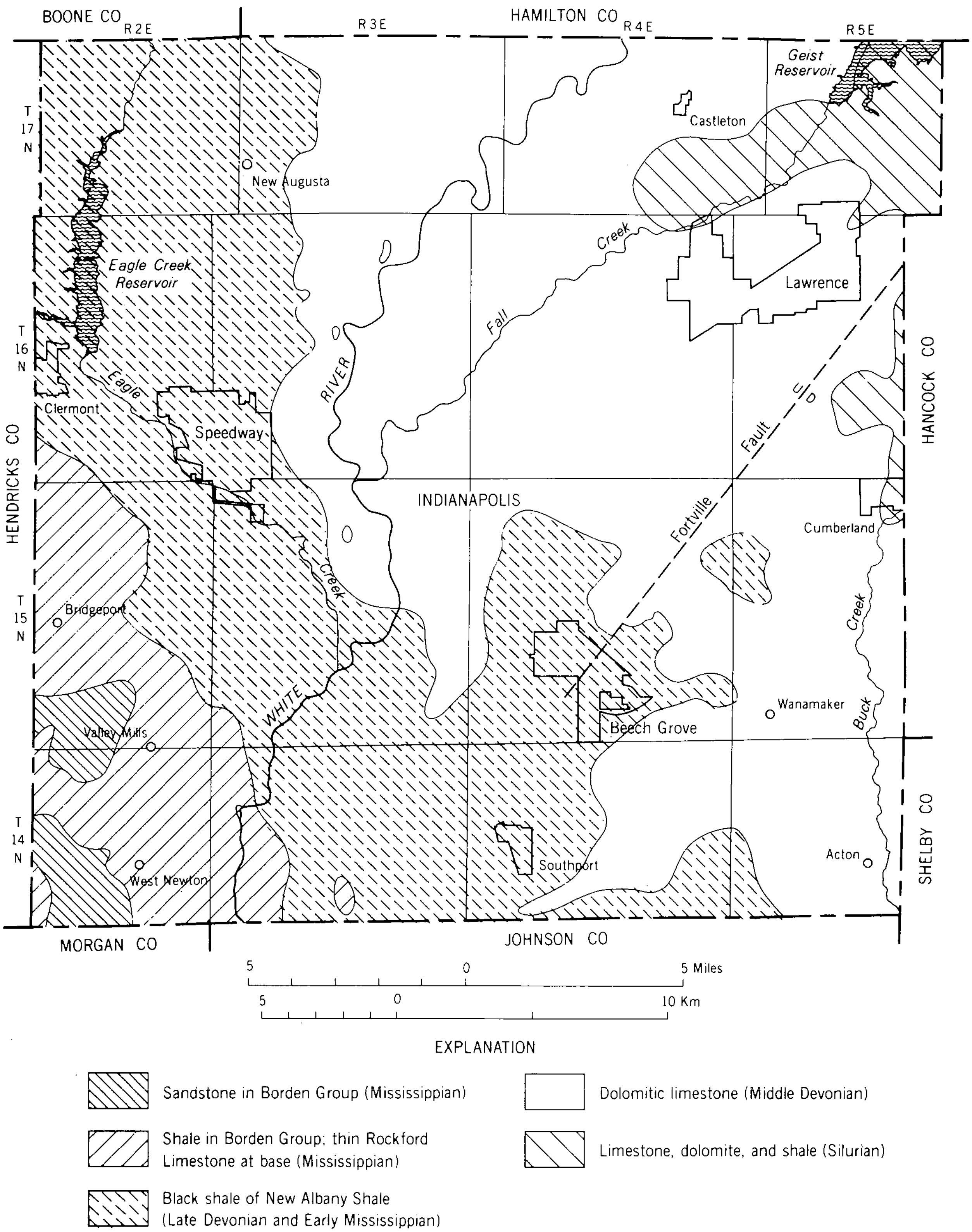
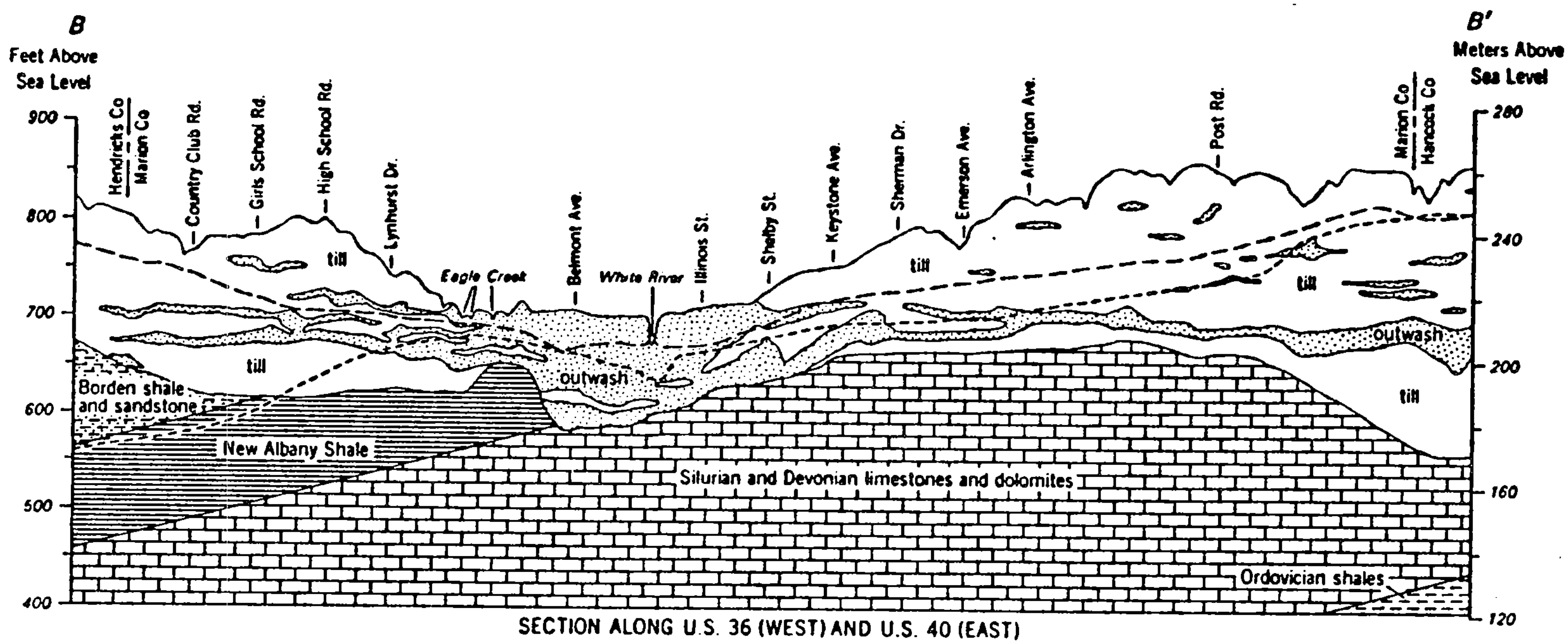
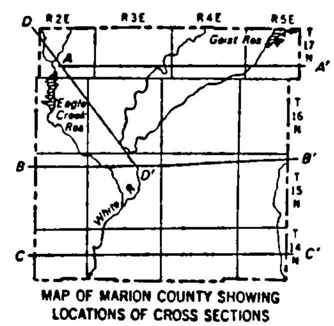


Figure 3 Map showing bedrock geology of Indianapolis, Indiana (after Hartke et al., 1980).



a.



b.

Figure 4. (a) Geologic cross section across Marion County. (b) Map showing location of cross section. (Both taken from Hartke et al., 1980.)

System	Rock unit		Composition	Thickness (ft)	Remarks
Mississippian		Borden Group	Sandstone, siltstone, and shale	0- 50	Present only in the southwest. Aquifer suitable for domestic and farm use. Not suited for liquid-waste injection.
		Rockford Limestone	Limestone		
Devonian		New Albany Shale	Shale	0-250	Present only in the west and south.
		North Vernon Limestone	Limestone		Not present in the northeast. Moderately productive aquifer. Not suited for liquid-waste injection.
	Jeffersonville Limestone	Vernon Fork Member	Dolomite		
		Geneva Dolomite Member	Dolomite		
Silurian	Wabash Formation	Liston Creek Limestone Member	Cherty dolomitic limestone	0-200	Contains reefs (Huntington Lithofacies). Moderately productive aquifer. Not suited for liquid-waste injection.
		Mississinewa Shale Member	Calcareous shale and argillaceous limestone		
		Louisville Limestone	Dolomitic limestone		
		Waldron Shale	Shale		
		Limberlost Dolomite	Dolomitic limestone		
		Salamonie Dolomite	Dolomite and dolomitic limestone		
		Brassfield Limestone	Limestone		
Ordovician		Maquoketa Group	Shaly limestone	~1,500	Aquifer and liquid-waste injection potentials unknown.
		Trenton Limestone	Dolomitic limestone		
		Black River Limestone	Limestone		
		Glenwood Shale and Joachim Dolomite	Shale, siltstone, and dolomite		
		Knox Dolomite	Dolomite	~1,700	
Cambrian		Davis Formation	Siltstone, shale, and limestone	~100	
		Eau Claire Formation	Shale	~700	Potential confining unit for liquid-waste injection.
		Mount Simon Sandstone	Sandstone	~1,200	Unit with greatest potential for satisfactory liquid-waste injection.

Figure 5. Generalized geologic column, Indianapolis, Indiana (after Hartke et al., 1980).

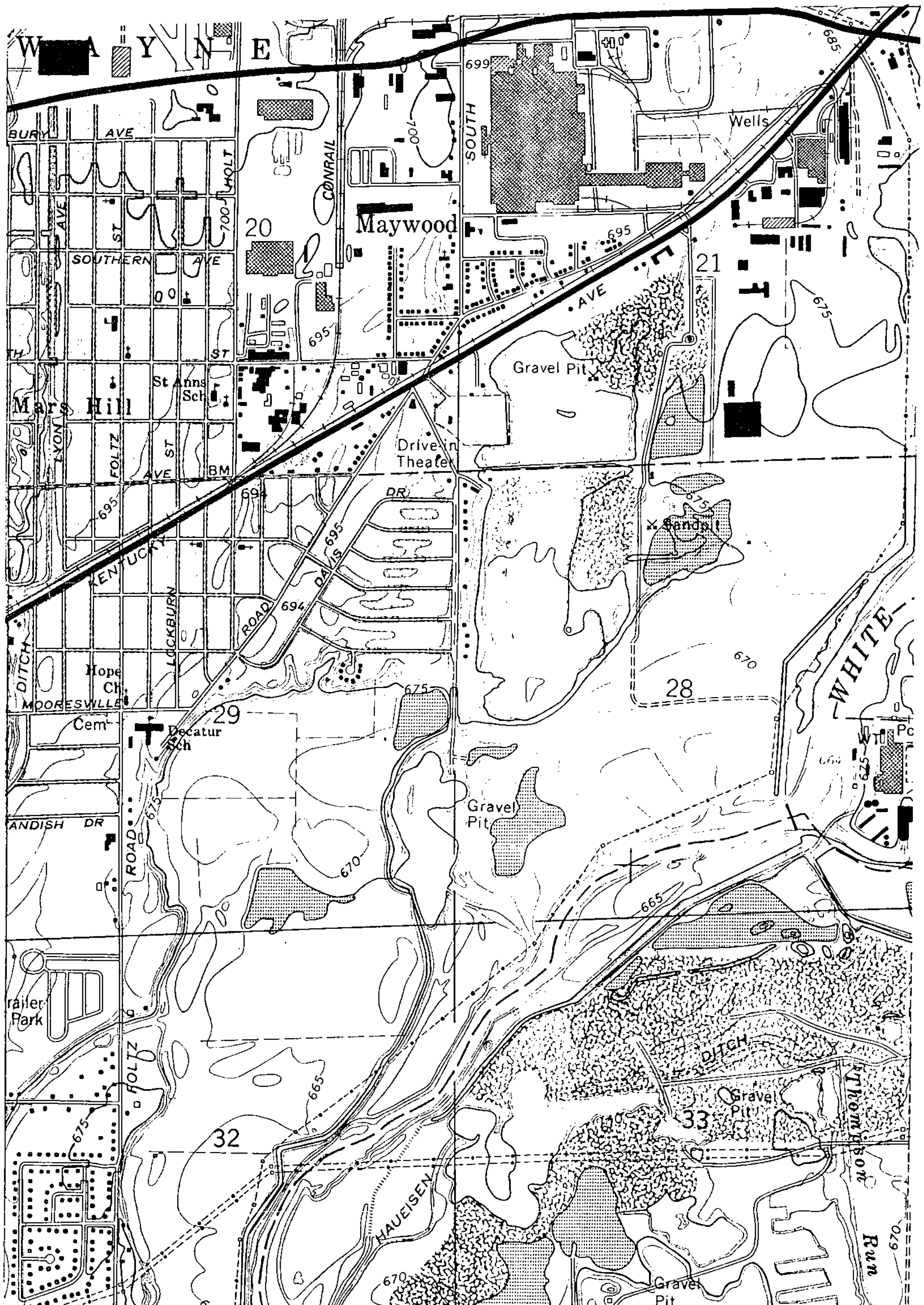


Figure 6. Map showing surface topographic features at research site (from USGS, 1967).

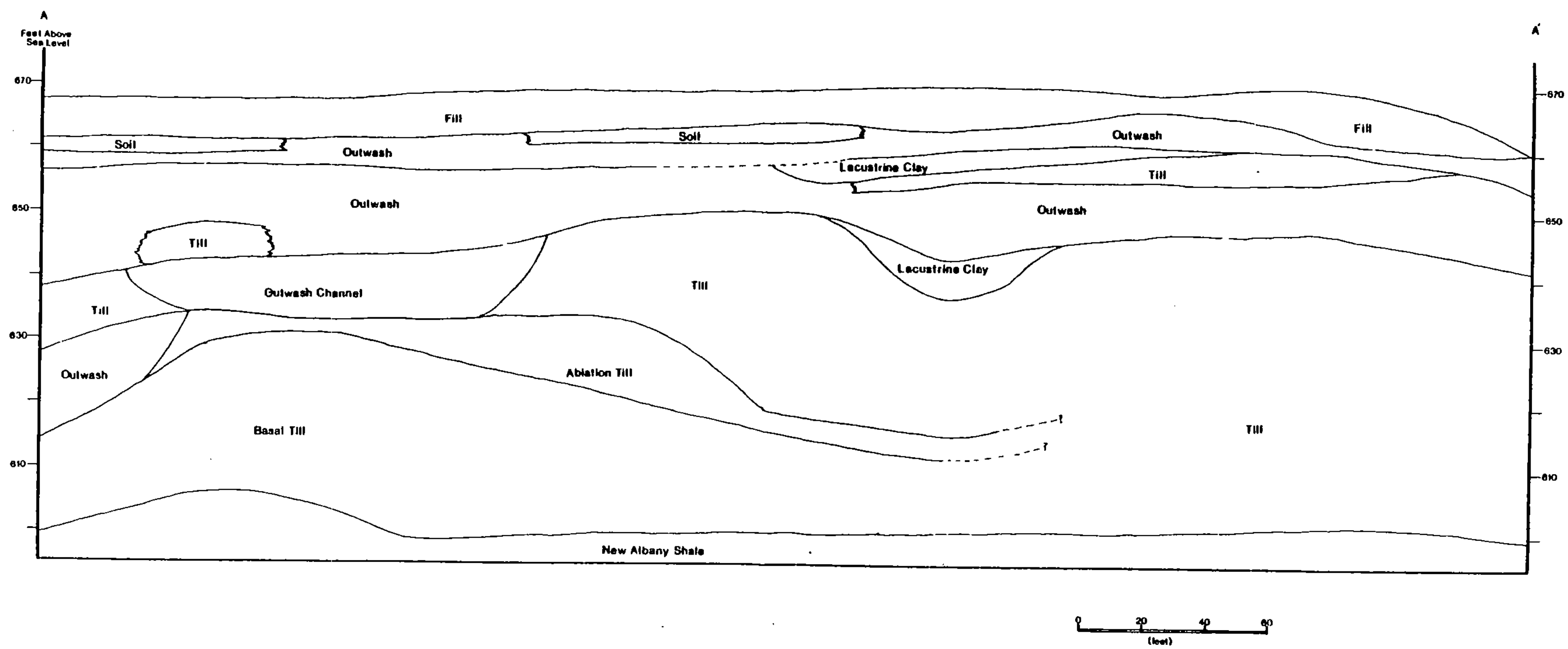


Figure 7. Cross section of terrace material at research site.

on drill log descriptions supplied by the gravel pit operator. As this area has been reworked periodically by the White River since glacial retreat, intermixed outwash, till and alluvial materials are to be expected.

Bedrock Geology

The rock which subcrops below the glacial drift at the site is the New Albany Shale. Figure 8 shows the bedrock topography map produced in this study based on water well and seismic refraction data. This map shows the location of the mine and a large valley developed in the shale immediately to the west. Although the bedrock immediately below the site is composed of the shale, the area of contact of the shale with the underlying limestone is fairly closeby. Based on this and on several seismic profiles taken a few miles to the south, the bedrock surface in the near vicinity could consist of limestone. Additional drilling or possibly geophysical investigation would be needed to confirm this. Such a stratigraphic relationship is a concern regarding ground water infiltration into the mine. This is considered further in the next section.

Ground Water

Because the mine lies below a river terrace, the potential for water infiltration would be a major concern were it not for the New Albany Shale. That unit acts as an impermeable layer which limits the amount of water which can reach the limestones below. The mine is reasonably wet, particularly in low areas and sections on the west to northwest side of the present mine. Water inflow is also likely to increase to the southeast as the shale also thins in that direction, moving away from the bedrock high. It is not anticipated that water infiltration will be a serious problem as long as some shale overlies the limestone unit.

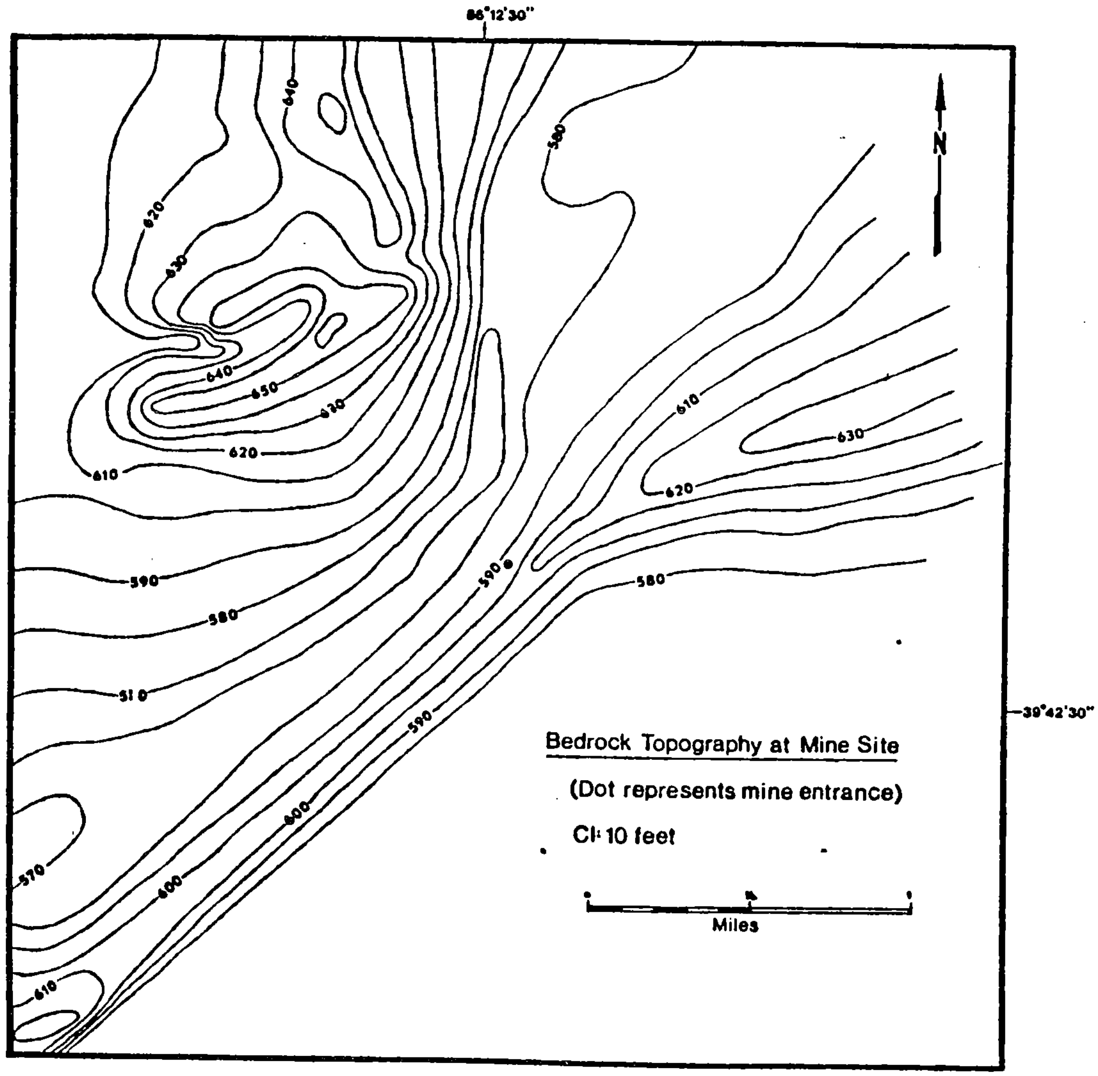


Figure 8. Map showing bedrock topography at research site.

Based on a report on ground water quality below and adjacent to landfills in Marion County by Pettijohn (1977), the two best sources of water in the unconsolidated materials at the mine site, are the aquifer just below the alluvial cover and another just above bedrock. Further, the regional ground water gradient is from northwest to southeast (towards White River as expected). The local gradient in the shallow aquifer (just below the alluvium) is to the southeast. Water also flows vertically, (leaks) from this aquifer to the deeper aquifer (just above bedrock). In contrast, the deep aquifer contains a ground water divide near the center of the landfill which existed in 1975. Thus, water flows to the northwest as well as southeast in the deeper aquifer. This condition is caused by heavy ground water pumping which draws water to the northwest. If pumping would cease the deeper aquifer would also flow to the southeast. During the spring, the ground water divide is lost owing to bank storage along the White River which causes water to flow to the northwest.

Two samples of water were collected to test for possible pollution of the ground water. One sample was collected in the mine from water seeping out of a pillar wall in the northwest area. The other was collected at the surface from a well located adjacent to the lunch room for the mine workers. This well is supplied by the lower aquifer, the one just above bedrock. Hence, one water sample was taken from below the shale and another from just above it.

The samples were tested for total organic carbon to indicate (by high levels of organic carbon) whether the groundwater was polluted by the landfill. Results indicated that the TOC for the surface sample was only 1 mg/l and for the mine sample, 0 mg/l. This indicates the ground water is not likely polluted by the landfill.

JOINT MAPPING IN THE MINE

The decision to map the roof joints led to a challenging problem. Since the roof of the mine is 22 feet high and lighting is poor, attempting to project a joint trace onto the mirror of a Brunton compass is not easy to accomplish. With this restriction a system was devised which would solve the problem, yet allow mapping to be accomplished by one person.

The equipment developed included a large, flat mirror (12 in x 18 in) which was attached to a board of slightly larger size. Connected to the bottom side of the board was a nut allowing the board (and mirror) to be attached by a threaded bolt to a tripod base. A plane table and tripod base were needed for drafting purposes. The only other pieces of equipment used were a 100 foot tape measure, Brunton compass and engineers scale.

The actual procedure used is described in the following steps:

1) The plane table was located directly below a survey station. The table would remain in this position while all joints within range (100 feet) were mapped. Data collected would be recorded and plotted on the base map attached to the plane table.

2) The mirror was placed directly below a joint and within 100 feet of the plane table. Using the bull's eye bubble level on the Brunton compass, the mirror was levelled.

3) A light was shone on the joint directly above the mirror so that the joint's image would be visible in the mirror. The joint orientation was measured simply by aligning the Brunton compass edge with the joint image visible in the mirror.

4) Before the joint could be mapped, the location of the joint relative to the plane table (i.e., survey station) was needed. The distance between the mirror and plane table was measured using the tape measure. (This distance was considered to accurately represent the distance between the two points measured along the roof line.) The Brunton compass was used to measure the direction of the mirror from the plane table. This represents the direction of the joint from the survey station. After the direction and distance were obtained, the location and orientation of the joint could be plotted on the base map.
5) It was important to map directly in the mine. This eliminated many errors since a visual check of the map was possible. Normally, two or three measurements were taken for each joint. These measurements were distributed along the length of the joint.
6) All joints within 100 feet of a survey station were mapped. The process was then repeated for the next survey station.

The mapping method fulfilled the necessary criteria because of its accuracy and because it could be performed by one person. However, three sources of error were discovered. 1) Distortions of the joint image in the mirror could occur if the mirror was not placed directly below the joint or the mirror was not levelled. The distortions could lead to errors in joint orientation measurement. 2) Since one person must measure the distance between two points, one end of the tape measure has to be hooked onto the tripod holding the mirror. To get an accurate measurement, the other end of the tape must be stretched out tightly, through the air. Inaccurate measurements resulted if this was not done. (Note: stretching the tape too tightly can cause the tripod to tip over.) 3) The Brunton compass is greatly affected by magnetic objects. Measurements should not be taken close to machinery, electrical junction boxes or rockbolts.

RESULTS OF JOINT MAPPING

Figure 9 is a map of the mine showing the joints in the mine roof. One of the concerns of this study was to determine if local roof falls were related to the joint pattern. It can be noted that the joint pattern is fairly consistent throughout the mine.

An implication of the consistent joint intensity and pattern is that the roof falls probably are not caused by a change in jointing alone. If jointing were the major factor promoting falls, then falls would be expected to occur throughout the mine not just in isolated groups. Therefore, it was concluded that the vertical jointing is not the main cause of roof falls.

Since the joint map shows that jointing had little effect on fall distribution, another line of investigation was selected. It was decided that rock structure mapping should be undertaken to determine if any consistent relationship existed between fall distribution and rock structure.

MAPPING OF ROCK STRUCTURE

The objective in this phase was to determine the rock structure throughout the mine. Since the mine roof is 22 feet high, more than enough space existed to accomodate surveying equipment. The plane table and telescopic alidade were chosen for surveying. Furthermore, a 14 foot stadia rod was used for the stadia method. The method selected was that described by Compton (1962) which unfortunately requires two persons to accomplish the work.

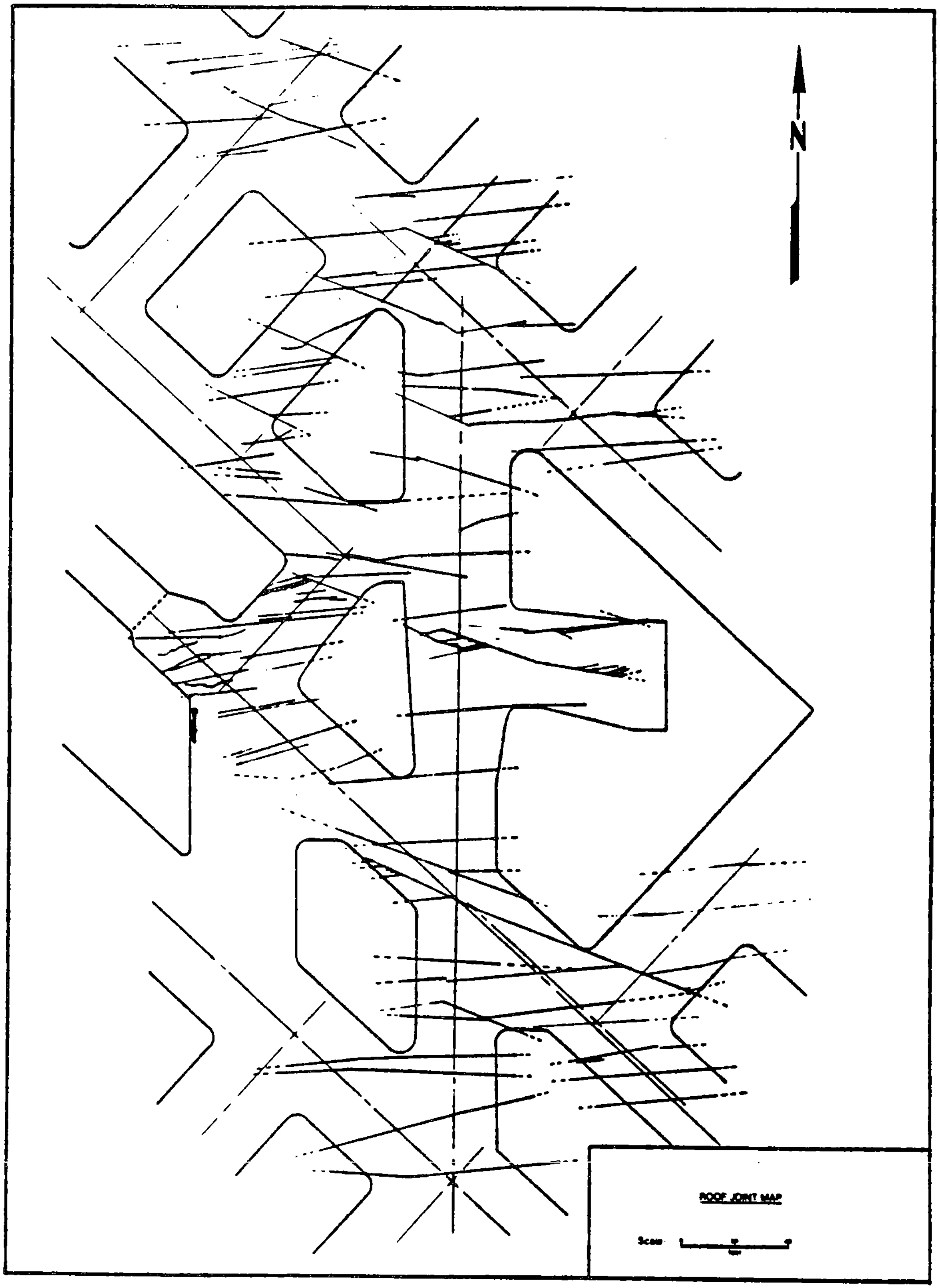


Figure 9. Map showing joints in the mine roof.

Before mapping began, it was recognized that in order to distinguish between rock structure and sedimentary thickening or thinning, two horizons at different elevations in the mine would have to be mapped. A comparison of the resulting maps would show if the roof rock could be assumed to have similar features as the rock below it. Owing to equipment limitations the roof horizon could not be mapped directly.

The two horizons chosen for mapping were the contact between the North Vernon Limestone and Jeffersonville Limestone, and a moist, clayey zone located in the North Vernon Limestone some 10 feet above the contact. The moist zone, henceforth called the moist layer, was chosen because it is visible and continuous throughout the mine. The moistness stood out well on the dry, dusty background of the pillar face. The contact between the two limestone units was visible and obvious throughout the mine.

Surveying was performed using the standard stadia method with two main modifications. The elevation of the contact between the two limestone units was determined by first using the stadia method to obtain the elevation of the floor directly below the contact and then adding on the distance from floor to contact. This modification was chosen because of the poor lighting conditions. Also, the moist layer elevation was determined by measuring the distance from the contact up to that layer using the stadia rod. This modification was necessary since the vertical angle of rotation for an alidade is limited and was too small to accomodate the mine conditions. It was concluded that little loss in accuracy occurred because of the necessary modifications.

RESULTS OF ROCK STRUCTURE MAPPING

Three maps were prepared to present the results. These included a structure contour map of the North Vernon-Jeffersonville Limestone contact, a structure contour map of the moist layer, and an isopach map for the interval between the moist layer and the contact.

The isopach map shows that there is some sedimentary thickening and thinning in the interval. However, the two structure contour maps are nearly identical. This suggests that the structure of the moist layer is nearly identical to that of the contact. It is believed that similar features observed on the two structure contour maps occur in the roof rocks as well. The isopach map shows that sedimentary thickening or thinning has had some but not a major effect on the structures observed.

An interesting relationship was observed between rock structure and the location of localized fall areas. The fall areas generally occur at locations where the dip of the rock structure is changing rapidly. Likewise, in areas where the rock is more flat, few falls are observed.

CONCLUSIONS

Several conclusions were drawn from this engineering geology study of the Martin Marietta underground limestone mine in Indianapolis.

1. Economic demand for quality crushed stone and the thick unconsolidated overburden in Marion County make underground mining of limestone economically feasible.

2. The New Albany Shale which overlies the limestone being mined reduces the ground water infiltration. In areas where the shale is reduced in thickness it is quite likely that water inflow will increase significantly.

3. Ground water in the aquifer above the shale and from below the shale within the limestone does not seem to be contaminated by the landfill leachate.

4. The fracture pattern in the roof of the mine has a strong nearly E-W trend and this pattern seems to be quite consistent throughout. No obvious relationship with fracture density and localized roof falls is apparent.

5. Some sedimentary thickening and thinning was observed between the moist line horizon and the contact surface between the North Vernon and Jeffersonville Limestones. The structural aspects of the two horizons is nearly identical.

6. Isolated roof fall areas generally occur at locations where the rock structure is rapidly changing (more steeply dipping); conversely where the roof is more flat few falls are observed.

7. Additional aspects concerning roof stability are considered in the complete report of research on the limestone mine (Fein, 1983).

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RETAINING WALL ALTERNATES

Robert M. Leary and Gary L. Klinedinst

Introduction

Recently, State highway agencies have shifted their program emphasis from construction on new location to projects involving reconstruction and widening of existing facilities. In order to minimize the impact of these projects on adjacent landowners and complete them within existing right-of-way limits, many of these projects include retaining walls to contain fill sections and to support adjacent land and buildings in cut sections. These retaining structures often amount to a large percentage of the total project cost.

Conventional cast-in-place concrete cantilever retaining walls are not only expensive but are very time-consuming to construct. In order to maintain traffic flow, it is often necessary to use temporary structures to support portions of the existing roadway during construction of the permanent wall. These temporary supports must remain in place while excavations are made, footings are placed, formwork is constructed, reinforcing steel is tied, wall concrete is placed and cured, forms are removed, and the wall is backfilled. These construction steps must often take place in confined areas with minimal access, thus increasing the cost even more, slowing the construction process, and keeping traffic in narrowed lanes or on detours with inadequate alignment for long periods of time.

Conventional cast-in-place concrete retaining walls cost at least $40 to $70 per square foot of wall surface area, depending on height, foundation conditions, and difficulty of construction. Recently introduced earthwork reinforcement and precast modular systems can be constructed for 30 to 50 percent less than conventional walls. Similar savings in construction time can also be realized by using these systems. Whenever earth retaining structures are called for in a project design, accepted engineering practice now demands that consideration be given to these alternate wall systems.

Recent Experience

Many different retaining systems are available and specific project constraints sometime dictate which should be considered. Three proprietary systems are now being routinely used by many State highway agencies (SHA's). These are Reinforced Earth and VSL's Retained earth, both systems of earthwork reinforcement, and Doublewal, a modular precast concrete bin system.

Since its introduction to this country in 1965, Reinforced Earth has been used to construct at least 700 walls in some 40 States, totaling over 6.5 million square feet of retaining wall. It has been used for both cut and fill roadway sections, landslide repair, and for abutment walls with the bridge supported either on piling through the fill or on spread footings directly atop the Reinforced Earth volume. Many SHA's now routinely consider a Reinforced Earth alternate whenever a project design calls for a retaining wall.

Until early in this decade, Reinforced Earth was either specified or bid as an alternate to conventional construction. To meet regulatory requirements concerning the use of proprietary products and to gain experience with this type of construction, most SHA's designated the first few Reinforced Earth structures in each State as experimental projects. Many such walls were instrumented and monitored to determine the adequacy of the design assumptions. Extensive performance data and construction case histories are available for Reinforced Earth.

As the use of Reinforced Earth became more widespread, agencies began to routinely bid this system against conventional cast-in-place retaining walls. Reinforced Earth was almost always selected because it was less expensive and usually saved significant construction time. As this contracting method became common, the price of the system, for whatever reason, began to rise to only slightly less than the conventional alternate. This situation set the stage for the introduction of additional competing systems of earthwork reinforcement.

During the design and construction of some of the first Reinforced Earth walls in this country, the California Department of Transportation (CALTRANS) began research into the mechanism of earthwork reinforcement. CALTRANS eventually developed a reinforcing mesh which they believed would be more efficient and applicable to a wider range of backfill materials. The CALTRANS system, called mechanically stabilized embankment, differed from the Reinforced Earth system in that the reinforcement consisted of a welded wire fabric constructed with 3/8 inch diameter bars, placed on 6 inch by 24 inch spacing. The few walls constructed with this system were carefully monitored and appeared to perform satisfactorily.

In 1980, the VSL Corporation adopted the mesh reinforcement principle used by CALTRANS. VSL developed its own hexagonally shaped concrete face panel and mesh connection detail and began to market the system as Retained Earth. At about the same time, the Doublewal Corporation, a Connecticut supplier of precast concrete bin-type retaining walls, licensed other precasters to manufacture its product and began to promote the product nationwide. These two products appeared to represent more realistic competition for Reinforced Earth than conventionally constructed concrete cantilever retaining walls. All three companies would assist in the development of the design plans, thereby eliminating the necessity for the State to prepare plans for conventional walls which were generally not bid. For these reasons, many SHA's began to include these systems as acceptable alternates to Reinforced Earth.

Most of these agencies restricted the first few uses of the two newer products to noncritical locations and designated these projects experimental. As more of these walls are completed and more agencies gain experience in their use, widespread adoption of these systems as equal alternates is expected.

In 1981, the Georgia Department of Transportation (GDOT) decided to build on the research done by CALTRANS and developed its own system for earthwork reinforcement. After a review of that research and discussions with CALTRANS engineers, GDOT decided to adopt the 3/8 inch diameter, 6 inch by 24 inch welded wire fabric reinforcement. By early 1982, GDOT had designed a face panel, developed a connection detail, and produced a computer program for internal and external design of what would be called GASE - Georgia Stabilized Earth. A series of laboratory tests were conducted to verify the pull-out resistance of the mesh in the backfill materials in common use in Georgia.

In April 1982, the GASE system was included as an alternate to Reinforced Earth, Retained Earth, and Doublewal on two major wall contracts. The successful bidder on these contracts chose to construct the GASE system at an average price of less than $23/square foot. Continued use of alternate wall plans in Georgia has shown that realistic competition will result in consistent retaining wall prices of between $22 and $28 per square foot. Successful bidders have generally chosen to use the GASE system in fill sections and Doublewal in cut sections although Reinforced Earth and Retained Earth are chosen occasionally.

Negotiations between GDOT and the Reinforced Earth Company resulted in an agreement that requires GDOT to pay a royalty of $1/square foot for each GASE wall constructed. This royalty is paid to Mr. Vidal, the holder of the original Reinforced Earth patents.

In other States, where Reinforced Earth, Retained Earth, and Doublewal have been bid as alternates, prices have generally been in the same range. A 1982 North Carolina project to build 40,000 square feet of retaining wall in 14 locations had six bids between $20 and $24/square foot to construct Doublewal. Three Florida contracts totaling 61,000 square feet of retaining walls had consistent bid prices bewteen $20 and $25/square foot. The successful contractors here chose mostly Retained Earth and some Doublewal. Two small 1983 Kentucky projects with about 18,000 square feet of wall are under construction for about $23/square foot. A 1983 Alabama bridge abutment project (10,000 square feet) is under construction for about $27/square foot. Both the Kentucky and Alabama projects are being constructed with Reinforced Earth.

Selecting Alternates

Many systems are available for retaining wall construction. Engineers often need to make comparisons of these various types to select a retaining wall which will perform satisfactorily and can be constructed at the lowest overall cost. To make an intelligent decision, an engineer should know what types of walls are available, which types will fit his design and construction constraints, and whether or not he should include alternate designs for bidding purposes. He should also know how to assure that alternate designs are equal and how to evaluate designs which are furnished by the wall suppliers.

Although many different wall types are available, they can be divided into four main types, based on the mechanism by which they resist external loads and restrain earth:

1. Gravity walls - mass concrete and reinforced concrete cantilever walls in which heel footing backfill provides most of the dead weight, gabion walls, crib walls of wood, concrete, or steel, and bin walls of steel or concrete.

2. Cantilever walls - walls constructed of sheet piling, soldier piles and lagging, and tangent or secant drilled shafts.

3. Anchored walls - walls that derive most of their ability to support horizontal loads from grouted rock or soil anchors or dead-man anchors connected to the wall facing with tension members. The wall facing and appearance is usually similar to that of a cantilever wall.

4. Reinforced Backfill Walls - Reinforced Earth, Retained Earth, Georgia Stabilized Earth, Mechanically Stabilized Embankments, Hilfiker Welded Wire Walls, and other walls constructed with geotextiles or geogrids that provide tensile reinforcement within the backfill material. The normal design assumption for this wall type is that the reinforced material acts as a block and that block acts as a gravity wall.

Particular project constraints and specific site conditions often dictate the types of retaining structures which should be considered. Lack of availability of materials, necessary service life, and environmental or aesthetic requirements often eliminate some types of wall systems from further consideration. Designers should also determine any special loading requirements, the anticipated settlements the wall will have to tolerate, ease and speed of construction, and adaptability to field changes of any wall system which is to be included as an alternate.

Alternate walls should be included in the contract documents whenever several different sytems meet all project constraints and appear economically competitive. This procedure will eliminate the preselection of a specific wall type based on erroneous information or estimates. It will also stimulate competition among the various suppliers and help to obtain a satisfactory retaining wall at the lowest possible cost.

Contracting Methods

Two options are available for including alternate retaining wall designs in a construction contract. The first option is to select a number of economically competitive alternates during the preliminary design stage. These alternates are then completely designed and included in the contract plans. The second option is to specify the location and size of a wall and leave the selection of wall type and detailed design of the wall to the contract bidders.

The first of these options has many advantages. They include maintaining control of the engineering by the contracting agency, integrating the wall design into the overall construction project, considering the necessary site specific conditions in each design, minimizing confusion concerning which alternates are acceptable to the contracting agency, and equalizing the various designs prior to the time the contractor prepares his bid. The main disadvantage of this option is the need for additional engineering time and money for the preparation of additional plans. Many suppliers of proprietary wall systems will prepare plans, however, thereby minimizing the additional engineering time.

The second option requires that detailed design and wall selection be handled by the contractor. The preliminary engineering effort expended by the contracting agency under this option is minimized. Competition is optimized by this option and contractor innovations are encouraged. This option's major disadvantage is that it shifts the engineering responsibility to the contractor. In order to eliminate possible disputes with the contractor and to assure that the wall will fit into the overall project, the contracting agency must provide sufficient detailed design parameters and site specific constraints in the plans and specifications to assure that each proposal is designed on an equal basis. This option also has the disadvantage of requiring a review and approval of the design during the bid analysis phase.

Generally, SHA's that have tried using contracting procedures which allow detailed design and wall selection by the contractor have changed to the procedure of including completely designed alternates in the contract documents. Most of those agencies which began contracting alternate walls using completely designed alternates have been satisfied to continue to use this option.

Insuring Equality

Since SHA's have begun to make extensive use of proprietary systems, the necessity to establish specific criteria to insure equity in the alternate designs has become important. Minimum physical dimensions of the wall must be set by the contracting agency. The top elevation of the wall, its beginning and ending points, and a maximum bottom elevation (based on foundation requirements) must be established and provided to each supplier who is preparing plans. Any necessary appurentances such as traffic barriers, coping, or light and sign standards, must be described to the designer. Specific design criteria should also be set by the contracting agency.

Some specific design criteria are:

1. A minimum safety factor against mesh pull-out should be specified. This safety factor should consider the loading criteria obtained from laboratory pull-out tests using the proposed reinforcement configuration, the site specific backfill material, and a maximum allowable deflection. Presently, a minimum factor of safety against mesh pull-out of 1.5 and a maximum allowable deflection of 3/4 inch is recommended.

2. A maximum allowable reinforcement stress should be specified. This is generally 0.55 FY.

3. A minimum design life should be stated. This is generally 75-100 years for highway applications.

4. Maximum allowable total and differential deflection limits should be set.

5. The magnitude and direction of external loads should be specified.

6. Any subsurface drainage required should also be specified.

7. The minimum design safety factor for overturning, sliding, bearing capacity, and stability of the overall slope must also be supplied.

8. Finally, establishment of the methods of payment and units of measurement must remain within the control of the contracting agency.

When designs are delivered by the wall suppliers, they must be carefully and fairly evaluated to insure that they meet the previously established criteria and that all designs are essentially equal. Some differences among alternates are unavoidable and are sometimes inherent in the various systems but overall equality must be insured if the designs are expected to compete as equivalent alternates.

Each wall supplier should provide technical data and calculations to show that his system meets the design criteria set forth by the contracting agency. The design package should include complete plans, construction specifications and special provisions compatible with the agency's standards, as well as a complete cost estimate with a breakdown of unit costs and item quantities. Site specific computations should also be provided by the designer and if a computerized design procedure is used, example computations showing how the program operates should be included.

All metal components of the wall system should be designed for corrosion based on a rational method that establishes section losses for the previously determined design life. All design procedures should be required to follow established engineering practice. Any deviations from standard practice should be supported by verifiable data from laboratory and/or full scale field performance tests.

Construction Monitoring

Some wall system suppliers have done a significant amount of monitoring to determine the actual field performance of their systems both during and after construction. Data from these instrumented walls can be used to verify design assumptions under various loading conditions, determine the constructibility in various applications, and assess long-term performance of the wall system.

Each contracting agency should review the results of these test walls to see if a wall similar in size, geometry, and application as the proposed project has been constructed or monitored by the supplier or by another agency. If not, the agency should include some construction monitoring in the special provision for that particular system. Although this requirement may add some cost to certain alternates, some wall suppliers are willing to absorb all or part of this cost in order to prove the validity of their design assumptions and to verify the performance of their system under new applications or loading conditions. Those suppliers unwilling to prove their systems or participate in extending the knowledge of their systems to the satisfaction of the contracting agencies, should not be considered as a viable alternate. Although this may limit the number of acceptable alternates for certain projects, it will keep the contracting agencies from absorbing the cost and liability of extending the applicability of a proprietary wall system.

Summary

Tremendous cost savings can be realized by providing alternate retaining wall systems from which the bidders can choose. In order to make alternate bidding viable, the contracting agency must maintain control of the engineering and must insure equality among all alternates.

To establish equity among the alternate systems, the contracting agency must determine what type of retaining walls will fit the particular project criteria and be economically viable. Further, the agency should establish specific design criteria for all wall systems being considered. State highway agencies have tried various contracting procedures and generally prefer to provide complete plans for all alternates. This allows the agency to maintain control of the engineering and at the same time it provides the best suited walls for each particular site. The extra engineering effort in this approach is often mitigated by the willingness of many suppliers to provide complete plans to the agency.

The approach of providing equitible alternate designs in the contract documents has been proven to stimulate competition and will result in large savings to the contracting agency.

INNOVATIONS
IN
ANCHORED RETAINING WALLS

Peter J. Nicholson(1)
and
Dennis L. Boley(2)

(1) President
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1.0 INTRODUCTION

The introduction and development of ground anchors as a civil engineering construction technique has proven to be one of the most significant innovations in the construction industry for many years. Over the last 25 years significant advances have been made in both the theory and practice of ground anchor design and construction to the extent that ground anchors are no longer regarded as just temporary construction expedients, but permanent installations. Permanent ground anchor installations are now commonplace and are incorporated into such major structures as dams, buildings and retaining walls.

Extensive renovation programs are being conducted on many of our state and federal highways. As a result of this activity many new designs and methods of construction are being developed and utilized to provide a more cost effective use of available construction dollars. This paper discusses two distinctively different methods of constructing retaining walls in metropolitan Atlanta, Georgia by utilizing permanent ground anchors.

2.0 WHEN TO USE ANCHORED RETAINING WALLS

Anchored or tied-back retaining walls have typically been used for temporary earth retaining systems but have not, until recently, been considered for use as permanent retaining walls. A permanently anchored retaining wall should be considered for use when a wall is required for a cut situation (e.g. a depressed roadway through a metropolitan area); the wall required is 15 feet or more in height; or in situations where temporary shoring is required for the construction of a conventional retaining wall. The permanently anchored wall will usually be the most cost effective solution in these situations. This has recently been the case in Atlanta where 13,500 square feet (surface area) of retaining wall was bid for construction, at the contractor's option, as either a conventional reinforced concrete cantilevered wall or as a permanently anchored retaining wall with a reinforced concrete facing. The permanently anchored wall bid was approximately $700,000 while the conventional wall bid was approximately $1,140,000 resulting in a cost savings of over $400,000.

3.0 PERMANENTLY ANCHORED RETAINING WALL-E1, ATLANTA, GEORGIA

3.1 Project Description

Widening of I-75 near the Penthouse Motel and the Coca-Cola Bottling Company required the construction of a retaining wall. The right-of-way limitations and potential effects of construction on the adjacent roadway and structures limited the type of retaining wall that could be built at this location. Two wall alternatives were considered as viable options for retaining walls at this location. The alternatives included a conventional reinforced concrete cantilevered wall and a permanently anchored retaining wall. The permanently anchored retaining wall was the chosen alternative due to its considerable cost savings.

Construction of the conventional concrete cantilevered retaining wall required the use of temporary shoring to permit Williams Street to remain open. Williams Street is located immediately behind, and is supported by, Retaining Wall-E1. The permanently anchored retaining wall however, utilized the shoring as an integral part of the permanent installation. The ability of the anchored walls to utilize shoring that would otherwise be only temporary, provides a significant cost savings.

3.2 Subsurface Conditions and Design Criteria

The subsurface conditions consisted of medium dense to dense micaceous sandy silt and silty sands. Standard penetration test resistances ranged from 11 blows per foot to 60 blows per foot with no penetration. The soil density generally increased with depth, however erratic layers of hard and soft materials occurred frequently. Slickensided surfaces were also present. The ground water table was located about 40 feet below the ground surface. The depth to competent rock varied from 45 to 65 feet. A soil profile is contained in Figure 1.

The walls were designed to include analyses of the states of stress in soldier piles, lagging and anchors at critical stages of construction using the earth pressure diagrams contained in Figures 2, 3 and 4. The soil design properties used in the analyses included an angle of internal friction of 30° and a coefficient of active earth pressures and a coefficient of passive earth pressures of 0.33 and 3.0 respectively. The conditions analyzed included:

a. Soldier Piles and lagging cantilevered during excavation prior to installation of the top row of anchors.

b. Intermediate excavations for subsequent anchor installations.

c. Final constructed condition assuming excavation for drainage facilities in front of the wall.

The anchor tendon size was determined so that the design load for the anchor did not exceed 60% of the guaranteed ultimate tensile strength of the tendon and the maximum load applied to the tendon did not exceed 80% of the guaranteed ultimate tensile strength of the tendon. The free length, or stressing length, of the anchor tendon was required not to be less than 15 feet. It was Nicholson's responsibility to determine the soldier beam size and spacing, number of anchors, length of anchors and anchor bond length necessary to develop the design loads selected for each anchor.

3.3 Wall Components

The wall height ranged from 13 to 33 feet, anchor loads were on the order 40 to 135 kips. The free length of the anchor tendon ranged from 15 to 20 feet, while bond length ranged from 25 to 45 feet.

One row of anchors was provided for wall heights between 13 and 20 feet. Two rows of anchors were provided for design heights of 25 to 29 feet, while three rows of anchors were provided for wall heights from 29 to 33 feet. The anchors were installed an an angle of 25° to 30° from the horizontal. A typical section of the wall is contained in Figure 5.

The anchor tendon utilized was a 0.6 inch diameter, 270 ksi, 7-wire strand. The number of strands per tendon ranged from 3 to 5, depending on anchor load. The tendon was sized based on a prestressing force of 53% of the guaranteed ultimate tensile strength of the steel strand. This was required to accommodate test loads of 1-1/2 times the design load of the anchor. The anchor grout consisted of 5 to 5-1/2 gallons of water per bag of Portland cement Type I or III. The soldier piles generally consisted of two wide flanged sections, spaced at 8 foot centers and encased in concrete. The piles and concrete were placed in pre-augered 30 inch diameter holes.

Double corrosion protection of the anchor tendon was provided by placing the full length of the tendon in 3-5/8 inch diameter corrugated plastic tubing and encapsulating the tendon and tubing in grout. The tendon free length was greased and sheathed. Corrosion protection of the wedge plate, bearing plate and anchor head were provided by the 12 inch thick reinforced concrete facing.

Shear studs were welded to the soldier beams to integrally connect the concrete facing to the soldier beams and lagging. The 12 inch thick reinforced concrete facing was designed to accommodate earth pressure loadings which could occur due to the deterioration of the wood lagging. The soldier beams functionally provided vertical reinforcement for the concrete facing.

Positive drainage for the wall was provided by placing a pre-formed drainage material (EnkaDrain) on the exposed face of the lagging at the mid-point between soldier beams. Drainage through the facing was provided by weep holes. The form for the concrete facing was designed to provide an architectural finish that would match existing walls in the vicinity.

3.4 Construction Sequence

The construction sequence is depicted schematically in Figures 6, 7 and 8 and is as follows:

1. Drill the 30 inch diameter hole for the soldier beams. Fill the socket portion with 3500 psi concrete, followed by lean concrete for the remainder of the pile height. Insert the soldier pile.

2. Excavate in front of the wall, placing wood lagging and soil anchors as the excavation is advanced.

3. Place the drainage fabric (EnkaDrain) at the mid-point between soldier piles. Place the shear studs on soldier piles, followed by the facing, reinforcing steel and concrete formwork. Cast in place the concrete facing in 24 foot wide sections. The 24 foot sections were placed every two days.

The soil anchors were installed as follows:

1. Drill a hole between double soldier beams at the design angle and to the design depth using 5 inch O.D. steel casing.

2. Disconnect the casing from drill head and flush casing with water until clean.

3. Tremie neat concrete grout until the grout overtops the casing.

4. Insert the anchor tendon complete with full length corrugated plastic tubing.

5. Reconnect the casing to the drill head and begin pressure grouting. Pressure grout at an injected pressure about 2 pounds per square inch per foot of overburden.

6. Slowly withdraw the casing as the grout pressure increases and casing rotation slows indicating grout take refusal. Continue this procedure for the full length of the anchor bond zone.

7. After pressure grouting is complete, remove the drill casing while maintaining the hole full of grout.

8. After the anchor grout is cured, place the stressing hardware and begin tendon stressing.

3.5 Testing and Instrumentation

The instrumentation and testing of the soil anchors and the performance of the completed wall are described by Mr. Lee W. Abramson in his presentation at this symposium entitled "Geotechnical Instrumentation of Modern Retaining Walls in an Urban Setting". These items, therefore, will not be discussed herein.

3.6 Conclusions

The permanently anchored retaining wall provided the following advantages:

1. Temporary shoring was utilized as a structural element of the completed wall.

2. The depth of excavation was reduced from that required for the conventional wall.

3. The developement width was minimized significantly since large spread footings were not required.

4. The architectural finish of the facing matched existing walls in the vicinity. In addition, less formwork was required to cast the wall facing than is required for a conventional wall.

5. Streets immediately behind the wall are permitted to remain open since the development width was reduced.

6. Most of all, an architecturally pleasing and structurally functional retaining wall was constructed at a considerable cost savings.

4.0 PERMANENTLY ANCHORED CYLINDER PILE WALL

4.1 Project Description

As part of the Interstate Highway Reconstruction Program in Atlanta, Georgia, it was necessary to construct a retaining wall to provide for the widening of I-85 at Peachtree Street. The proposed design consisted of about 360 feet of conventional reinforced concrete cantilever wall and another 210 lineal feet of a specially designed 'L' type wall. The specially designed 'L' wall was needed because a portion of the wall was required to be located within 1 foot of an existing 12 story reinforced cast-in-place concrete frame office building with an attached 4 level parking deck of similar construction. The proximity of the existing structure precluded placing a footing behind the proposed cantilevered wall stem. Also, at this location I-85 is twenty feet lower than Peachtree Street (ground floor of the parking garage).

A temporary bracing system was required to construct the conventional wall. The proposed bracing system consisted of driving sheet piling between the existing structure and the proposed retaining wall. A system of wide-flange walers, compression rakers, concrete deadmen and temporary continuous sheeting behind the deadmen was required to hold the sheeting in position. In addition, staged excavation and preloading of the rakers was necessary. The raker spacing was about 6 feet. Three tiers of rakers were required and separate deadmen were to be used for each set of 3 rakers.

Calculated wall deflections during construction and for the completed wall were on the order of 1-3/4 inches, with the majority of this deflection occurring during construction. This magnitude of deflection was alarming due to the proximity of the adjacent building and the potential for adverse affects on the structure. Therefore, alternative methods of construction were considered.

In light of the above, Nicholson Construction Company developed a construction technique and permanent wall installation that would reduce wall deflections considerably and would permit construction of the retaining wall with minimimal ground disturbance and within available right-of-way. This concept is called a permanently anchored cylinder pile wall. The wall is as yet unbuilt and our final designs are currently being reviewed by the Georgia Department of Transportation. Our anchored wall is unique because ground anchors were not permitted to extend beneath the adjacent building. In addition, wall deflections were to be limited to a maximum of 3/4 of an inch at the top of the wall, while accommodating building surcharges and at rest earth pressures. As mentioned previously, the space available for construction was limited in that the right-of-way was as narrow as 4 feet and the pavement edge of Interstate I-85 was within 10 feet of the wall.

4.2 Wall Components

A schematic section of the building and the proposed wall is presented in Figure 9. The wall consists of 42 inch diameter, 40 foot long cylinder piles, bearing on rock, located at 42 to 57 inches center to center spacings. Prestressing steel strands or tendons are eccentrically draped in the caisson and are anchored in 20 to 25 feet of rock. Tendons composed of 23 to 28, 0.6 inch diameter, 270 ksi, 7-wire strands are required to accommodate the design loads. The steel tendons are subsequently post-tensioned with a load of about 700 to 870 kips. A 14 inch eccentricity in the tendon locations was provided to essentially pull the top of the caisson back towards the soil and thus reduce wall deflections. The anchored cylinder piles are essentially post-tensioned beams and allow the piles to function as a cantilevered retaining wall. In addition, the anchored piles form a relatively rigid wall system to control deflections.

Positive drainage of the wall is provided by placing pre-formed drainage material (EnkaDrain) on the exposed soil between the caissons. The wall is completed by constructing a reinforced concrete facing on the exposed pile surfaces. The facing surface is formed to provide an architectural finish identical to walls in the vicinity.

4.3 Construction Sequence

The sequence of construction is as follows:

1. Auger the 42 inch diameter cylinder pile holes.

2. Place the reinforcing steel cage in the piles. Attached to the rebar cage is an 8 inch I.D. conduit. The conduit is positioned to provide the proper eccentricity for the steel tendons.

3. Place the concrete in the piles.

4. Drill the rock sockets for the steel tendons by drilling through the 8 inch I.D. conduit.

5. Grout the steel tendons in place to form a rock anchor.

6. Stress the steel tendons after the concrete pile has reached 4000 psi compressive strength.

7. Upon completion of anchor stressing begin excavation in front of the wall.

8. As excavation is progressed place pre-formed drainage fabric on the exposed soil face between the piles; embed shear studs for the concrete facing in the piles; place reinforcing steel and formwork for the concrete facing.

9. Cast the concrete facing to complete the wall.

4.4 Conclusions

There are several advantages associated with our permanently anchored cylinder pile wall, including:

1. The wall requires minimal space for construction.

2. Temporary shoring is not required for construction.

3. The magnitude of wall deflections is minimized and are calculated to be 3/4 of an inch or less.

4. Construction time is reduced.

5. Lateral support for adjacent structures exceeds that of conventional methods.

6. The amount of excavation required is about 50% less since the proposed conventinal wall was to bear on rock.

5.0 SUMMARY

Recent advances in the technology related to permanently anchored structures are truly remarkable. Not only have anchor loads become progressively greater, but the range of suitable anchoring strata has widened to include materials that but a few years ago were considered to be very difficult if not impossible to utilize. Simultaneously, there has been a growth in awareness and confidence in the use of permanent ground anchors by the engineering community. Therefore, permanent ground anchors are now seen as an important engineering tool.

And what of the future? There will be continuing improvements in the understanding of the behaviour of soils under load and thus a widening of the range of soil types and anchor loads possible. Drilling techniques and equipment will continue to improve, resulting in improved anchor performance and rapid installation. Even though these advances will be made, the most noticable developments will probably occur in anchor applications. The two examples cited above are just the tip of the iceberg. The future holds great promise for extending the use of tensioned ground anchors into many novel and exciting applications.

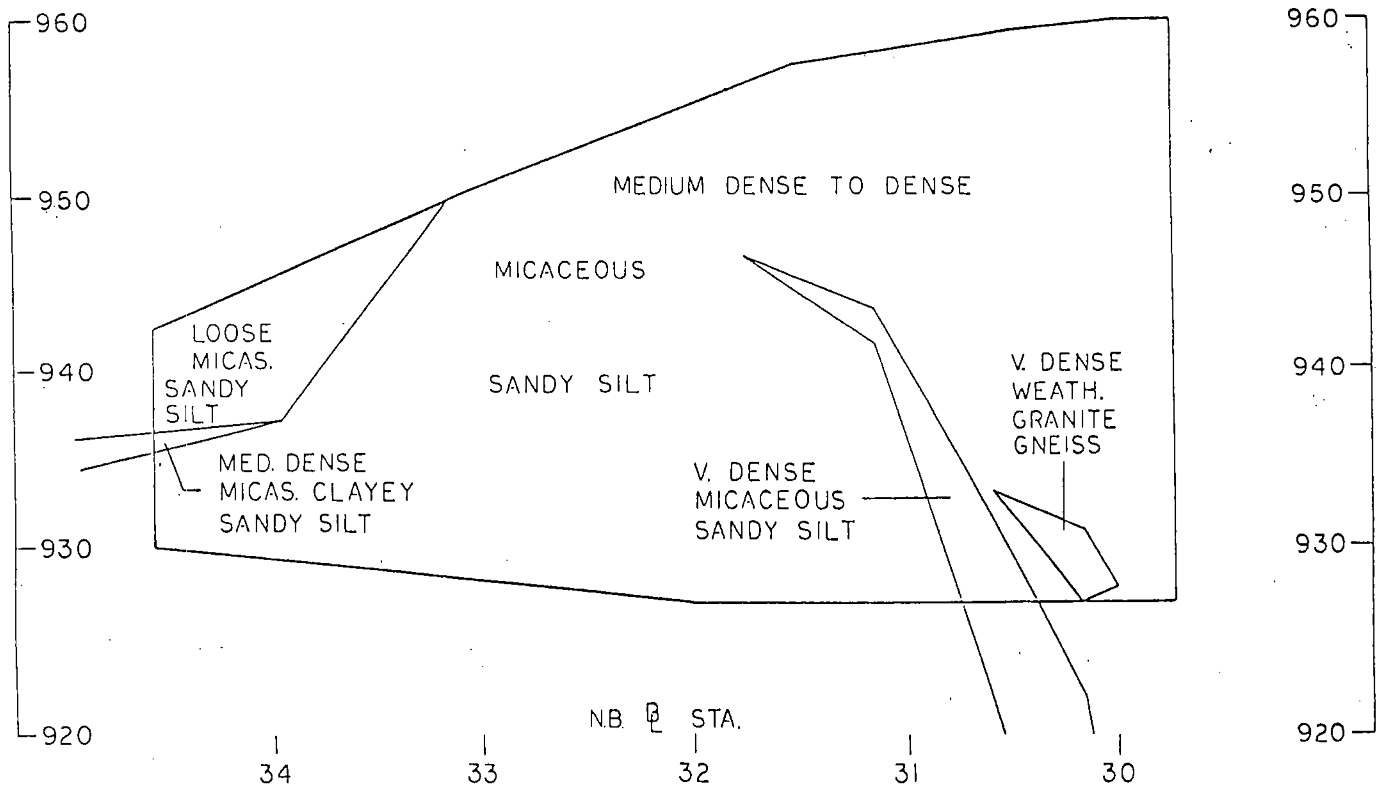
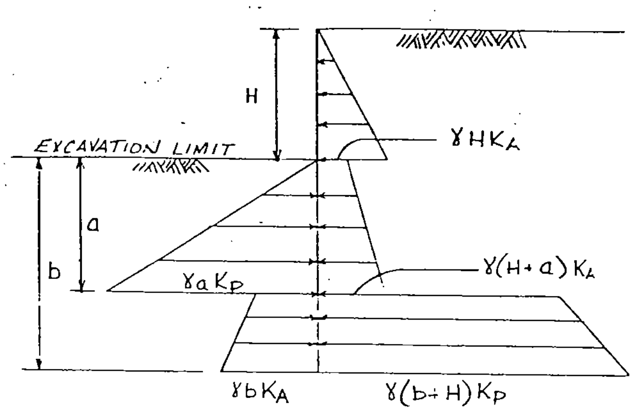


FIG. I SOIL PROFILE WALL E-1



LEGEND:

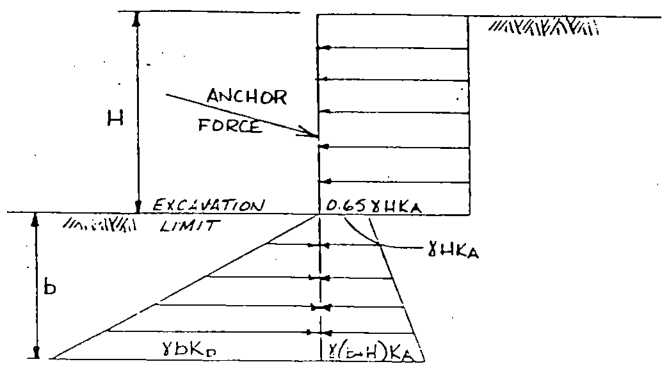
γ — UNIT WEIGHT OF SOIL
K_A — COEFFICIENT OF ACTIVE EARTH PRESSURE
K_P — COEFFICIENT OF PASSIVE EARTH PRESSURE

NOTES:

1. THE PRESSURE DIAGRAM PRESENTED IS FOR COHESIONLESS SOIL.
2. WHERE LAGGING IS IN PLACE, ACTIVE EARTH PRESSURE ACTS OVER ENTIRE WALL SURFACE. BELOW LAGGING ACTIVE EARTH PRESSURE ACTS ONLY ON THE SOLDIER PILE WIDTH AND PASSIVE EARTH PRESSURE IS GENERATED OVER THREE TIMES THE SOLDIER PILE WIDTH.
3. IN ADDITION TO THE ABOVE SOIL LOADINGS A 250 P.S.F. SURCHARGE SHOULD BE PLACED BEHIND THE WALL.

FIGURE 2

—DESIGN PRESSURE DIAGRAM FOR CANTILEVERED CONDITION PRIOR TO ANCHOR INSTALLATION—



LEGEND:

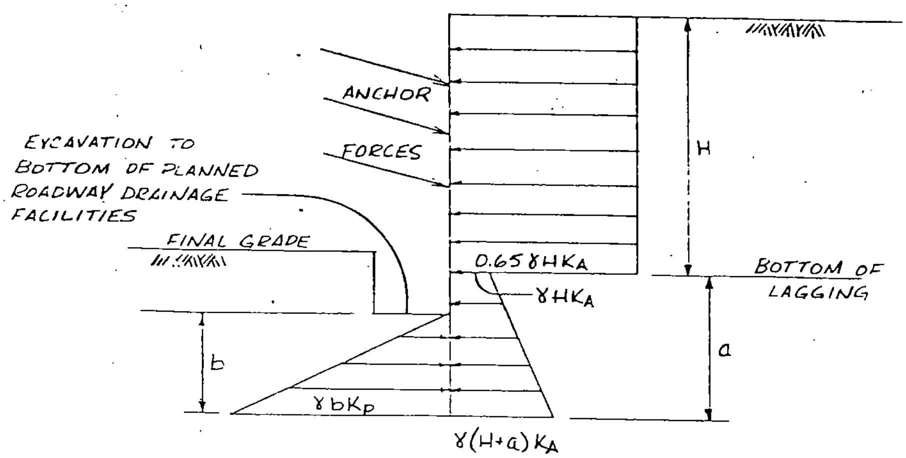
ɣ — UNIT WEIGHT OF SOIL
K_A — COEFFICIENT OF ACTIVE SOIL PRESSURE
K_P — COEFFICIENT OF PASSIVE SOIL PRESSURE

NOTES:

1. THE PRESSURE DIAGRAM PRESENTED IS FOR COHESIONLESS SOIL
2. WHERE LAGGING IS IN PLACE, ACTIVE EARTH PRESSURE ACTS OVER ENTIRE WALL SURFACE. BELOW LAGGING, ACTIVE EARTH PRESSURE ACTS ONLY ON THE SOLDIER PILE WIDTH AND PASSIVE EARTH PRESSURE IS GENERATED OVER THREE TIMES THE SOLDIER PILE WIDTH.
3. VERTICAL COMPONENTS OF ANCHOR FORCES MUST BE RESISTED BY EMBEDDED LENGTH OF SOLDIER PILES BELOW ASSUMED EXCAVATION.
4. IN ADDITION TO THE ABOVE SOIL LOADINGS A 250 P.S.F. SURCHARGE SHOULD BE PLACED BEHIND THE WALL.

FIGURE 3

— DESIGN PRESSURE DIAGRAM
FOR INTERMEDIATE EXCAVATIONS AND SUBSEQUENT
ANCHOR INSTALLATIONS —



LEGEND;

γ – UNIT WEIGHT OF SOIL
K_A – COEFFICIENT OF ACTIVE EARTH PRESSURE
K_P – COEFFICIENT OF PASSIVE EARTH PRESSURE

NOTES;

1. THE PRESSURE DIAGRAM PRESENTED IS FOR COHESIONLESS SOIL.
2. WHERE LAGGING IS IN PLACE, ACTIVE EARTH PRESSURE ACTS OVER ENTIRE WALL SURFACE. BELOW LAGGING ACTIVE EARTH PRESSURE ACTS ONLY ON THE SOLDIER PILE WIDTH AND PASSIVE EARTH PRESSURE IS GENERATED OVER THREE TIMES THE SOLDIER PILE WIDTH.
3. VERTICAL COMPONENTS OF ANCHOR FORCES MUST BE RESISTED BY EMBEDDED LENGTH OF SOLDIER PILES BELOW ASSUMED EXCAVATION.
4. IN ADDITION TO THE ABOVE SOIL LOADINGS A 250 P.S.F. SURCHARGE SHOULD BE PLACED BEHIND THE WALL AND A 10 KIP HORIZONTAL LOAD SHOULD BE PLACED AT THE TOP OF THE WALL TO ACCOMODATE WIND AND IMPACT LOADS.

FIGURE 4
—DESIGN PRESSURE DIAGRAM FOR FINAL CONSTRUCTED CONDITION—

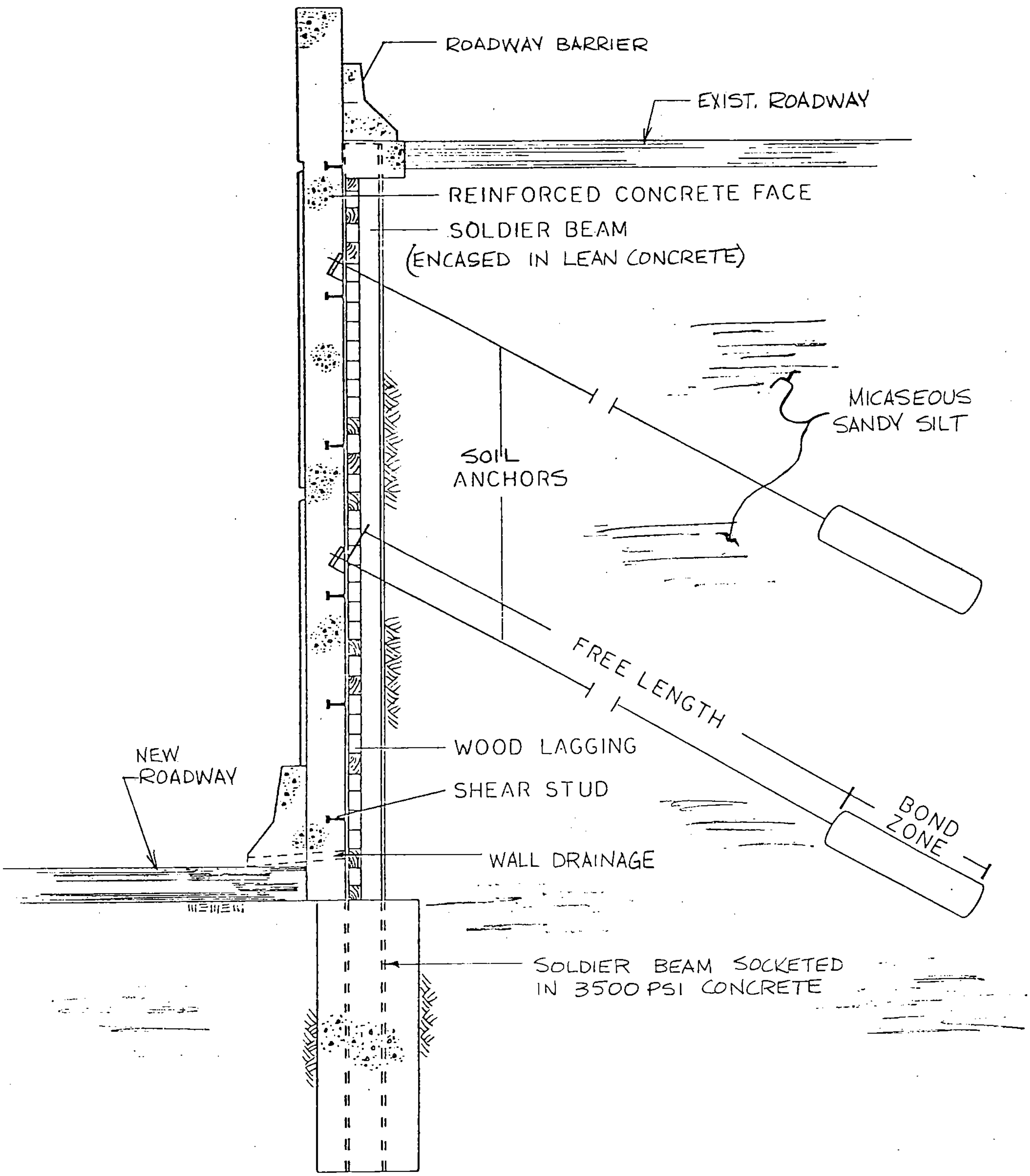


FIG. 5

—SCHEMATIC SECTION - WALL E-1—

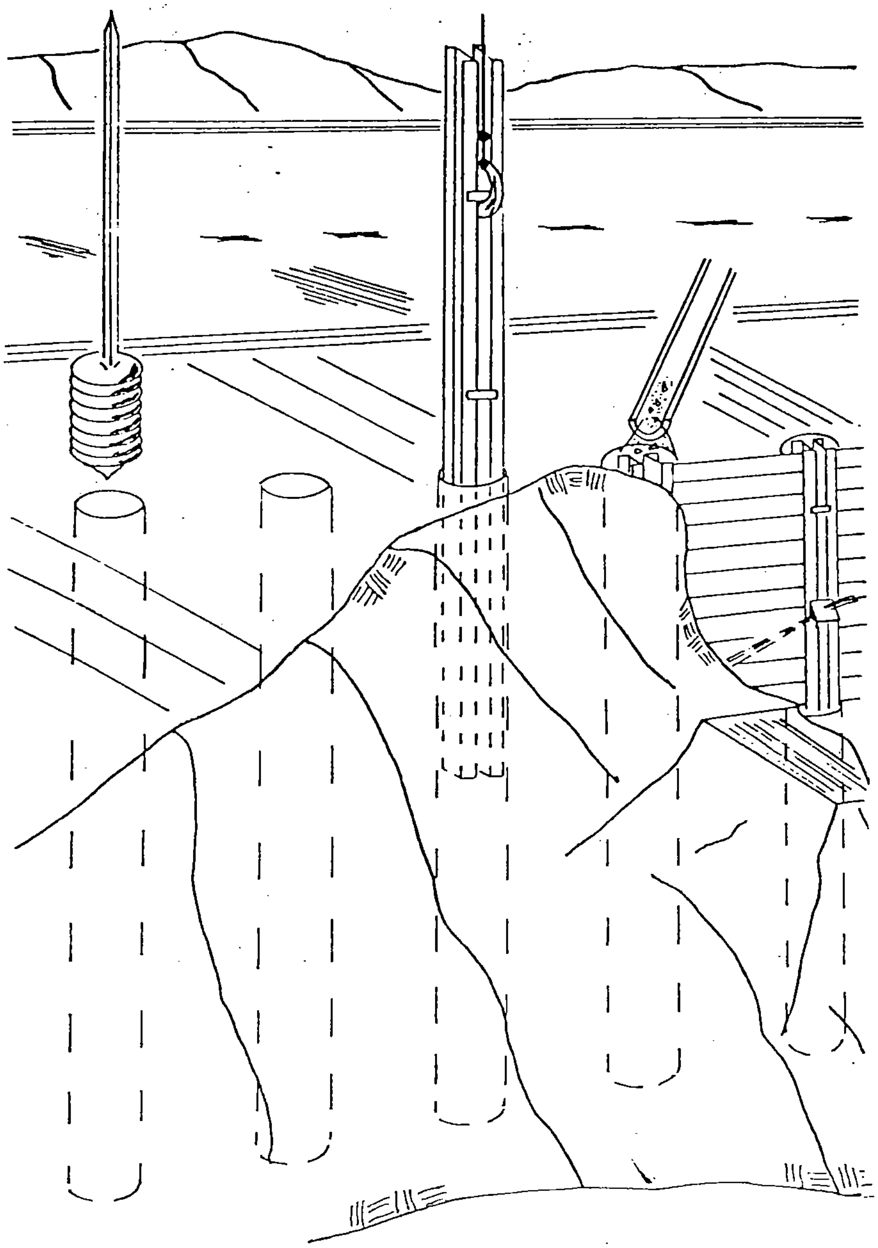


FIG. 6

CONSTRUCTION SEQUENCE 1

— DRILLING AND PLACING OF SOLDIER BEAMS —

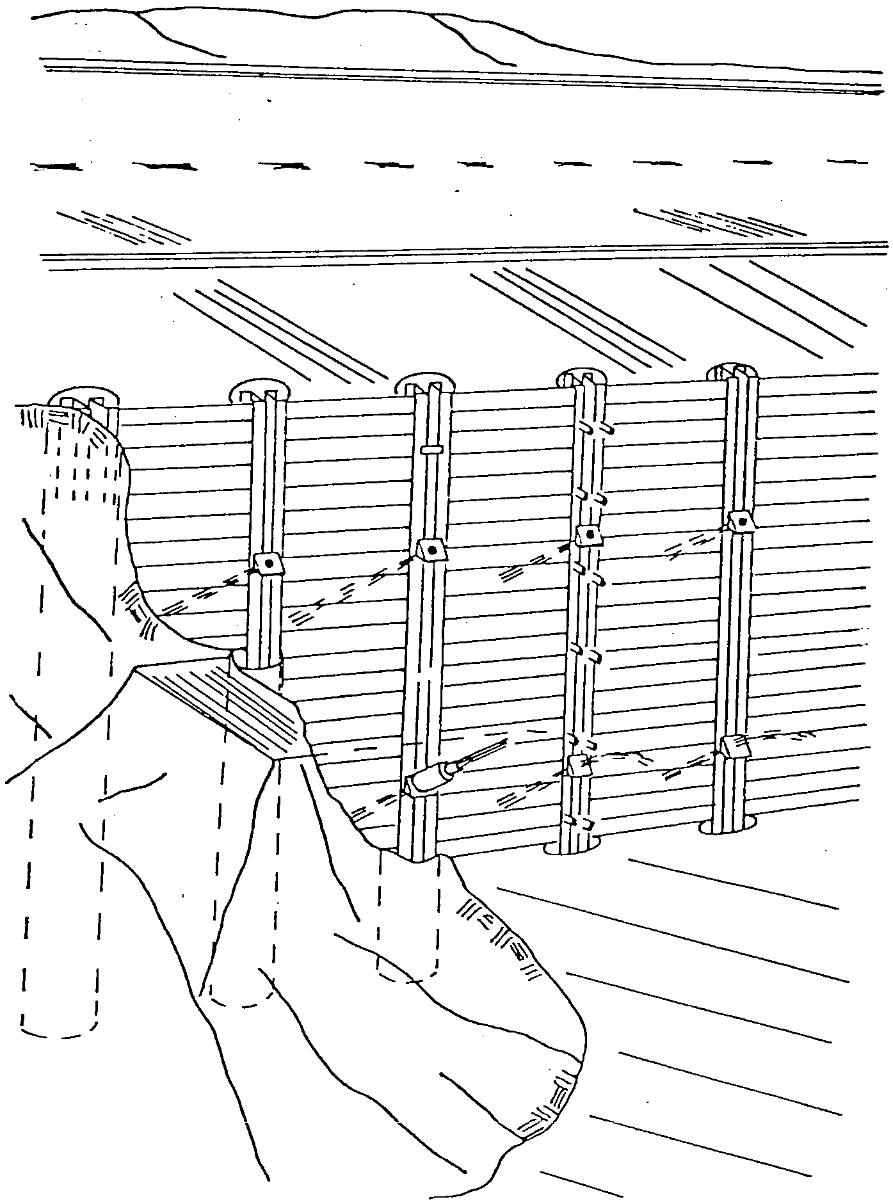


FIG. 7

CONSTRUCTION SEQUENCE 2

—EXCAVATION AND PLACING OF LAGGING AND SOIL ANCHORS—

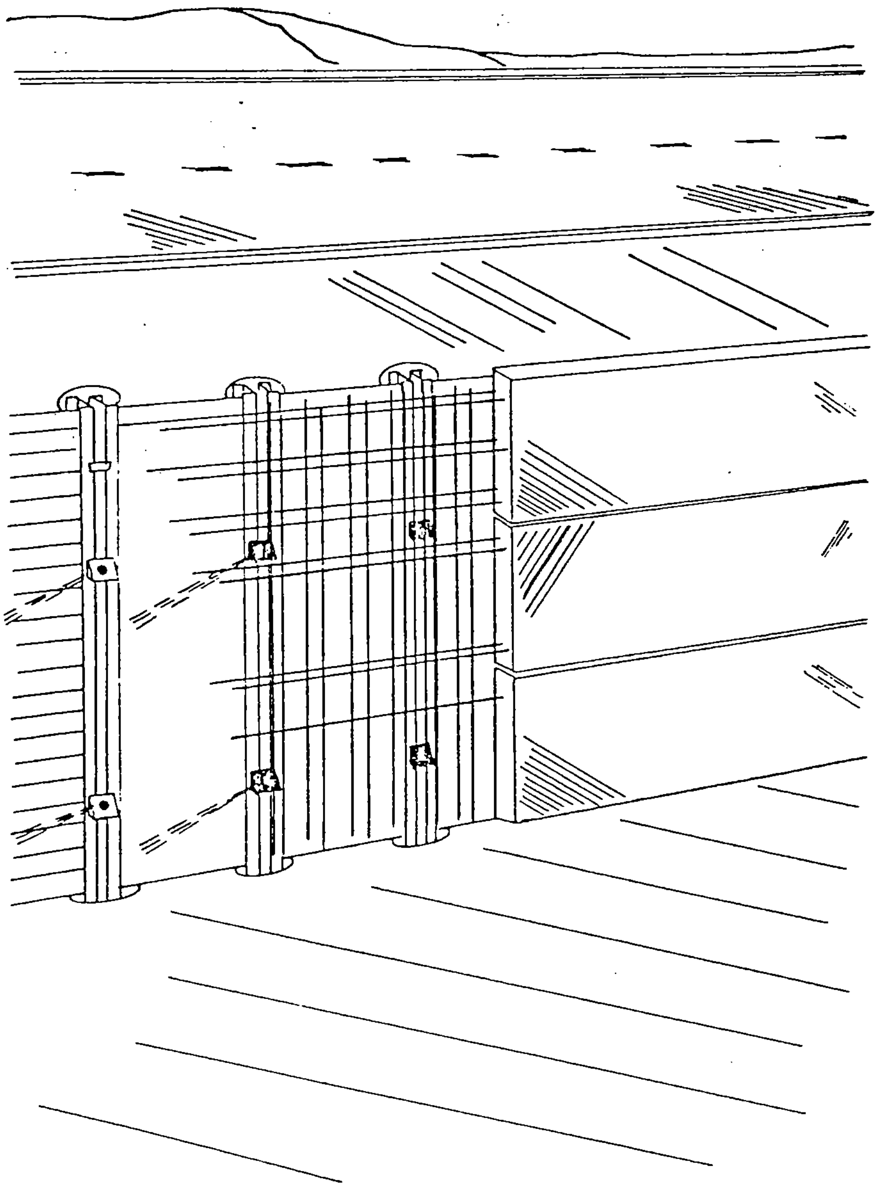


FIG. 8

CONSTRUCTION SEQUENCE 3

—PLACEMENT OF REINFORCING STEEL AND CASTING OF CONCRETE FACING—

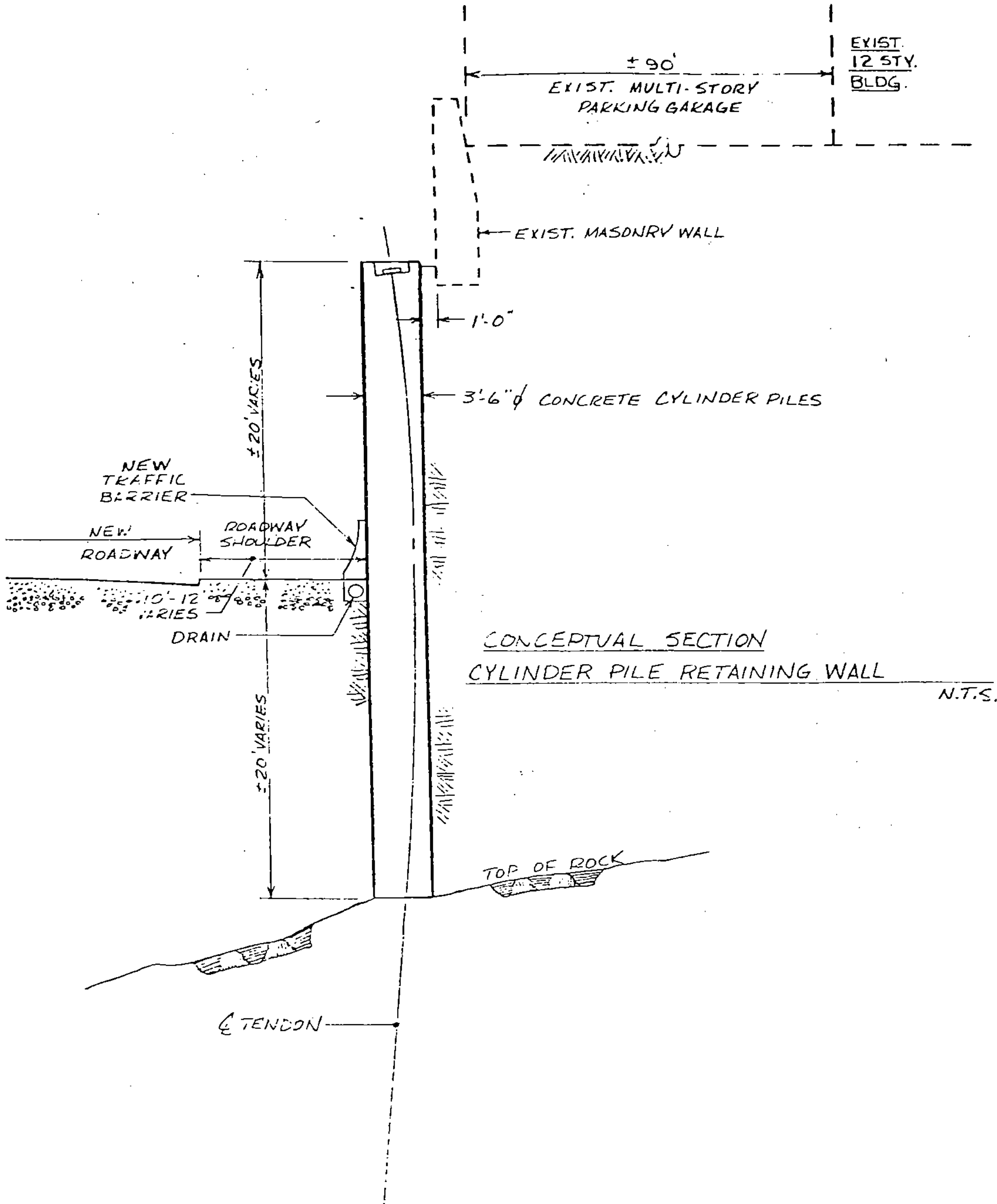


FIG. 9 CONCEPTUAL SECTION
—PERMANENTLY ANCHORED
CYLINDER PILE RETAINING WALL—

GEOTECHNICAL INSTRUMENTATION OF MODERN RETAINING WALL DESIGNS IN AN URBAN SETTING

Lee W. Abramson, Project Manager
Law/Geoconsult International, Inc.*
Marietta, Georgia

PREFACE

This paper was written to present data from two very interesting Ga. D.O.T. projects at the time of the 34th Annual Highway Geology Symposium (May, 1983). The two projects consist of an instrumented permanent tieback wall and an instrumented earth retaining system developed by Ga. D.O.T. Instrumentation monitoring is scheduled to be a long-term endeavor and will continue beyond May, 1983. This paper should in no way be construed to present final results and/or conclusions. The information contained herein is the opinions of the author (prior to completion of the project) and not necessarily the opinions of Ga. D.O.T., F.H.W.A., Law/Geoconsult International, Law Engineering Testing Co., the Contractors or the Consultants. The interested reader should contact Ga. D.O.T. for the final results of these instrumentation programs.

*Presently employed by Parsons Brinckerhoff, New York

GEOTECHNICAL INSTRUMENTATION OF MODERN RETAINING WALL DESIGNS IN AN URBAN SETTING

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INTRODUCTION

An extensive refurbishing program is being conducted on Atlanta's highways. Many new designs and methods of construction are being used to develop a modern and economical interstate system through an urban environment. Two innovations in retaining wall design and construction include permanent tieback retaining walls where space limitations preclude the use of conventional designs and stabilized earth embankments where high economical cuts or fills are required. One of each of these retaining structure types are being instrumented for performance monitoring and subsequent enhancement of design philosophy. Data has been collected during construction of the walls and will continue to be collected for a long period of time following construction. This paper describes the monitoring methods and results of the data gathered during the first six months of the projects. The purposes of instrumenting the permanent tieback wall were to monitor loads in the anchors, to examine the load transfer characteristics of soil anchors and to monitor horizontal deflection of the retaining system and the earth behind it. The stabilized embankment was monitored to examine stresses developed in the steel reinforcing mats, strains developed in the earth mass and vertical and horizontal earth pressures developed under and behind the stabilized embankment. The permanent tieback wall will be discussed first followed by the stabilized earth embankment.

PERMANENT TIEBACK WALL E-1

Project Description

Earth anchored tieback walls have previously been used in Georgia for temporary earth retaining systems but have never been used as permanent retaining walls. Widening the I-75 highway near the Penthouse Motel and the Coca Cola Bottling Company required the construction of a retaining wall (Figure 1). The right-of-way limitations and potential effects of construction on the adjacent structures limited the type of retaining wall that could be built at this location. The normal solution to such a problem would be to construct a temporary earth anchored tieback wall and to then construct a permanent cantilevered retaining wall in front of the temporary system. However, with the ap-

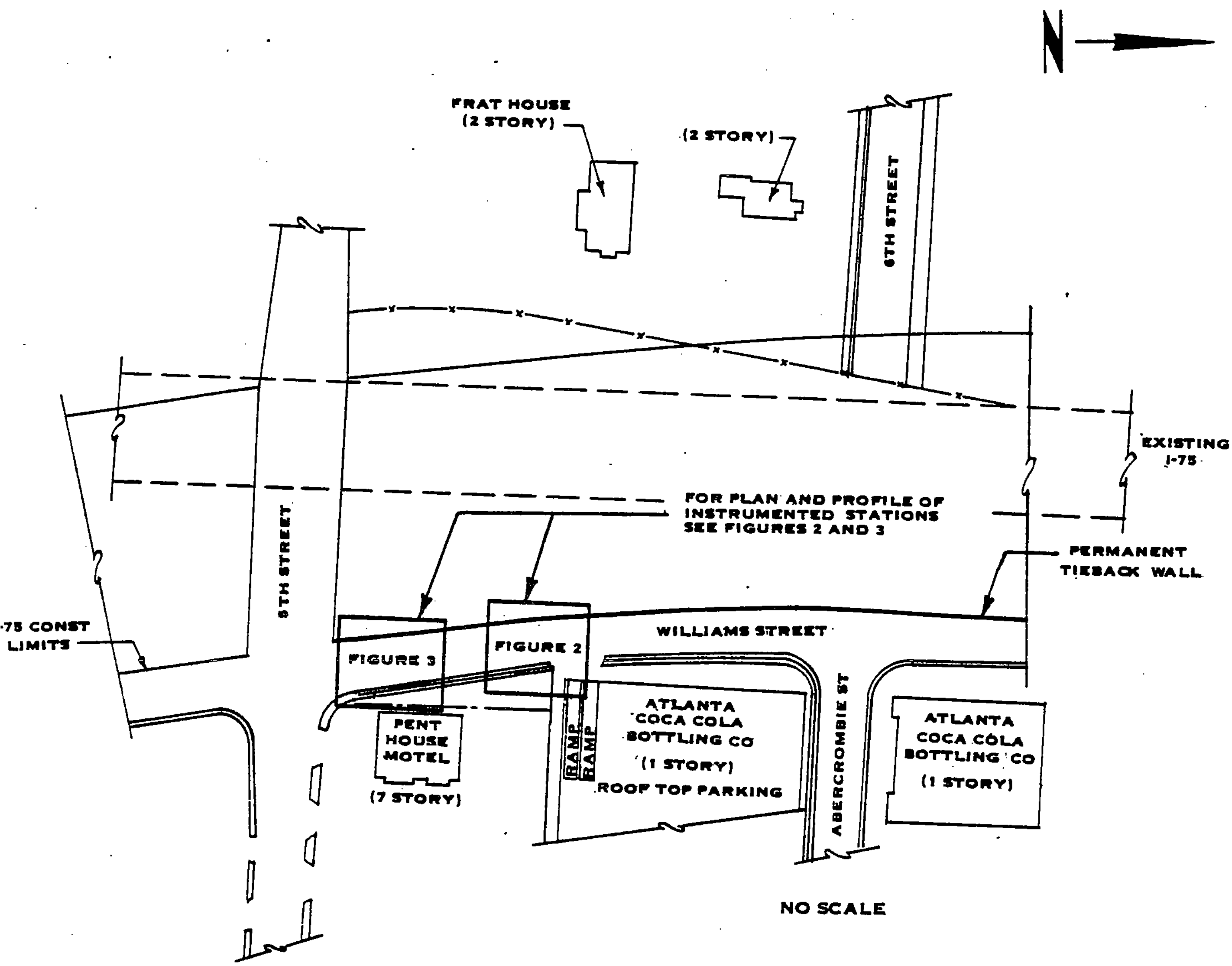


FIGURE 1 - SITE PLAN OF GEORGIA DEPARTMENT OF TRANSPORTATION INSTRUMENTED PERMANENT TIEBACK WALL

proval from the Federal Highway Administration (FHWA), the Georgia Department of Transportation (DOT) chose to construct one of Georgia's first permanent earth anchored walls at this location. This wall combined the temporary tieback wall and the permanent retaining wall and resulted in significant cost savings. Since the use of the permanent earth anchored wall is relatively new, the FHWA chose the wall as a Federal demonstration project. The wall was instrumented to determine the long term performance of such a system.

Method of Construction

The method of construction used in this tieback wall involved several components and sequences. First, the soldier piles were placed in a row along the proposed wall face in augered holes extending from the top of the wall to a designated point below the bottom of the wall. The members used for the soldier piles ranged from 1-W18 x 50 steel beam at the lowest portion of the wall to 2-W16 x 40 at the highest portion. Concrete was poured in place around the piles from the bottom of the hole to the proposed ground line. Lagging was installed between the piling as the earth in front of the wall was excavated. At designated levels, holes were drilled through the piling into the earth behind the wall, at calculated lengths and angles. The 7 wire - 5 strand (0.6 inch diameter) tendons were then installed and grout was pressure injected at 2 pounds per square inch per foot of overburden around them.

After curing, the anchors were load tested and post tensioned to a predetermined load. The process of lagging installation and anchor installation was repeated until the excavation in front of the wall was complete. The final sequence was to install strips of drainage fabric along the lagging and cast concrete face panels over the pile, lagging and tieback retaining system. The concrete was attached to the soldier piles by a series of studs embedded in the concrete.

The predetermined lock-off load for the anchors is normally 80% of the "design load". Occasionally conditions in the field were different from that assumed in design and the anchor would not hold the load for which it was designed. When this happened, the "design load" was reduced to reflect conditions in the field and the designers assessed what affect the change in anchor load might have on the retaining system.

Subsurface Conditions

The soils immediately affecting the wall at this site are medium dense to dense yellowish-brown and pinkish micaceous sandy silts and silty sands. Standard Penetration Test resistances range from 11 blows per foot to 60 blows with no penetration. Soil density generally increases with depth but hard and soft layers occur frequently. Large slickensided surfaces were observed in the field. Groundwater was encountered approximately

40 feet below the ground surface. The depth to hard rock varies from 45 to 65 feet. Boring logs and a more detailed description of site geology may be found in the Appendix.

Instrumentation

The instrumentation of the wall consists of many different monitoring devices. The internal performance of the wall is being monitored through ten anchors that have been instrumented with some combination of rod telltales, wire telltales and permanent load cells. Conventional strain gages could not be used with the 7 wire - 5 strand anchors because of the inherent difficulty of mounting the gages to the tendons. Telltales were chosen as an alternative with a custom fitted fixation to the strand. The external movement of the wall system and adjacent structures are being monitored with slope inclinometers and optical survey points. The instruments are to be monitored over a three-year period to evaluate the performance of the wall.

Two monitoring stations, spaced approximately 135 feet apart, were used to monitor the wall performance at the highest portions of the wall (Figures 2 and 3). Instrumentation for the primary instrumented anchors (Figure 4) consisted of one rod telltale (RT) fixed to one of the strands of the 5-strand anchor at the interface of the unbonded and bonded zones (instrument position designation 1). Also to this strand, five wire telltales (WT) were fixed within the bonded zone spaced approximately 7.5 feet apart (instrument position designations 1 through 5). A second rod telltale was fixed to one of the remaining four strands at position 5. A cross section of a primary instrumented anchor is shown in Figure 5. When the anchor was installed, a permanent load cell (LC) was mounted at the anchor lock-off point. The secondary instrumented anchors had only one rod telltale fixed to one of the 5 strands at position 1. No permanent load cell was used. It was assumed that all 5 strands in the anchors behaved the same and instrumentation of one strand would predict the behavior of the entire anchor.

Each instrument was given a unique code for identification. This code is presented below.

W - XY - Z

where W = Instrument Type
- RT = Rod telltale
- WT = Wire telltale
- LC = Load cell
- SI = Slope inclinometer

X = Pile Number
See Figures 2 and 3

Y = Anchor Locations
U = Upper or L = Lower

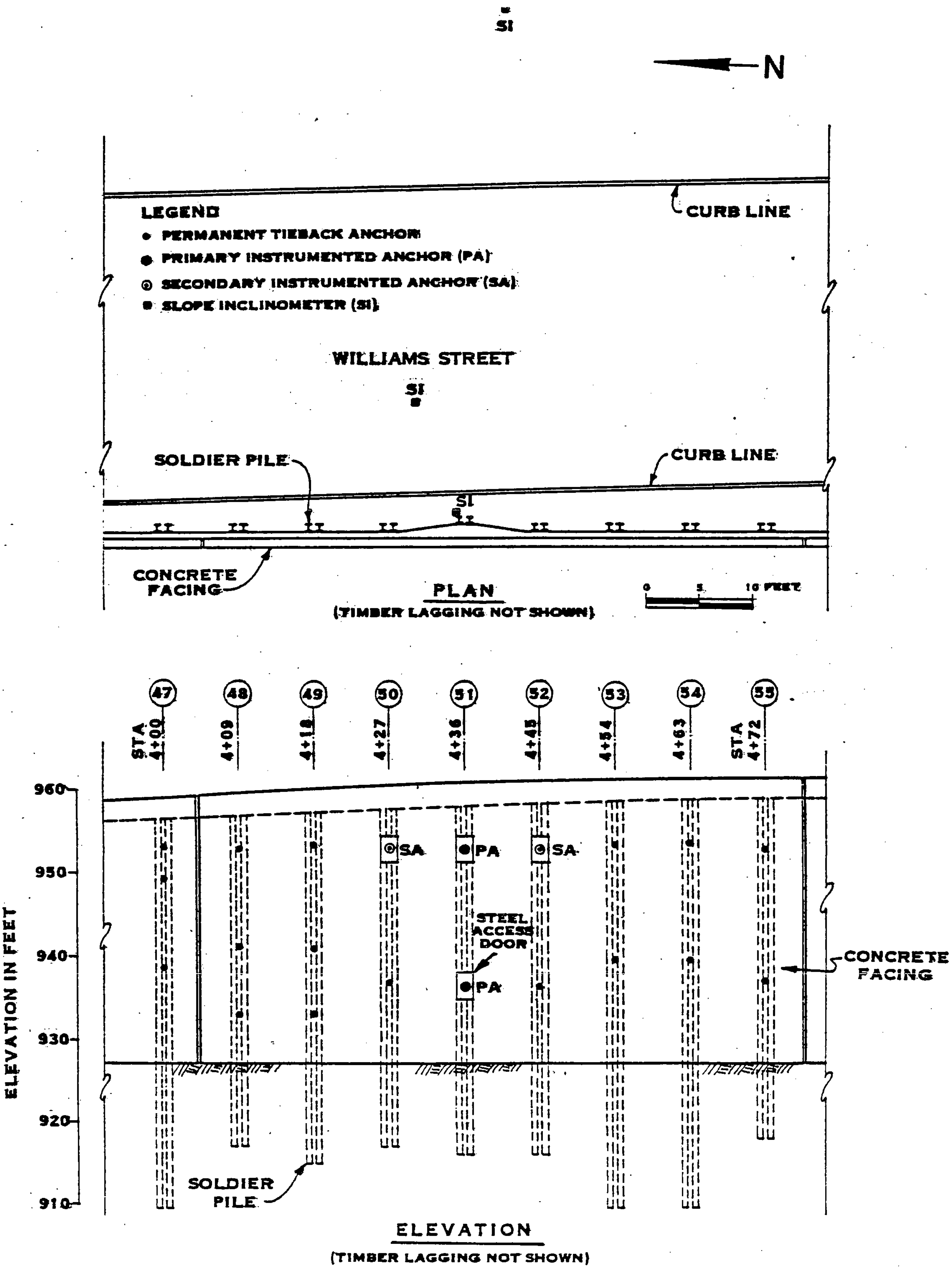


FIGURE 2 - PERMANENT TIEBACK WALL INSTRUMENTATION

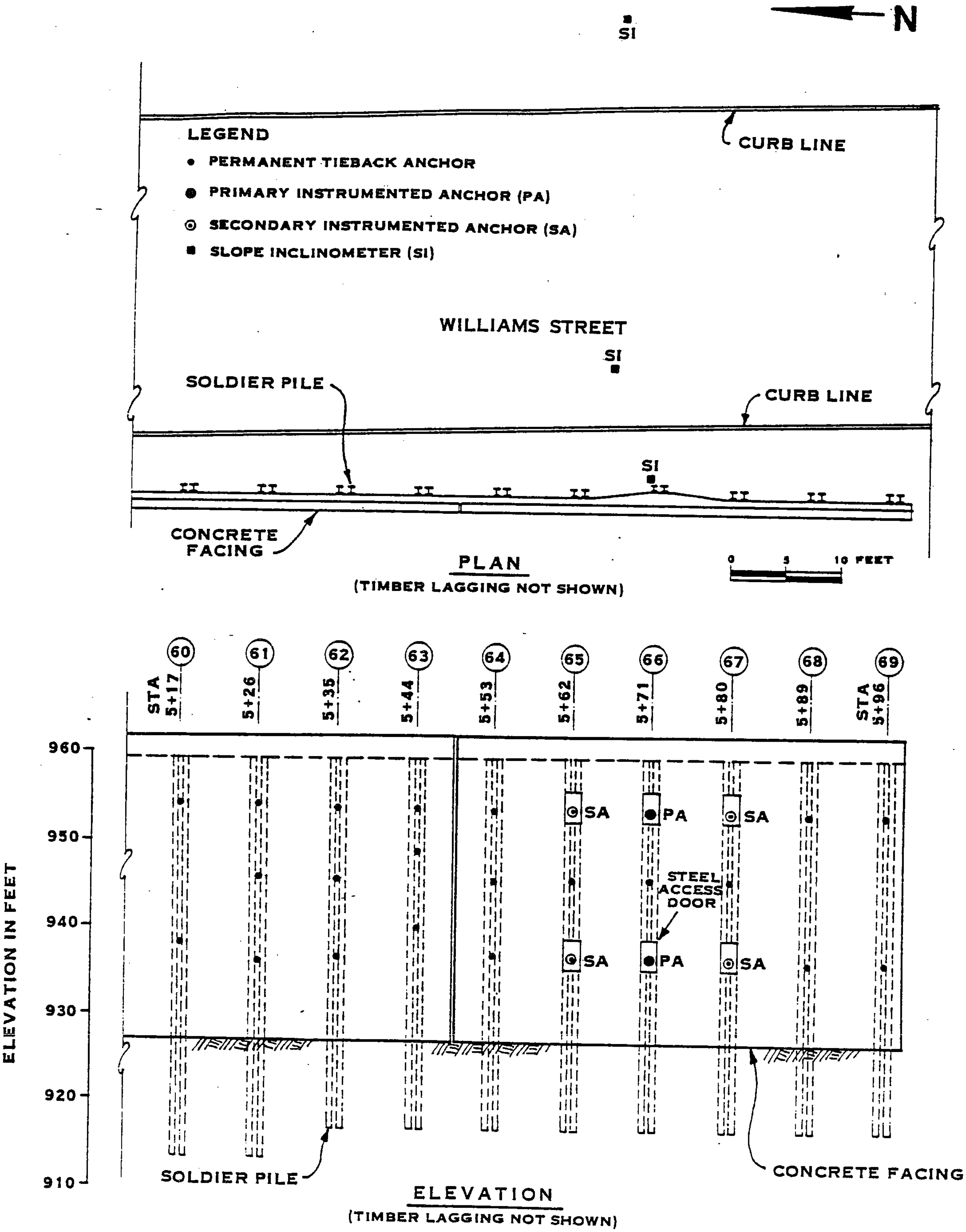


FIGURE 3 - PERMANENT TIEBACK WALL INSTRUMENTATION

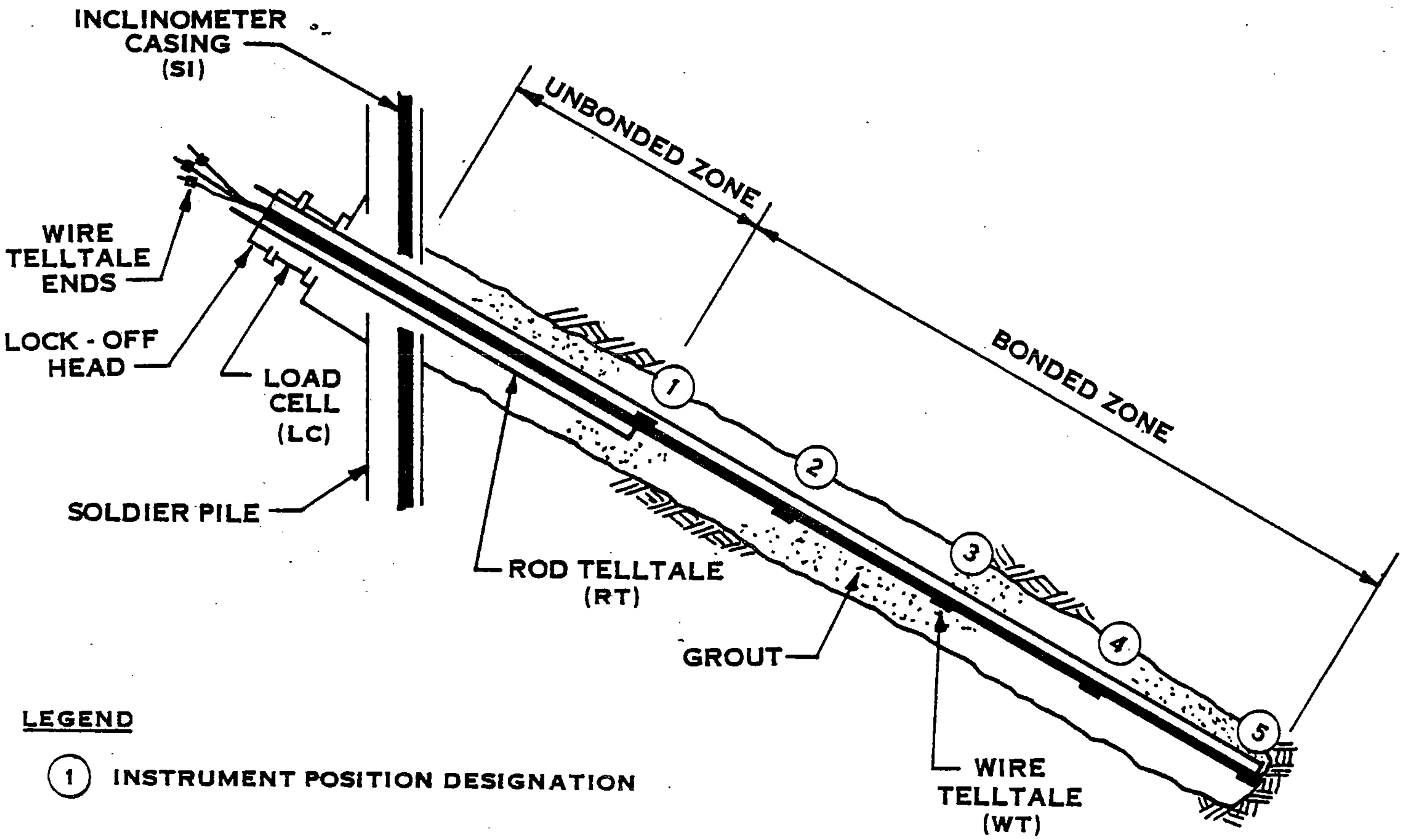


FIGURE 4 - SCHEMATIC OF TYPICAL INSTRUMENTED TIEBACK ANCHOR

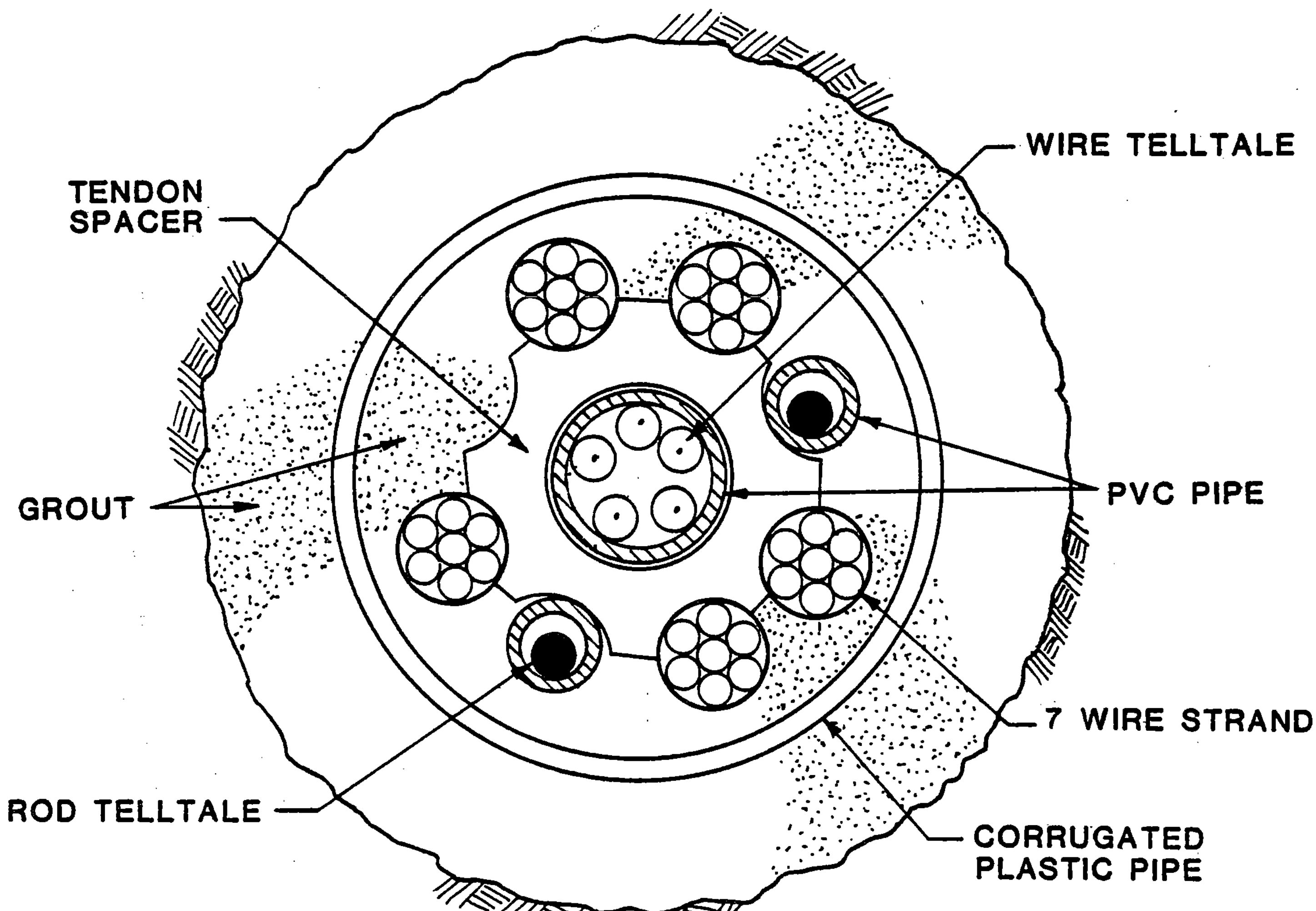


FIGURE 5 - CROSS SECTION OF INSTRUMENTED TIEBACK ANCHOR

Z = Position Designation
See Figure 4

At each monitoring station there are two primary instrumented anchors on the same soldier pile (upper and lower). Each of these primary instrumented anchors has a secondary instrumented anchor on both adjacent soldier piles, except for the lower primary anchor on Pile 51. The secondary anchor instrumentation at this location did not survive installation and was abandoned.

Testing Procedure

The instrumented anchors were proof tested by loading the anchor in increments to approximately 150% of its predicted design load. This was accomplished using a 150 ton capacity hydraulic jack and electric pump for the secondary anchors and a 300 ton jack for the primary anchors. The primary anchors required a larger jack because the center hole in the 150 ton jack was not large enough to accommodate the instrumentation and the anchor. A load cell was used to monitor the anchor loads and displacement of the loading head was measured with a dial gage.

The acceptance criteria for the anchors is based on the elastic movement of the anchors during proof testing and creep movement at the maximum proof test load. An anchor is acceptable if:

a. The total elastic movement obtained exceeds 80% of the theoretical elastic elongation of the free length and is less than the theoretical elastic elongation to the free length plus 50% of the bond length.

b. The creep movement does not exceed 0.080 inches during the 5 minute to 50 minute time increments regardless of tendon length and load.

If the proof test fails, the anchor is subject to redesign. If it passes, the anchor is unloaded and locked off at 80% of the design load.

Normally the jack pressure gage is used to measure load during a proof test. The anchor is cycled back to the seating load so it can more accurately be loaded to the lock-off load using the gage. The instrumented anchors were loaded using a load cell. Therefore, load cycling was not required. The gage pressure was recorded at each load increment and a comparison of jack pressure load to load cell reading will be made later in this paper.

Test Results

The load variation in ten instrumented anchors has been monitored since lock-off. Four of the anchors have load cells mounted permanently on the anchor heads. All of the anchors have short (position 1) rod telltales. Once the anchor has been locked off, any position change of the rod telltale theoretically corresponds to a change in load. Data from the four anchors with load cells was studied to determine the most reliable method of predicting anchor load change, after lock-off, using the short rod telltale.

Hooke's Law proved to be the best indicator of load change. A load factor was calculated for each anchor using Hooke's Law. The strand modulus and steel area used were 28.5 x 103 kips per square inch and 1.075 square inches, respectively. The length of unbonded anchor zone in each anchor was used for the stressed length. The load factor, expressed in kips per inch, is calculated by multiplying the strand modulus with the steel area and dividing by the stressed anchor length. By applying this load factor to any change in short rod telltale position, a change in anchor load can be calculated.

Approximately 5 months of load variation data has been collected for all ten instrumented anchors since lock-off. Based on the apparent accuracy of the data the anchors have been grouped into one of three categories: Less than 95% of lock-off load, 95% to 105% of lock-off load and Greater than 105% of lock-off load. The bulk of anchor load variation took place in the first two months after lock-off (Figure 6). After two months, the anchor loads have stayed relatively constant. Twenty percent (20%) of the instrumented anchors are holding greater than 105% of the lock-off load. Seventy percent (70%) are holding between 95% and 105% of the lock-off load and ten percent (10%) are holding less than 95% of the lock-off load. The flexibility of the retaining system allows for some variation in anchor loads and readjustment of the loads does in fact take place. Therefore, minor variation in the anchor loads, in itself, should not be a cause for concern.

Load distribution in four of the instrumented anchors was measured using a series of five wire telltales in each of the bonded anchor zones. By taking the difference between the amount of movement at two adjacent wire telltale locations, the elongation of the segment of strand in between can be theoretically determined. Then, by using Hooke's Law to convert the elongation to load and accumulating the loads starting from the deepest end of the anchor, load distribution or transfer can be determined in the bonded zone (Figures 7 and 8). Some relatively small negative (compressive) loads were calculated in the tendons but are not shown in the figures and probably reflect the level of inaccuracy in the instrumentation system. Data taken during unloading of the primary anchors is currently being processed. No conclusions can be drawn at this time.

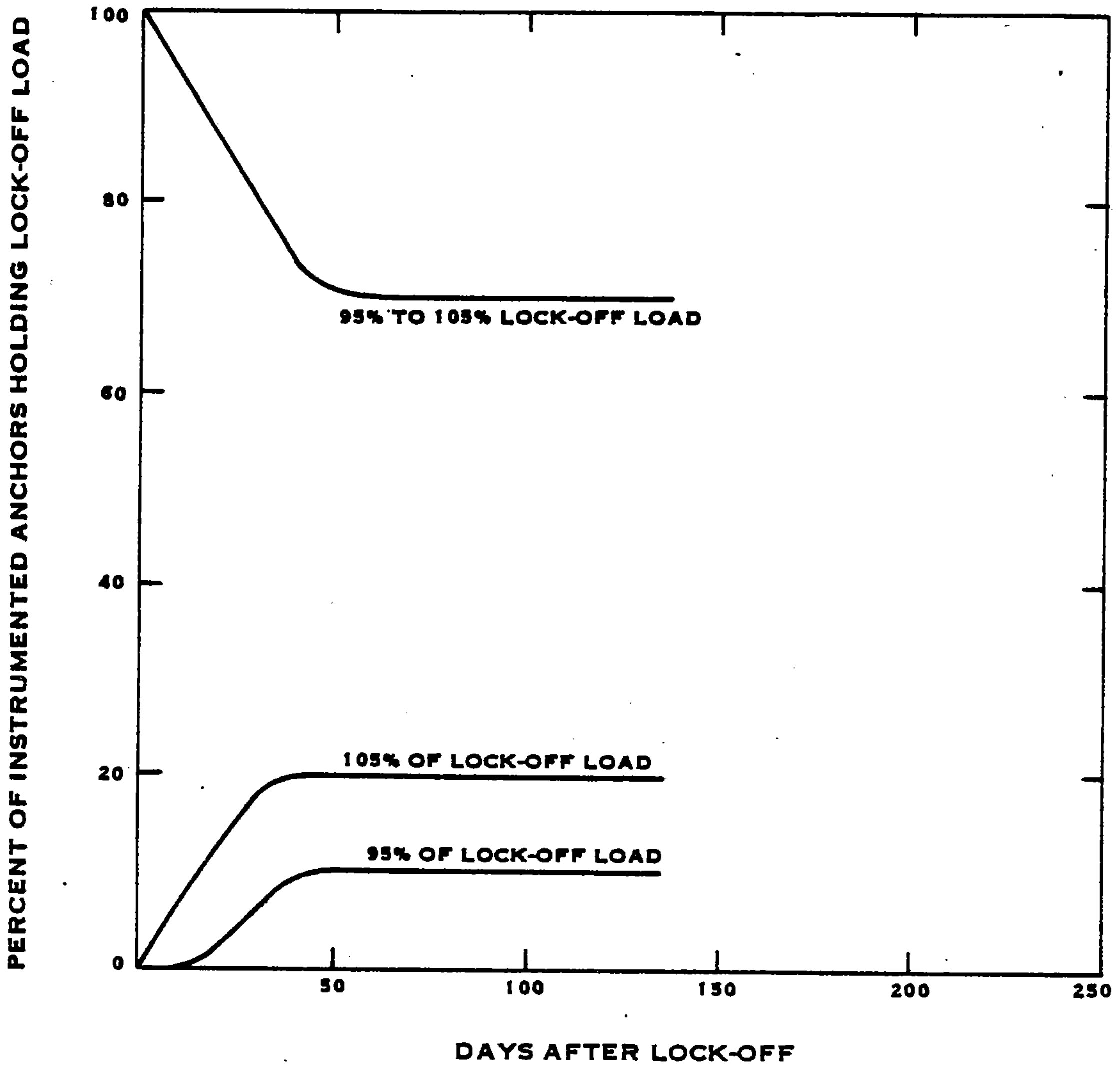


FIGURE 6 - LONG TERM INSTRUMENTED ANCHOR PERFORMANCE

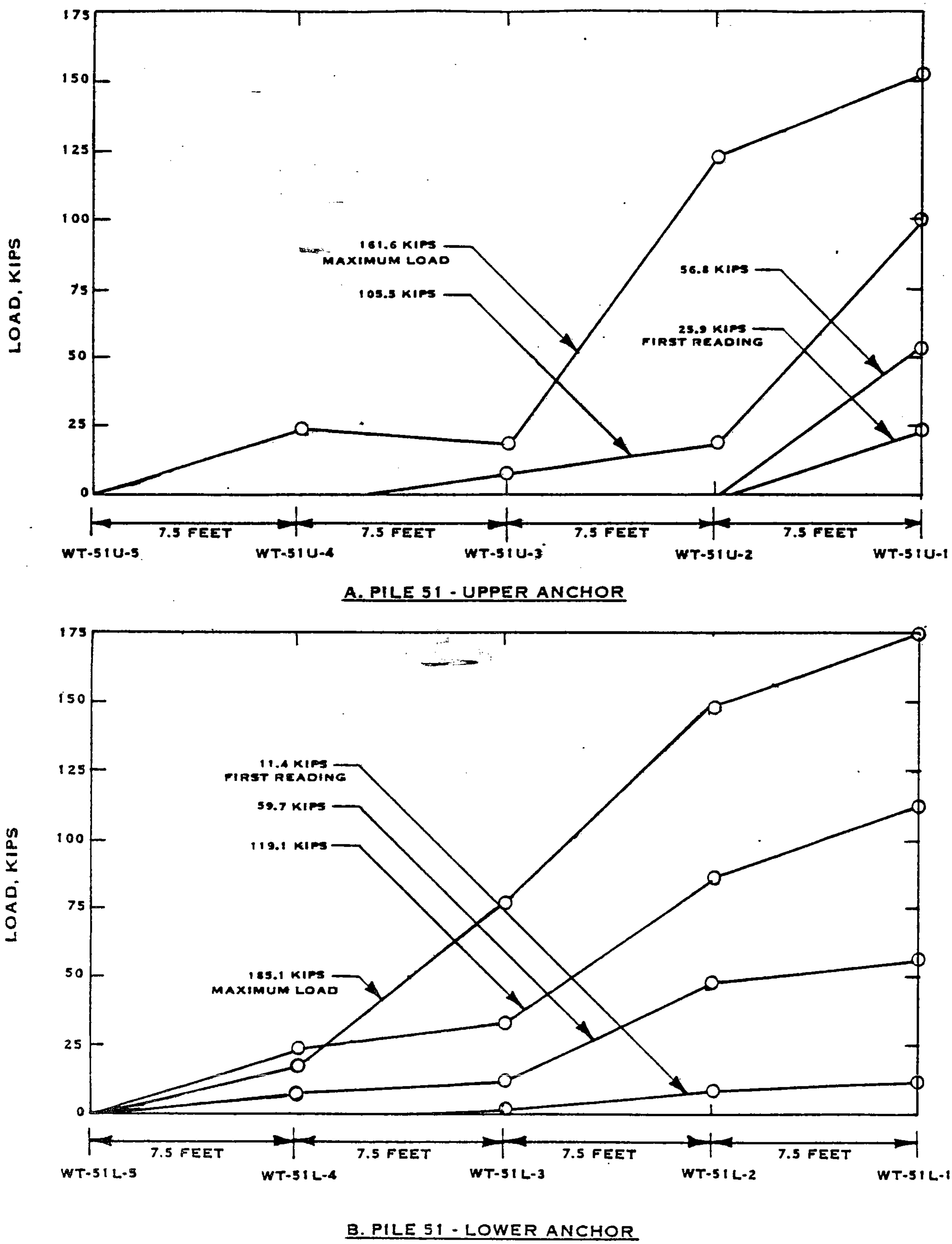


FIGURE 7 - BONDED ZONE LOAD DISTRIBUTION

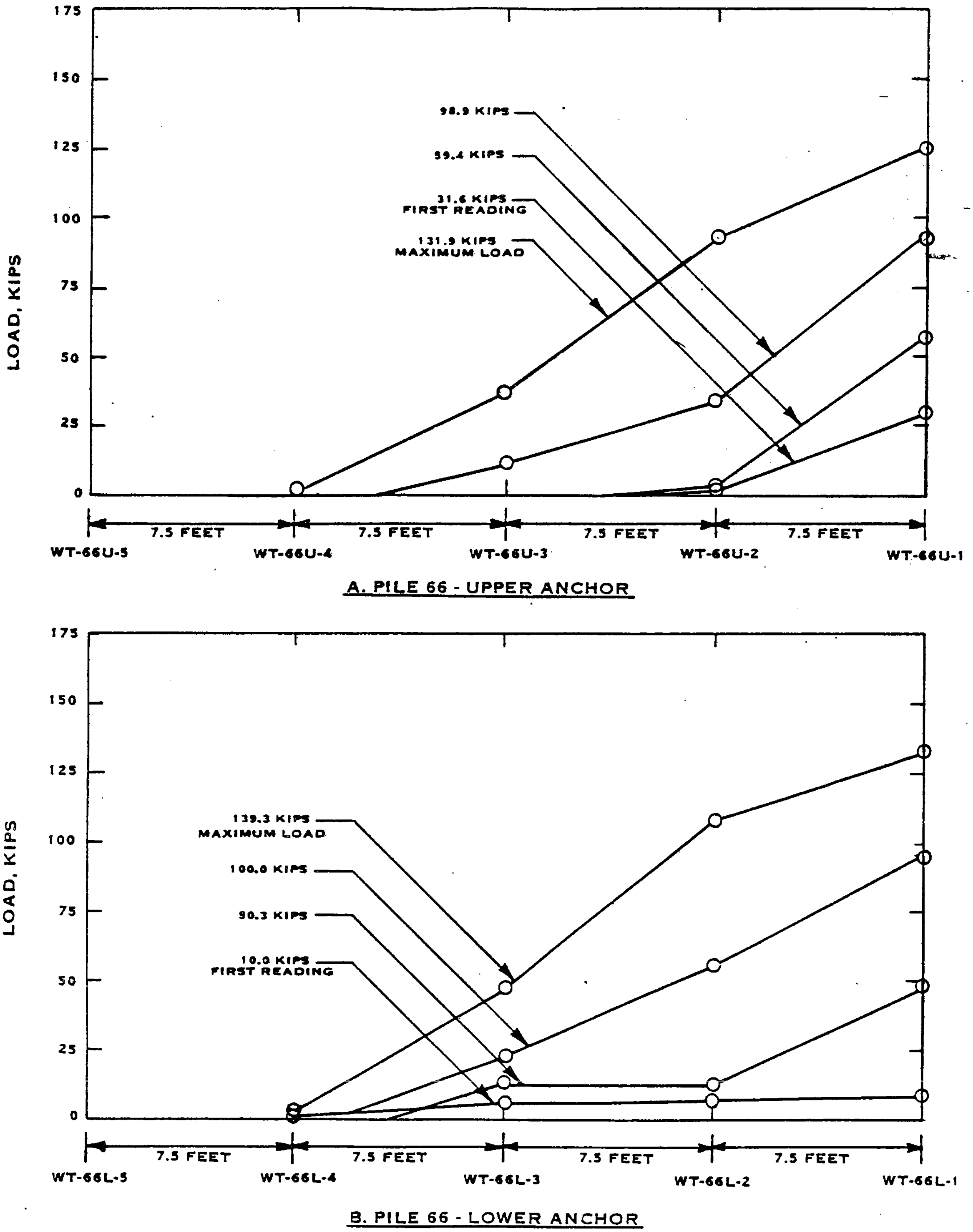


FIGURE 8 - BONDED ZONE LOAD DISTRUIBUTION

All of the wire telltales located at position 1 (i.e. at the interface of the bonded and unbonded zones) did not perform reliably during testing. This is surprising, since the rod telltales fastened at the same location worked well. Without excavating the instrumented anchor the reasons for this are difficult to assess. The rod telltales are stiffer than the wires and therefore aren't as sensitive to installation procedures, rotation of the strands during stressing and stress concentration at the bonded and unbonded zone interface. As a result, the data gathered from the No. 1 wire telltales was ignored. To compensate for this loss of valuable data, the load at position 1 at the top of the bonded zone was assumed to be 95% of the load measured at the anchor head. A 5% load loss due to friction is consistent with tests run on grease coated-plastic sheathed tendons like that used for the tieback anchors (PIC, Incorporated). Analysis of data currently continues. As data reduction methods and anchor behavior become more apparent in the near future, the No. 1 wire data will likely be incorporated into the load distribution analyses.

The bonded zone load distribution in the Pile 66 anchors apparently attenuates over a shorter distance than in the Pile 51 anchors. The subsurface conditions near Pile 66 tend to be better (denser) in general than Pile 51 (see Appendix). This could account for the difference in load distribution characteristics. The Standard Penetration Resistance averages 29 to 35 and 19 to 21 blows per foot near Piles 66 and 51, respectively. Groundwater may also account for the difference in load distribution. It is inferred from the subsurface information available that part of the Pile 51 - lower anchor bonded zone is below the groundwater table. Groundwater could reduce the load transfer capacity of the soil surrounding the anchor.

It can be seen in Figures 7 and 8 that at low anchor loads, only a small portion of the bonded zone becomes loaded. As the anchor load is increased, the length of stressed bonded zone also increases. This increase apparently takes place when the load transfer (slope of load distribution line) reaches some limiting level which is most likely related to the direct shear strength characteristics of the soil. This has been observed in the instrumented anchors (Figure 9).

As the anchor loads increased, load transfer between the first and second wire telltales generally increased to some peak level and then decreased to a residual level. When this peak level between wire telltales 1 and 2 was reached, load transfer between wire telltales 2 and 3 began to increase at a faster rate. If anchor loading continued, load transfer between wire telltales 2 and 3 probably would have peaked and a greater load transfer would have occurred between wire telltales 3 and 4.

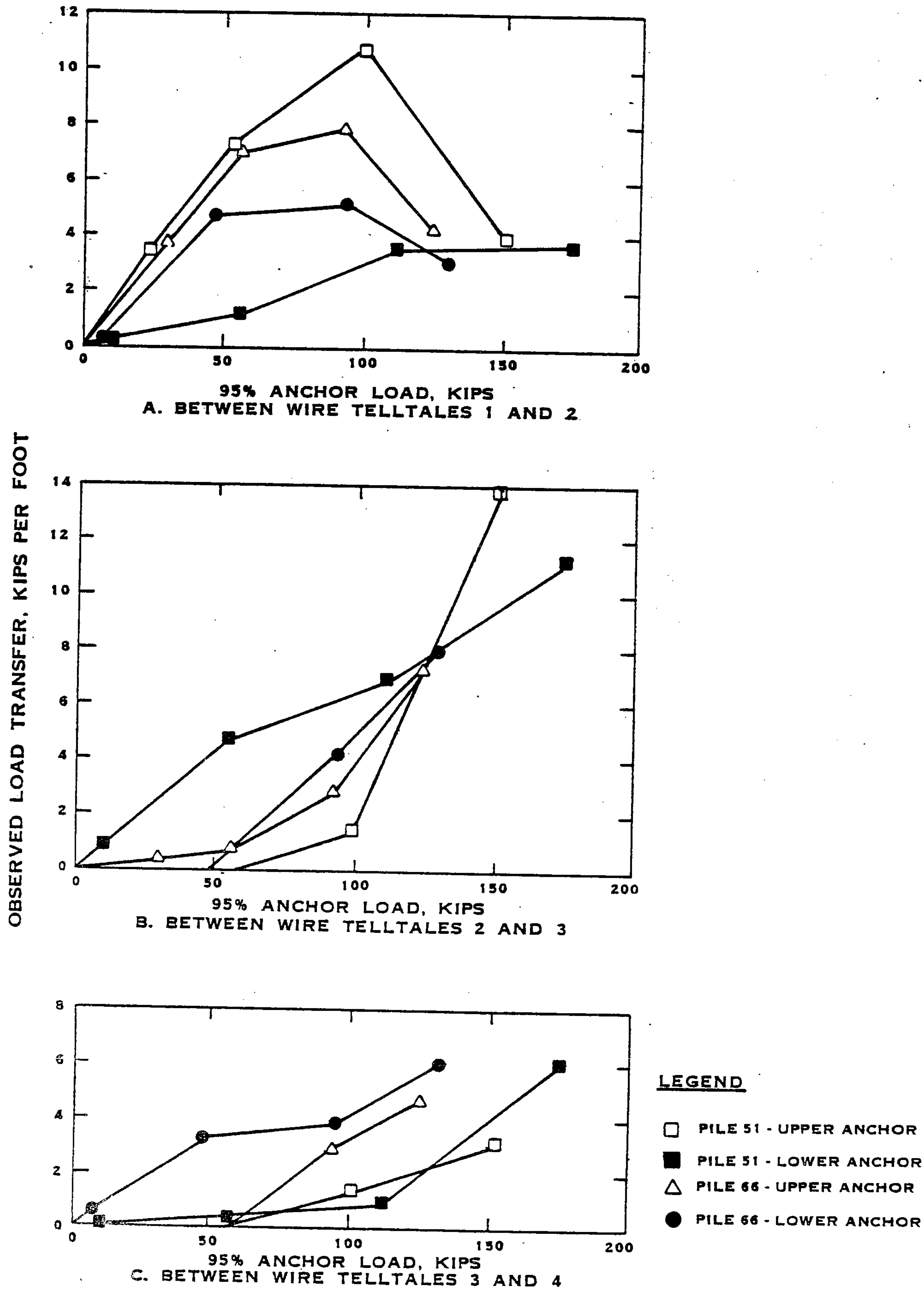


FIGURE 9 -- VARIATION IN LOAD TRANSFER WITH ANCHOR LOAD

Tieback anchor proof testing is commonly conducted using a jack pressure gage for the anchor load indicator. It is commonly thought that the pressure gage is not a reliable indicator of anchor load in the unloading mode. A load cell was used to proof test every instrumented anchor. A comparison of jack pressure gage load to load cell reading can be made. Such a comparison is presented in Figure 10. This data confirms thoughts about unloading and suggests that even in the loading mode the load cell and pressure gage do not agree. In the loading mode, the pressure gage over predicted the anchor load by approximately 5% at the 50 to 150 kip load levels.

Three slope inclinometers (SI) are located behind each instrumented station (Piles 50-52 and 65-67). The number 1 inclinometers are installed in a steel pipe welded to the soldier piles. The numer 2 and 3 inclinometers are located in the soil, approximately 10 and 46 to 51 feet behind the retaining wall, respectively. The horizontal inclinometer deflections observed at different stages of construction are shown in Figures 11 and 12. The maximum horizontal retaining wall movement detected by the inclinometers was 3/4 inches toward the excavation. This corresponds to approximately 0.2% of the wall height. Goldberg, Jaworski and Gordon (1976) reported a range of normalized horizontal movements of 0.1 and 0.6% for tieback walls in sand and gravel. The magnitude of horizontal movements 10 feet behind the wall (number 2 inclinometers) are similar to that of the retaining wall. The distribution of the movement is somewhat different. Inclinometer SI-66-2 deflected slightly more than SI-66-1. The former is located behind timber lagging whereas the latter is attached to the soldier pile and probably accounts for the difference. The deflection of number 3 inclinometers was not significant. Horizontal movement continued to occur after the excavation was complete but generally seems to be slowing down with time. Survey data showed the same trends as the inclinometers.

GEORGIA STABILIZED EMBANKMENT (GASE) WALL 12

Project Description

GASE is an earth retaining system developed by the Georgia D.O.T. In March of 1982, the Georgia DOT began development of an instrumentation plan for analyzing the performance of GASE walls, with the objective of gaining information for improvement of wall design. The GASE wall chosen for instrumentation was Wall 12, located on I-75 near Northside Drive in northwest Atlanta. The wall retains the fill for the new location of the northbound exit ramp at Northside Drive and is approximately 650 feet long and 55 feet high at its highest point. The main components of the system are concrete face panels, steel stabilizer mats and backfill material assembled to form an earth retaining system. The

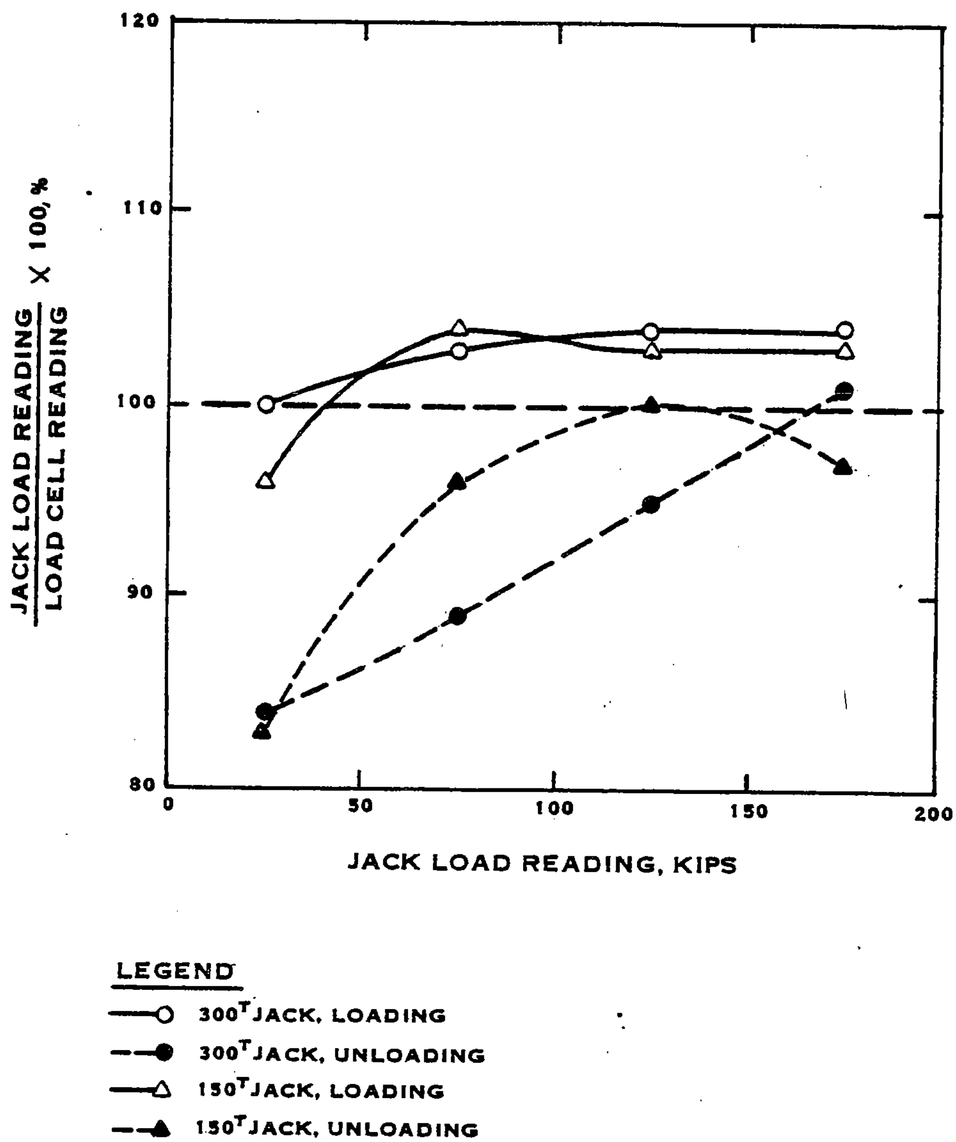
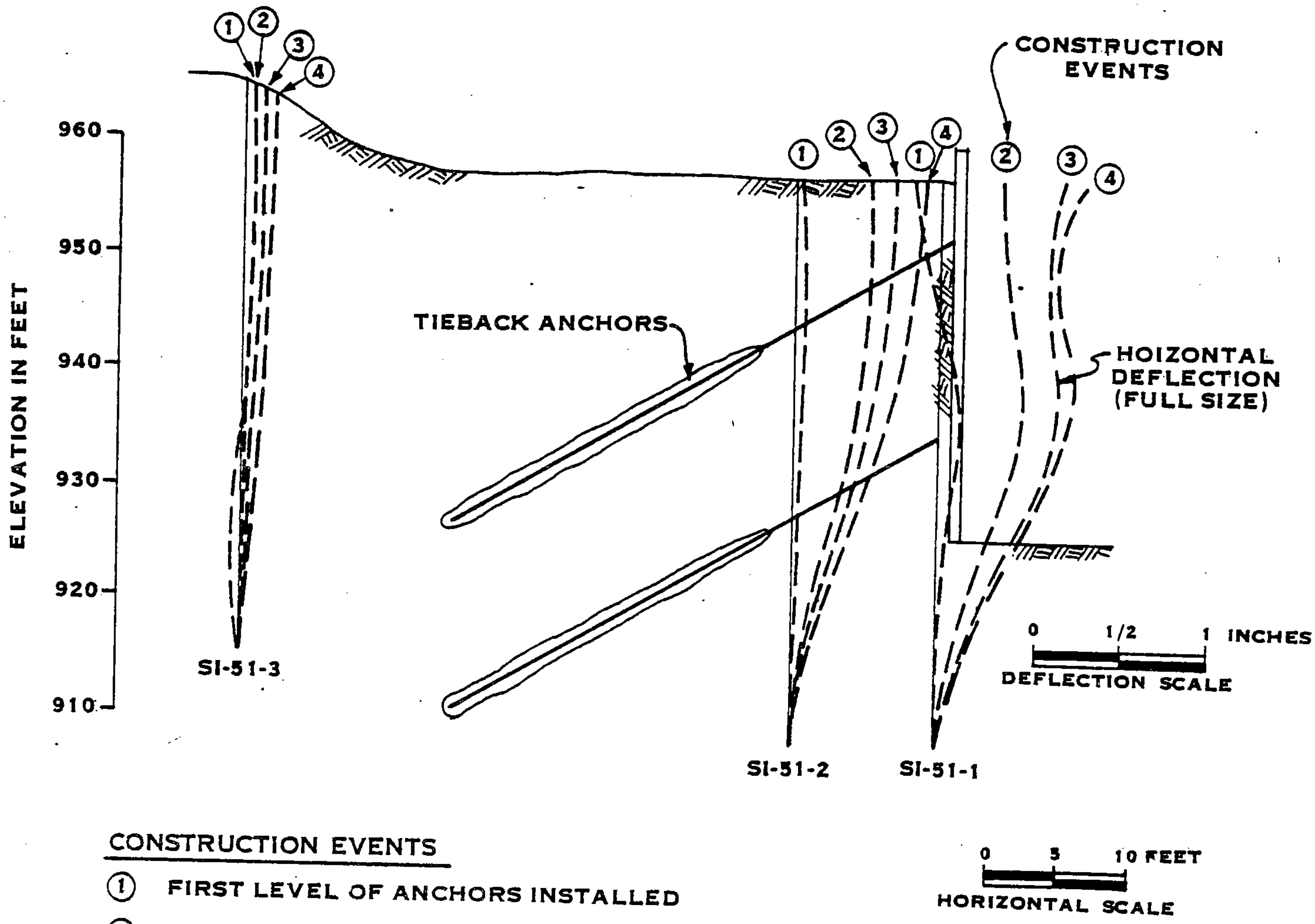


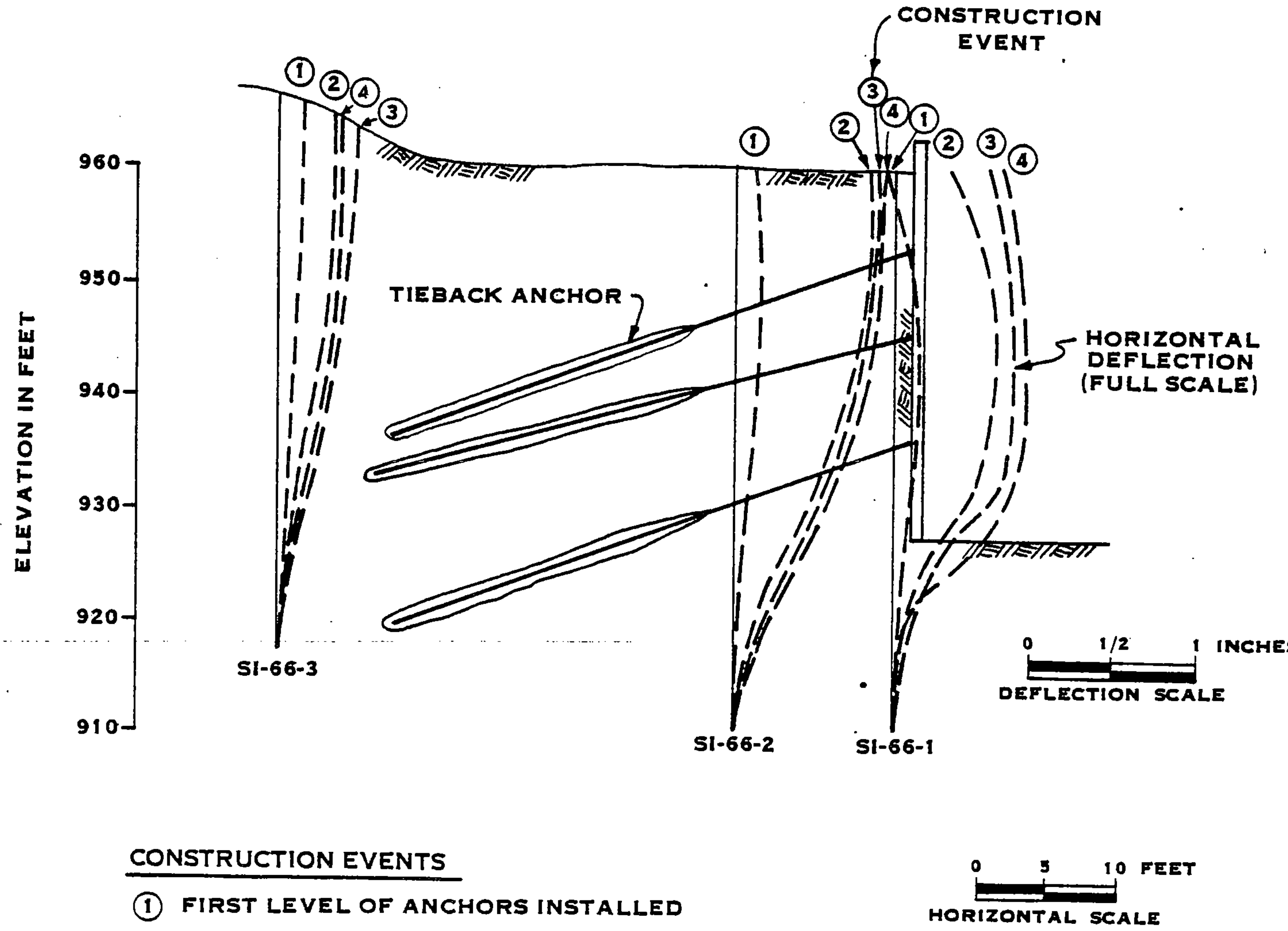
FIGURE 10 – COMPARISON OF LOAD CELL READINGS TO LOAD CALCULATED FROM JACK PRESSURE



CONSTRUCTION EVENTS

1. FIRST LEVEL OF ANCHORS INSTALLED
2. SECOND LEVEL OF ANCHORS INSTALLED
3. TWO MONTHS AFTER SECOND LEVEL INSTALLED
4. FOUR MONTHS AFTER SECOND LEVEL INSTALLED

FIGURE 11 - HORIZONTAL DEFLECTIONS BEHIND PERMANENT TIEBACK WALL (PILE 51)



CONSTRUCTION EVENTS

(1) FIRST LEVEL OF ANCHORS INSTALLED

(2) SECOND LEVEL OF ANCHORS INSTALLED

(3) TWO MONTHS AFTER SECOND LEVEL INSTALLED

(4) FOUR MONTHS AFTER SECOND LEVEL INSTALLED

FIGURE 12 - HORIZONTAL DEFLECTIONS BEHIND PERMANENT TIEBACK WALL (PILE 66)

goals of the study are to determine what stresses and strains are developed in the steel mats and earth mass, to evaluate different backfill materials, and to evaluate the overall performance of the wall.

Method of Construction

The GASE system is composed of concrete face panels, steel wire stabilizer mats and backfill material as shown in Figure 13. There are other minor components in the wall which will not be discussed. The method of construction is to set the first row of face panels in position on a leveling pad and to hold it in position temporarily with braces and clamps. The wire mats are then attached to the back of the face panels and a layer of granular backfill placed over the mats and compacted. Panels are set upon panels and the process repeated until the wall is completed.

The basic face panel for the GASE wall is shown in Figures 14 and 15, although panels can be modified to fit various geometrical conditions of the wall.

The stabilizing mat is a 64,000 psi welded wire steel mesh approximately 2 feet wide. There are four 3/8 inch diameter longitudinal bars. Transverse bars are 3/8 inch in diameter, 2 feet long and generally located on two foot centers. There are four uniformly spaced mats per face panel. The mats are attached to the face panel as shown in Figures 15 and 16.

The backfill material for the wall consists of a free draining granular material. Clean sands and crushed stone with low corrosion potential are commonly used. In the Atlanta area it is common to see a No. 4 crushed granite stone used as a backfill material. To evaluate the effect that different backfills would have on the wall, two types of backfill were used. The fine backfill material (No. 4 crushed granite stone) was used in most of the embankment. A coarser backfill having grain sizes ranging from 1 to 6 inches with almost no fines was placed in a small section of the embankment for comparison.

Subsurface Conditions

The soils immediately affecting the wall at this site are loose to very dense brown micaceous sandy silts and silty sands. Standard Penetration Test resistances range from 7 blows per foot to 60 blows with no penetration. Soil density generally increases with depth but hard and soft layers occur frequently. During excavation groundwater springs were encountered in the existing embankment. Groundwater was encountered during the subsurface exploration between elevation 832 and 852 feet above mean sea level. The depth to hard rock below the wall foundation ranges from 6 to 36 feet. A subsurface profile and description of site geology may be found in the Appendix.

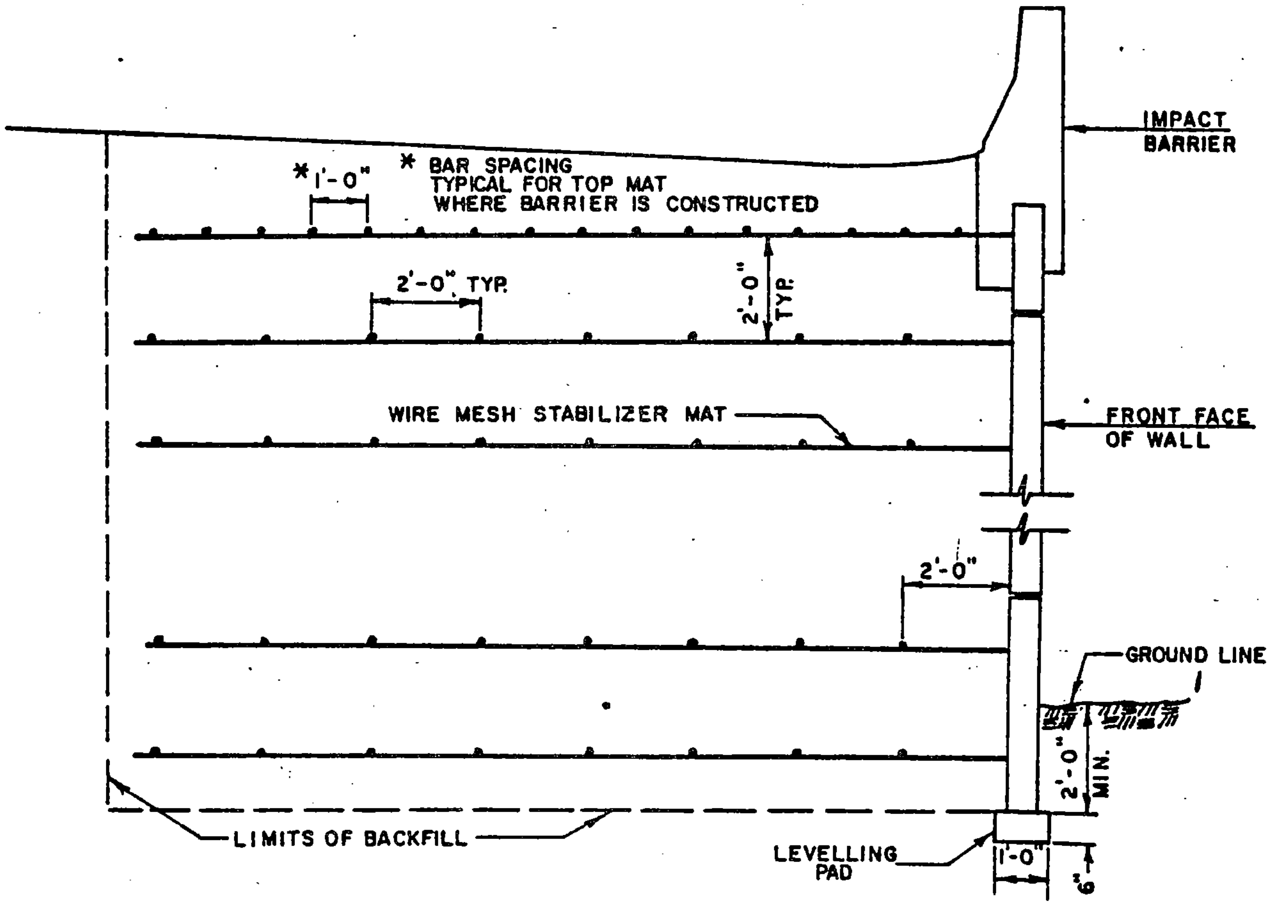


FIGURE 13–TYPICAL CROSS SECTION OF GASE

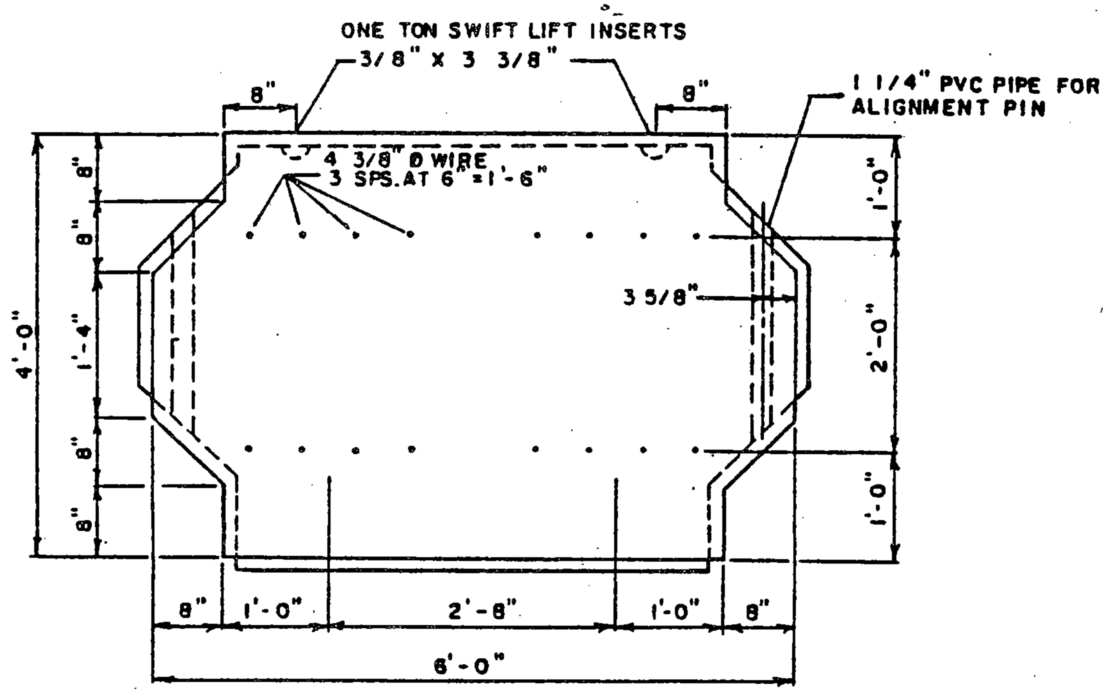


FIGURE 14–STANDARD TYPE A PANEL

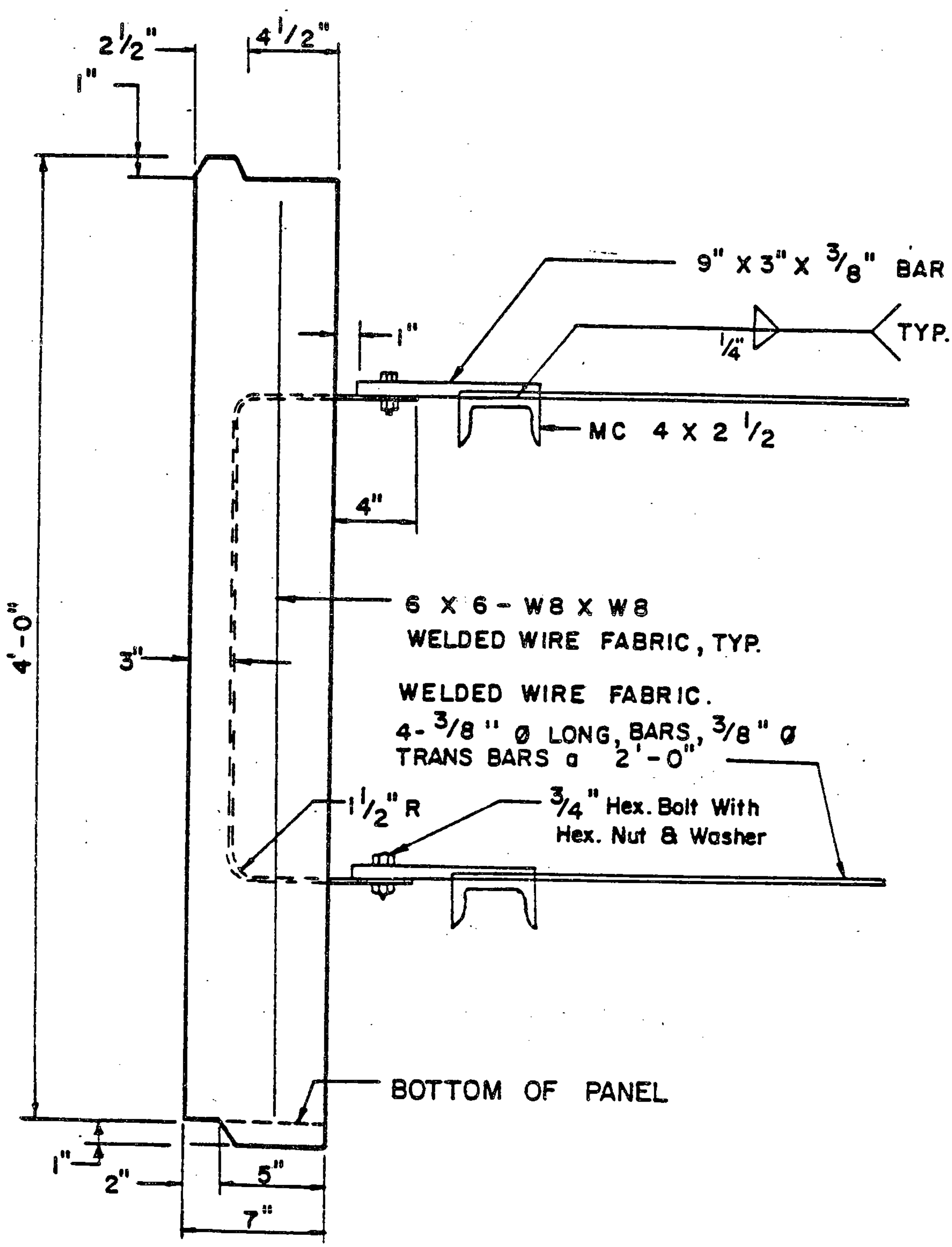


FIGURE 15 - TYPICAL PANEL CROSS SECTION AND MAT CONNECTION

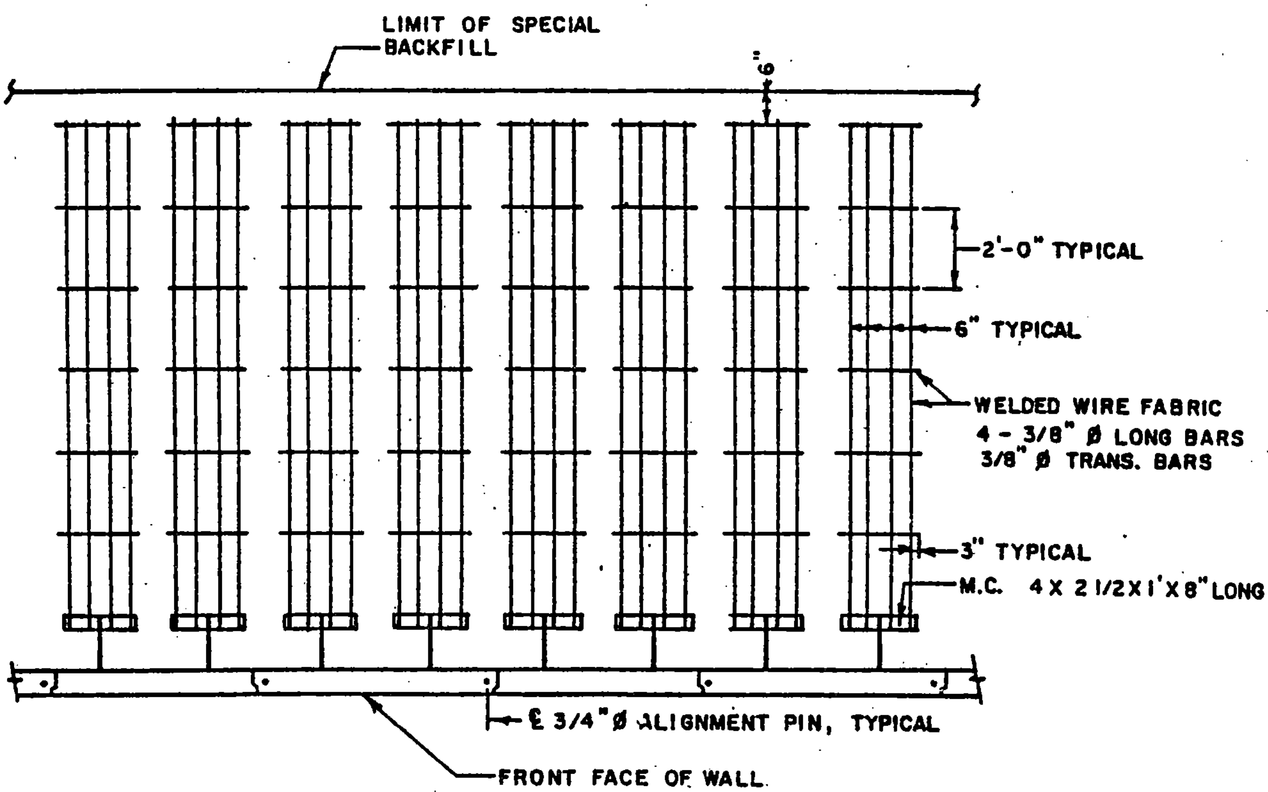


FIGURE 16 – PLAN VIEW OF TYPICAL MAT ARRANGEMENT

Instrumentation

The instrumentation system consists of a combination of strain gages, extensometers (EX), pressure cells (PC) and survey points. The relative locations of these instruments are shown in Figure 17. One instrumented station (Section A) is located in the fine backfill material (No. 4 crushed granite stone) and the other (Section B) in the coarse backfill material (having grains ranging in size from 1 to 6 inches).

Strain gages were installed on several of the mats to observe the stress levels that develop in the steel during and after construction. The gaged mats are shown in Figures 17 and 18. As shown, four of these mats will be used for full scale pull-out tests to determine the ultimate pull-out capacity of a 12 and 24 foot long mat in both the fine and coarse backfills.

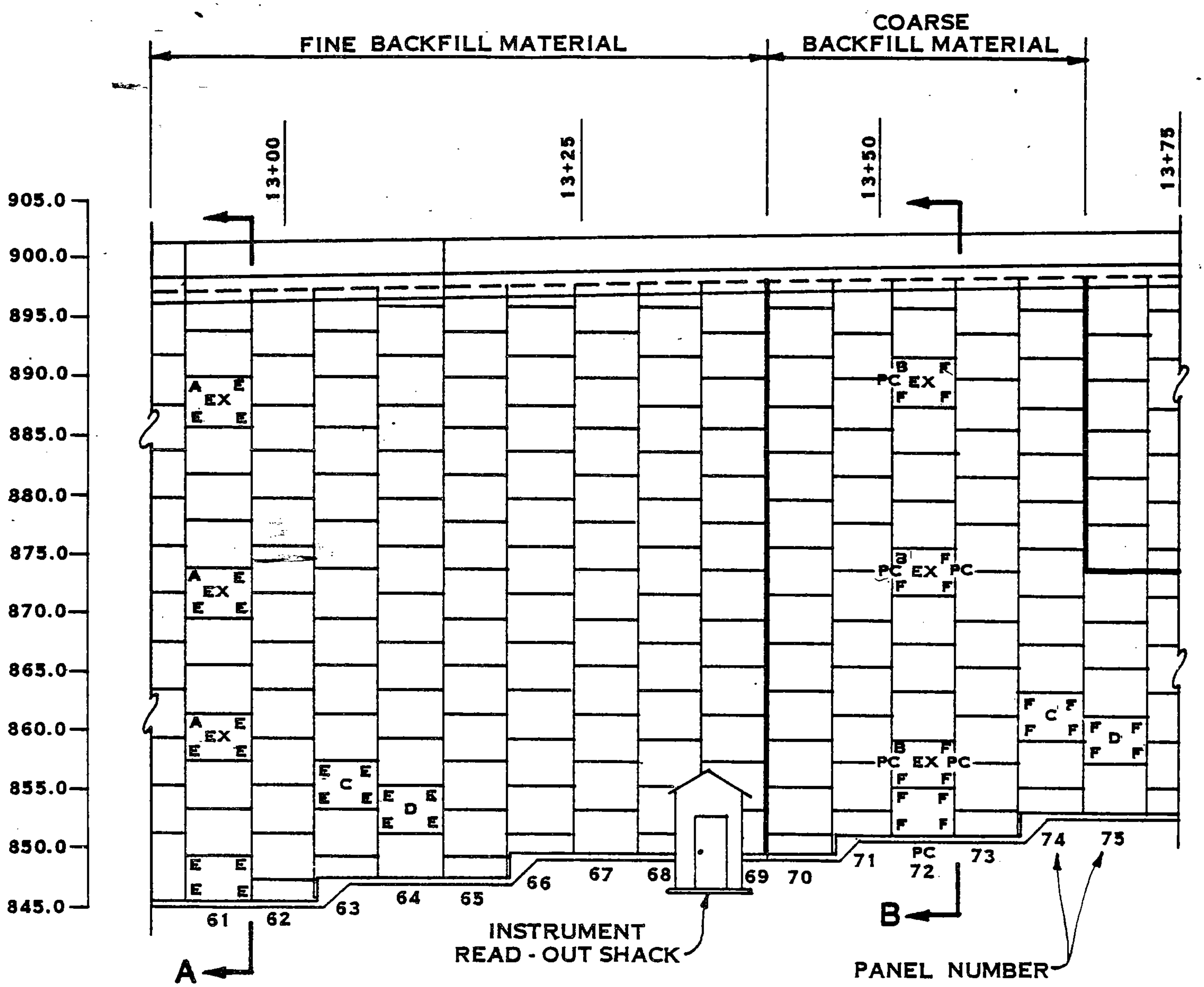
It would have been preferred to place the pressure cells in the fine backfill material (at Section A). The foundation materials here are very dense and it was feared that stress reflection could be a problem. It was assumed that backfill grain size would not influence the distribution of vertical and horizontal earth pressure and the cells were located at Section B instead. The vertical pressure cells were located beneath the embankment to determine the distribution of foundation pressures. The horizontal pressure cells were installed between the stabilized embankment and the earth fill behind it to determine the lateral earth pressures against the embankment. Cross sections of the Section A and B instrumentation may be found in Figures 19 and 20, respectively.

Multiple point extensometers were installed at six locations. The purpose of these devices is to help correlate internal movement of the backfill material with the stress distribution within the stabilizer mats.

All of the instrumentation was installed during construction of the fill and required a significant effort by the Contractor (Pittman Highway Construction), Ga. D.O.T. personnel, and Law/Geoconsult to facilitate this installation. Survey points were established by D.O.T. to monitor settlement and horizontal movement (and tilt) of the wall. All instruments were monitored during and after construction and monitoring will continue for many years.

Monitoring Results

At the time this paper is written (August, 1983) the project is not yet complete. The following is a description and current interpretation of the data collected thus far. The author urges Ga. D.O.T. to make the long term data accessable to the profession when it becomes available. The interpretation presented herein should be considered preliminary at best.



LEGEND (SEE FIGURE 14)

A 38 FOOT FULLY GAGED MAT
B 36 FOOT FULLY GAGED MAT
C 24 FOOT FULLY GAGED MAT (PULL-OUT TEST)
D 12 FOOT FULLY GAGED MAT (PULL-OUT TEST)
E 38 FOOT TONGUE ONLY GAGED MAT
F 36 FOOT TONGUE ONLY GAGED MAT
PC EARTH PRESSURE CELL
EX EXTENSOMETER

FIGURE 17 - PROFILE OF GEORGIA STABILIZED EMBANKMENT INSTRUMENTATION

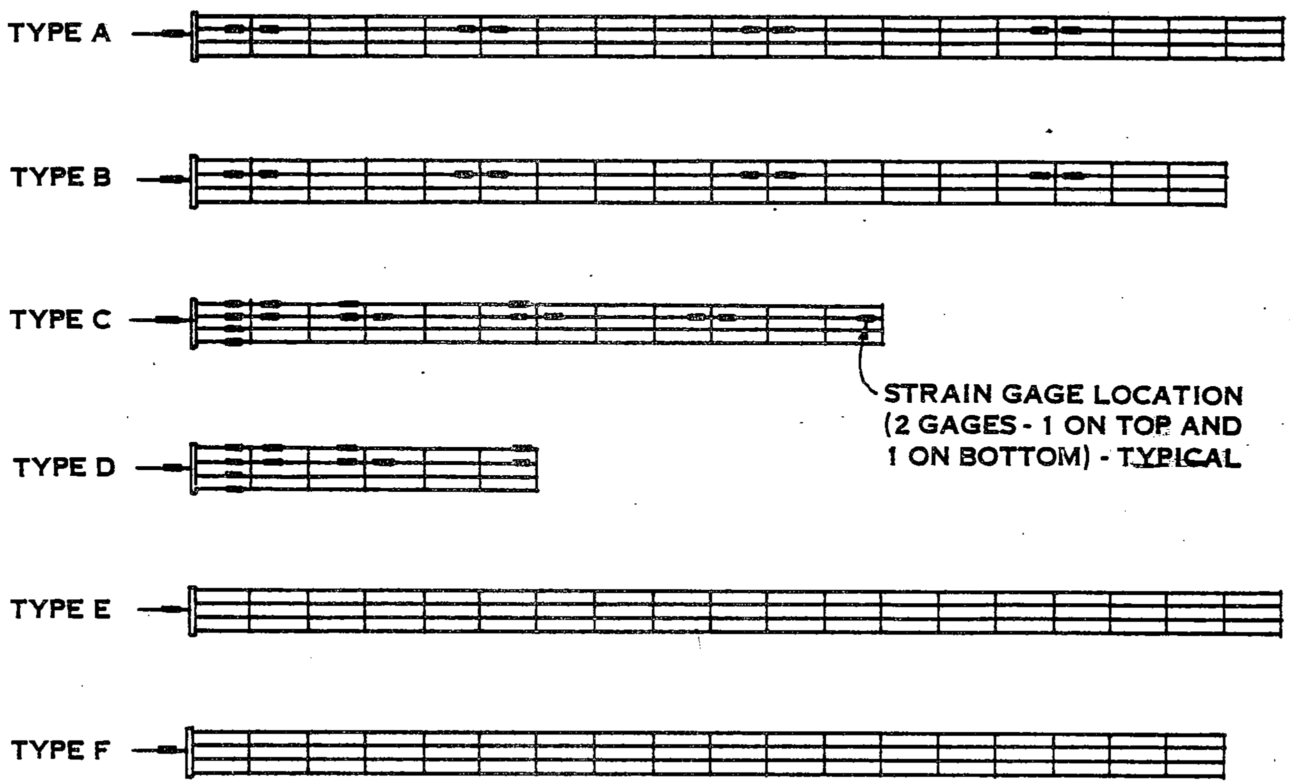


FIGURE 18 - STRAIN GAGED MAT DETAIL

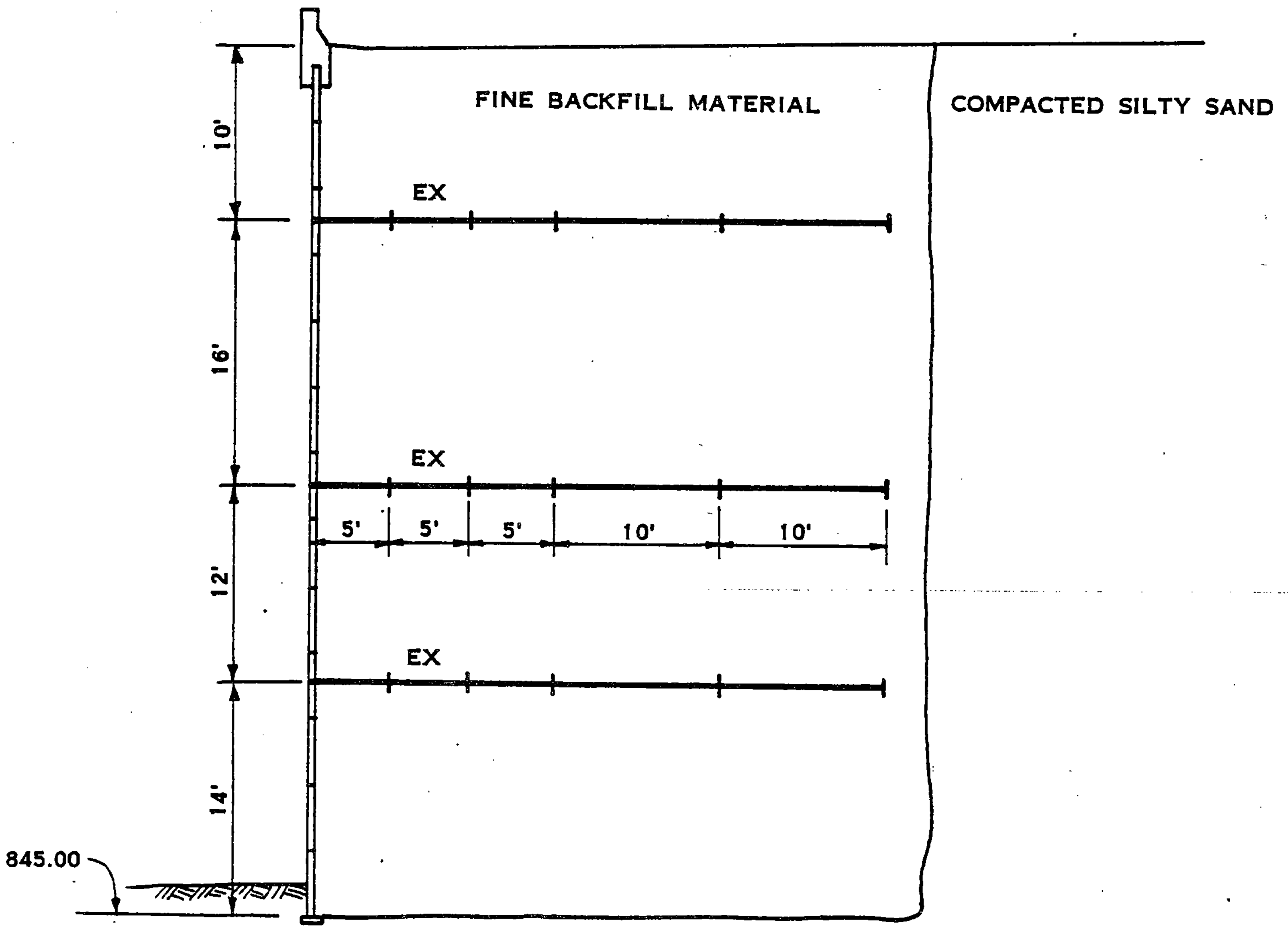


FIGURE 19 - CROSS SECTION OF INSTRUMENTED SECTION A
(MATS NOT SHOWN)
SCALE 1" = 10'

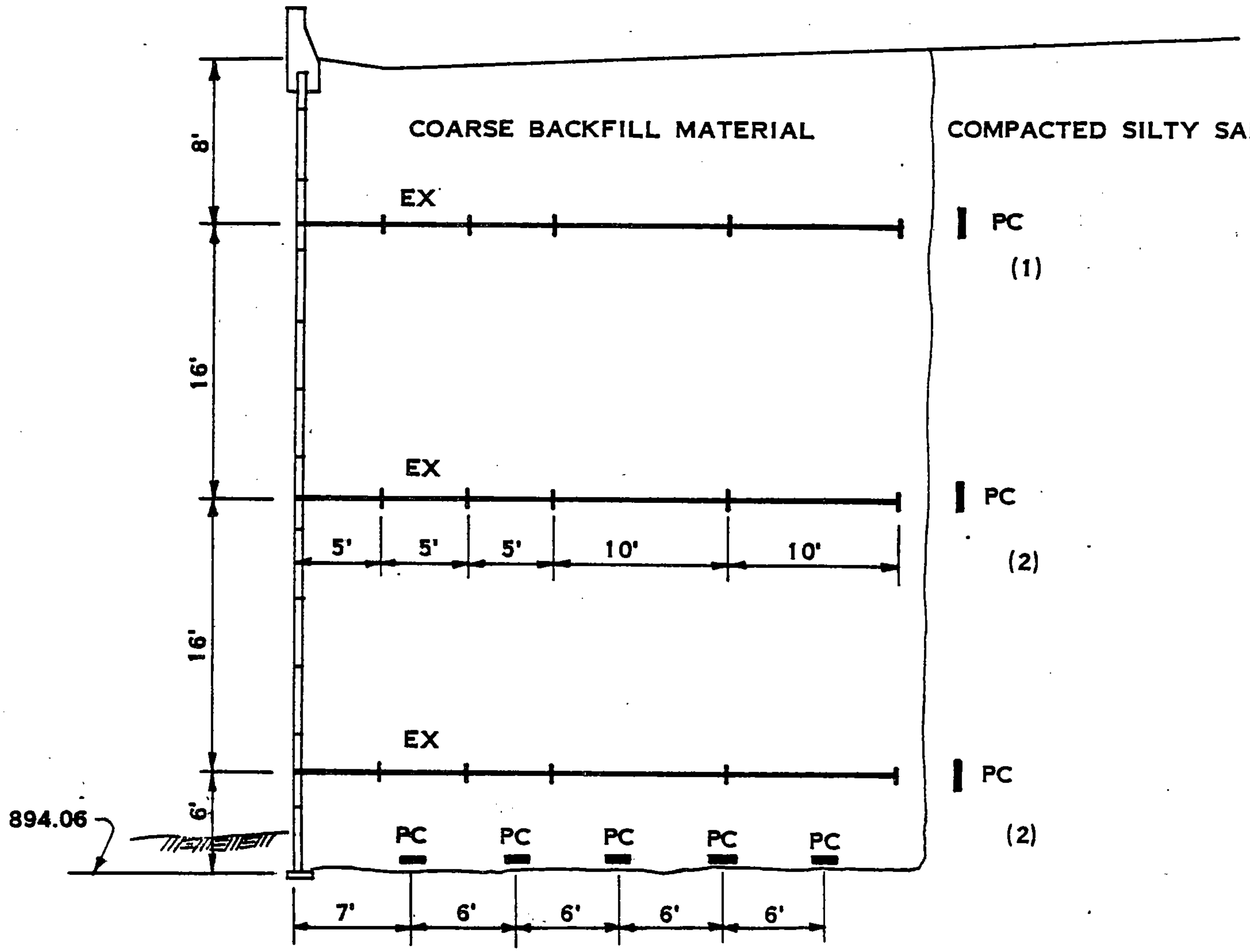


FIGURE 20 - CROSS SECTION OF INSTRUMENTED SECTION B

(MATS NOT SHOWN) SCALE 1" = 10'

Vertical earth pressure increased proportionally with fill height (Figure 21) during construction. The pressure cell farthest from the wall (PC-72-31) registered values of earth pressure close to what would be calculated by multiplying the height times the density of the fill (overburden). The two cells closest to the wall (PC-72-7 and 13) registered pressures greater than the theoretical overburden and the two cells under the middle of the embankment (PC-72-19 and 25) registered pressures less than the theoretical overburden. This variation in base pressure resembles that of a retaining wall (Peck, Hanson and Thornburn, 1974) with a moment applied due primarily to horizontal earth pressure behind the wall. In the case of the GASE Wall, the granular stabilized embankment acts like a gravity retaining wall with horizontal earth pressure loads from the silty sand backfill placed behind it. The vertical base pressure distribution is shown in Figure 22. Pressure cell PC-72-31 apparently is far enough back from the wall to not be effected by the horizontal earth pressure load (moment).

Horizontal earth pressure increased proportionally with fill height as the vertical earth pressure did (Figure 23). Only the three horizontal pressure cells shown in the figure survived construction. The increase in horizontal earth pressure against the embankment with fill height is similar to what one might calculate for the Rankine at-rest condition. The soil density and angle of internal shearing resistance for the silty sand backfill placed behind the GASE wall has been assumed to equal 110 pounds per cubic foot and 22 degrees, respectively. These values are consistent with similar soils observed in the Atlanta area.

A comparison of measured horizontal earth pressures on the back of the GASE wall embankment has been made with the average horizontal pressure distribution on the concrete face panels due to the stabilizing mats (Figure 24). The latter is calculated by summing the forces on the concrete wall panel due to the four steel mats and dividing by the area of the panel. The calculated at-rest conditions have been plotted for the sand backfill behind the GASE wall and the granular stabilized embankment. A density of 104 pounds per cubic foot and an angle of internal shearing resistance of 40 degrees was used to calculate the stabilized embankment at-rest earth pressure. This information is based on field density testing and laboratory tests. The at-rest lines are plotted for comparison to observed data. Sowers, Robb, Mullis and Glenn (1957) reported a similar earth pressure distribution trend in compacted sands and clays. Figure 24 also confirms that the concrete panels do not merely serve as architectural skin but actually resist some horizontal earth pressure somewhat like a tieback or "dead man" anchor retaining system.

The stress distribution in the steel stabilizing mats has been plotted in Figures 25 and 26. No apparent trend is obvious concerning the relationship between stress distribution in the mats and depth below ground surface. It is possible that compaction of the fill masks the affect of depth. Future monitoring and analysis should bring a relationship between mat stress and depth to light.

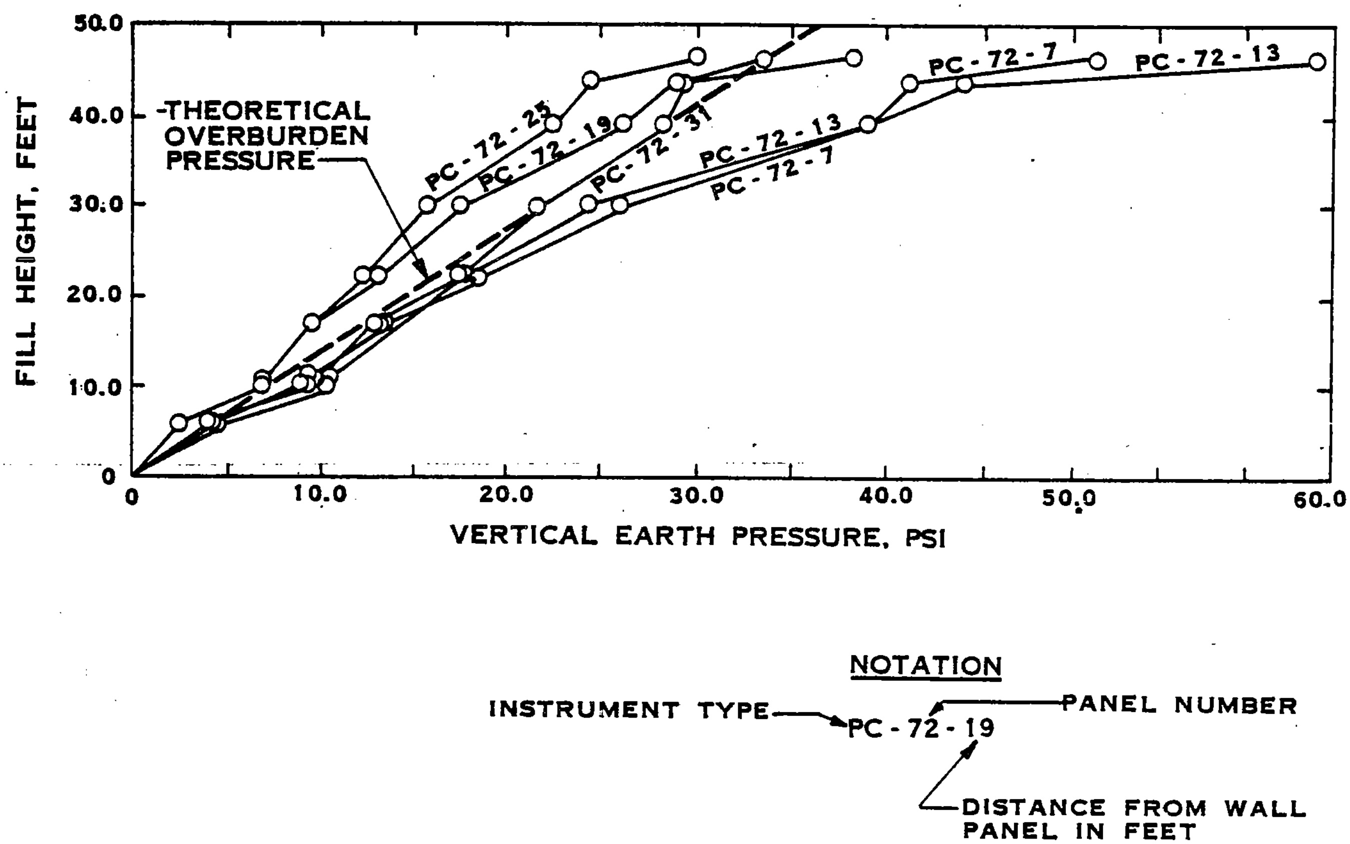


FIGURE 21 - VERTICAL EARTH PRESSURE DISTRIBUTION DURING COARSE FILL PLACEMENT

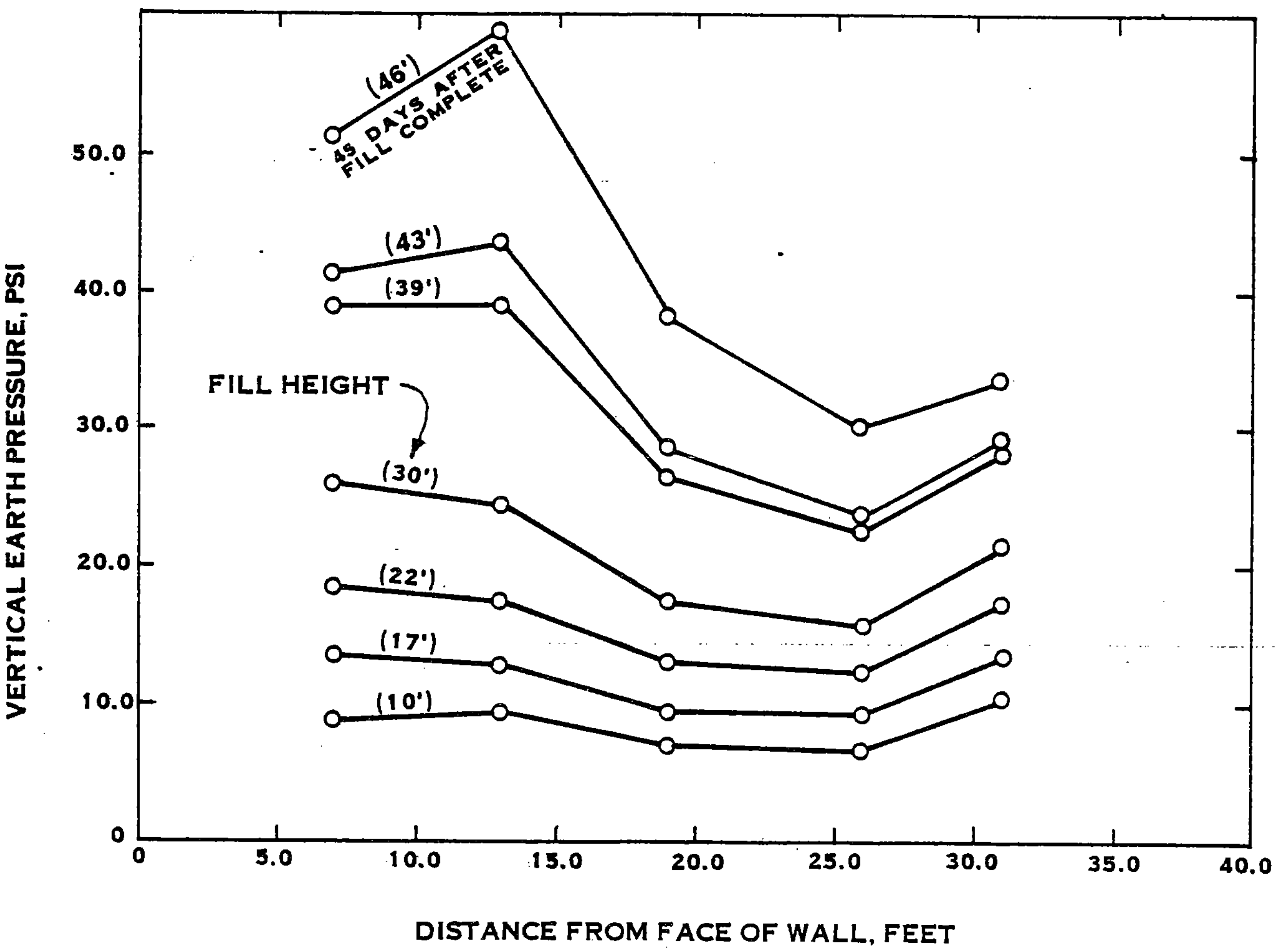
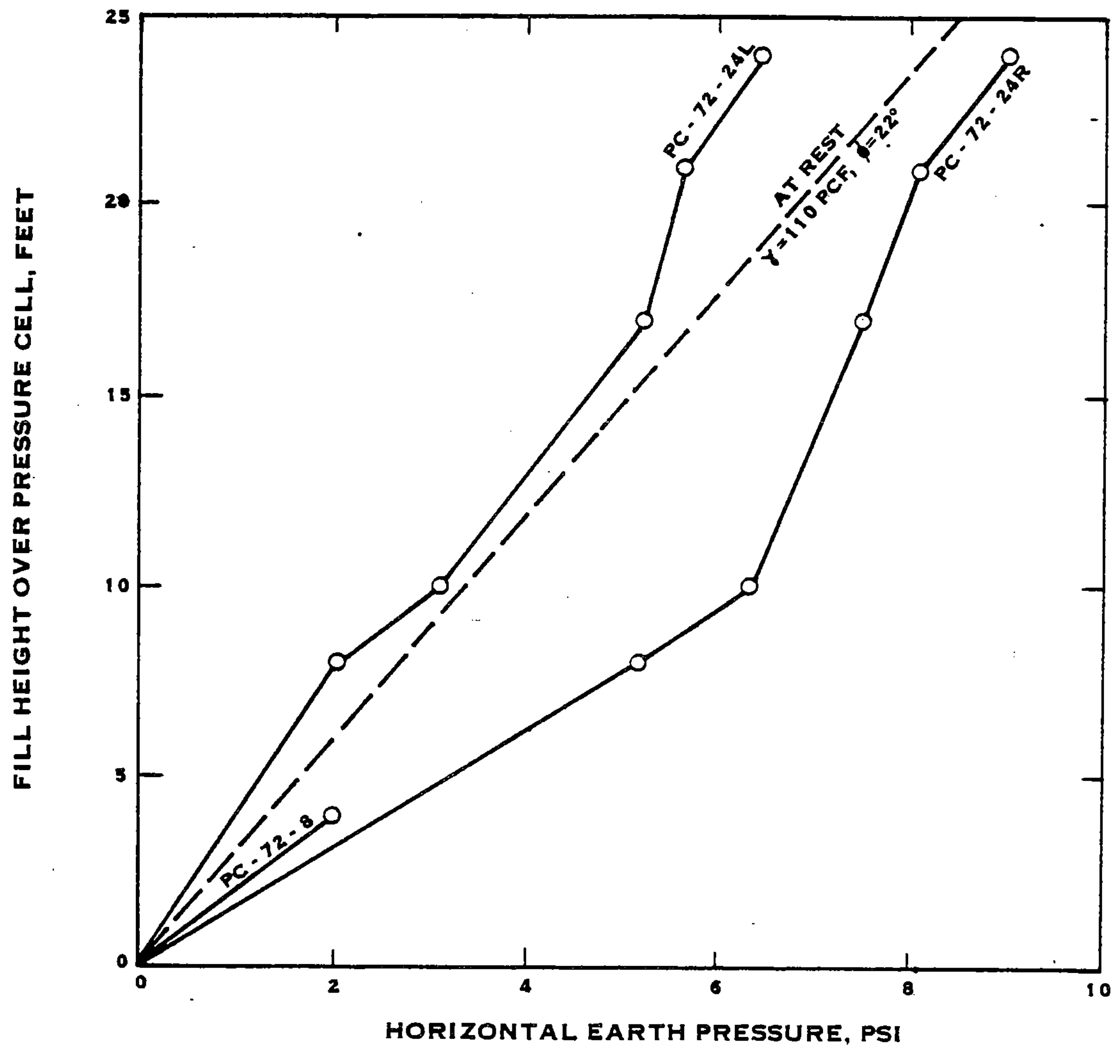


FIGURE 22 - VERTICAL EARTH PRESSURE DISTRIBUTION AT SECTION B



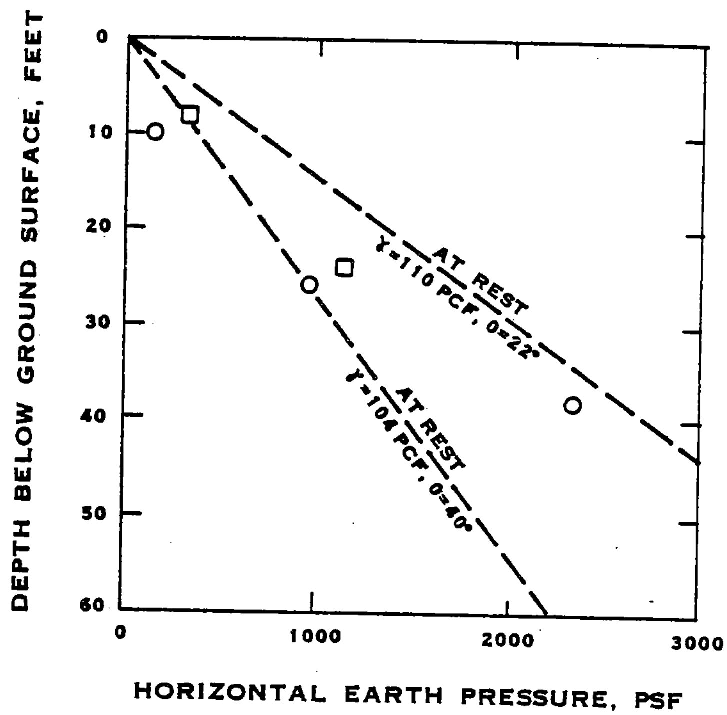
NOTATION

INSTRUMENT TYPE — PANEL NUMBER

PC - 72 - 24L

DEPTH BELOW GROUND SURFACE AND LEFT (L) OR RIGHT (R) SIDE

FIGURE 23 - HORIZONTAL EARTH PRESSURE DURING COARSE FILL PLACEMENT



LEGEND

○ AVERAGE CALCULATED PRESSURE ON WALL PANEL

□ MEASURED HORIZONTAL EARTH PRESSURE AT BACK OF GASE WALL

FIGURE 24 - COMPARISON OF PRESSURE ON WALL PANELS TO EARTH PRESSURE AT BACK OF GASE WALL

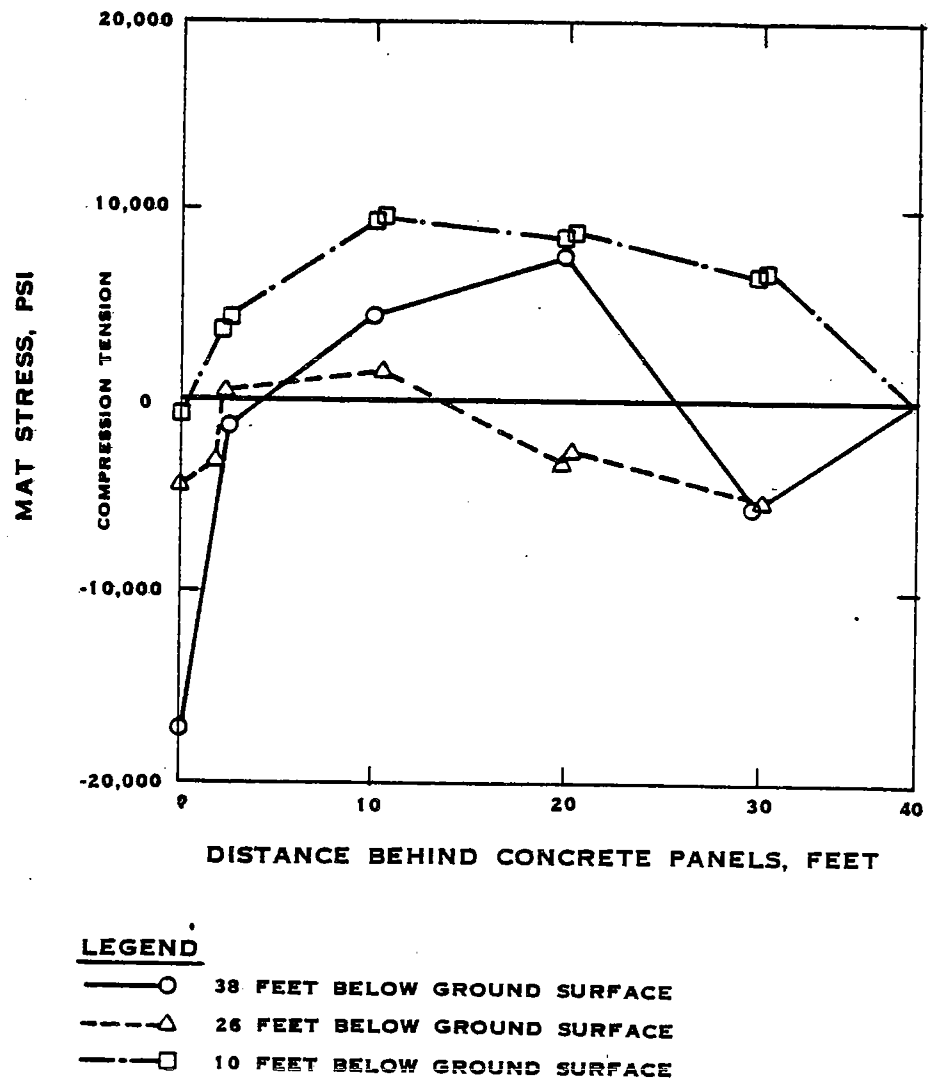


FIGURE 25 - MAT STRESS DISTRIBUTION AT SECTION A

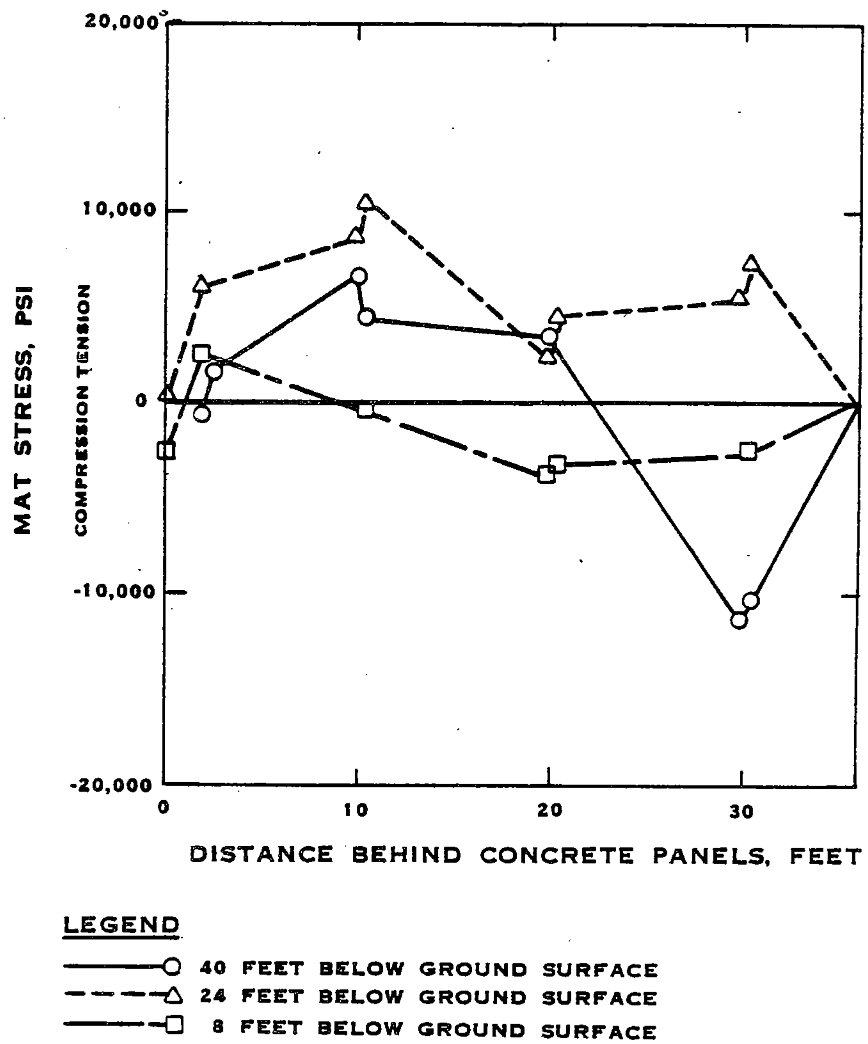


FIGURE 26 - MAT STRESS DISTRIBUTION AT SECTION B

Gages were placed on the mats not only to measure the distribution along the entire mat length but also to examine the affect of the cross bars on the stress distribution and pull-out resistance. To do this, gages were mounted in pairs on either side of the cross bars. Unfortunately not all of these pairs survived construction. Where the gage pairs survived, the average tensile stress increase between the front gages (closest to the wall panels) and the back gages was computed. In the fine and coarse granular backfills, the tensile stress increase averaged 1,000 and 1,600 pounds per square inch, respectively.

The method of GASE wall construction can cause bending of the stabilizing mats during embankment fill dumping and compaction or relative settling of the fill with respect to the concrete wall panels. The strain gages are mounted on the 3/8 inch mat bar connections where maximum bending stresses could develop. This apparently has happened, rendering some of the gages useless. The mat stresses are calculated by multiplying the observed strains by the modulus of elasticity of the steel. Once the extreme fiber of the bar exceeds the yield strength, permanent (plastic) strain occurs and this simple elastic method for calculating stress is no longer valid. For this reason, it is difficult to assess what maximum stresses have developed in the stabilizing mats and how much of the bar cross-section is in that maximum state of stress.

Table I shows the average observed axial bar stresses at different depths in the fill. These values do not include bending stresses and do not include data from gages that are fastened at the extreme fiber of bars that have yielded.

TABLE I - MAXIMUM OBSERVED AVERAGE AXIAL BAR STRESS

	Maximum Stress			
Depth Below Ground Surface Feet	Fine Backfill Tension (KSI)	Fine Backfill Compression (KSI)	Coarse Backfill Tension (KSI)	Coarse Backfill Compression (KSI)
0 - 25	13.8	12.0	22.6	9.2
25 - 50	42.8	34.1	24.9	20.5

There is a scarcity of data published on the subject of stresses developed in the stabilizer mats. Chang, Hannon and Forsyth (1981) published much lower values than what was observed in the GASE wall. The fills used in that report are different than the fills used for the GASE wall and could be one reason for the difference in observed stresses.

The strain distribution in the two instrumented embankment sections (A and B) after construction has been studied. Embankment strains are predominantly extension although compressive strains do occur toward the front of the granular backfill (near the wall panels). No obvious relationship between embankment strain and depth is apparent. The observed range in values after complete fill placement is shown in Table II.

TABLE II - OBSERVED RANGE IN EMBANKMENT STRAIN

Distance From Wall Panel Feet	Strain, $\frac{in.}{in.}$ x 10 -6 Fine Backfill	Coarse Backfill
0 - 10	-1,100 to 1,600	-1,483 to 1,483
11 - 20	- 517 to 1,367	- 433 to 917
21 - 30	250 to 2,125	342 to 850

NOTE: Positive strain = extension and negative strain = compression.

Laboratory pull-out resistance tests have been performed on short (5 feet long) GASE mats in backfill materials similar to that used in construction (CALTRANS, 1982). The results of these tests may be found in Figure 27. To date, one full scale, field pull-out test has been run on a 24 foot long mat. The results of this test are also plotted on the graph. The full scale test seems to agree with the results of the laboratory tests. Three more full scale tests will be conducted to investigate this further. The pull-out behavior of full size mats and the affect of mat length and depth will also be evaluated.

ACKNOWLEDGEMENTS

The author wishes to recognize the agencies who participated in these projects and helped prepare this report. The Georgia Department of Transportation is leading the giant effort to renovate the Georgia highway system of which the retaining walls discussed are a part. Most of the construction and research is being funded by the Federal Highway Administration. A joint venture of Law/Geoconsult International, Inc. and Law Engineering Testing Company is the geotechnical consultant to the Ga. D.O.T. The permanent tieback wall was built by Nicholson Construction Company and the GASE wall was built by Pittman Highway Construction. The author gratefully acknowledges the many people working for these agencies who supplied help when needed to make these successful studies of the performance of two modern retaining wall systems. A special acknowledgement goes to Mr. John Dunnicliff, Geotechnical Instrumentation Consultant for the tieback wall, whose professional and personal guidance has proved to be invaluable to the author in executing both projects.

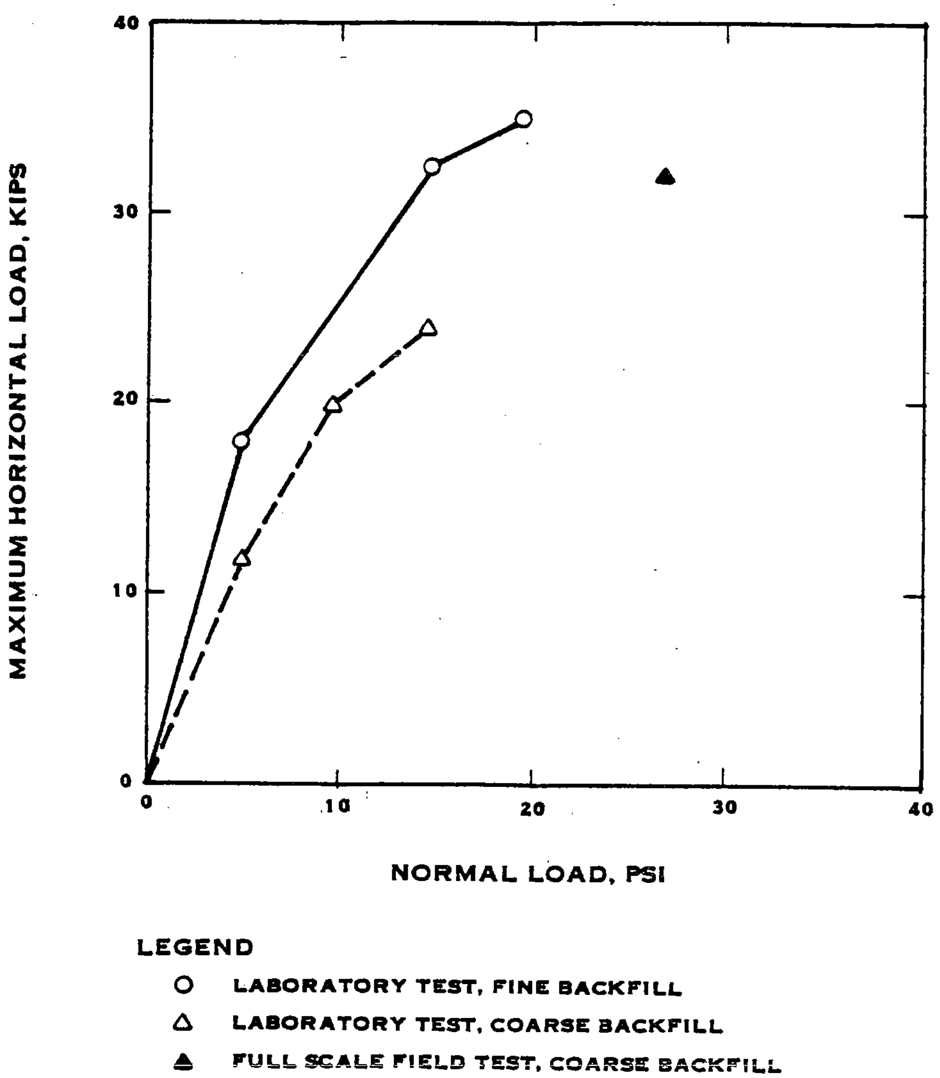


FIGURE 27 - MAT PULL-OUT RESISTANCE TESTS

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APPENDIX

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND RESEARCH, FOREST PARK, GEORGIA
GEOTECHNICAL ENGINEERING

PROJECT I-75-2(41) *COUNTY* FULTON *DATE* 2/13/79

LOCATION RETAINING WALL "E-L" *BORING NO.* EL-2

BENT NO. ______ *FOOTING* 168' RT. STA. 30+16 ℄ I-75 / 88' RT. STA. 29+80 ℄ PROP. NBL *GROUND ELEV.* 959.62

PROPOSED FOOTING ELEV. ______ *PARTY CHIEF* SPORNBERGER

ELEV.	BORING LOG		BLOW	UNIFIED	γ	W	Gs	% 200	% CLAY	LL	PI	C	ø	
	GR. EL. Asphalt													
	Mltc Micas Sandy Silt													
950														
		1u												
	Med. Dense Same	2s	19											
940		3u												
	Med. Dense Same	4s	18											
		5u												
	Dense Same	6s	47											
930	V. Dense Weathered Granite Gneiss	7s	60=.8											
	Dense Same	8s	43											
920		9s	60=.0'											
	V. Dense Same													
910														
	Practical Refusal on Rock													
900														

The Department of Transportation in making this foundation report available to contractors assumes no responsibility for its accuracy.

No claim will be considered if the contractor relies on this information in his bidding or in his construction operations and finds that it is inaccurate.

This foundation investigation report is not considered as a part of the Plans and Specifications or Contract on the job.

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2 (41)256 COUNTY Fulton DATE 6-6-74

LOCATION RETAINING WALL "E1" BORING NO. E1-1

BENT NO. ______ FOOTING 170' RT. STA. 29+90 EXISTING ℄ / 85' RT STA. 29+51 ℗ PROP NBL GROUND ELEV. 959.46

PROPOSED FOOTING ELEV. ______ PARTY CHIEF Bowling

EV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C.	Ø	BC	LL	PI	% 200	% CLAY	e
60	Gr. El.	4" Asphalt													
		7" Concrete													
	Dse. Mltc. Micas. Sdy. Silt	1s	25												
		2s	14												
		3s	11												
50															
		4s	13												
		5s	20												
	Med. Dense Mltc. Micas. Sandy Silt	6s	16												
40		7s	11												
	Dense Same	8s	30												
30	Med. Dense Mltc. Micas. Sandy Silt	9s	16												
	Dense Same	10s	26												
20		11s	17												
	Med. Dense Mltc. Micas. Sandy Silt	12s	11												
10		13s	32												
	Dense Same	14s	49												
00															

cont.

The Department of Transportation in making this [illegible] assumes [illegible] accuracy. [illegible] This foundation investigation report is not [illegible] as a part of the Plans and Specifications [illegible] the job.

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT ______ COUNTY ______ DATE ______

LOCATION ______ BORING NO. El-1 cont.

BENT NO. ______ FOOTING ______ GROUND ELEV. ______

PROPOSED FOOTING ELEV. ______ PARTY CHIEF ______

ELEV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C.	Ø	BC	LL	PI	% 200	% CLAY
	Dense Mltc. Micas. Sandy Silt	15s	50											
890		16s	60=.5'											
	Very Dense Same													
880														
	~~sli. weathered to hard rock~~													
	Practical Refusal													

The Department of Transportation in making this foundation report available to contractors assumes no responsibility for its accuracy.

No claim will be considered if the contractor relies on this information in his bidding or in his construction operations and finds that it is inaccurate.

This foundation investigation report is not considered as a part of the Plans and Specifications or Contract on the job.

SUBSURFACE CONDITIONS

Permanent Tieback Wall E-1

The site geology was investigated by the Georgia D.O.T. Materials and Research Laboratory under the direction of Mr. David A. Mitchell. Four borings from the investigation closest to the instrumented tieback section are attached. The D.O.T. engineers described the site geology in the following way.

"The rock underlying the wall is part of the Stonewall Formation, which is a medium grained biotite gneiss interlayered with fine grained hornblende-plagioclase amphibolites and dark red clayey soil while the amphibilite weathers to a blocky ocherous saprolite and the schist weathers to a pinkish micaceous soil. The soils immediately affecting the wall at this site" (Figure A.1) "are medium dense to dense yellowish-brown and pinkish micaceous sandy silts. Due to differential weathering, the depth to hard rock varies from 45 to 65 feet." (Mitchell, Leary and McLemore, 1983)

DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND RESEARCH, FOREST PARK, GEORGIA
GEOTECHNICAL ENGINEERING

PROJECT I-75-2(41) *COUNTY* FULTON *DATE* 12/15/78

LOCATION RETAINING WALL "E" *BORING NO.* E1-3

BENT NO. ______ *FOOTING* 167' RT. STA. 31 + 17 ℄ I-75 / 103' RT. STA. 30+83 ℄ PROP. NBL *GROUND ELEV.* 959.69

PROPOSED FOOTING ELEV. ______ *PARTY CHIEF* SPORNBERGER

ELEV.	BORING LOG		BLOW	UNIFIED	γ	W	Gs	% 200	% CLAY	LL	PI	C	Ø		
	GR. El.														
	Asphalt & Base														
	Med. Dense Mitc Micas Sandy Silt	Is	20												
950		2s	12												
	V. Dense Same	3s	60=.8'												
940		4s	20												
		5s	15												
930	Med. Dense Same	6s	19												
		7s	16												
920		8s	22												
910															
	Dense Material														
900															
	End Boring														
890															

The Department of Transportation in making this foundation report available to contractors assumes no responsibility for its accuracy.

No claim will be considered if the [illegible] relies on the information in this boring log in his construction operations and finds that it is inadequate.

This foundation investigation report is not considered as a part of the Plans and Specifications or Contract on the job.

.DOT 490

DEPARTMENT OF TRANSPORTATION

OFFICE OF MATERIALS AND TEST, FOREST PARK, GEORGIA
SOILS ENGINEERING AND GEOLOGY BRANCH

BRIDGE SUBSURFACE INVESTIGATION

PROJECT I-75-2 (41) 256 COUNTY FULTON DATE 1-10-74

LOCATION RET. WALL "E-I" BORING NO. EI-4

BENT NO. ______ FOOTING 118' RT. STA. 32+02 EXISTING ℄ / 65' RT. STA. 31+76 ℄ PROP. NBL GROUND ELEV. 957.7

PROPOSED FOOTING ELEV. ______ PARTY CHIEF SIMMONS

ELEV.	BORING LOG	SAMPLE	BLOW	UNIFIED	W	γ	Gs	C	Ø	BC	LL	PI	% 200	% CLAY
	GR. EL. 7													
		1u												
950														
		2u												
	MED. DENSE MLTC. MICAS. SANDY SILT	3u												
940		4s	18											
		5s	20											
		6s	18											
		7s	21											
930		8s	28											
		9s	25											
	DENSE SAME													
		10s	27											
920														
WT		11s	19											
		12s	22											
910	MED. DENSE MLTC. MICAS. SANDY SILT													
		13s	20											
		14s	22											
900														
		15s	24											
890	SLI. WEATH. TO HARD ROCK													

PRACT. REFUSAL

The Department of Transportation ... report is not considered a part of the Plans and Specifications or Contract on the job.

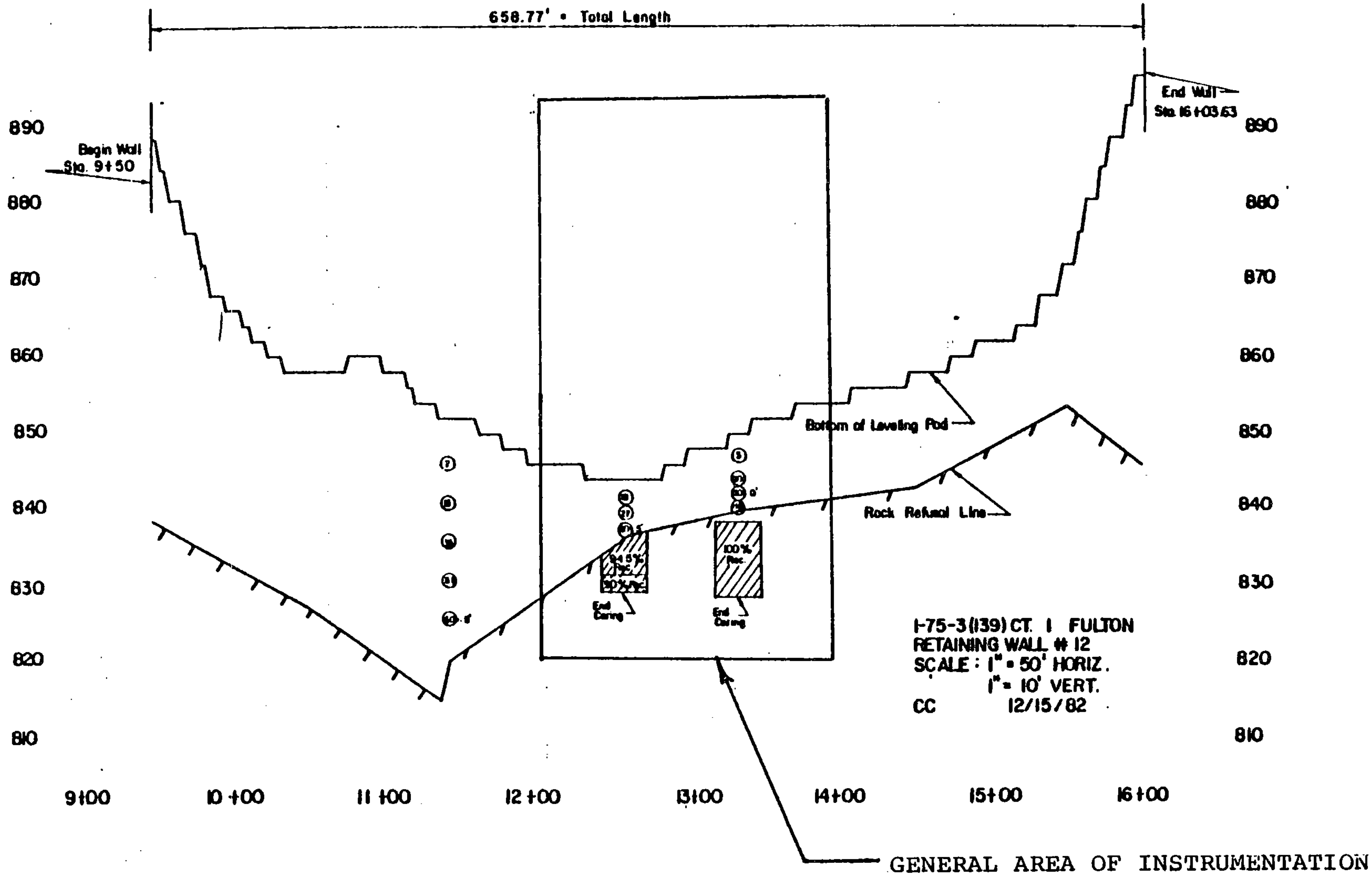

FIGURE A.2-SUBSURFACE CONDITIONS NEAR GASE WALL INSTRUMENTATION

SUBSURFACE CONDITIONS

GASE Wall 12

The site geology was investigated by the Georgia D.O.T. Materials and Research Laboratory under the direction of Mr. David A. Mitchell. A subsurface profile at the site is attached. The D.O.T. engineers described the site geology in the following way.

"This area is underlain by the Norcross Gneiss" (Figure A.2) "which is a light gray epidote-biotite-muscovite-plagioclase gneiss. The Norcross Gneiss weathers to a grayish-white rounded boulders and finally to an orangish-pink saprolite and micaceous sandy silty soil." (Mitchell, Leary, and McLemore, 1983)

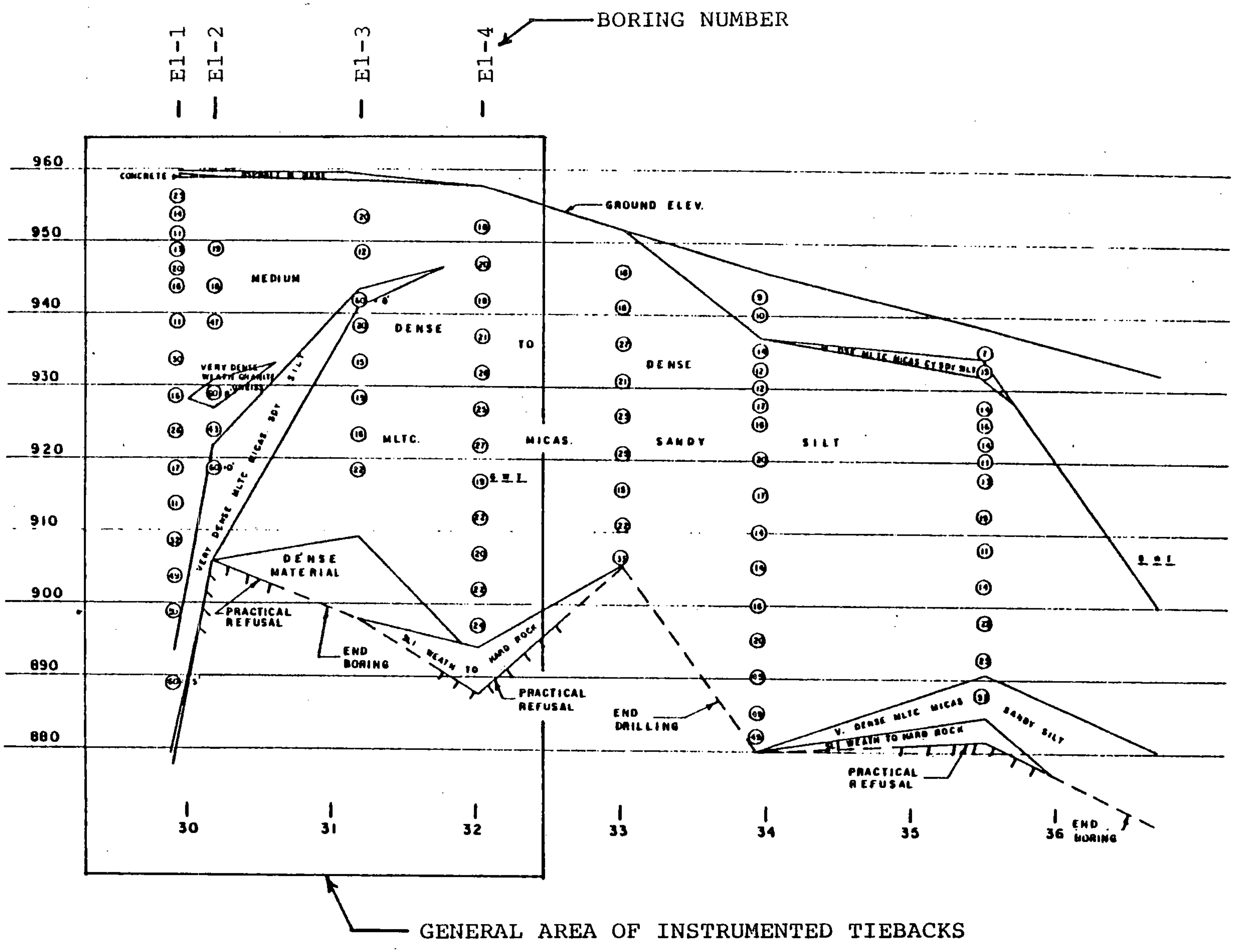

FIGURE A.1-SUBSURFACE CONDITIONS NEAR INSTRUMENTED TIEBACKS

IMPROVEMENT OF MARGINAL URBAN SITES USING STONE COLUMNS AND RIGID CONCRETE COLUMNS

by

Richard D. Barksdale
and
Tom Dobson

Paper Presented at the
34th Annual Highway Geology
Symposium and Field Trip
at
Stone Mountain, Ga.
May 1-3, 1983

IMPROVEMENT OF MARGINAL URBAN SITES USING STONE COLUMNS AND RIGID CONCRETE COLUMNS

by

Richard D. Barksdale[1]
and
Tom Dobson[2]

INTRODUCTION

Alignments for highway construction in urban areas underlain by good soils are rapidly disappearing. Undoubtedly, in the future the utilization of poor sites such as swamps, marshes and hillsides alignments subject to landslides will become even more common than today. Ground reinforcement using relatively stiff columns offers a valuable potential alternative to conventional methods of construction at many of these marginal sites.

Marginal sites can be improved by reinforcing them with relatively closely spaced columns of material stronger and more rigid than the in-situ soil. The installation of such reinforcement results in a reduction in settlement and increase in resistance to shear type failures.

The purpose of this paper is to present potential applications, introduce selected methods of construction that can be used, and summarize a case history. A summary of the advantages and disadvantages of each method is also given. Since stone columns are most widely used at the present time, emphasis perhaps is placed on this technique.

OVERVIEW OF TECHNIQUES

Ground reinforcement techniques have been known for many years with the major advancements taking place in Europe and Japan during the 1950's. The reinforcement systems were developed and are generally performed by specialty contractors.

To date, the use of these processes in North America has been limited to the stone column system. However, the specialist equipment and expertise necessary to construct several of the other systems described in this paper are now available in the U.S. The ground reinforcement systems currently available are summarized as follows:

---

1. Professor of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia, 30332.
2. President, GKN Keller, Inc., Tampa, Florida, 33614.

1. Stone Columns - Stone columns are generally constructed in very soft to firm, basically cohesive soils using a coarse relatively uniformly graded gravel or crushed stone. Stone columns have been successfully used in Europe on a wide range of projects over the last 25 years. This technique is an extension of the Vibro-Compaction or Vibroflotation system. It was introduced in the United States in 1972 when a five million gallon water storage tank was supported on soil reinforced with stone columns [1].

   Stone columns are used to support loads from embankments or structures placed on very soft to firm clays and silts having undrained shear strengths greater than about 150 to 200 psf (7-10 kN/m^2). Marginal silty sands having silt contents greater than about 15 percent can also be improved with respect to bearing capacity and settlement for the support of structural loads. Conventional stone columns should not in general be used where peat layers are present greater in thickness than the diameter of the stone column. Where peat layers are encountered, two or more vibrators are sometimes attached together (Fig. 1) to give a larger diameter column. Improvement of silty sands for the support of bridge bents, retaining walls, abutments and similar structures offer a potentially important application of stone columns. The full potential of this type of application, however, has generally not been realized at the present time.

2. Rigid Concrete Columns - Rigid concrete or grout columns resulted from a modification of the stone column construction technique in Europe in about 1976. At first, rigid columns were constructed by injecting a portland cement grout into the stone as the column was constructed. The rigid column is now constructed by using a pumped concrete or grout introduced to the tip of the vibrator through a tremmie pipe attached to the vibrator. This requires the use of specially adapted vibrators, and a machine to support the vibrator. Rigid columns are constructed in a fully supported hole.

   Rigid concrete columns can be used to replace stone columns in most applications. Rigid columns offer important advantages over stone columns where very soft organic soils or peat layers are encountered. Rigid columns are also a good design alternative where settlements must be minimized beneath structures such as bridges or where a dry technique is required. Rigid concrete columns are equally applicable for reducing settlement or improving stability. A local bearing failure behind the rigid column would often determine the maximum lateral load which a rigid concrete column can carry. A local bearing failure may also, however, limit the lateral load which a stone column, sand or stone compaction pile can carry.

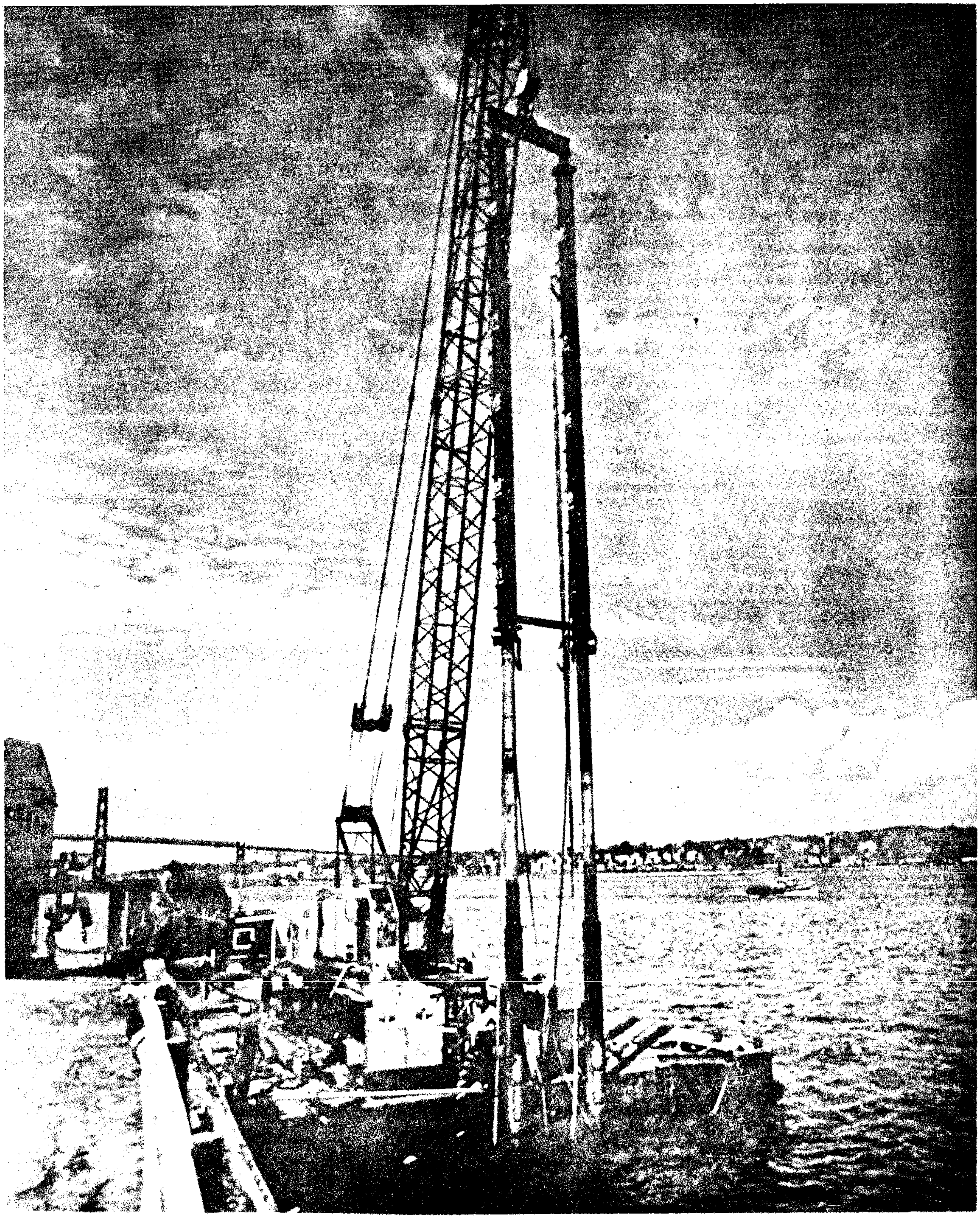

FIGURE 1. TWO VIBRATORS ATTACHED TOGETHER USED FOR DENSIFYING A GRAVEL BLANKET UNDER WATER.

3. <u>Sand Compaction Piles and Stone Compaction Columns</u> - Other methods of ground reinforcement include sand compaction piles and stone compaction columns. Sand compaction piles have been used extensively in Japan for stability applications. Sand compaction piles are constructed using sand in a fully cased hole. They have the important advantage that sand is often cheaper than stone. On the other hand, sand compaction piles have less strength than stone columns and result in greater settlement. Stone compaction piles are constructed in a fully cased hole using the same pull-down construction equipment as for rigid columns. These techniques of ground improvement are to be discussed in a subsequent paper.

APPLICATIONS

Important potential uses of ground reinforcement techniques for highway applications are as follows:

1. <u>Embankments</u>. The use of stone columns or rigid concrete columns offers a practical alternative for the support of highway embankments where conventional embankments cannot be constructed due to stability considerations (Fig. 2a). Potential applications include moderate to high fills on very soft to firm cohesive soils (typically $150 \leq C_u \leq 800$ psf). The term C_u is the undrained shear strength of a saturated, cohesive soil. Ground reinforcement techniques are particularly attractive where removal of very soft soils is not permitted due to environmental restrictions. Stone columns were used at Hampton, Virginia [2] due partly to environmental factors.

   A considerable amount of widening and reconstruction work will be done in future years. Some of this work will involve building additional lanes immediately adjacent to existing highways constructed on moderate to high fills over soft cohesive soils such as those found in marsh areas. For this application differential settlement between the old and new construction is an important problem in addition to embankment stability. Support of the new fill on stone columns offers a viable design alternative to conventional construction (Fig. 2b).

2. <u>Landslides</u>. The stabilization of either potential or active landslides (Fig. 3) is a very important application of ground reinforcement techniques (stone columns, rigid concrete columns or other methods).

3. <u>Bridge Approach Fills</u>. Ground reinforcement techniques can be used to support bridge approach fills, to provide stability, and to reduce the costly maintenance problem at the joint between the fill and bridge. Stone columns have been used at Lake Okoboji, Iowa and Mobridge, South

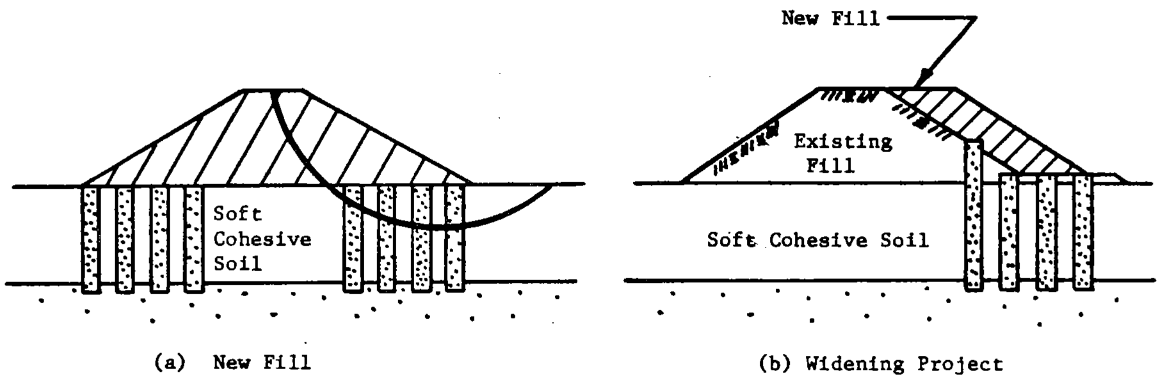

FIGURE 2. FILL SUPPORT APPLICATIONS OF STONE COLUMNS - REDUCE SETTLEMENT AND INCREASE STABILITY.

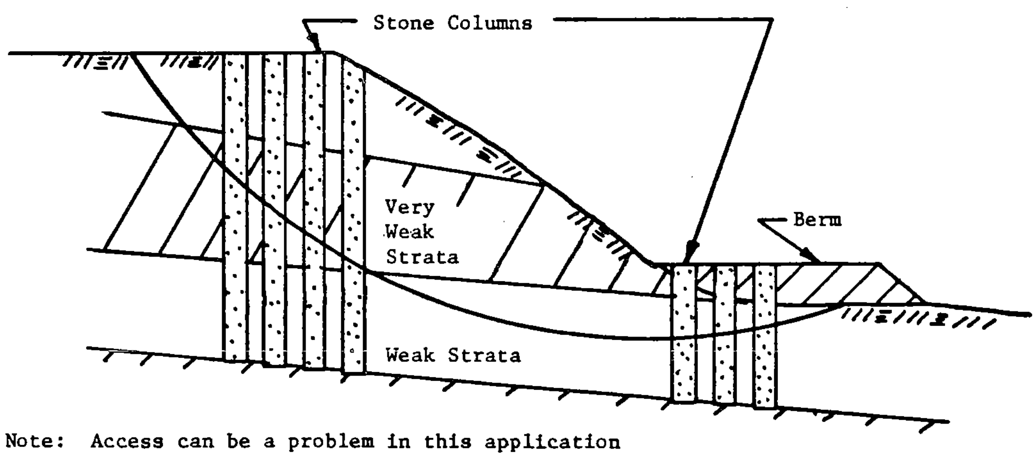

FIGURE 3. LANDSLIDE STABILIZATION APPLICATION.

Dakota for a bridge approach and embankment, respectively. At Sioux City, Iowa, stone columns were used for an interchange [1].

Under favorable conditions stone column supported embankments can be extended outward over wide, soft marsh areas and along rivers and lakes further than a conventional approach embankment. The potential therefore exists of reducing the length of costly bridge structures by ground reinforcement techniques.

4. Retaining Structures. Stone column supported Reinforced Earth retaining structures have been used at Clark Fork, Idaho [3], Jourdan Road Terminal, New Orleans [4], and Rouen, France [5]. At Jourdan Road Terminal, a Reinforced Earth wall tested to failure underwent a total movement of about 4.5 ft. (1.4 m). After failure, the Reinforced Earth wall panels were found to be generally in good condition, the embankment and wall having failed as a rigid block.

   This example nicely illustrates that stone column supported Reinforced Earth abutments or retaining structures results in a very compatible, flexible construction which can withstand large settlements. Undoubtedly stone column supported structures (either conventional or Reinforced Earth) offers an important potential application of stone columns and other ground reinforcement techniques.

5. Bridge Structures. Stone columns can be used to support interior bridge bents, integral end bent/abutments, and end bents on sloping earth abutments. Settlement considerations would generally govern whether a given site is suitable for improvement with stone columns. From settlement considerations, cohesive soils should generally be stiff, having shear strengths greater than about 1 ksf (50 kN/m^2). Stone columns should not be used for bridge bent support at sites underlain by deposits of peat. Stone columns can also be used to improve slightly marginal sites underlain by silty sand having silt contents greater than about 15 percent.

Stone column reinforced ground has also been used to support many other type structures such as a hospitality station, box culvert, tanks, parking garages, seven-story library and other structures [6].

STONE COLUMN CONSTRUCTION - DRY METHOD

In the past only the wet (vibro-replacement) method has been used in the U.S. In the wet method large quantities of water are used which, in an urban setting, is environmentally objectionable. On the other hand, the dry (vibro-displacement) method does not use water and hence is environmentally much more appealing. An uncased hole is used for both techniques of construction.

In both the wet and dry method a vibrating probe is used typically 14 to 18 in. (356-457 mm) in diameter (Fig. 4). Heavy follower tubes are connected to the vibrator as required so that the tubes protrude from the ground when the probe has reached the final elevation. The vibrator is suspended from a crane. Lateral vibration is caused by eccentric, counter rotating weights driven by either electric or hydraulic motors.

The wet (vibro-replacement) method of constructing stone columns has been described elsewhere [6,7]. Therefore, only the dry (vibro-displacement) method is discussed.

Vibro-Displacement (Dry) Method. The dry (vibro-displacement) method is used in partially saturated soils which will remain open during construction. The hole is constructed "dry". Water is not used to either advance the hole or stabilize it during construction. Air, however, is generally used throughout construction to (1) prevent a suction from developing in the bottom of the hole when the vibrator is withdrawn, (2) help support the hole, (3) cool the vibrator water, and (4) help prevent sticking of the vibrator. An air pressure of 40 to 90 psi (3-6 bars) is used.

The hole is formed by displacing the soil laterally by the vibrator as the hole is advanced (Fig. 5). The weight of the vibrator and follower tubes also help advance the hole. Dry (vibro-displacement) stone columns can be constructed using the same vibrators as for wet columns. Rather than using water, however, air is hooked up to the jet pipes. The vibrator is kept running throughout column construction.

The construction technique in the dry process differs from the wet method in that the vibrator is completely withdrawn prior to adding each increment of stone. Charges of stone sufficient for about 3 to 5 ft. (0.9-1.5 m) of column are added. The vibrator then repenetrates the stone to full depth. The vibrator is then raised, and the recently placed lift of stone repenetrated several more times. If good densification is achieved, the depth of vibrator repenetration will decrease for each successive repenetration. Dry stone columns can also be constructed using the pull-down rig described in the next section.

Soft to stiff cohesive soils ($400 < C_u < 1,000$ psf) can be reinforced using this method. Stiffer soils can be reinforced or penetrated using this or other methods by predrilling the hole. Predrilling is often performed in landslide stabilization work.

The most important advantage of the dry method is that water is not used during construction. A water supply is therefore not required. Also of more importance, large quantities of silty water do not have to be disposed of. This is a very important consideration in an urban setting, or other areas which have strict environmental regulations. Dry stone columns have also been used in partially saturated clay fills which might be weakened by water from the wet process. In one instance in Canada the dry process was used during the winter under low temperature conditions where freezing of jetting water would have created many problems.

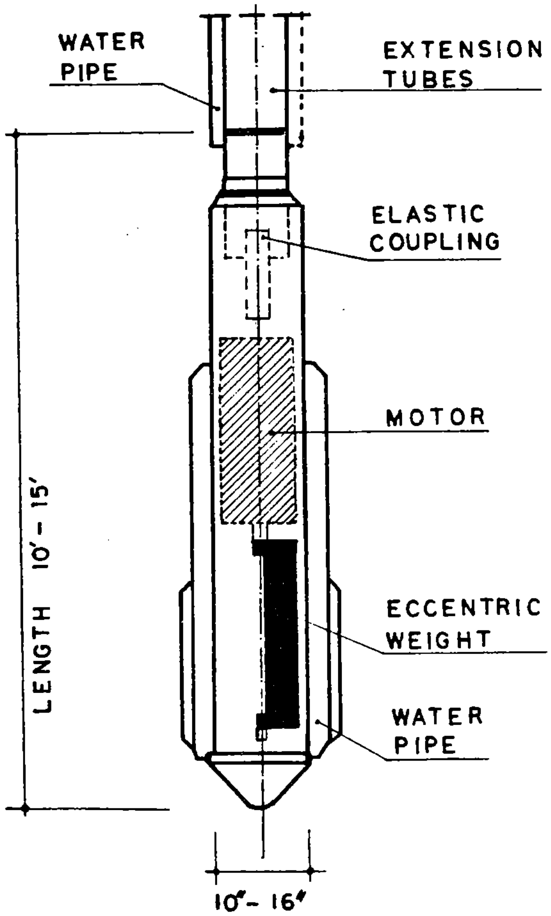

FIGURE 4. TYPICAL VIBRATOR USE FOR STONE COLUMN CONSTRUCTION.

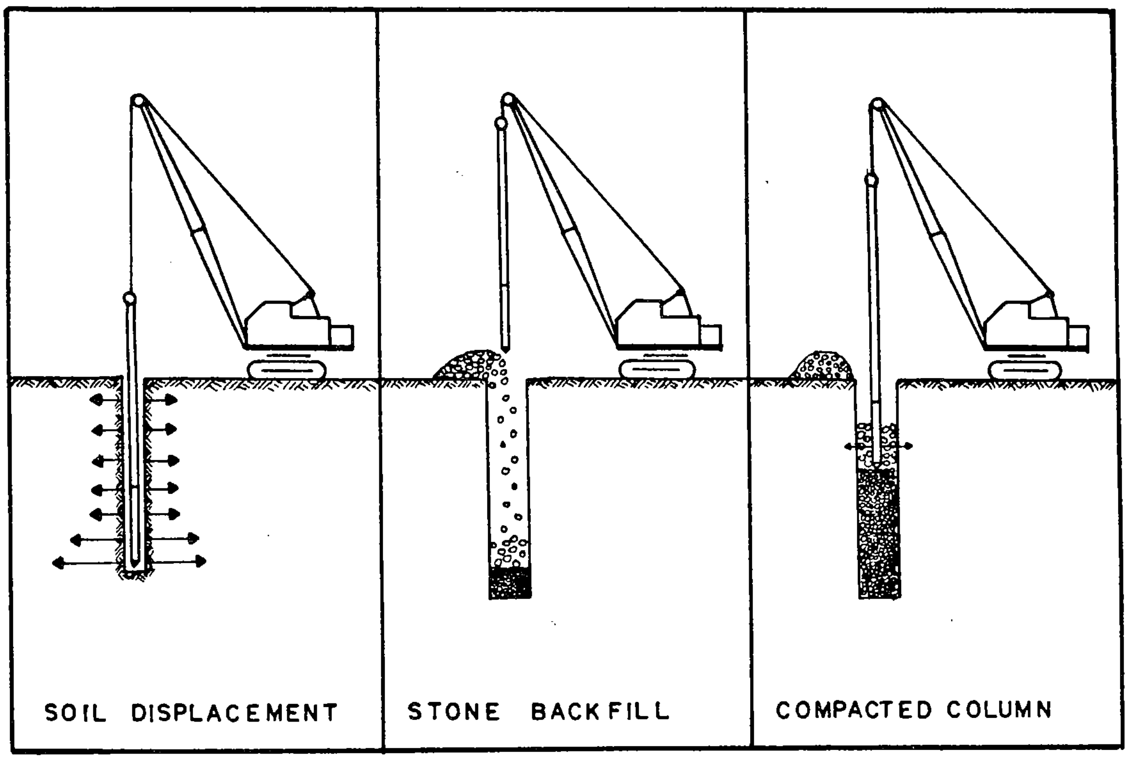

FIGURE 5. CONSTRUCTION OF STONE COLUMNS BY THE DRY (VIBRO-DISPLACEMENT) METHOD.

The cost of constructing dry columns tends to be less than vibro-replacement wet stone columns. The diameter of a dry stone column is usually smaller than a wet column constructed in the same soil. Typically, a dry column has a diameter of 2 to 3 ft. (0.6-0.9 m) compared to 3 to 4 ft. (0.9-1.3 m) for a wet column. Against this smaller diameter, however, the displacement of the soil during penetration using the dry process results in greater lateral support for the column. Engineers frequently use a smaller load carrying capacity for the vibro-displacement (dry) column than the vibro-replacement (wet) column. The uncertainty of a completely unsupported hole requires that this method be used only for appropriate subsurface conditions; this restriction is an additional factor in the use of reduced load capacity for dry columns.

RIGID CONCRETE COLUMNS - CASED HOLE

A special pull-down rig such as the one illustrated in Fig. 6 is used to construct rigid concrete columns. Construction of stone or concrete columns using a pull-down rig is well suited to an urban environment since water is not used. This rig can also be used to construct dry (vibro-displacement) stone columns, stone compaction piles and sand compaction piles.

The pull-down rig can exert a downward force of 12 to 15 tons. An important feature of this rig is that the soil is fully supported by the vibrator and follower tubes throughout the construction sequence. This eliminates the uncertainties associated with either the vibro-replacement (wet) or vibro-displacement (dry) methods of construction. The pull-down rig is mounted on a platform which moves on crawler tracks. The vibrator is supported by leads which can be laid down during transportation. Columns up to 48 ft. (15 m) in length can be constructed with this equipment. The pull-down rig is mobile, and requires little set-up time.

The same vibrator and follower tubes are used as employed for vibro-replacement (wet) stone column construction. Both air and water can, if desired, be used during construction. A 5.7 in. (145 mm) diameter tube is attached to the outside of the vibrator and follower tubes on the pull-down rig. Concrete or grout is pumped at a pressure of 200-800 psi (15 to 60 bars) down this tube to the bottom of the hole. The material which is fed through the pipe is limited to about 1.4 in. (36 mm) in diameter to prevent hanging up in the tube.

Using the pull-down equipment, a rigid column is constructed as follows: (1) The tremmie pipe and special nose cone are first filled with concrete to form a plug. (2) The vibrator and follower tubes are rapidly pulled down to the required elevation by a cable and winch system. The vibrator is left running throughout the construction. (3) Before pulling up the vibrator, concrete is pumped through the pipe. (4) The running vibrator is pulled up about 2 ft. (0.6 m), and the freshly placed concrete is then repenetrated to enlarge the base of the column. (5) The vibrator is then slowly and continuously extracted as the concrete is pumped.

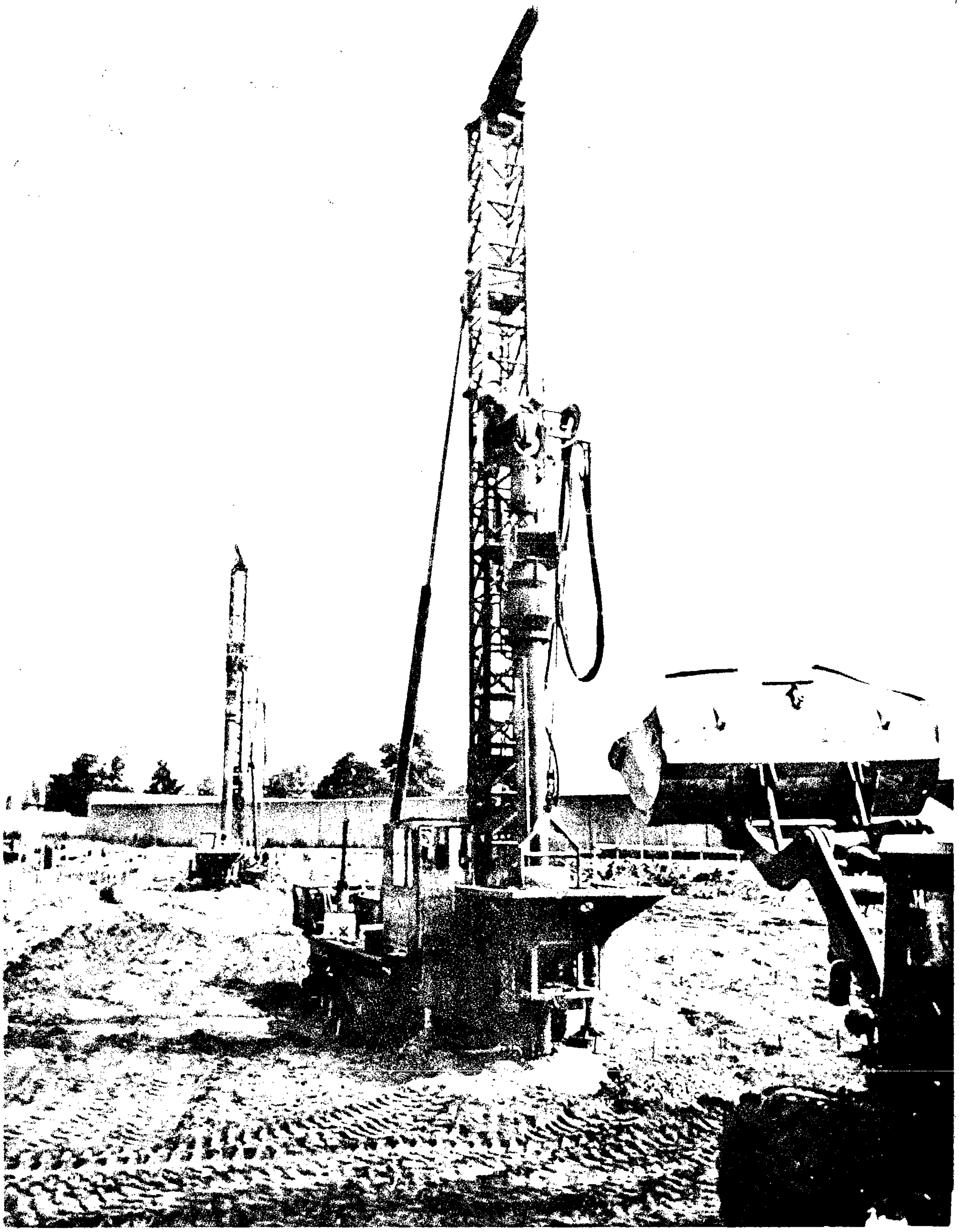

FIGURE 6. VIBRO-CAT PULL-DOWN RIG USED FOR RIGID CONCRETE COLUMNS.

Because of the presence of the feed tube adjacent to the vibrator, an egg-shaped column is constructed. The minimum diameter column constructed in soft cohesive soils is about 12 by 22 in. (300 mm x 550 mm). This represents the actual size of the vibrator and is an absolute minimum. The concrete used is usually ready-mix having a strength of 3500 to 5000 psi (24-34 kN/m^2). For stability applications (lateral bending) a single large steel reinforcing bar is pushed down the center of the column after it has been completed.

Because of their smaller size and rapid construction obtained using the pull-down rig, rigid columns have about the same cost as stone columns.

CASE HISTORY - RIGID STONE COLUMNS

Rigid stone columns were used to support a 25 ft. (7.6 m) high embankment fill for a high speed railway near Munich, Germany. Because of the presence of a thick peat layer conventional stone columns were not feasible. The embankment was constructed immediately adjacent to an existing railway embankment as a result of construction of the Rhine-Main-Danube Canal and highway interchange (Fig. 7).

A typical boring log from the site is shown in Fig. 8. The ground-water table at the site was near the surface. A 1 to 15 ft. (0.3-4.6 m) thick layer of very soft peat having a shear strength of only 100 psf (5 kN/m^2) was encountered at the surface over most of the site. Alternating strata of soft silts and firm clays were found beneath the peat to the boring termination depth of 50 ft. (15 m). A very loose gravel layer 5 to more than 10 ft. (1.5-3 m) in thickness was frequently present at a depth of 6 to 15 ft. (2 to 4.7 m).

Originally, removal and replacement of the peat was planned to increase stability and reduce long-term settlement of the embankment. This alternative involved constructing a temporary sheet pile wall along the edge of the existing adjacent embankment for support during peat removal. The sheet pile wall was to be tied back into the existing embankment. Use of rigid stone columns offered the following advantages over replacement: (1) the sheet pile wall was not required, (2) embankment fill quantities and working area were reduced since the peat was not removed, (3) construction time was decreased, and (4) rigid stone columns offered an economic advantage over replacement.

To stabilize the site, 866 rigid stone columns were constructed using the bottom-beed system. The rigid columns were carried down through the loose gravel strata and terminated in the stiff clay at an average depth of 21 ft. (6.5 m). The design load on each rigid stone column was 45 tons with the measured ultimate load being greater than 130 tons (Fig. 9). The rigid columns varied from 20 to 22 in. (510-560 mm) in diameter. An equilateral triangular pattern of columns was used with the spacing varying from 5.2 to 7.2 ft. (1.6-2.2 m). Each rigid column had a total tributary area of 30 to 42 ft.^{2} (2.8-3.9 m^2) depending upon the embankment height. The corresponding area replaced varied from 6 to 8 percent which is much less than usually used for conventional stone columns. Reported settlement of the embankment was

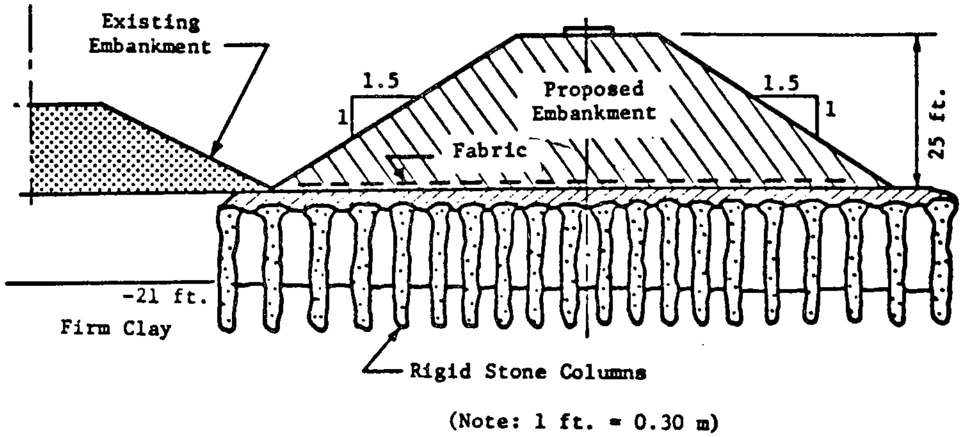

FIGURE 7. EMBANKMENT SECTION AT MUNICH, GERMANY - RIGID STONE COLUMNS.

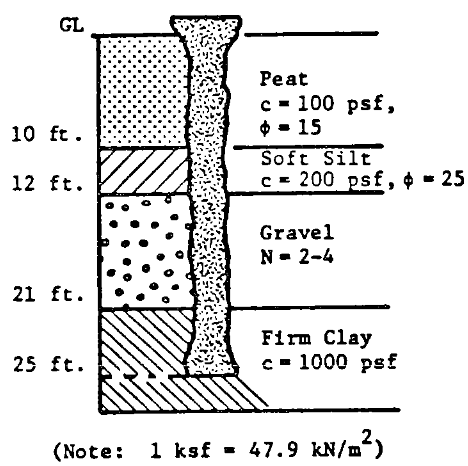

FIGURE 8. TYPICAL SOIL PROFILE AT MUNICH, GERMANY - RIGID STONE COLUMNS.

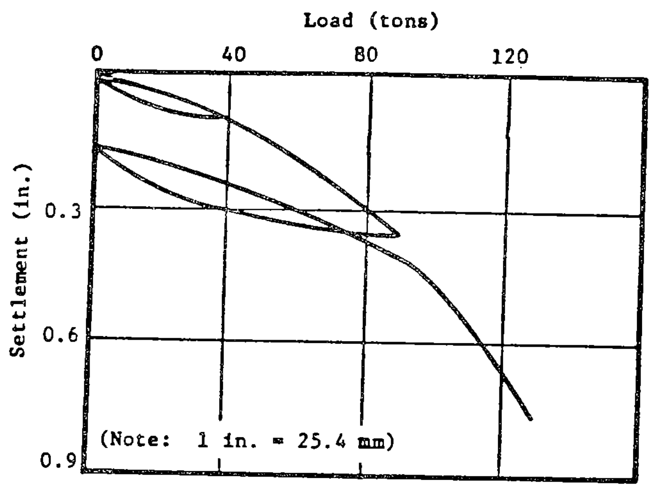

FIGURE 9. VERTICAL LOAD-SETTLEMENT RESPONSE OF RIGID STONE COLUMN AT MUNICH, GERMANY.

less than 0.25 in. (6 mm).

The rigid stone columns were constructed using a ready mix concrete which was pumped to the bottom of the hole through the small feeder pipe attached to the outside of the main vibrator tube. The feeder pipe was approximately 5.8 in. (145 mm) in diameter. The concrete had a maximum aggregate size of 1.8 in. (32 mm), and an unconfined compressive strength of 5,000 psi (34,000 kN/m^2). After pushing the probe to the final elevation with the vibrator running, the tubes were lifted about 1 ft. (0.3 m). Enough concrete was then pumped into the bottom to fill this space, and the concrete was repenetrated by the vibrator. The tube was slowly and continuously withdrawn (with the vibrator running) as concrete was pumped into the hole left by the tube. Running the vibrator as the tube was withdrawn densified the concrete and pushed it into the surrounding soil. A rigid column constructed in this way is quite similar to a conventional cast-in-place concrete or auger cast pile. Conventional piles, however, are not subjected to the high level of vibration that a rigid stone column undergoes.

A 1 to 2 ft. (0.3-0.6 m) thick granular blanket was placed over the rigid columns. A fabric layer having a tensile strength of 1 to 2 tons/ft. (3-6 tons/m) was laid at the interface between the granular blanket and the embankment to resist horizontal embankment forces. Use of a granular blanket and fabric over rigid stone columns is a common practice in Germany.

DESIGN

The ratio of stress in the stone column to stress in the surrounding soil is called the stress concentration factor n. The stress concentration factor is used in both stability and settlement analyses, and accounts for part of the beneficial effect of using a rigid reinforcing member in a soft material. For stone column stability analyses a value of n from 2 to 2.5 is recommended. A detailed discussion of the theoretical aspects of stone column design including stress concentration, stability and settlement analyses has been given elsewhere [6-9].

For stability analyses an angle of internal friction ϕ of 38 to 42° is recommended for gravel, and 42° to 45° for good quality crushed stone. For sand compaction piles a friction angle of 30 to 35° is appropriate.

CONCLUSIONS

Presently available ground reinforcement techniques include wet and dry stone columns, rigid concrete columns, stone compaction piles and sand compaction piles. These techniques offer the designer a number of alternatives to conventional designs, particularly in an urban setting. To select the most suitable alternative, a thorough subsurface investigation is needed to define the type soils present, and the extent of their strength and variation across the site. For the type of ground improvement methods discussed, it is very important to determine the presence of peats or organic soils.

Particularly promising applications of these methods appear to be for (1) stabilizations of embankments and slopes, (2) support of retaining structures including Reinforced Earth construction, (3) bridge approach and widening work, and (4) prevention of liquefaction during earthquakes. Foundation support of bridges and other structures is another potential use at sites where settlements without remedial work would be slightly excessive. Under these conditions reinforcement of slightly marginal silty sands having a silt content greater than about 15 percent is quite attractive.

Those involved with design and inspection should become familiar with the details of constructing these reinforcement systems. Careful field inspection is particularly important in the construction of vibro-replacement (wet) and vibro-displacement (dry) stone columns which are constructed in uncased holes.

ACKNOWLEDGEMENT

This paper is an extension of an FHWA-sponsored research project on stone columns conducted by R. D. Barksdale and R. C. Bachus. The project technical monitor was Jerry DiMaggio.

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"BLASTING VIBRATIONS IN AN URBAN ENVIRONMENT

by

Charles W. Trettel, President
VIBRA-TECH SOUTH

Presented To

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Stone Mountain, Georgia
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"BLASTING VIBRATIONS IN AN URBAN ENVIRONMENT"

Charles W. Trettel
Vibra-Tech South

The use of explosives, when possible, on rock excavation projects is perhaps the most economical process to use. Mechanical or the new chemical processes, being the only alternative in some cases, are expensive and time consuming.

Explosives therefore, are commonly used to obtain minerals from the earth, to mine raw materials for industry and agriculture and to build utility and transportation facilities to move goods to support our economy and provide us with jobs.

Demands set forth by environmental concerns have necessitated to an ever increasing extent, the need for vibration measurements to determine the effects of blasting on surrounding structures.

For a geotechnical engineer to do an effective job on projects requiring blasting, a knowledge of explosives and explosives effects is necessary. The purpose of this paper is to discuss the ground vibration effects of blasting and to suggest procedures to be followed for measuring and evaluating these effects.

Consideration of these effects in the design and specification writing stages of a project is time well spent in preventing damage to neighboring or job site structures and serious public relations problems during construction.

In order to disseminate this information, we will review some basic principles of blasting mechanics, elastic waves, vibration classification, damage criteria and new techniques of vibration analyses.

I hope when you leave the room this morning you will remember the following points:

- Particle Velocity and Frequency (hz) are the most important considerations in examining a vibration problem.

- Input energy into a structure can be greater or less than the measurements within the structure. Thus, outside measurements may not be a true reflection of what is going on inside.

- A single number vibration damage criteria is not a valid method to treat damage potential.

- The effects of blasting vibration on nearby structures, particularly on a construction project, are not as critical as the actual effects of the displacement and heaving of the shot rock.

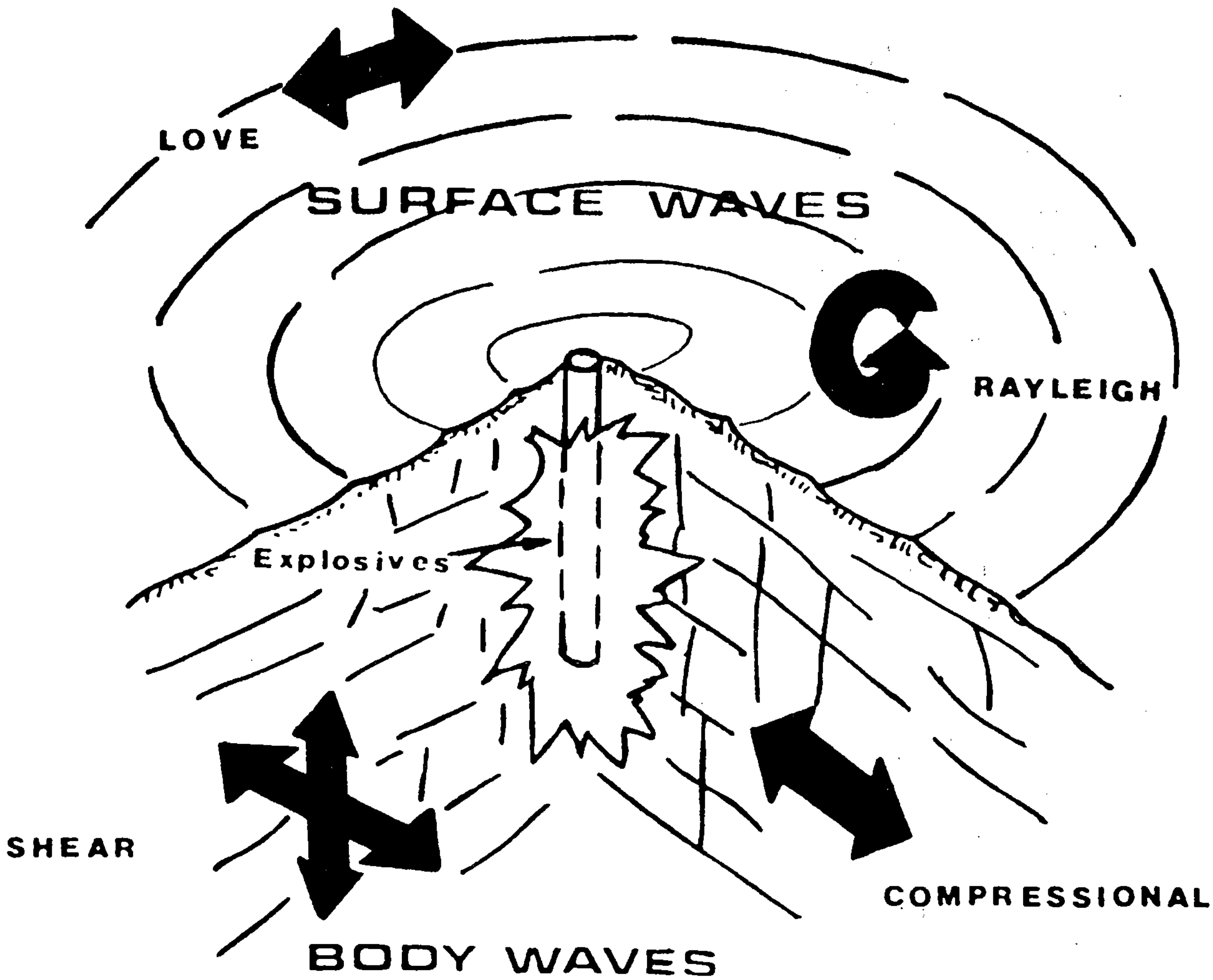
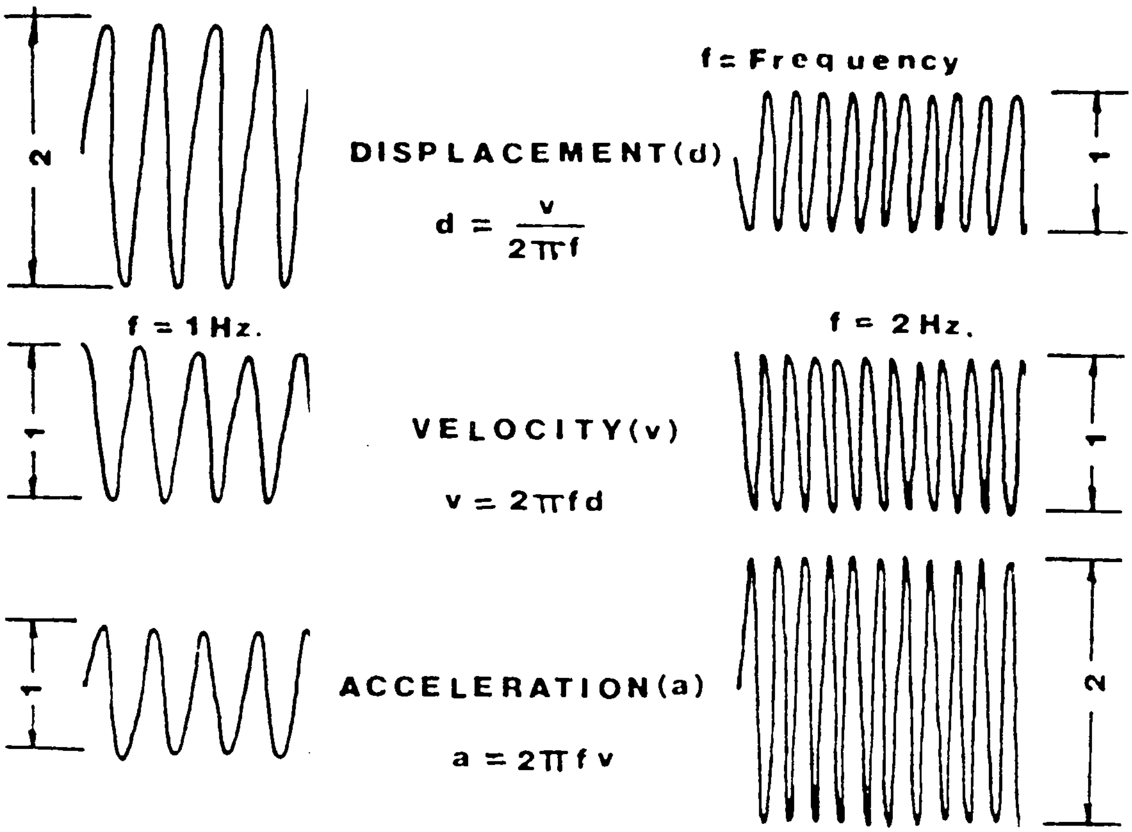


FIGURE I - Elastic Waves Generated by Blasting

The physical processes that take place in the vicinity of an explosives filled drill hole immediately after detonation are violent and extremely complex. The explosives perform work by breaking and heaving rock. Beyond this immediate zone, energy is transferred into the surrounding rock as an elastic wave. The passing of this wave out away from the blast site develops many types of seismic waves. Compressional and shear waves travel through the body of the rock and surface waves are developed along the ground surface. As these body and surface waves propagate through the earth, movements take place within the elastic limit of the materials through which they travel. These materials return to their original shape after the waves have passed through them. The seismic energy attenuates rapidly with distance due to geometric spreading and loss mechanicisms associated with the transmission of energy. The effect, real and imagined, of these waves on neighboring structures, is the core of the blast vibration problem.

Blasting vibrations are classified in terms of displacement, velocity, acceleration and frequency of the motion. Displacement is the aplitude of the motion usually expressed in inches. Velocity is the rate of change in displacement expressed in inches per second. Acceleration is the rate of change in velocity in units of inches per second^2. Frequency is the number of oscillations that take place per unit of time expressed in cycles per second of Hertz (1 cycle/second). Blasting seismology today is primarily concerned in measuring velocity and frequency.

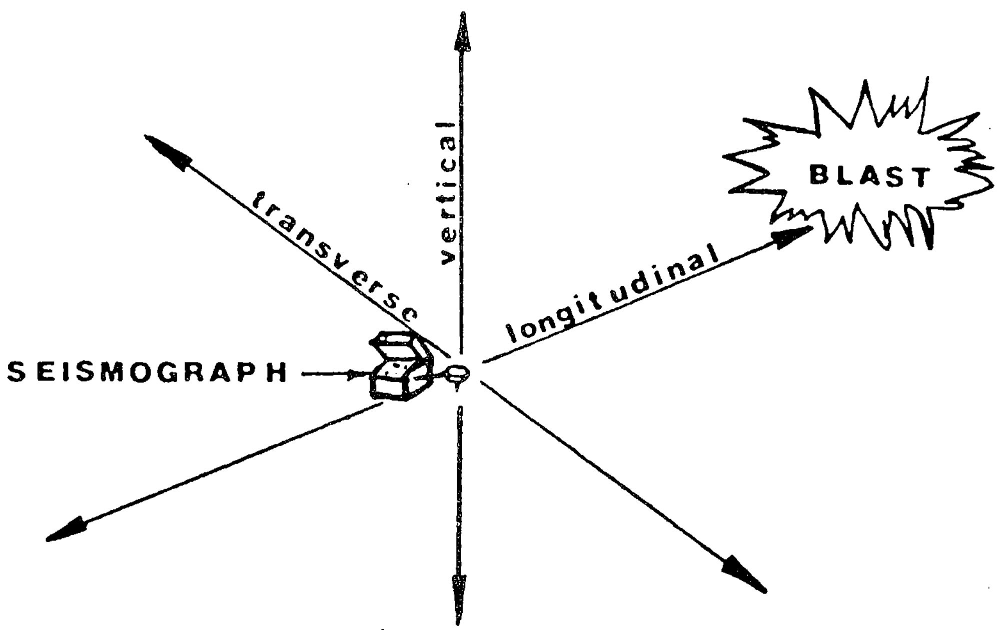
For harmonic motion the relationship between the displacement, velocity, acceleration and frequency can be seen below:



It can be seen that for a constant velocity, that doubling the frequency of the motion will reduce the displacement by one half and increase the acceleration by two times. Two vibrations of the same velocity could produce entirely different displacements and accelerations. Frequency is, therefore, a very important factor when considering the blasting vibration effects on structures.

Most blasting seismographs in use today measure the velocity (called particle velocity) of the blast vibrations. The particle velocity is measured by recording output of a velocity transducer. This output is generated by a coil moving through a magnetic field. The voltage induced in the coil is directly related to the relative velocity between the coil and the magnetic field. Having the coil or the magnetic field remain stationary while the other moves as the ground surface moves will generate a voltage directly related to the ground motion. The greater the velocity of ground motion, the higher the induced voltage.

Blasting vibrations are recorded using three transducers oriented in three mutually perpendicular planes (two horizontal and one vertical). The horizontal plane connecting the seismograph location and the blast site is called the longitudinal, or radial component. The second horizontal plane, called the transverse component, is perpendicular to the longitudinal plane. The third transducer lies in the vertical plane.

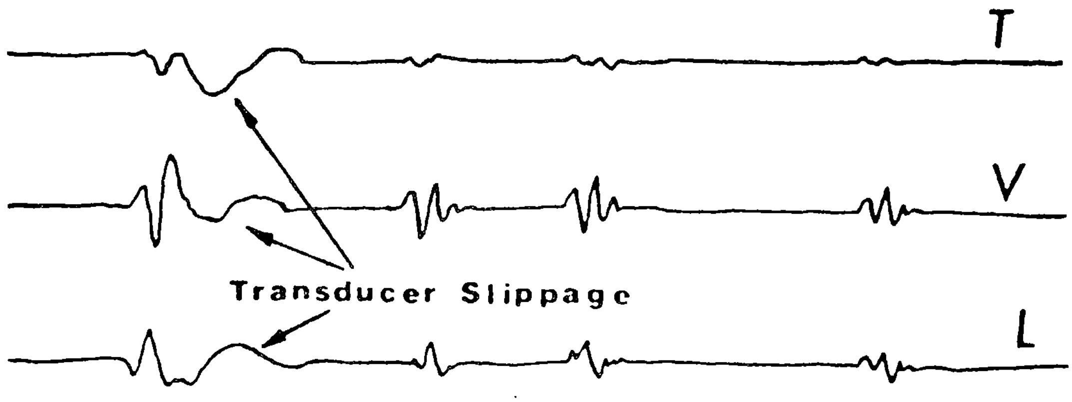


In addition to the triaxial transducer, a blasting seismograph contains a timing system to produce an accurate time reference signal and a recording medium such as photographic paper or magnetic film. The United States Bureau of Mines recommends that seismographs used for recording blasting vibrations have a minimum frequency range from 2 to 200 Hertz.

Care must be taken to insure good transducer coupling to the surface upon which is is placed. The transducer's purpose is to measure the motion of the surface upon which it is located. Any slippage or independent movement of the transducer will therefore result in an incorrect and misleading recording. One method frequently used when there is a danger of slippage is to place a sandbag on top of the transducer to insure good coupling to the ground.

Full waveform velocity seismographs provide a three dimension time history of the ground motion particle velocity. These wavesforms enable a much more complete and relevant analysis of the blast vibration effects.

Transducer slippage can be readily detected on the waveform.



In discussing blasting vibrations, there are two key factors: how far away is the closest structure or point of concern and secondly, what size blasts are being planned. These two factors determine what the vibration levels will be at a specific location. Obviously, the closer the blasting and the larger the amount of explosives detonated at one time, the greater the vibration levels will be. There is not much one can do so far as distance is concerned but there is a lot of flexibility in the shot design which will vary the amount of explosives being detonated at one time.

One way of expressing this relationship between distance and explosive quantity is called Scaled Distance.

$$\text{Scaled Distance} = \frac{\text{Distance (feet)}}{\sqrt{\text{Maximum Explosives/Delay (lbs)}}}$$

This relationship between scaled distance and particle velocity is shown by:

$$\text{Particle Velocity} = H\ (\text{Scaled Distance})^{b}$$

(H and b are site constants)

When particle velocity is plotted on log-log paper as a function of scaled distance, H is the intercept when the scaled distance equals 1.0 and b is the slope or regression coefficient.

In 1942, Bulletin 442 was developed by the U. S. Bureau of Mines. It classified vibration predictability with respect to overburden and rock density vs. total weight of explosives. The recommended limit of vibrations in Bulletin 442 was a ground displacement of 0.032 inches. Later studies in 1965 and 1971 produced Bulletin 656 in which the maximum pounds of explosives per delay vs. distance was considered a better criteria to predict ground vibration.

A particle velocity of 2.0 inches per second was recommended as a safe vibration limit for all structures. Many states and regulating agencies have adopted 2.0 inches per second as a maximum vibration limit. This is the case in the state of Georgia.

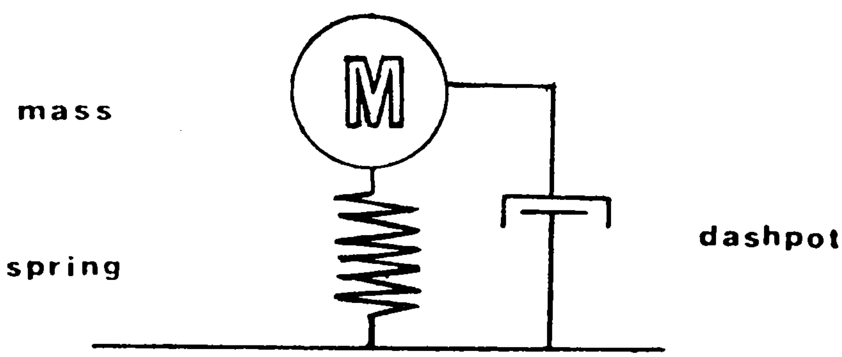
If no seismograph recordings were taken, a scaled distance of 50 was used as a guide in planning blasts to insure the particle velocity would not exceed 2.0 inches/second.

In recent years, the use of a one number peak particle velocity criteria to determine blast vibration damage potential has been increasingly criticized. The basis of this criticism has been that the one number criteria ignores frequency content of the ground motion relative to the natural frequency of the structure.

In 1976, the National Crushed Stone Association, the Institute of Manufacturers of Explosives, along with several state stone producing organizations funded a study to re-examine the blast vibration criteria in light of this criticism. The study undertaken by Kenneth Medearis Associates, Fort Collins, Colorado recommended a more rational criteria based upon response spectra. Response spectra is a technique originally introduced in earthquake engineering and used extensively in nuclear blasting seismology. The response spectra takes into account the amplitude and frequency of the seismic signal as well as the natural frequency and damping of the structure.

The response spectrum technique involves computing the response of a structure to a ground motion forcing function. This method calculates how much energy is coupled into the structure, a quantity which may or may not be reflected by peak particle velocity alone.

A structure is represented as a single degree of freedom system consisting of a mass, spring and dashpot.



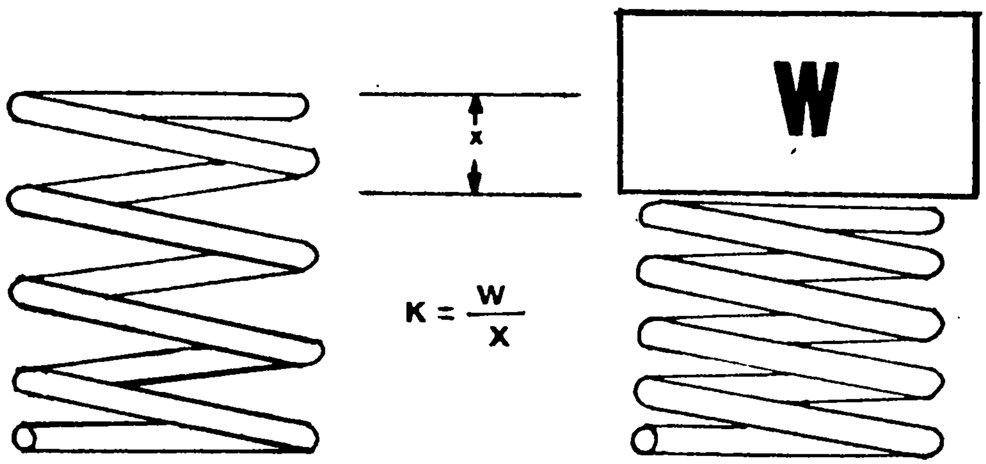
The natural frequency of the system can be determined from:

$$F = \frac{1}{2\pi}\sqrt{\frac{K}{M}}$$

K = equivalent spring stiffness

M = mass

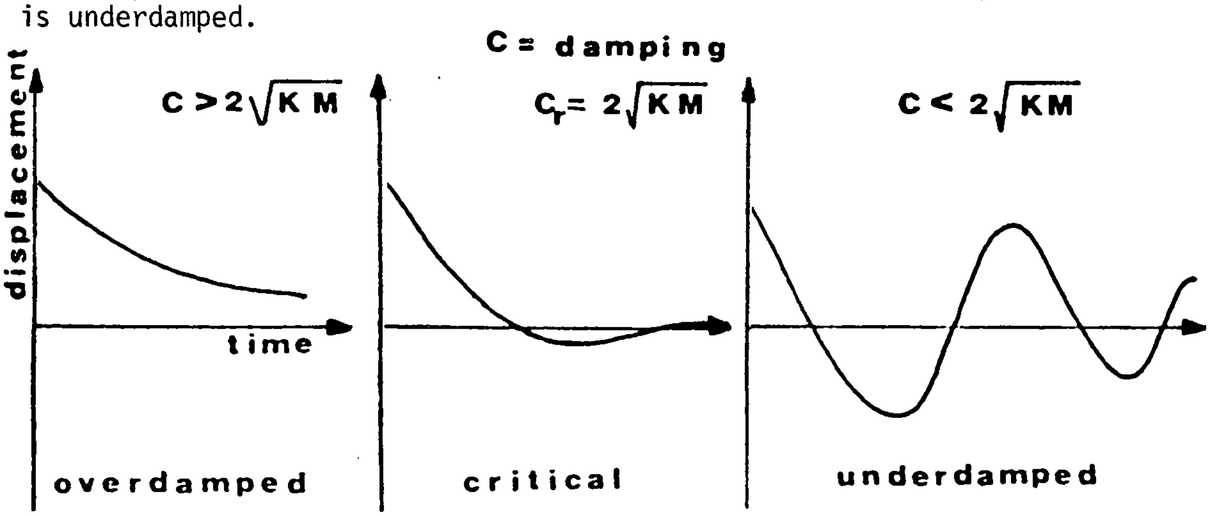
The equivalent spring stiffness can be visualized by picturing a spring being deformed because of a force being applied to it.



Structures like springs have stiffness which resists deformation. From the above equation for natural frequency, it can be seen that the stiffer a structure, the higher the natural frequency. Tall buildings, being more flexible, will have a lower natural frequency than shorter, stiff buildings. The more mass to a structure, the lower its natural frequency.

Vibratory motion in the single degree of freedom system encounters resistance from the dashpot. The dashpot dampens or slows down the motion, resulting in the eventual dying out of the oscillations. The damping force is directly proportional to the velocity of the vibrations having units in pounds/inches/second.

Damping exerts a force which acts to oppose the motion of the mass. When the mass will not oscillate, when it is displaced, and the released, but only returns to its equilibrium position, it is said to be critically damped. When the mass does not return to its equilibrium position, it is overdamped. When the mass oscillates about its equilibrium position, it is underdamped.



It is common to refer to damping as percent of critical damping or damping ratio:

D = damping ratio

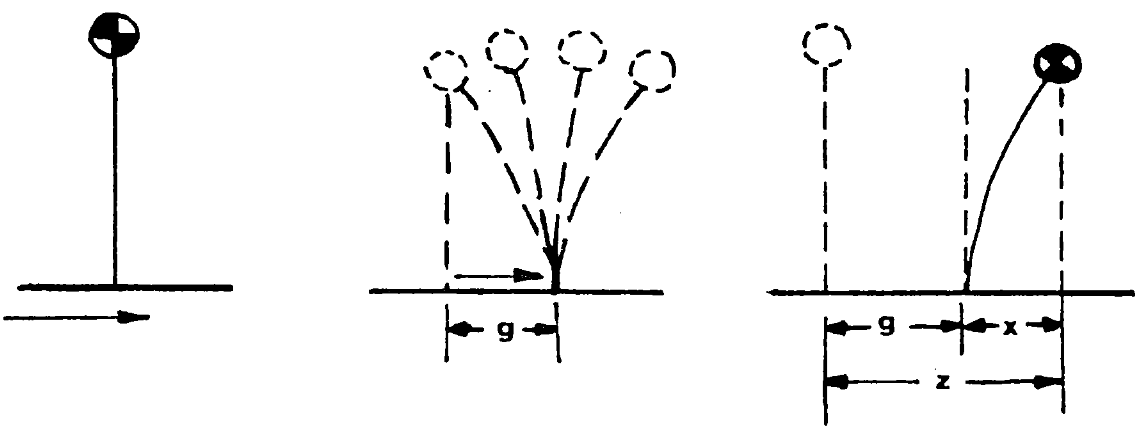
$$D = \frac{C}{C_r} = \frac{C}{2\sqrt{KM}}$$

Damping and stiffness produce forces which oppose a vibrating forcing function. A positive displacement of the mass will create a negative restoring force (k x d) and a negative damping force (c x v). From Newton's second and third law of motion the following relationship exists:

$$ma = -cv - kd$$

$$ma + cv + kd = 0$$

The following figure shows a single degree of freedom system about to receive and then respond to an incoming vibration forcing function at its base. As the base moves initially, the mass tends to remain stationary, which causes a deflection in the system, the mass then responds and vibrates about the new base position.



Modeling the above system as a building produces the following relationships:

z = **absolute building displacement**

g = **ground displacement**

x = **relative displacement of building**

$$z = g + x$$

$$\ddot{z} = \ddot{g} + \ddot{x}$$

The equation of motion for the system is then expanded to include the effects of the ground motion, natural frequency and damping of the structure.

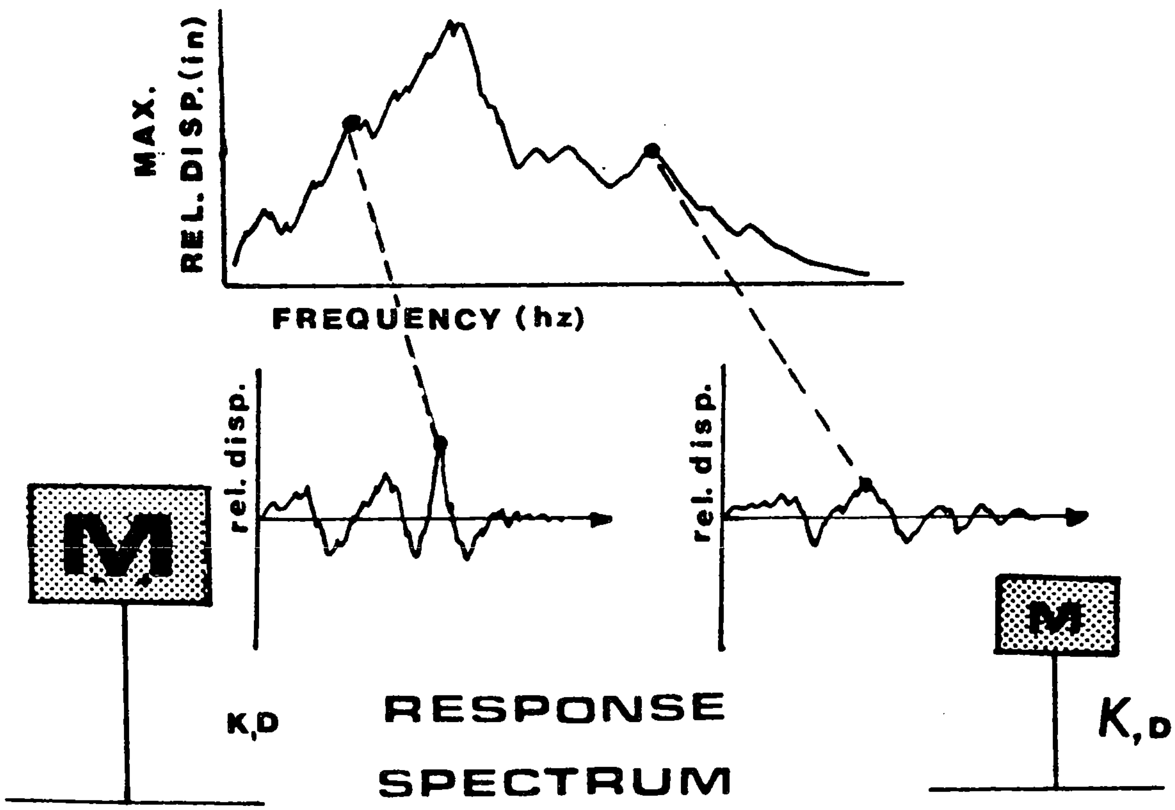
$$m\ddot{z} + c\dot{x} + kx = 0$$

$$m(\ddot{g} + \ddot{x}) + c\dot{x} + kx = 0$$

This equation is solved for x, the building deflection or relative displacement between the ground and the equivalent concentrated mass. This calculation is done using the seismograph recording of the ground motion and assigning a value for the structure's natural frequency and damping.

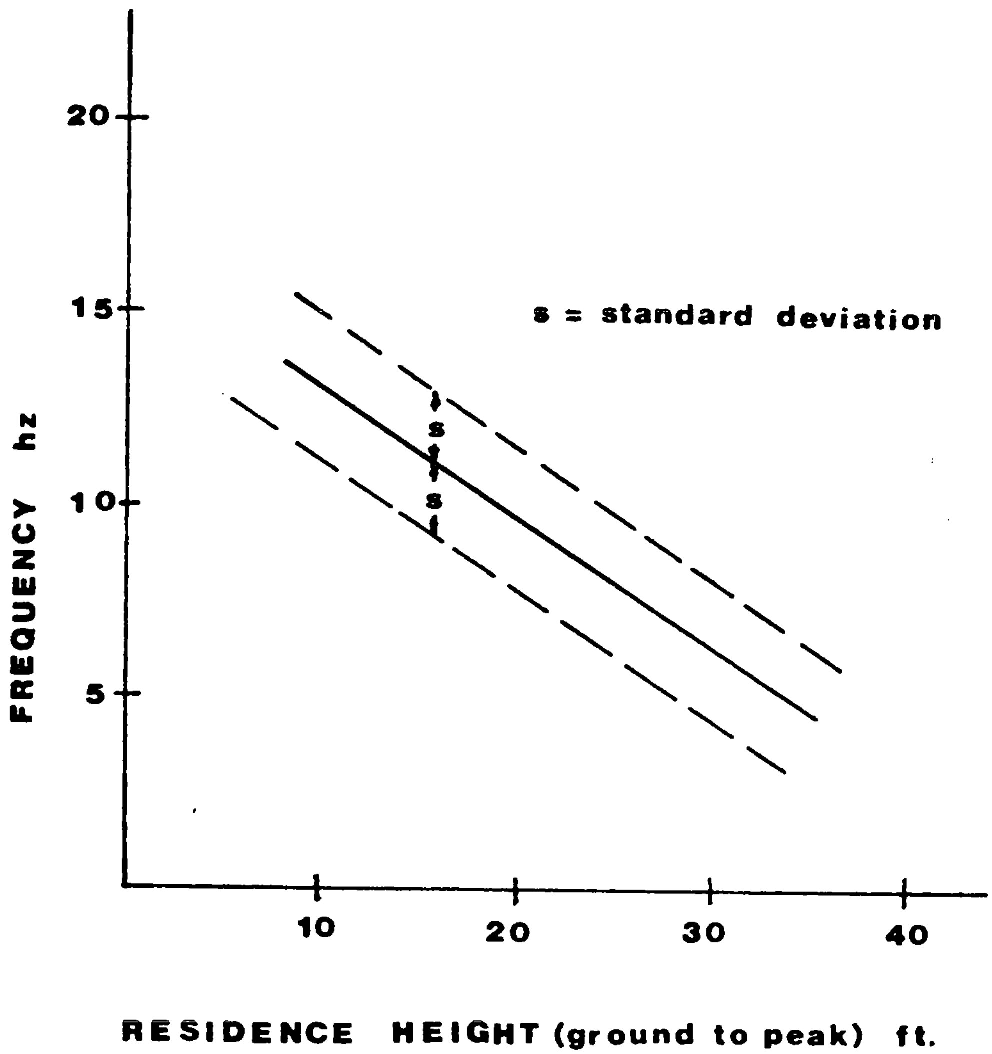
For the calculations to be done efficiently, a digital computer is required and a magnetic tape recording of the ground motion available.

Determining the maximum relative displacement for a given fundamental natural frequency gives one point on a reponse spectrum curve. Holding the damping ratio constant, but varying the natural frequency of the system, the calculation to determine the maximum relative displacement is again carried out. This process is carried out over a wide range of frequencies. The curve generated by plotting the maximum relative displacement versus natural frequency is the response spectrum. A series of curves could be plotted for several different values of damping. A set of these curves is called a response spectra.



The relative displacement can be converted to pseudo spectral relative velocity by multiplying the relative displacement by $2\pi f$. The pseudo spectral relative velocity is closely related to the maximum energy transferred into the structure by the ground shaking and therefore, it is the maximum energy the structure has to be able to dissipate without damage.

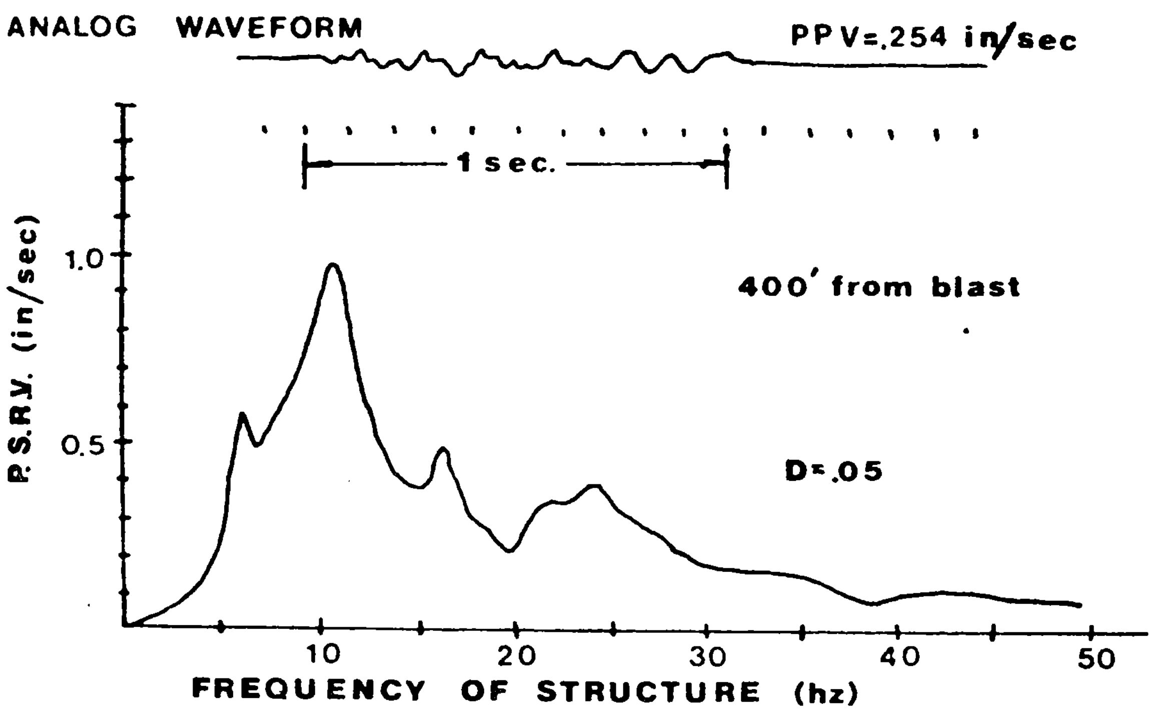
The Medearis study indicated that the natural frequency of residential structures can be estimated from their height.



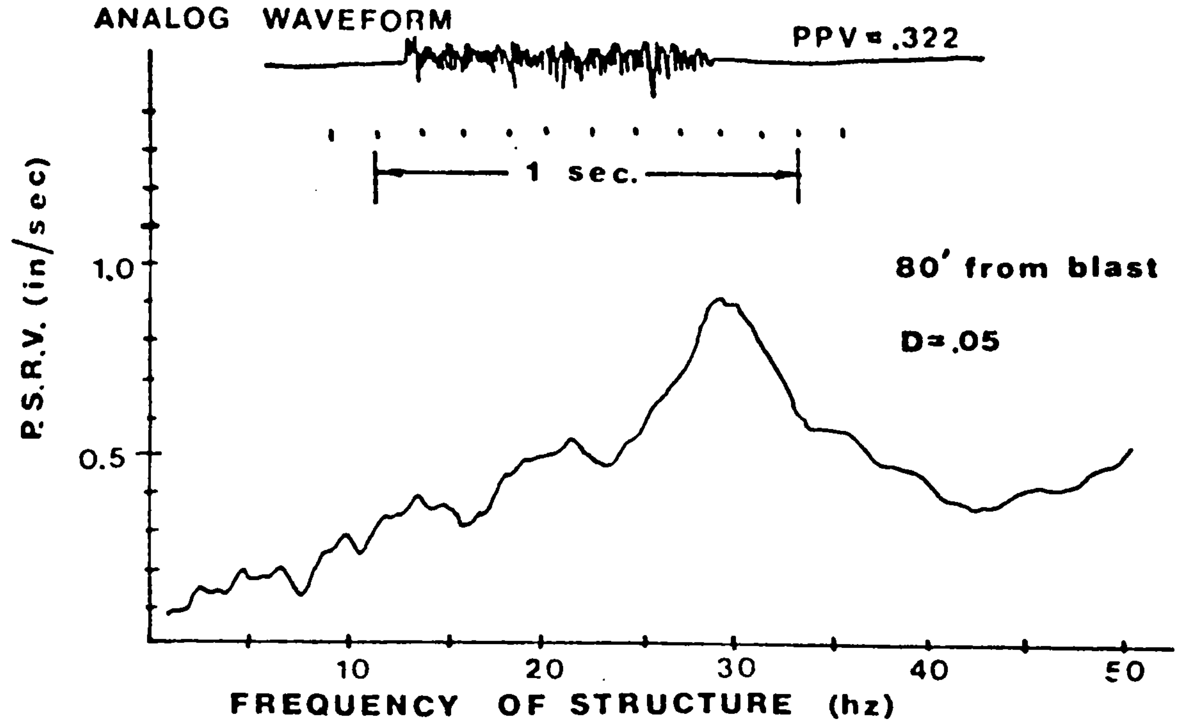
(From Medearis Study)

Medearis recommends a 5% critical damping ratio as more relevant for residential structures than the 2% spectra, particularly at damage inducing levels of ground motion. Reviewing past studies of blast vibration damage, Medearis concludes that a measured pseudo spectral relative velocity of 1.5 inches/second has a damage probability estimated at no more than 1%.

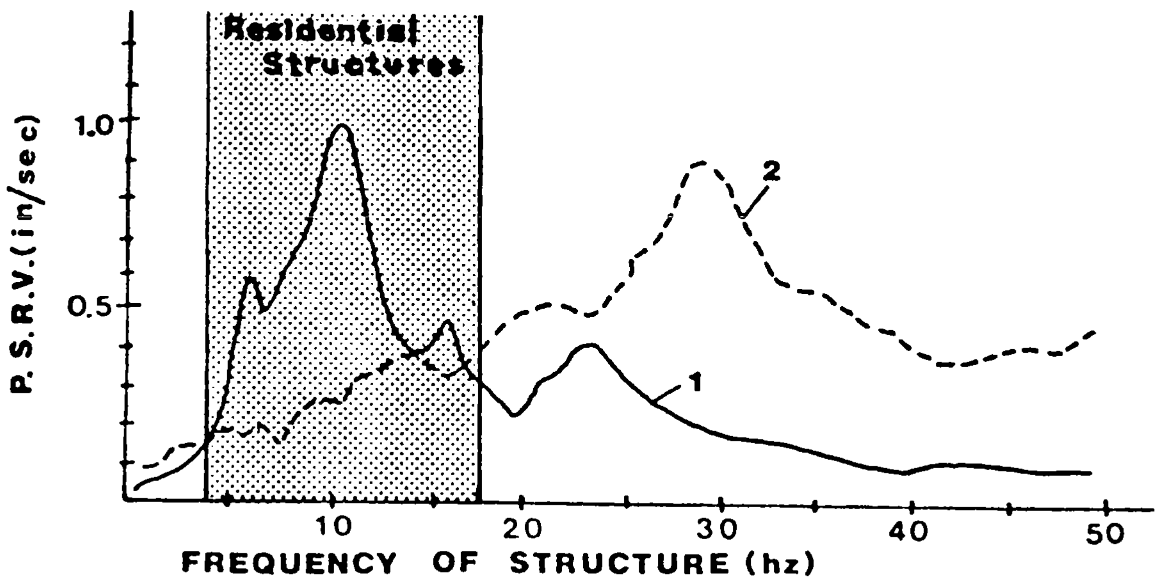
Blasts at two different civil engineering projects are compared to demonstrate the usefulness of the response spectrum technique. Project No. 1 involved blasting approximately 18 feet of weathered limestone in an area underlain by 15 to 30 feet of overburden. The overburden had been removed from the rock to be blasted. The closest structure was 400 feet from the blasting. The blast consisted of 102 holes, 3½ inches in diameter, 18 feet deep with a maximum charge per delay of 178 pounds.



On Project No. 2 approximately 9 feet of trap rock was being blasted for an office building foundation. The trap rock was overlain by a thin soil cover. The closest structure to the blasting was 80 feet. The blast consisted of 54 holes, 2½ inches in diameter, 9 feet deep with a maximum charge per delay of 4.5 pounds.



While the particle velocities for the two blasts were similar, the frequency content was very different. By comparing the response plot for each blast, this difference becomes very relevant. The blast vibrations in Project No. 1 introduced much more energy into neighboring residential structures. A potential public relations problem is present and if the particle velocities on future blasts are increased, the potential for damage exists. The response spectrum has alerted the geotechnical engineer to a situation that was not obvious from peak particle velocity alone.



Criticism and questions relating to a one number damage criteria has led to more recent Federal Regulation governing the surface mining of bituminous coal and has established varying particle velocity limits based upon distance from the blast site:

Distance from Blast (feet)	Maximum Particle Velocity (inch per second)	Scaled Distance w/o monitoring
0 - 300	1.25	50
301 - 5,000	1.00	55
5,000 +	0.75	65

As noted from the above, greater particle velocities are permitted closer to the blast source. This of course, is predicted on the fact that the seismic wave frequencies (hz) are higher closer to the blast and less apt to be in the frequency range of existing structures.

Prevention of damage is only one aspect of the blast vibration problem. Many vibration consultants will tell you that establishing and maintaining good public relations is equally or more important.

Most adverse public reaction to blasting can be related to the subjective human response to vibrations. Studies have shown that people are sensitive to vibrations that are far below damaging levels and will judge them to be severe or objectionable. The lack of trust and credibility toward the explosive user, justly or unjustly created, is often the root of the problem.

The solution, as is the solution to most problems involving people, is creating lines of communication and establishing credibility and trust. Public meetings, before the start of the project, explaining the project, blasting procedures and steps to be taken to insure proper vibration levels are being transmitted into the neighborhood, are the beginning of a good public relations effort.

A pre-blast inspection serves the double function of not only documenting any existing defects in a structure, but also makes the homeowner aware of their existence before blasting takes place.

Measurement of the vibration levels in the presence of the property owner has been an effective way to show there is nothing being hidden. Peak meters on the recording seismograph enable the public to see the vibration levels being produced.

In summary, blasting is an economic necessity for most construction projects involving rock excavation. The protection of nearby structures from damage and the maintenance of good public relations can be done by proper measurement and analysis of ground vibrations from blasting. In order to take full advantage of the benefits offered by using explosives, the geotechnical engineer should be familar with vibration measurement and analysis procedures.

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SOUTHWEST CORRIDOR PROJECT, BOSTON, MASSACHUSETTS

by James R. Lambrechts(1)

INTRODUCTION

Many geotechnical challenges have been encountered during design and construction of the Massachusetts Bay Transportation Authority's (MBTA) 4-1/2 mile long Southwest Corridor Project in Boston, Massachusetts. The project will provide two tracks to relocate part of the MBTA's subway system, plus three tracks for MBTA commuter rail service and AMTRAK's high-speed Northeast Corridor.

Although most major transportation projects in urban areas encounter similar types of problems such as lateral support of excavations, protection of nearby structures, the effects of groundwater and its control, and existing utilities, it has been Boston's unique geologic setting and patterns of historical development that have made our share of these problems particularly challenging.

Presented first in this paper is a brief overview of the Southwest Corridor Project, including a description of the project and background on its evolution. Next the geology, historic urban development, and the subsurface stratification and engineering properties of soils along the corridor are summarized. Finally, five of the many geotechnical problems which confronted the designers and contractors are discussed; these problems include:

1. Cut-and-cover tunnel construction through an area underlain by soft organic soils with adjacent structures generally 4 to 6 feet away.
2. Preventing the new corridor structure from creating a barrier to cross alignment groundwater flow.
3. Measures, to resist hydrostatic uplift, particularly the use of rock anchors.
4. The Stony Brook Conduit reconstruction and mitigation of effects of filling above a marginally stable section of the conduit.
5. Underpinning part of an eight story caisson-supported building to allow the new corridor structure to pass beneath.

(1) Senior Engineer, Haley & Aldrich, Inc., Cambridge, Massachusetts

PROJECT DESCRIPTION

The Southwest Corridor Project is one of the largest construction projects ever undertaken in the city of Boston. It involves the construction of a 4-1/2 mile long, 80 to 120 ft. wide railroad/rapid transit structure from the edge of downtown Boston to Forest Hills. Construction is currently underway along all of the 4-1/2 mile-long alignment. The new structure will provide two tracks for rapid transit and three tracks for MBTA commuter rail service and AMTRAK's high-speed Northeast Corridor. Estimated total project cost is $783 million.

The Urban Mass Transportation Administration (UMTA) has provided 90% of the funding through a transfer of funds previously earmarked for an extension of interstate highway I-95 into Boston. In the late 1960's, plans for the Southwest Expressway (I-95) were abandoned in the wake of strong local opposition, but not before most structures within the proposed right-of-way had been razed. Redevelopment in these cleared areas is important to the revitalization of neighboring communities along the Southwest Corridor Project alignment.

The corridor follows the former Penn Central railroad alignment southwest from Boston, and passes through the Boston neighborhoods of Back Bay, Roxbury and Jamaica Plain, as indicated in Figure 1. Along the alignment there will be eight new transit stations, three of these will have new commuter rail stations, and one will also have an AMTRAK station.

Haley & Aldrich, Inc. is the geotechnical consultant to Kaiser Engineers, Inc./Fay, Spofford & Thorndike, Inc., designer of the Back Bay section, and to PRC Harris, Inc., designer of the Roxbury section. The designer for the Jamaica Plain section has been Howard, Needles, Tammen and Bergendorf with Goldberg, Zoino & Associates, Inc. as geotechnical consultant.

The new corridor structure will be generally depressed below ground surface to remove the barrier and eyesore of the former railroad right-of-way, much of which was elevated on a 10 to 25 ft. high embankment. About 30% of the new structure will be decked-over and buried, primarily through the Back Bay area and next to housing projects where brick rowhouses and apartment buildings are immediately adjacent, to further reduce noise and vibration. The remainder of the corridor structure will not have a roof, but has been designed to accommodate a future deck.

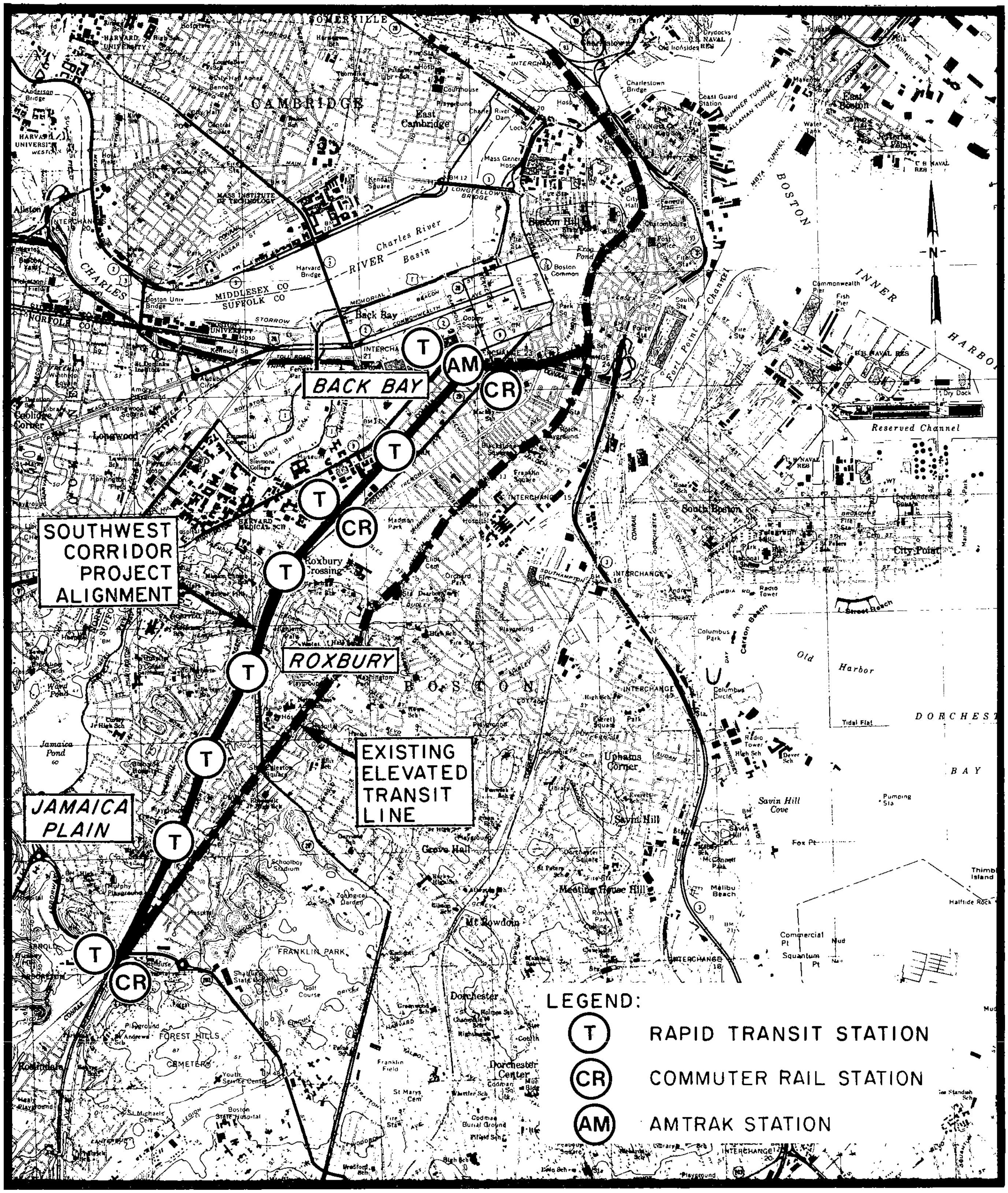


Figure 1 Location of Boston's Southwest Corridor Project

The new rapid transit line will replace a nearby, aging, elevated transit structure, the last in the MBTA's system. During construction, commuter rail and AMTRAK services have been diverted to an upgraded freight alignment through another area of Boston.

REGIONAL GEOLOGY

Boston lies in a lowland area known as the Boston Basin, which is surrounded by upland areas to the north, west and south, and the waters of the Boston Bay to the east. The Boston Basin consists of a very complex bedrock downwarp or structural synclinorium with which several major faults are associated. Bedrock is Roxbury Conglomerate and younger Cambridge Argillite.

The Boston area was significantly affected by glaciation, as was most of New England. The topographical low associated with the Boston Basin was created largely by glacial erosion of previously deposited sediments and considerable amounts of soft fine-grained Argillite bedrock. Over most bedrock surfaces there is a thin layer of very dense glacial till, later blanketed with redeposited sediments and other scoured debris. Meltwater streams flowing from the retreating glaciers deposited outwash sands and gravel over the till. In the upland areas where shallow lakes formed in areas behind knobs of bedrock or glacial till, fine sands and silts were deposited in lacustrine environments. Deposits of such fine sands and silts occur in several areas along the corridor in the Roxbury and Jamaica Plain sections.

The thick deposits of fine, marine sediments, known locally as the "Boston Blue Clay", were formed in the quiescent waters of the Boston Basin as the continental ice sheet retreated and sea level rose. In some areas, the marine clay filled deep valleys which were up to 200 ft. deep. The Boston Blue Clay is found throughout the Back Bay section of the Southwest Corridor. A minor readvance of the glacial ice sheet caused a drop in sea level, exposing the surface of the marine sediments to weathering, dessication and erosion. The present stiff crust of the Boston Blue Clay stratum is the result of this dessication. Glacial meltwater streams deposited granular outwash sporadically over the marine sediments.

Post glacial accredation has created alluvium deposits along local streams. With the slow, continued rise of sea level over the past 11,000 years, deposits of silts and fine sands have formed in flooded embayments and estuaries in and around the Boston Basin. These deposits usually have high organic contents and occasionally peaty layers due to the marshy environs that developed around the edges of the embayments.

HISTORICAL DEVELOPMENT

In colonial times, Boston occupied only a peninsula composed of several drumlins and glacial overthrust deposits. The peninsula was connected to the mainland by a narrow neck of land which became submerged under extreme high tide. The approximate shoreline of colonial Boston is indicated in Figure 2. Large tidal embayments with marsh areas existed on both sides of the neck. The embayment south and west of the Boston peninsula was known as Back Bay. These areas have, for the most part, been filled. The northern third of the Southwest Corridor alignment cuts through one such filled area of Back Bay.

Upland areas lie to the south and west of the Back Bay. The Stony Brook meandered through a valley which lies between several drumlins. The valley is believed to be the surface expression of the Stony Brook fault zone.

Development of Boston's Back Bay area began in the early 1800's with the construction of a dam and mill works along approximately the area's present northern edge to harness tidal water power. In the 1830's, the first railroad access into Boston was along the Southwest Corridor alignment. It was specifically routed through the Stony Brook valley to take advantage of the gently falling topography. To cross the Back Bay, earth embankments were constructed for the railroad. Sections of trestle were included to allow passage of water from areas isolated by the embankment; they were, however, ineffective in preventing stagnant, smelly conditions. For reasons of health and land development, the Back Bay tidal areas were filled. Filling occurred principally between the 1850's and 1880's, in a series of private and public development efforts. Millions of tons of sand and gravel were imported by rail from Needham, Massachusetts, about 15 miles away.

Building construction, most of which was residential, quickly followed the filling operations. Typically, Back Bay rowhouses are 3 or 4 stories high, constructed of brick, and founded on timber piles driven through the fill and organic silt to bearing in the crust of the marine clay or, where present, in the overlying sand layer. Consolidation of the organic silt and marine clay under the rapidly applied fill and building

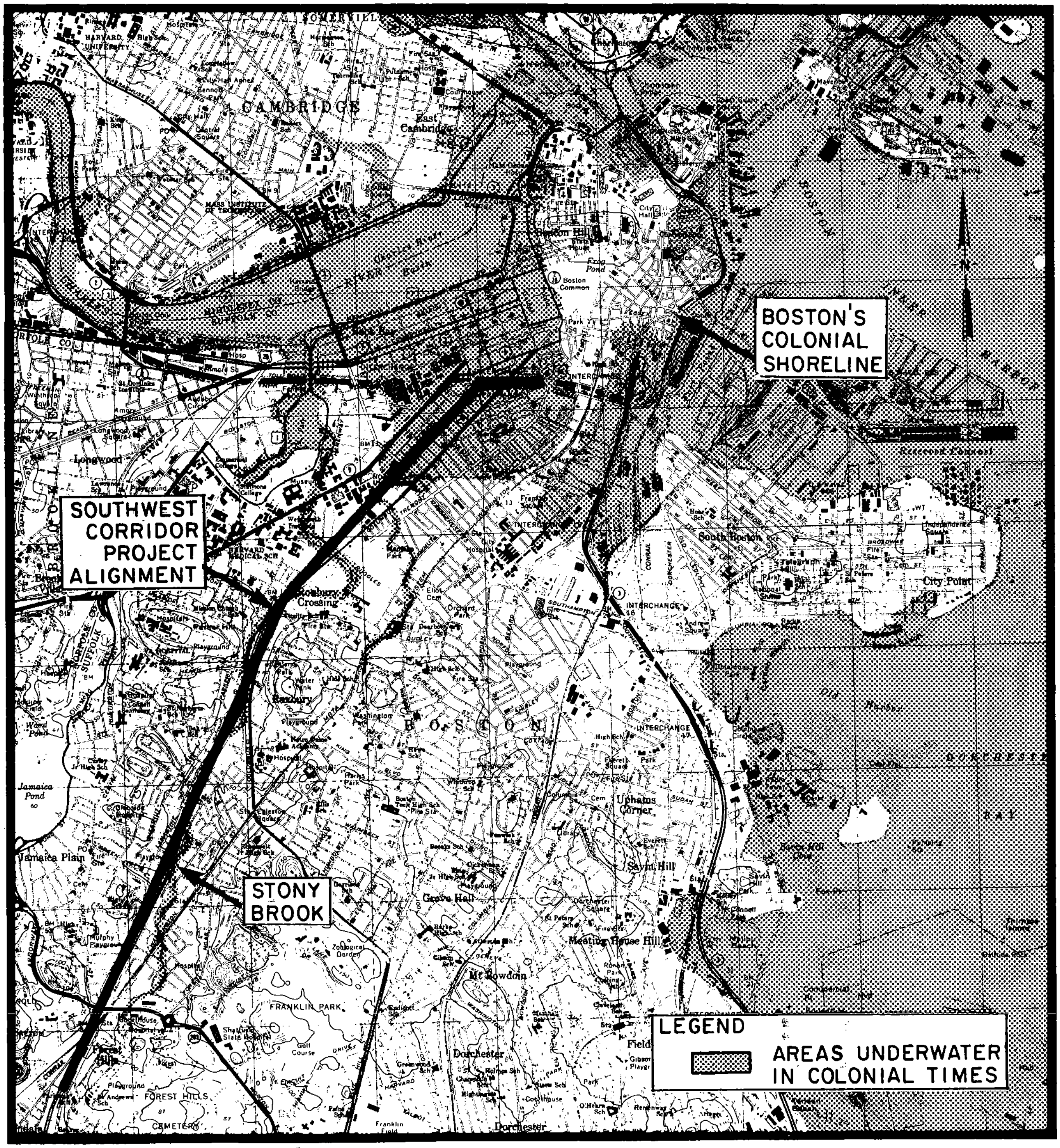


Figure 2 Southwest Corridor Project Location in Relation to the Shorelines and Streams of Colonial Boston

loads caused settlement which is on-going to this day. Buildings along the Back Bay portion of the alignment that are founded on the top of the clay stratum have in recent years been observed to be settling at a rate of about 1/16 to 1/8 in./year.

The fresh water and available power potential of the Stony Brook attracted industrial and residential development. The Stony Brook, however, soon became polluted with industrial and residential waste. It was also subject to occasional, severe flooding that occurred due to inadequate hydraulic capacity of the railroad bridges that crossed the meandering Stony Brook in several locations. These problems hastened stream channelization projects which by the late 1880's saw the completion of the Stony Brook Conduit, that extended several miles from an area south of Jamaica Plain to the west side of the Back Bay area. Along the Corridor, it is a horseshoe-shaped brick masonry structure 17 ft. in diameter and 15-1/2 ft. high.

Around 1900, grade separations between the railroad and cross streets were finally accomplished in Roxbury and Jamaica Plain by raising the railroad upon a 10 to 25 ft. high embankment retained in places between massive walls of granite blocks. The Southwest Corridor Project has been removing this century old barrier as the new below-grade structure is constructed.

See references 1, 2 and 3 for further discussions of regional geology and historical development in Boston and along the Southwest Corridor.

SUBSURFACE CONDITIONS

Two distinctly different geologic conditions are present along the Southwest Corridor alignment as shown in the soil stratification illustrated in Figure 3. Within the Back Bay area there are the deep deposits of Boston Blue Clay with overlying organic silts and "recent" granular fill. Typical properties of these soils are presented in Figure 4. The compressibility of the organic silt varies widely, being significantly greater for the higher organic content soils. Organic contents of some peaty samples exceeded 15%. Layers of very peaty silt up to 5 ft. in thickness have been encountered in the tunnel excavation.

The Boston Blue Clay along the alignment is not unlike that found throughout much of the Boston Basin. It is generally a gray silty clay of fairly uniform medium plasticity. It usually has a stiff to hard crust, commonly mottled yellow-gray

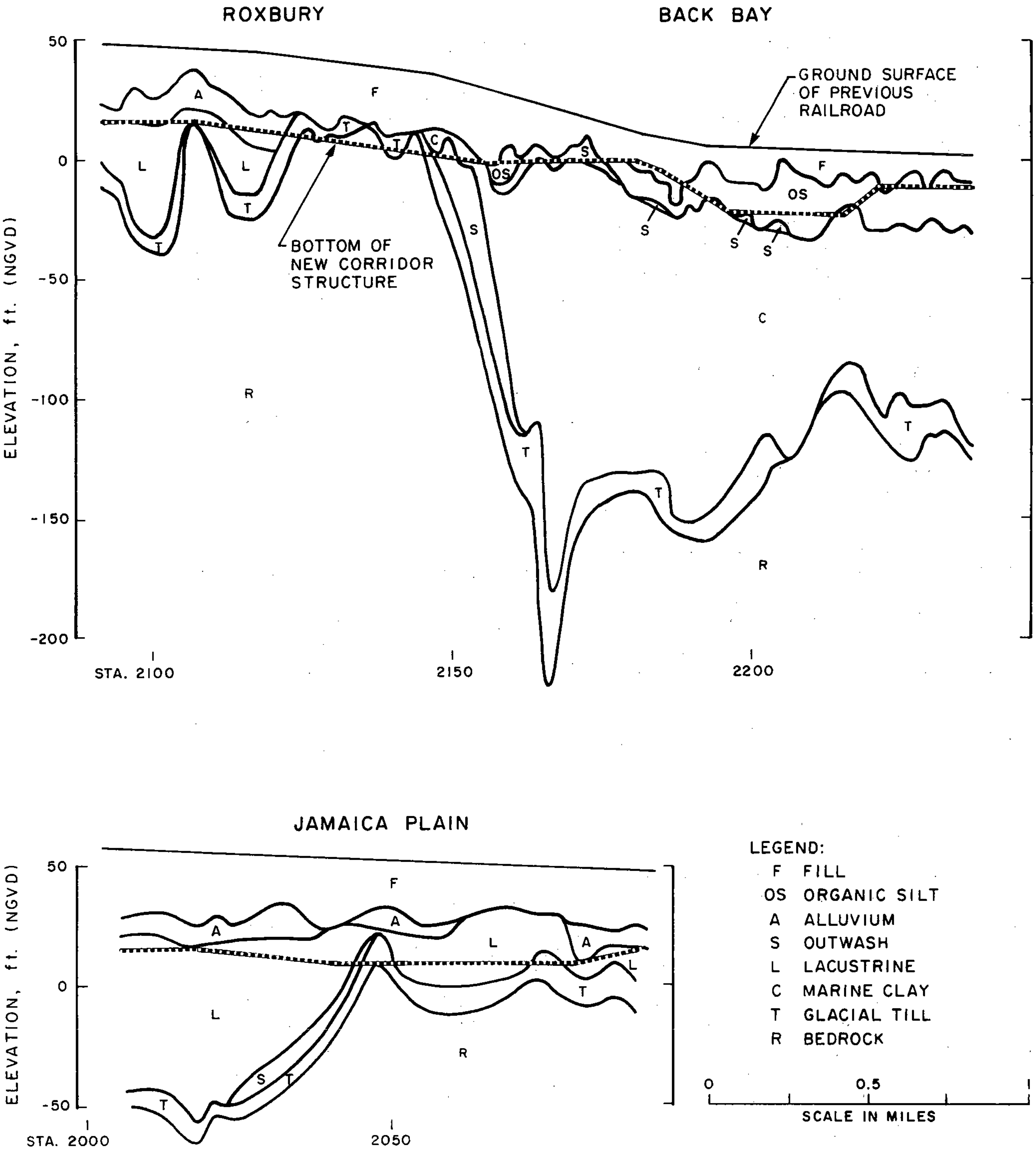


Figure 3 Profile of Subsurface conditions along the Southwest Corridor Project

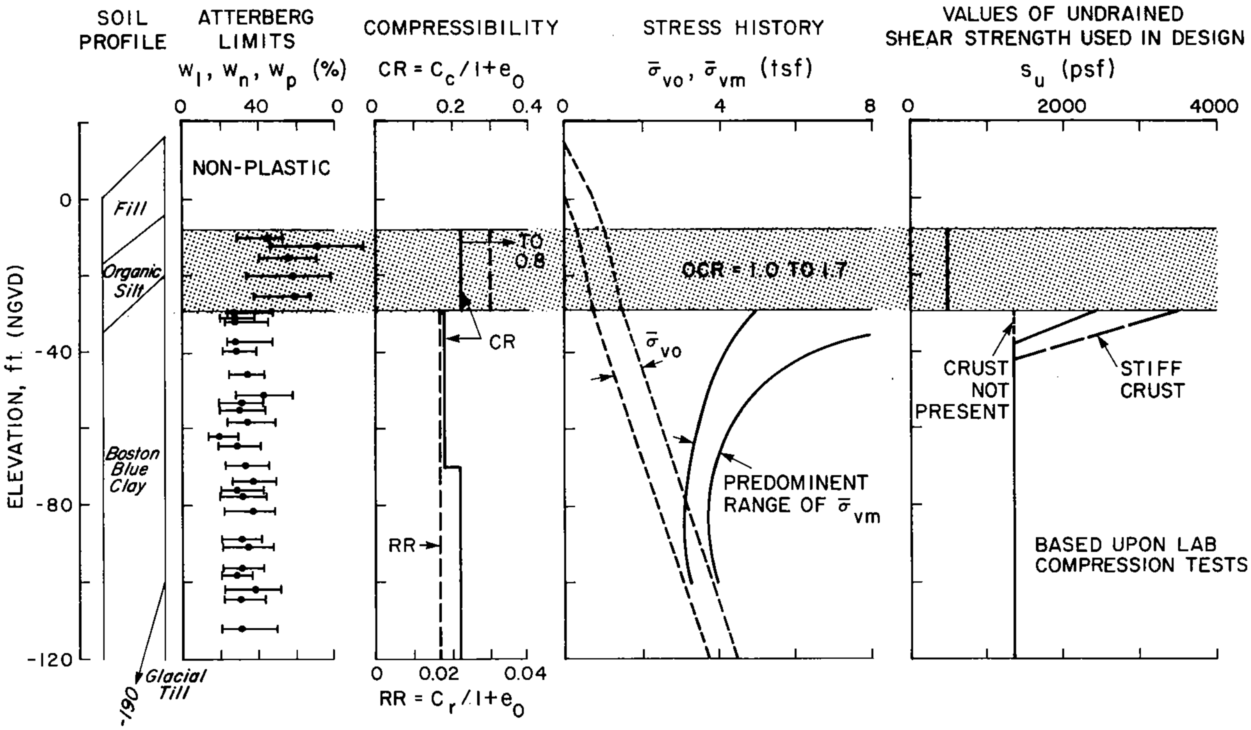


Figure 4 Properties of Boston Basin Soils along the Southwest Corridor Project

in color. The strength of the crust decreases with depth as shown in Figure 4. In some areas a crust is not apparent, possibly having been removed by erosion when the clay was exposed by the lower sea level, long ago. Over-consolidation of the Boston Blue Clay is most pronounced in its upper 50 ft., although it is considered to be slightly precompressed throughout.

In the upland area, the soils are predominantly granular. Thin clayey layers were encountered only occasionally in the lacustrine soils. The indicated top of the fill stratum was the top of the former railroad embankment; it had been 10 to 20 ft. above surrounding grade. The natural granular soils are medium compact to dense and of moderate to high permeability. The glacial till of the upland area is coarser than that found in the Boston Basin, perhaps reflecting the parent bedrock from which the till was partially derived.

GEOTECHNICAL CHALLENGES TO DESIGN AND CONSTRUCTION

Problem 1: Lateral Support for Cut-and-Cover Tunnel: The urban environment of the Back Bay area and right-of-way limitations dictated that the new corridor structure be below ground. This is being accomplished using a 3,000 ft. long cut-and-cover tunnel. The configuration of the new tunnel in relation to the previous railbed and the proximity of adjacent buildings is illustrated in Figure 5. Throughout this section, clearance between adjacent structures and the tunnel walls varied from 2 to 6 ft., but averaged about 4 ft.

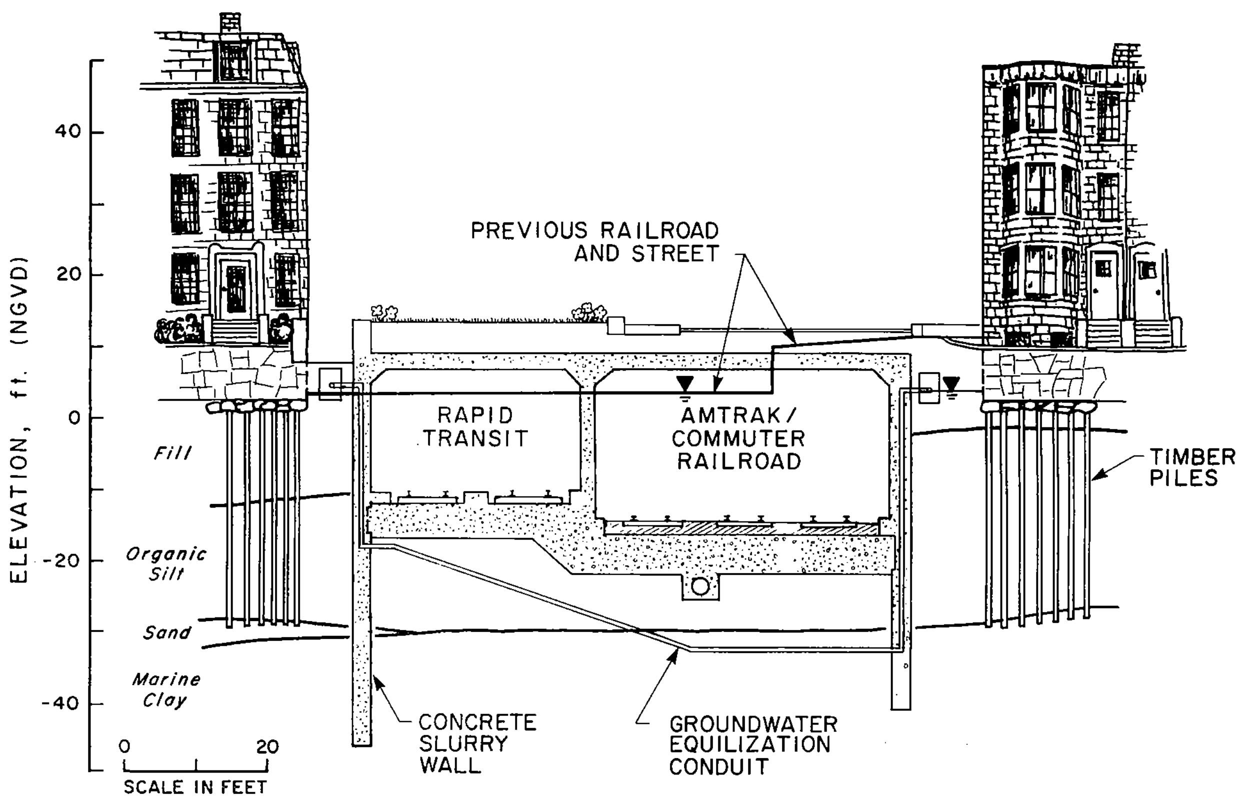


Figure 5 Typical Section of the Tunnel Structure through Back Bay

About 2,100 ft. of the tunnel is supported by a concrete invert slab bearing on compacted granular fill which was used to replace unsuitable organic silt. Excavations 24 to 38 ft. deep have been required for the removal of the organic silt. Three foot thick, reinforced concrete slurry walls were required for lateral support of the sides of the excavation in this portion of the tunnel, rather than steel sheet piling, because; (1) they are more rigid and better able to restrain the adjacent soft soils thereby limiting adjacent ground and building movements and (2) concrete slurry walls are are essentially watertight, thus preventing noticeable groundwater drawdown, an important feature in the Back Bay area where most structures are supported on untreated timber piles.

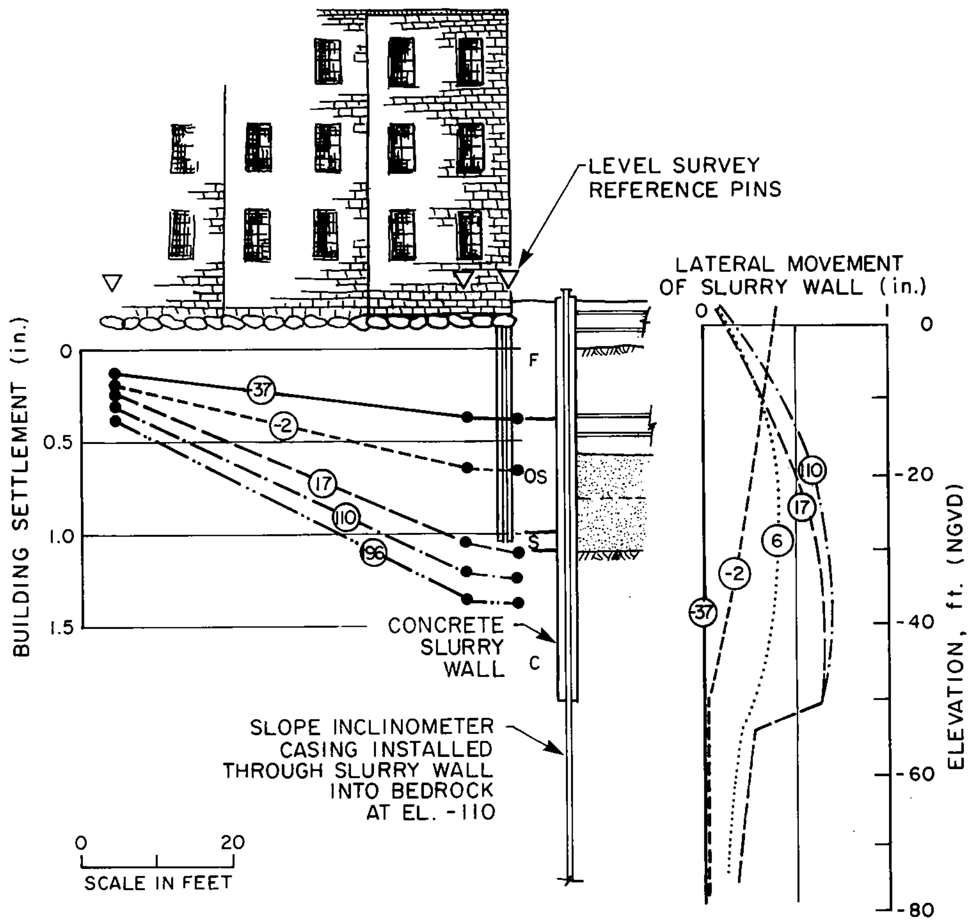
The slurry walls are being used as permanent outside walls for the tunnel structure. Structural connection between the walls and the invert slab has been provided by keyways formed into each wall by styrofoam blockout panels. The blockout panels also covered reinforcing bars that were later bent down into position to structurally connect the wall and invert slab. The completed structure is supported primarily by the invert slab.

The concrete slurry walls were typically braced at three levels: by steel wide flange beam struts at about 4 and 17 ft. below ground, and at the bottom of excavation by the crust of the Boston Blue Clay. Later the compacted granular fill used to replace organic silt also provided support to the bottom of the wall. An 8 ft. penetration into the stiff crust of the clay was required to provide passive resistance for lateral support of the wall bottom. In areas where the stiff clay crust was absent and vertical wall loads could not be supported by end bearing, wall penetrations of about 15 ft. were necessary to develop adequate adhesion support.

Movements of the slurry walls, adjacent ground, and buildings have been monitored throughout construction. Slope inclinometers were installed through sleeves in the slurry walls for monitoring lateral wall movements. In 3 locations they were installed in soil between the walls and adjacent structures. To provide fixed reference positions, all inclinometers casings were anchored 5 ft. into the underlying glacial till. Some of the inclinometers were over 160 ft. long.

Periodic level surveys of reference pins installed on structures adjacent to the excavation and others further away have been made using a Lietz micrometer level and Invar rod. Resulting measurements are accurate to $\pm$ 0.06 in. All level references have been to deep benchmarks installed into bedrock.

A sequence of measured lateral slurry wall movement and settlement of adjacent buildings at various times during excavation and invert slab construction is presented in Figure 6. The magnitudes and pattern of the indicated movements are typical of those which occurred along much of the slurry wall supported tunnel section.



<u>LEGEND:</u>

6 Indicates number of days since main excavation passed by the slope inclinometer. Negative number indicates days prior to excavation.

<u>CHRONOLOGY OF EXCAVATION/CONSTRUCTION</u>

- 37 Days : Main excavation over 150 ft. away.

- 2 Days : Main excavation 20 ft. away, top strut installed after partial excavation to El. -2.

6 Days : Excavation to El. -17; bottom strut installed.

17 Days : Excavation had been to top of marine clay, El. -32. Compacted granular fill placed to El. -24.

110 Days : Compacted granular fill placed to El. -17. Bottom strut removed. Concrete invert slab placed.

196 Days : Construction status same as +110 days.

Figure 6 Typical Movements of Slurry Wall and Adjacent Buildings During Tunnel Excavation ettlement of adjacent structures associated with installation of adjacent slurry wall panels was generally 1/4 to 3/8 in. (indicated in Figure 6 by the settlement line for time -37 days). Inclinometers in soil between the slurry walls and the buildings indicated lateral movement toward the slurry filled trenches in only the organic silt and fill strata. This could indicate that structure settlement at this stage of construction was caused primarily by drag on timber piles.

During slurry trench excavation, the bentonite slurry level was seldom above local groundwater level and therefore could not provide required stabilizing pressure to the sides of the trench. Some sloughing of the sides of the trench occurred in the fill stratum. Several of the resulting concrete protrusions extend over 2 ft. inside the intended face of the wall.

The main excavation was made in one pass with a large backhoe that was equipped with a 1-3/4 cu. yd. bucket. The backhoe operated from a bench slightly below original grade. A Gradall, working beneath the bottom strut on timber mats on the excavation bottom, "cleaned" organic silt from the top of the clay stratum. The overall excavation slope was kept at about 1 vertical to 1-1/2 horizontal.

The data in Figure 6, shows that as the excavation moved by a location, the top of the slurry wall would first move in toward the excavation. But after the top had been restrained by the struts, the bottom of the wall would move inward as passive resistance in the clay was mobilized. Later, when the bottom struts were removed to permit unobstructed construction of the transit side (west) invert slab, the slurry walls were observed to move in another 1/4 in. as additional passive resistance was mobilized to make up part of the removed strut support.

Settlement of adjacent rowhouses lagged behind wall movement, but was generally of comparable magnitude. Settlement is believed due to a combination of factors, including drag on timber piles, settlement of the clay beneath pile tips due to deep slurry wall movements, and on-going settlement of structures supported on the Boston Blue Clay which is typical throughout Back Bay.

Slurry wall movements could have been lessened by reducing the unsupported area of the walls and/or the length of time that the exposed wall areas were not supported. Procedures by which this could have been accomplished include installation of the top struts in advance of the excavation and use of a clamshell to allow more expeditious installation of bottom struts. Excavation by backhoe reduced costs with the penalty of greater movements. It is interesting to note that wall tops generally moved 3/8 in. or more before the excavation had advanced to a given section. This is believed due to the longitudinal rigidity of the concrete slurry walls which caused a squeezing in on unexcavated soils.

Problem 2: Avoiding Cut-off of Groundwater Flow: In two areas along the Southwest Corridor, the new, below grade structure forms a barrier to cross-corridor groundwater flow through existing granular strata. In the Back Bay area, it is essential that the untreated timber piles, upon which most of the rowhouses are founded, be kept saturated to prevent rot and consequent structure damage. In the 1930's a portion of the nearby Boston Public Library experienced significant settlement due to deterioration of many of its timber piles when the local groundwater level was unknowingly drawn down by a nearby leaking sewer.

In the Back Bay section, the concrete slurry walls of the tunnel section and the cast-in-place walls of the adjacent approach sections penetrate into the clay or organic silt strata and would cut off cross-corridor flow. In some areas, a preconstruction gradient of 2 ft. in 100 ft has been inferred from contour plans of observed groundwater levels.

To allow groundwater movement across the corridor, a groundwater equalization underdrain system is being installed. A typical installation is illustrated in Figure 6. In the case of the slurry walls, 8 in. diameter galvanized steel pipes were cast into the walls. Later, during excavation and backfilling, the two wall pipes were connected. Outside the slurry walls, perforated longitudinal header pipes surrounded in crushed stone wrapped in filter fabric were installed.

In an area in Roxbury where the structure is founded on bedrock, a similar groundwater equalization system was installed. Here the problem was not possible rotting of timber piles, but the need to prevent excessive unbalanced hydrostatic pressures from developing on the upgradient side of the corridor structure.

Problem 3: Hydrostatic Uplift Resistance: Eliminating the barrier formed by the former railroad embankment required the new corridor structure to be 15 to 25 ft. below grade. In many areas groundwater levels are near ground surface and uplift pressures are large. Buoyancy has generally been resisted by only the mass weight of the corridor structure. In many areas, this has meant thickening the concrete invert slab by as much as 3 ft. over that needed for structural reasons.

In a half mile long section of the corridor structure in Roxbury, where the invert slab is founded on bedrock, nearly 700 permanent, prestressed high capacity tiedown rock anchors have been used instead of mass concrete to reduce structure weight and cost. Figure 7 illustrates typical use of rock anchors, the resulting reduction in slab thickness, and some of the details of a typical anchor. Anchors used were 1-3/8 inch diameter threaded bars made of high strength steel (f_y = 150 ksi). Required minimum design capacity load in each anchor was 125 kips.

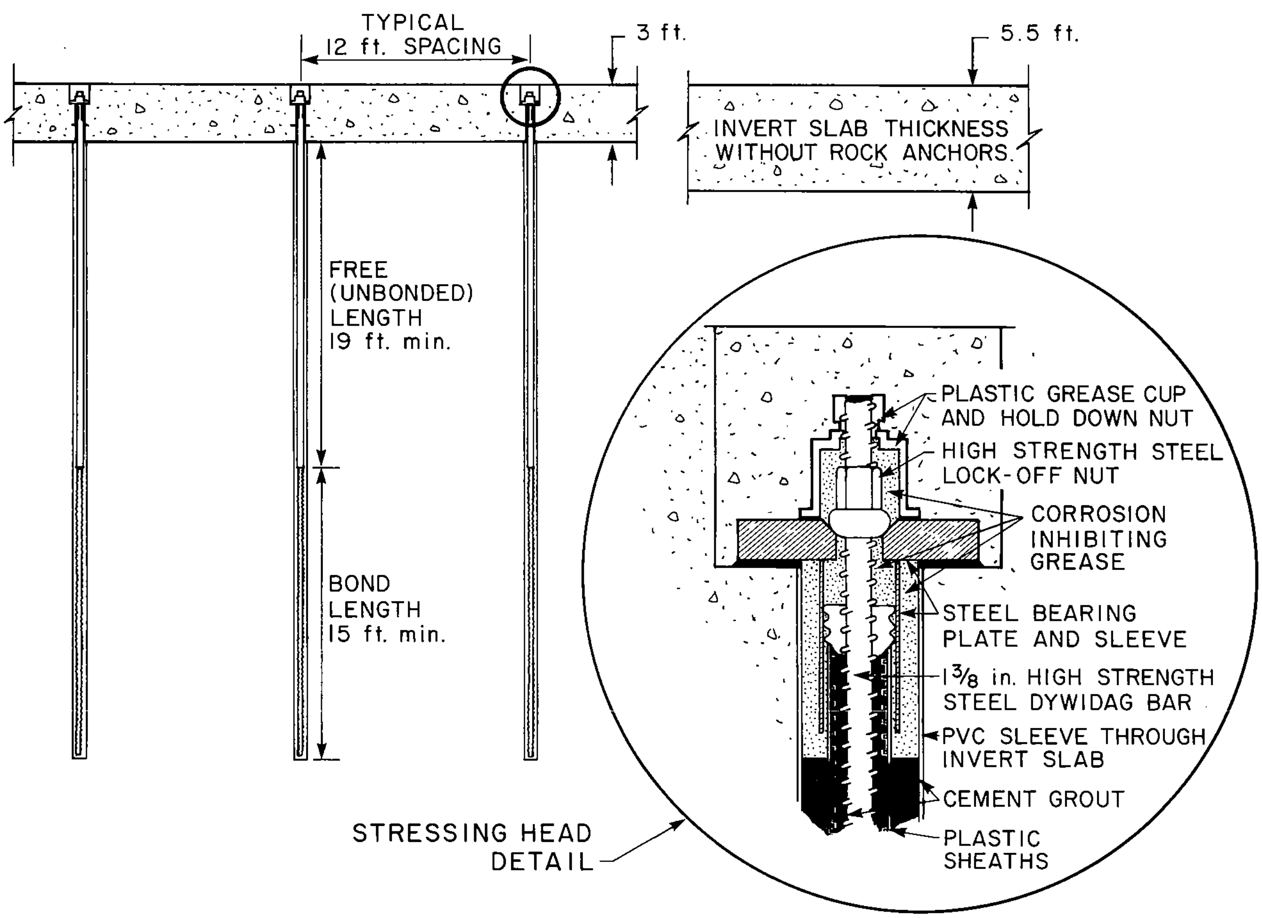


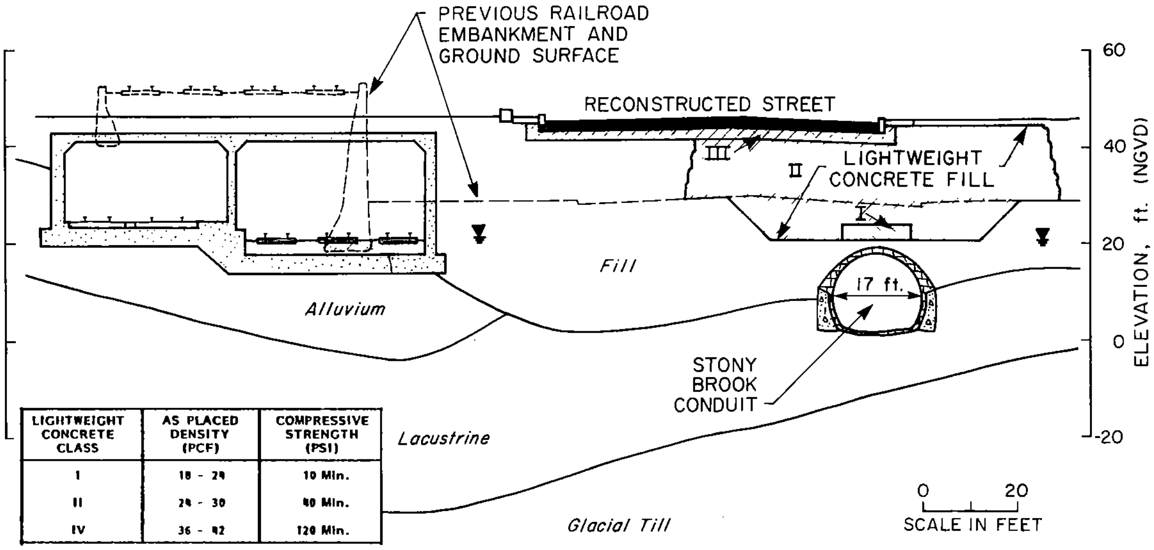
Figure 7 Typical Tiedown Rock Anchor Installation and Stressing Head Detail

To prevent possible corrosion due to stray electric current effects from the electrified transit system, a system which physically isolates the anchors was installed. A double corrosion protection system of corrugated PVC grouted onto the steel bar was required over the entire length of the bars below the stressing head assembly. A special isolation system, shown in detail in Figure 7, was designed for the stressing head. The complexity of the corrosion protection system has made installation difficult.

Problem 4: Stony Brook Conduit: The large, century-old Stony Brook Conduit is parallel to the corridor structure through much of Roxbury and Jamaica Plain. In most areas, the conduit is 17 ft. wide by 15-1/2 ft. high and is of brick masonry construction (3 to 5 layers of brick). In three areas where the conduit crosses the alignment, profile conflicts have necessitated replacement of horseshoe shaped conduit with wide, low concrete box culverts. On several occasions, the construction site at one reconstruction area was flooded out as run-off from heavy storms caused the Stony Brook to overflow the banks of the temporary open diversion channel.

In another area bridge grade requirements at street crossings over the new Southwest Corridor structure necessitated raising the grade of the parallel major arterial street from 10 to 25 ft. over the Stony Brook Conduit. Fortunately, the conduit was founded on glacial till or bedrock for much of the grade raise area and can withstand the increased loads. However, along a 700 ft. long section, the Stony Brook Conduit was founded on lacustrine fine sands and silts. Analyses indicated that the lacustrine soils were not stiff enough to take the increased thrust from the conduit's arch roof that would result from the additional 20 ft. of fill required for the grade raise.

To minimize additional load on the marginally stable portion of the conduit, lightweight concrete fill was used. The cross-section in Figure 8 shows the location of the "balanced" fill over the conduit and the various classes (or densities) of lightweight concrete used. This geometry provided no net change in calculated overburden load on the conduit.



LIGHTWEIGHT CONCRETE CLASS	AS PLACED DENSITY (PCF)	COMPRESSIVE STRENGTH (PSI)
I	18 - 24	10 Min.
II	24 - 30	40 Min.
IV	36 - 42	120 Min.

Figure 8 Use of Lightweight Concrete Fill to Minimize Load on the Stony Brook Conduit

The lightweight concrete is essentially a cement paste with a high entrained air content. Air entrainment is achieved by a proprietary product manufactured and supplied by Elastizell Systems. The unit weight of the lightweight fill used varied from 18 to 42 lb/cu. ft. Its strength varies with density and ranged from 10 psi to over 120 psi. A 3 ft. thick cap of low density/low strength fill concrete was placed above the conduit crown to act as a cushion and prevent a "hard spot" from occurring over the crown. Instrumentation, in the form of earth pressure cells and strain gages, installed above and on the conduit crown, verified that the vertical pressures were generally as assumed in design and that the strains in the conduit crown were within acceptable limits.

Problem 5: Underpinning an 8-Story Building: In the Back Bay Station area there will be a major AMTRAK station , along with the rapid transit and commuter rail stations. The corridor structure will be significantly wider than the previous railroad right-of-way to provide platforms for all five tracks. To accommodate the new station, an adjacent street and a portion of the basement and first floor of a neighboring eight-story building are being taken, as indicated in Figure 9. Portions of five exterior building columns are in the track area and must be removed. Three nearby interior columns will pass through one of the platforms and do not have to be removed.

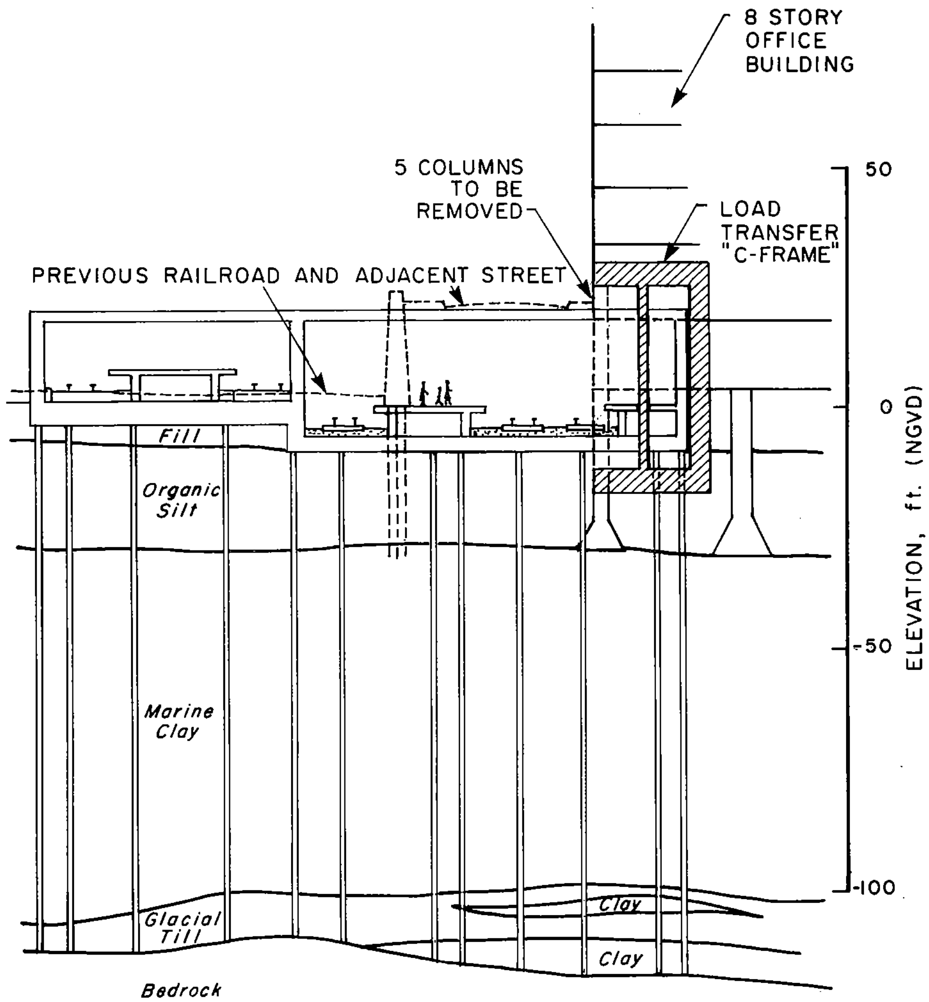


Figure 9 Cross-Section of the Corridor Structure Showing Required Building Underpinning in Back Bay Station Area

Different foundation types between the building, supported on belled caissons founded on the crust of the clay stratum, and the corridor structure, which will be supported by deep end-bearing piles, complicate an otherwise straightforward underpinning problem. The underpinning scheme has to allow for gradual, continuing, long-term building settlement, due to secondary compression within the deep underlying clay.

To avoid installing new foundation units for the building, large load transfer units that resemble giant C-frames are to be installed. These will carry building column loads down around the corridor structure, to existing caissons, as shown in Figure 9. Each C-frame has a compression and a tension member to provide the moment capacity required in the cantilevered frame.

Unforeseen field conditgions have further complicated the underpinning work. The tops of several caissons were found to be offset from the columns, one by as much as 1.4 ft. Several caisson shafts were inclined, one by about 10 degrees from vertical. These deviations from conditions assumed in design have required modifications to the intended underpinning scheme.

The load-transfer units are to be installed such that the caissons do not experience significant changes in load. Each frame will be preloaded as the load is transferred from the columns. Large decreases or increases in caisson/column loads would be expected to cause heave or settlement, and could cause structural distress. Changes in column load are being monitored with vibrating wire strain gages mounted on each of the 5 columns that will be underpinned and on 12 adjacent columns. Level surveys are also being performed on these 17 columns to measure structure vertical movement. The surveys are done in the building's second floor, above the underpinning work. Carrying the level reference into the second floor, from a fixed, deep benchmark across the street, has been a difficult and time consuming procedure in itself.

Precast concrete piles are generally being used for corridor structure support in this area, but due to the limited headroom, segmentally installed pipe piles will be used under the overhanging building. The pipe piles are 16 in O.D. by 0.375 in. wall thickness and have a design load of 100 tons. They will be installed open-ended and cleaned out before each successive increment is added because of concerns for possible building settlement or heave if the underlying clay is displaced. Each pipe pile will finally be filled with concrete after final pile driving penetration resistance is attained in glacial till or bedrock and all soil within the pipe is removed.

CLOSING REMARKS

The Southwest Corridor Project's urban setting and the widely varying geologic conditions have combined to produce geotechnical challenges which have been met with innovative and cost-effective solutions. This paper has presented five of the more interesting geotechnical problems encountered in the Southwest Corridor Project.

Sincere thanks are expressed to the Massachusetts Bay Transportation Authority for permission to present the Southwest Corridor Project to the geotechnical community in general and to the 34th Annual Highway Symposium in particular.

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Preloading Peat for Foundation Use

by

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for

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Areas of peat and organic soil are commonly avoided as sites for engineered construction. This becomes progressively more difficult to accomplish in urban areas, as adjacent lands are developed and property values appreciate. If the organic accumulation is relatively shallow, excavation and replacement is feasible. However, for deeper deposits other alternatives need be considered, including the preloading technique discussed here.

Preloading both strengthens the peat, so that it can safely carry the intended structural load, and achieves long term compression of said peat within a construction (or short preconstruction) period. Prediction of the settlement of the peat under both the service load and the preload is important. This paper proposes methods for doing this, involving both laboratory testing and field measurements, with a highway embankment as the engineered structure. Case studies are presented.

Introduction.

Building highways over peat and other highly organic deposits has been avoided by engineers whenever possible. It has been customary to go around peatlands when planning a highway, and this is still the preferred solution. However, as land becomes more scarce and areas become populated, the choice of highway routes becomes more restricted. This is especially true in large cities. As the city grows, property that was once considered unsuitable is again being considered for development. In populated or industrial areas the cost of right-of-way for roads around the peat deposit may be very high, whereas the price of property in the peat deposit often will be low. Due to these and other reasons, passing the highway alignment over the deposit may be the most desirable alternative.

When these deposits are relatively shallow (less than 5 meters), excavation and replacement by granular materials is commonly employed. However, when the deposits are deeper or of a large lateral extent, special foundation treatment is usually required.

Preloading.

One such treatment is preloading. As a result of expansion into areas with poor foundation soils, preloading techniques through surcharging have been developed with some success as a means of in-situ improvement of the soil properties. Preloading strengthens the deposit so that an embankment can be supported without failure or excessive settlement.

A major drawback to preloading peat has been the inability to predict the deformation characteristics of the peat under loading. This lack of knowledge is apparent in the determination of surcharge magnitude and duration required to accelerate settlements. The time rate and magnitude of settlements to be expected with peat are at best uncertain. Methods currently used give poor results when applied to large strain materials with significant secondary compression effects, i.e., peats. Thus, after a preload has been applied, it is often uncertain how much settlement will occur and consequently, the required duration of the surcharge period is unknown.

This paper presents a technique to accurately control the duration of the preloading period, so that construction may be completed in the minimum amount of time.

The Gibson and Lo Model.

Gibson and Lo (1961) proposed a rheological model which applies to large strain soils which exhibit secondary compression. This theory assumes the structural viscosity of the soil to be linear. For large values of time, the deformation behavior, $\varepsilon(t)$ may be written as

$$\varepsilon(t) = \Delta\sigma \left[a+b(1-e^{-(\frac{\lambda}{b})t}) \right] \qquad , t > t_a \qquad (1)$$

Where a, b, and λ are empirical parameters which can be determined from deformation-time data; $\Delta\sigma$ is the increase in vertical stress; and t_a is the time after which the stress has become fully effective. This model has been shown to closely model both laboratory and field behavior of peat (Edil and Dhowian, 1979, Gruen and Lovell, 1983).

Dhowian (1978) derived the following method for determining the rheological parameters a, b and λ. If equation (1) is differentiated with respect to time, the rate of strain obtained is:

$$\frac{\partial \varepsilon(t)}{\partial t} = \Delta\sigma e^{-(\lambda/b)t} \qquad (2)$$

Taking the logarithm of both sides in equation (2), the following linear relation is obtained:

$$\log_{10} \frac{\partial \varepsilon(t)}{\partial t} = \log_{10} \Delta\sigma\lambda - 0.434 \frac{\lambda}{b} t \qquad (3)$$

Which in a simplified form is the following straight line:

$$Y = C + D(t) \qquad (4)$$

Where

$Y = \log_{10} \frac{\partial \varepsilon(t)}{\partial t}$ = log of strain rate,

$C = \log_{10} \Delta\sigma\lambda$ = line intercept,

$D = -0.434 \frac{\lambda}{b}$ = slope of the line.

The parameters are determined by plotting the logarithm of strain rate against time from compression results for a particular soil. A straight line is then drawn through these points. The slope (D) and the intercept (C) of this line yields the values of b and λ. The primary compressibility parameter a, is found by substituting the known quantities into equation (5).

$$a = \frac{\varepsilon(t)}{\Delta\sigma} - b + be^{-(\lambda/b)t} \quad (5)$$

Application.

The Gibson and Lo model may be used to extrapolate field settlement curves and predict field settlements under various stress levels. The actual surcharge embankment is constructed in the field and settlement data are recorded. After a short period (normally less than three months), the load has become fully effective and sufficient data are available to determine the rheological parameters. This method has been computerized (Gruen and Lovell, 1983) such that data can be entered as they are collected, refining the rheological parameters to a greater accuracy as settlement progresses. Once these parameters have been determined for a given deposit, the settlement behavior can be extrapolated to any time.

In a similar manner, using equation (1), the stress change term ($\Delta\sigma$), can be chosen to predict the settlement behavior under other loads. Varying the stress change term in equation (1), while using one set of rheological parameters (a, b, and λ) assumes that these parameters are constant with stress level and that strain is a linear function of stress at a given time. This is not the case with peat, however, Gruen and Lovell (1983), have shown that for the stress change levels involved in the preloading of peat, the violation of these assumptions causes small and acceptable errors.

Case study.

Edil (1981) showed that this method is valuable in analyzing the duration of preloading by an actual application. A shopping center was to be built near Madison, Wisconsin, over a peat deposit. Results of the subsurface exploration are shown in Figure 1. The peat was preloaded and settlement data were recorded. After approximately 9 months, the parameters for use in Gibson and Lo's model were determined from the settlement data. The settlement under the preload was extrapolated using the model and compared very well with the actual settlements which subsequently occurred (see Figure 2). The slight discrepancy during the first few months is attributable to the fact that a single stress increase ($\Delta\sigma$) was used in evaluating Equation (1), while in fact the stress change took place in a number of steps as shown in the upper part of Figure 2. The model was applied to estimate the settlement curve for the case if the building was placed without preloading

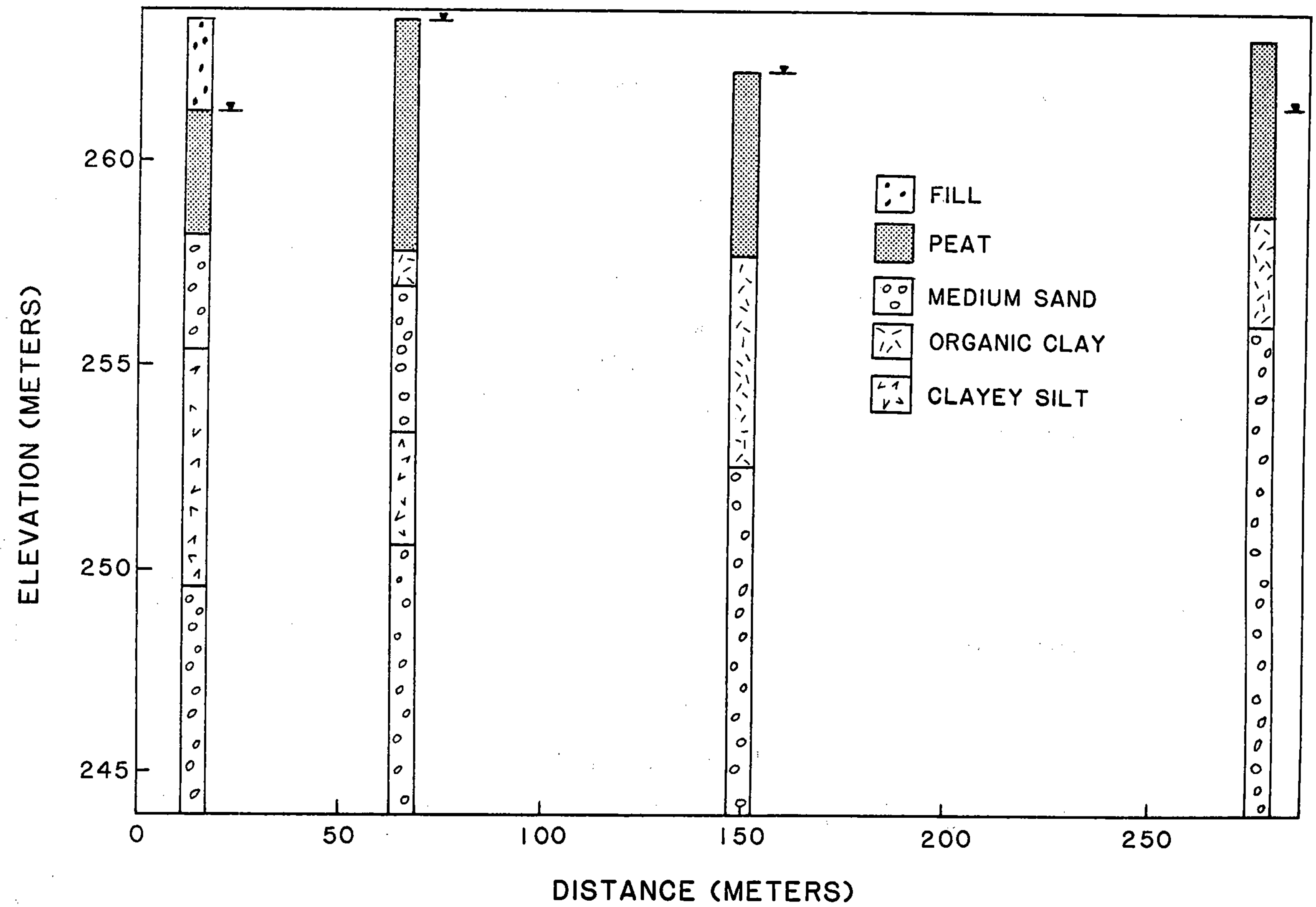

Figure 1. Longitudinal subsurface section. From Edil (1981).

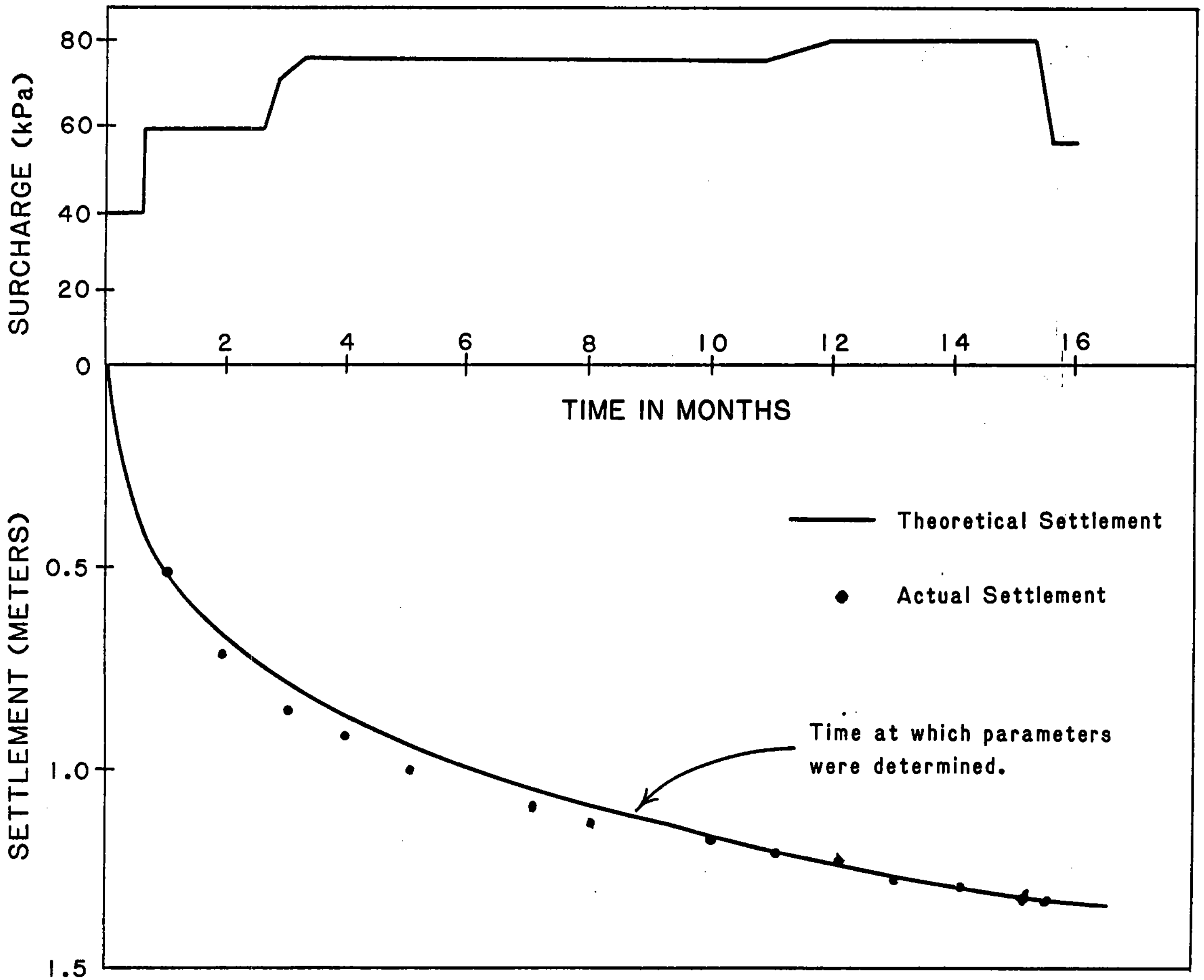

Figure 2. Settlement data under preload. After Edil (1981).

(see Figure 3). In this case, the parameters a, b and λ were assumed to be the same as those calculated from the preload case. The stress change used was that caused by the building only. As shown in Figure 3, the surcharge was intended to eliminate the settlements expected under the load of the structure over its useful life. The theoretical building settlement curve was extrapolated to 30 years and compared with the preload curve. At that time, a marginal situation existed and the design engineer decided to postpone construction until the following spring. Edil (1981) reports that after more than 3 years since construction, there are no known problems associated with settlements of this shopping center.

Conclusion.

When peat is to be used directly as a foundation material, its properties must be improved by preloading. Using preliminary settlement estimates, the magnitude and duration of preloading can be calculated and the surcharge applied. After the primary strain portion under the surcharge load has occurred, Gibson and Lo's theory can be applied to determine the rheological parameters used for the model. According to Landva (1980) the field settlements under embankment loading, have normally entered the secondary strain portion within 3 to 4 months. Knowing these parameters the surcharge settlement curve can be extrapolated, and the settlement curve for the final design load can be estimated. These two curves can be compared so that the duration of preloading is sufficient to accelerate the anticipated settlements under the final design load.

Using Gibson and Lo's theory in this manner will give much more accurate control over preloading than other methods currently used. This could be considered somewhat of an observational method, in that the Gibson and Lo model gets more and more accurate as settlement continues, providing more data for determination of the parameters. The determination of the rheological parameters and settlement predictions have been simplified by use of the computer program given by Gruen and Lovell (1983).

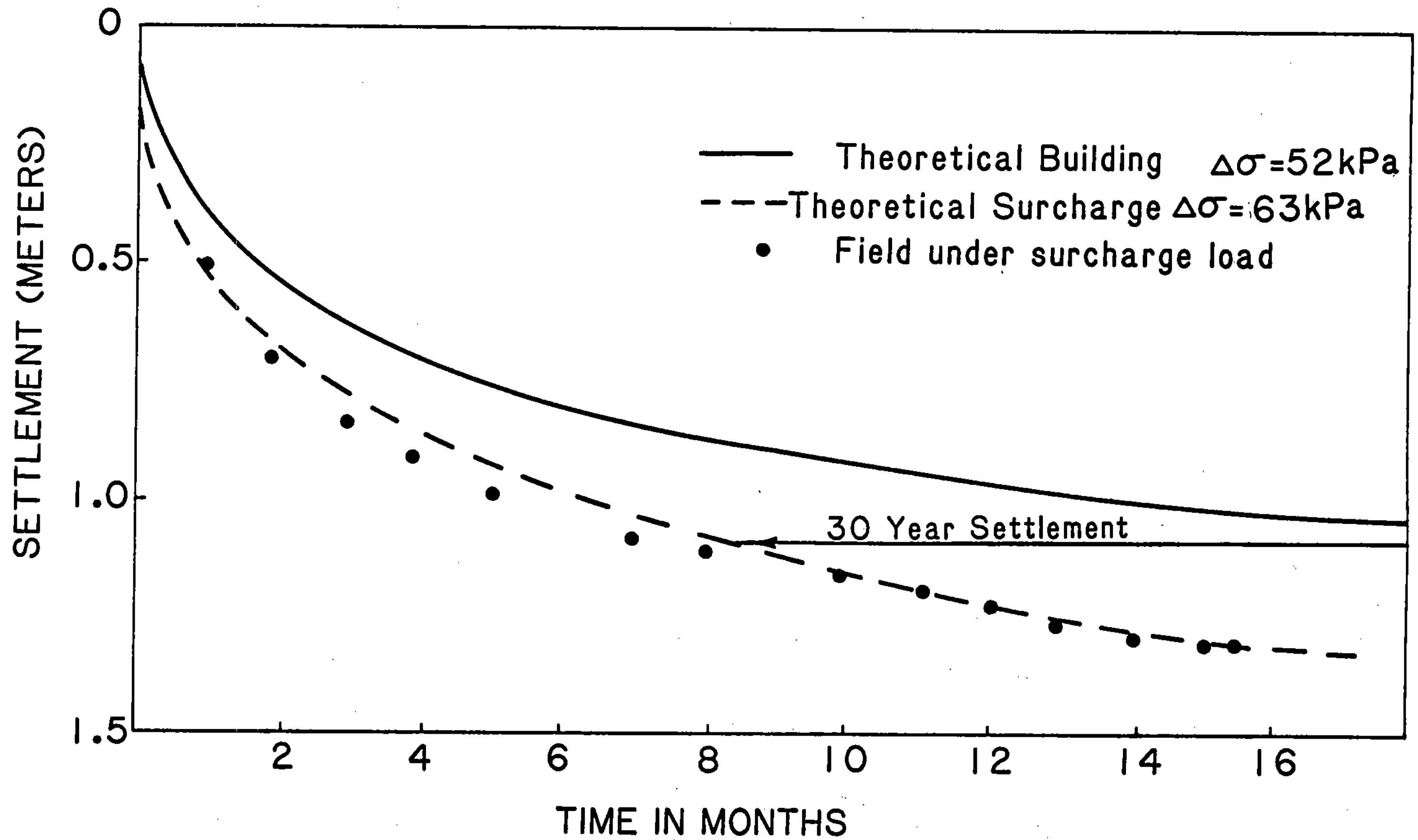

Figure 3. Theoretical predictions and field settlement data. From Edil (1981).

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Measurement of Construction Influences on Adjacent Structures

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ABSTRACT

As construction increases in our urban areas, the engineering geologist is increasingly called upon to evaluate the response of adjacent structures to construction produced affects. The purpose of such study is twofold: both as an attempt to minimize the possibility of damage claims and to reduce, as much as feasible, the undesired environmental effects.

Geologic forces effecting the local environs can be caused by fluctuation of the groundwater table, displacements from construction excavation or vibrations created in the construction process. To examine the conditions, a monitoring system must be designed and implemented to measure rotational, lateral and vertical displacements. Such a program may use land surveying methods, geophysical instrumentation, passive measurement devices such as pins or bi-axial displacement gauges, or more often a combination of these methods.

Certainly, the basis of any measurement program is the evaluation of existing facilities prior to construction. Techniques combining scalar photography, diagrammatic and

narrative notes of inspection tours, and installation of passive recording stations can provide an effective and economical program.

The final analysis will be based on all field data gathered according to a time-based schedule. The reduction of this data will serve to determine critical areas, as well as, to possibly determine damage thresholds. Anomolies in the ambient vibration and noise levels will also be indicative of construction effects.

This discussion will attempt to provide insight into establishing such a system and interpreting the data by conventional methematical methods, as well as, through the use of three dimensional models.

MEASUREMENT OF CONSTRUCTION INFLUENCES ON ADJACENT STRUCTURES

The practice of evaluating construction effects at the boundary of a project by various geotechnical means is an accepted and established procedure. By making physical measurements of environmental effects such as vibration and air overpressure from pile driving and blasting, deformation of adjacent soils from mass excavations, groundwater elevation fluctuations, and resulting forces on associated retaining structures some evaluation and documentation of the stability of adjacent properties are maintained. Although research and experience has allowed good correlation of these seismic, displacement, and strain values with damage thresholds, a higher degree of accuracy can be attained by concurrently evaluating those forces both at the project boundary and on the structure being scrutinized. This information becomes invaluable in the event of litigation.

This presentation then will endeavor to present a suggested procedure for assessing a building's structural integrity and then pursue the various systems available to monitor any structural response. Chronologically, this normally entails the following steps:

- Evaluation of existing and proposed site conditions.
- Inspection and cataloging of existing structural distress.
- An appraisal of the forces generating the observed flaws.
- Prediction of construction influences.
- Selection of appropriate monitoring systems.
- Schedule of data collection and analysis.

Site Evaluation

Review of project design criteria, any special provisions applicable to the geotechnical phasing of the project, along with examination of subsurface boring data and/or geophysical prospecting results will yield a wealth of information concerning anticipated problem areas. A special concern will be information regarding soils to be encountered, groundwater elevations and the presence of rock in areas requiring excavation. In order to gain a perspective of long term effects, the project should be evaluated with respect to construction methodology as well as new environmental effects which the completed project may introduce to the structure(s) being studied.

Pre-Construction Inspection

The basis of any measurement program is evaluation of the existing facilities prior to construction. Not only will this allow the opportunity to document any existing structural or cosmetic damage but upon examination of the data, the overall condition of the building, current stress points and their possible causes will become evident. This will form the basis for establishing a structural monitoring system.

Inspections may be performed using several mediums of documentation. Traditionally, 35 mm or larger formatted photographs, either color, black and white prints, or color transparencies, have been used for notation of exterior flaws. Flash photography was prone to reflection and over exposure on interior surfaces and frequently resulted in over exposure. With the advent of higher speed, more light sensitive films, however, many interior inspection tours can be accomplished on film. When using photographs it is important to include a scale of measure in the photo, either from the natural setting or added, and to note in an accompanying log, the date photographed, orientation of the camera and subject area, as well as film roll and frame number.

These supporting notations as well as documentation of structural flaws where poor lighting, angle, etc. prevent photographing, may be detailed in writing using narrative and diagrammatic notes or by using a tape recorder. In either case, a detailed account of the date, structure, room, orientation and sufficient quantitative information about the size and appearance of the defect to allow reexamination is mandatory. A general description of the age, composition, and orientation of the structure and surrounding topography should also be recorded. Video tape recordings of existing conditions have been tried with mixed results depending on the resolution and light gathering capabilities of the particular system and consistency of the operator. Regardless of the medium used and attention to detail given the survey, its results must be taken as only representative due to human and mechanical limitations.

Natural Environmental Effects - Through an evaluation of the data obtained from the pre-construction inspection, apparent causes of existing defects and localized stress concentrations can be determined. It must first be realized that most structures crack. This is due to a variety of movements produced by both internal and external influences. The primary internal causes of these movements are temperature and moisture. These cyclic motions acting against fixed points and cyclic motions between dissimilar materials are responsible for most of the minor cosmetic damages in structures. Actual cracks and defects noted will depend to a great extent on the size, age, and condition of the building. Some new construction methods and materials exhibit higher degrees of elasticity but at the same time can be subjected to greater stresses through longer unsupported members in the superstructure, integral heating and air conditioning systems, and newer foundation methods. We present below a catalog of various structural components, the most recurring signs of natural distress associated with each, and the most common causes.

Roof Structues - Roofs can transmit temperature and wind induced loads and cause damage in both the ceiling and upper walls of a structure. Truss systems often exhibit substantial changes in length under temperature variations. If the supporting system is not designed to allow for this, i.e. fixed at both ends, severe cracking of the upper walls can occur. In residential structures such action can normally be verified due to the opening of corner joints in the cornice. Rafters can additionally transmit wind and snow loads through the supporting knee walls and braces into the ceiling joists creating cracks in the ceiling just below these braces.

Floor Systems - The differential movements between horizatonal elements of a superstructure (for example, top floor and roof) can create cracks in walls especially where the wall strength is decreased by a window or door opening. Likewise, deflection of the floor system, particularly over a long span, can result in wall cracks. Due to expansion and contraction of the horizontal members (especially wood and steel) due to temperature and humidity.

Doors and Windows - Shrinkage and expansion of lintels will create cracks above openings which will have a tendency to open and close seasonally. If lintels are of insufficient strength to support the material bearing on them, lintel cracks can also develop. Related to these openings is an occurence observed in structures with brick veneer. As the wood frame moves with temperature and moisture changes, window and door casings will be pulled and pushed, often tearing the original caulking.

Interior Surfaces - Older, plaster finshies tend to display "crazing", that is, an irregular pattern of cracking. This is caused by internal differences in tensile stresses of the material. More recent buildings will generally be constructed using drywall. Due to physical properties inherent to the paper and gypsum construction of drywall and the installation procedure, these materials are more apt to display "creep". This is the time dependent deformation that a material undergoes following an initial elastic deformation. This process is also related to movement due to moisture exchange in materials and is also displayed in concrete and block construction.

Foundations - Together with temperature related movements previously discussed, the differential motions in foundations are the most serious threats to a structure's integrity. Unless founded on unweathered rock, some settlement of a structure's foundation is probable. This can occur as a result of soil deformation, compaction, (particularly on previously filled areas) and consolidation. The real problem again is differential motion and is especially evident where a foundation is cut into a hillside and where a drastic difference in structural bearing and temperature/moisture differences occur from one end to the other. Further problems are displayed in shallow footings where freezing can occur under the footing lifting the structure or by the other extreme where heat evaporates moisture from the soil by capillary attraction. Here, a tilting of the foundation occurs creating horizontal cracks above the foundation. Deep foundation walls,can be severly effected by lateral earth and soil pressures (including groundwater) bearing on the wall. Should the floor system at ground level provide sufficient resistance to movement, an outward motion may occur above the ground level with some probability for horizontal cracks occuring both in the foundation and first floor wall.

Construction Influences

With the effects of various physical aspects identified from the field inspection, some prediction can be made as to the consequences of proposed construction methods. Generally, where mass excavation is required within close proximity of adjacent structures. The soils and rock encountered will determine the construction methods. Soldier pile and lagging systems, slurry walls, and interlocking steel sheet piles are some of the alternatives. By whichever method, excavation will result in changes in stress in the soil mass which can in turn create lateral and vertical ground movements. The amount of movement will depend on local geology, excavation procedure, timing of support installation, and the type of supports used. Shallow excavation of any residual material will result in a horizontal deflection in the bracing system of a cantilever nature accompanied by some degree of settling. As bracing or tiebacks are installed and excavation goes deeper, further settlement may develop and be accompanied by bulging of the lower wall. In deep excavations, some basal heave of the bearing soils may occur. The extent of these deformations can be controlled by spacing of support elements.

Environmental influences also need to be addressed. These will include any effects of watertable displacement as well as vibration intensities to be anticipated from pile driving, heavy equipment operation, or blasting operations. The proper selection of vibration parameters to be used will require not only observation of vibration intensity levels for structural safety but an assessment of anticipated frequency ranges and the possible effects on any vibration sensitive equipment and the building inhabitants.

Monitoring Systems

Fatigue and damage in a structure may first be noted by widening or extension of existing cracks. A monitoring program should be designed to provide high resolution data in those areas discovered during the preliminary inspection as exhibiting stress cracks or where stresses might be anticipated. Observation of these locations will enable early detection of structural responses or provide a more complete overview of the reaction of the entire structure.

Exterior settlement and lateral motion detection in the structure may be accomplished by first order surveying methods. Measurements from an established benchmark to a point on the building will provide good vertical control. Maximum horizontal control will be gained using an electronic distance measuring (EDM) device. Benchmark locations should be sufficiently distant from the site to assure isolation from construction effects. It is important to evaluate various points on the adjacent structure so as to allow checking for any differential movement. These motions can also be visualized through the use of inclinometers, tiltmeters, extensometers and other physical measurements.

A crack measurement program should be established prior to any excavation. Several methods are available for allowing periodic interrogation of the areas. The applicability of the chosen components will vary depending on the material, accessability, and anticipated reaction of the building members.

Since cracks will open or extend as an initial reaction to dynamic loading, simply marking the extent of the crack and with a pen and color coding (or noting) dates of future observed advances on the member, will allow visual tracking. Scalar photography can also be used for maintaining a record of such movement. Widths of crack development can be analyzed to a higher tolerance by installation of pins, or surface mounted reference points attached with epoxy or other cement on either side of the crack. Measurements can then be accomplished using a caliper for measurements to 0.0001". Such attachments should be of brass or other material of sufficient properties to resist wear and temperature changes. Should pins be desired, care must be taken to avoid penetrating the surface under investigation (in the case of a wall) and anchoring the device into a dissimilar material. Simple visual gages capable of providing two-directional monitoring of cracks have recently come available. Made of Liexan plastic the two member device attaches to the surface with cement or screws and provides hairline and graph means of plotting crack progress.

studies requiring the highest resolution, a variety of strain gage configurations including electrical resistance and vibrating wire types are available. It is a good idea to correlate the date and time of measurements with local temperature and humidity measurements.

Plumbness of walls may be checked with plumb and line systems or perhaps more conveniently through the use of a clinometer such as those integral to most field compasses.

Other measurements such as wall displacement between wall and ceiling, wall and floor, and corner wall junctions, can be evaluated using variations of these methods, customized to individual project needs. The basic causes and guidelines for establishing such a program will remain the same.

Analysis of Data

A project schedule for taking readings at these stations will depend on construction sequencing but should provide a time referenced base for proper analysis of data. Interpretation can be plotted onto an architectural rendering, plans, cross sections, or vector motion graphs and compared to other available geotechnical and geophysical instrumentation outputs in order to substantiate observed data. Other measurement systems may be implemented as necessary to monitor short period disturbances such as blasting. Based on an accurate monitoring program of structural effects from construction activities and comparison with anticipated environmental effects proper geotechnical procedures can be scheduled thereby facilitating construction and protecting all parties from unwarranted damage litigation.

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Plumbness of walls may be checked with plumb and line systems or perhaps more conveniently through the use of a clinometer such as those integral to most field compasses.

Other measurements systems may be implemented as necessary to monitor short period disturbances such as blast vibrations.

Analysis and Conclusion

A project schedule for taking readings at these stations will depend on construction sequencing but should provide a time referenced base for proper analysis of data. Good correlation between natural and externally induced construction effects can be developed by allowing the observation period to extend prior to and following the project sequence. Interpretation can be plotted onto an architectural rendering, plans, cross sec tions, or vector motion graphs and compared to other available geotechnical and geophysical instrumentation outputs in order to substantiate observed data. Based on an accurate monitoring program of structural effects from construction activities and comparison with anticipated environmental effects proper geotechnical procedures can be scheduled thereby facilitating construction and protecting all parties from unwarranted damage litigation.

STRENGTHS AND WEAKNESSES OF SLOPE STABIILITY ANALYSES

by

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ABSTRACT

Currently, there are numerous computer programs for the analysis of slopes. These programs consider limiting equilibrium for the computation of the overall factor of safety, with assumptions for the interslice forces. However, these factors of safety are dependent on the method of analysis. A comparison of these factors using different analyses is presented to illustrate their variations. It has been shown that the simplified Bishop and Janbu analyses for circular and non-circular surfaces, respectively, give consistent results. A computer program for the analysis of slopes, called STABL, has been developed at Purdue University during the past seven years and uses the Bishop and Janbu methods of analysis.

The program can handle slope profiles having multiple ground surfaces and up to ten piezometric surfaces. The trial surfaces are generated in a random manner and may be either block, circular or irregular in shape or alternatively, the user may specify the trial surfaces. The output is presented in a graphical manner to allow the designer to determine if adequate space has been explored for the most probable minimum factor of safety. Examples showing the flexibility of this program are also presented.

INTRODUCTION

The general objective of an assessment of slope stability requires that an adequate factor of safety be maintained against the large deformations associated with a shear failure. In order to achieve this objective, today's designer has at his or her disposal over 20 different Limit Equilibrium Methods of analysis and over 100 computer programs which will calculate a factor of safety. This factor of safety relates to a selected, potential failure surface and is commonly defined as the ratio of available strength to the magnitude of strength required to maintain equilibrium. Currently, all these analyses are for two dimensional sections but considerable progress is being made towards formulating the more realistic three dimensional problem.

However, since assumptions have to be made in computing such a factor of safety, a unique value cannot be calculated from a closed form solution. Thus, numerous potential failure planes, with possible shapes ranging from circular to block, have to be examined to determine the "most" probable minimum factor of safety. Hundreds of such surfaces can be examined using available programs and today's efficient computers. However, this analysis is only one of the "links" of a chain of the engineering design process (Harr, 1977). The soil sampling, testing and experience parts of the chain must also be accommodated in order to achieve a successful design. Thus, the designer must become aware of the strengths and weaknesses of the available analytical tools, since this is one "chain-link" which can be controlled.

In this paper, after a brief discussion of some common limit equilibrium methods, a comparison of these methods is presented to illustrate the outcomes of different analyses. Then, with the analytical aspects clarified, a slope stability analysis program, STABL, is discussed to show its relative advantages and uniqueness, in overcoming the difficulties associated with trying to determine the most probable minimum factor of safety.

ANALYTICAL METHODS

In this paper, only the analytical part of design is considered, since it requires considerable judgment in insuring that the "right" result is obtained. Although, sometimes the "wrong" analysis may lead us to the correct answer, we must endeavor to use the "right" analysis to determine the correct answer. Primarily, there are five common methods of analysis which are listed below:

1. Ordinary or Fellenius Method of Slices
2. Simplified Bishop Method
3. Simplified Janbu Method
4. Spencer's Method
5. Morgenstern-Price Method

The method of slices is used by all these methods to "subdivide" the slope above the selected potential failure surface into small elements or slices. Then by application of the Mohr-Coulomb failure criterion along the potential failure surface and by differing assumptions regarding side forces on a typical slice (see Figures 1 and 2), the factor of safety for limiting equilibrium may be computed.

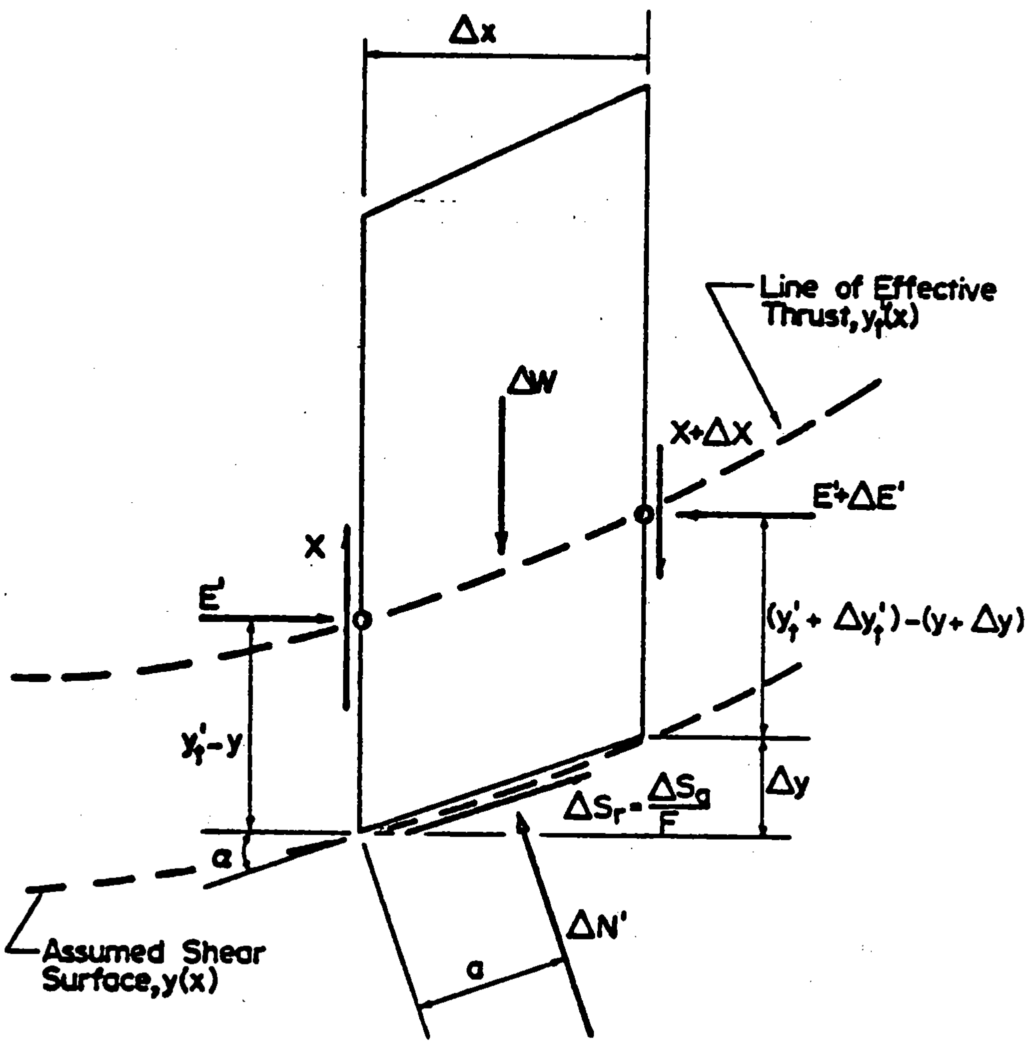


Figure 1, Forces on a Typical Slice (Siegel, 1975)

Ordinary or Fellenius Method of Slices

This method neglects all interslice forces (Fellenius, 1927) by summing forces normal to the base and by assuming that the direction of action of the normal interslice forces (E) is parallel to the base of the slice (see Figure 2). Since the angle of the base of a slice varies from one slice to the next, equilibrium between adjacent slices cannot be satisfied and may result in an error of the factor of safety by as much as 60% (Whitman and Bailey, 1967). The factor of safety is computed from the summation of moments about the center of the potential failure arc.

Simplified Bishop Method (Bishop, 1965)

The interslice shear forces, X_L and X_R, are neglected and the forces are summed in the vertical direction. When compared to the Ordinary Method, the direction of this summation is the only difference, since the factor of safety is derived by summing moments about the center of the proposed failure arc. The final equation, however, includes an implicit factor of safety and requires an iterative approach for the solution.

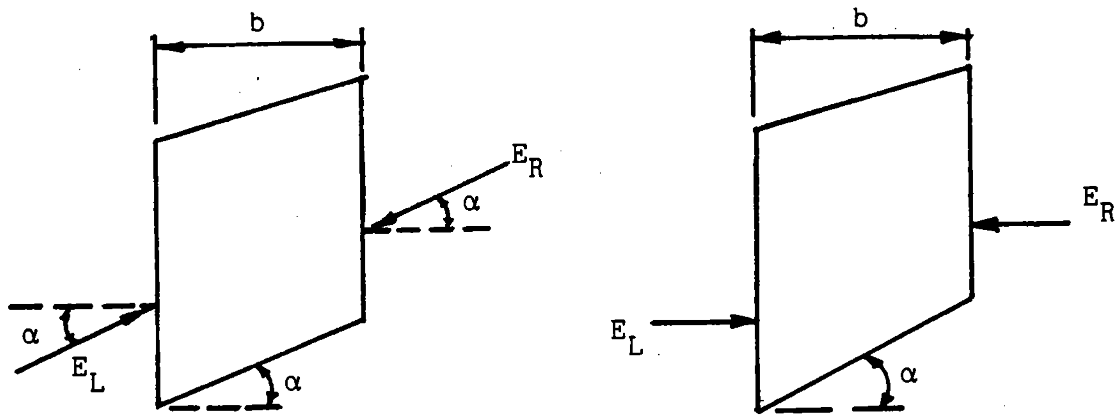
Janbu's Simplified Method (Janbu, 1956)

This method is applicable to irregular potential failure surfaces and considers overall equilibrium of horizontal forces, as well as vertical forces for each slice. The shear forces, X_L and X_R, are neglected and the factor of safety is computed from the overall horizontal equilibrium of the forces. An iterative process is also required to solve the final equation. To compensate for the assumptions, Janbu proposed a correction factor (Figure 3), based on the slope geometry, to arrive at the final factor of safety. These correction factors having been obtained from a more rigorous method of analysis.

Spencer's Method (Spencer, 1967)

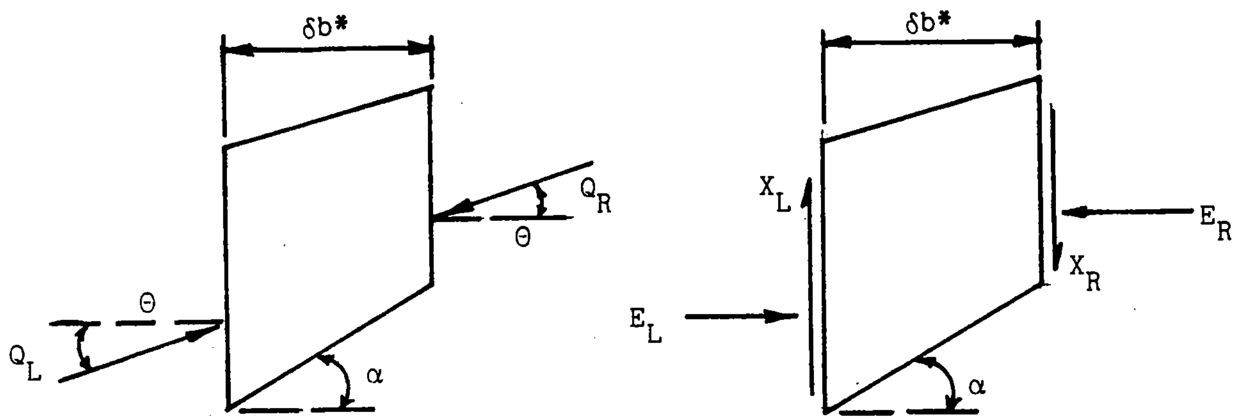
All interslice forces are considered in this method of analysis, which is applicable for circular, potential failure planes. The ratio of the normal and shear forces on the sides of the slices is maintained constant and these forces are subsequently simulated by their resultant. This approach results in two factors of safety being computed from the equation resulting from:

1. Summation of moments about the center of the arc,
2. Summation of forces parallel to the interslice resultant force.



(a) Ordinary or Fellenius Method of Slices

(b) Simplified Bishop Method

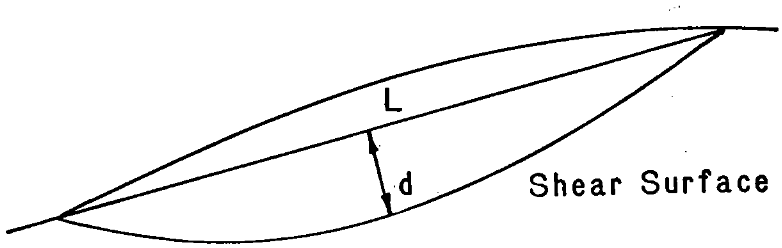


* infinitesimal slice width

(c) Spencer's Method

(d) Morgenstern-Price

Figure 2, Interslice Forces Assumed for Limit Equilibrium Analyses



$F = f_0 F_0$

$F_0 =$ Factor of Safety by The Simplified Janbu Procedure

$F =$ Corrected Factor of Safety

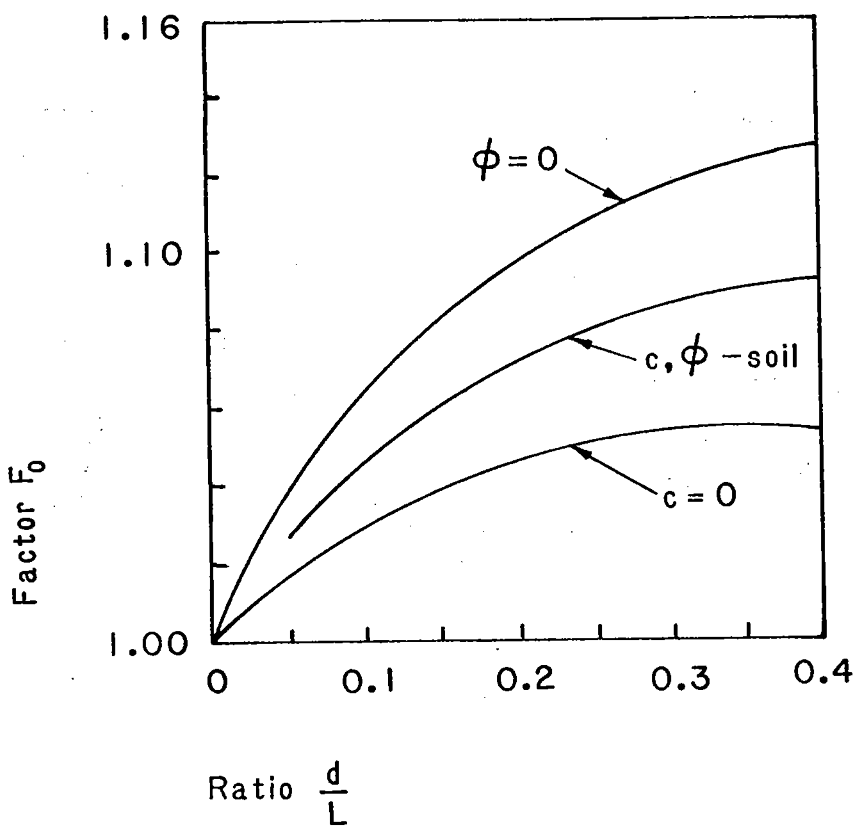


Figure 3, Janbu's Correction Factors
(after Janbu et al, 1956)

Then by varying the constant ratio of the side forces, a unique factor of safety satisfying force and moment equilibrium may be determined iteratively. Although this method was developed for circular surfaces, it can be extended to analyze irregular shapes (Wright, 1972).

Morgenstern-Price Method (Morgenstern and Price, 1965)

This method of analysis is similar to the Spencer Method except that a mathematical function is used to describe the direction of the interslice forces. The derivation of the factor of safety is applicable to any potential failure shape.

A summary of the type of equations, ie. either moment or force equilibrium, used for the computation of the factor of safety is presented in Table 1, below.

TABLE 1, Comparison of Factor of Safety Equations (after Fredlund, 1978)

Method	Factor of Safety Calculated by	
	Moment Equilibrium	Force Equilibrium
Ordinary or Fellenius	X	
Simplified Bishop	X	
Simplified Janbu		X
Spencer's	X	X
Morgenstern-Price	X	X

COMPARISON OF ANALYSES

A study to compare these methods of analysis has been performed by Fredlund and Krahn (1977) in which the same slope was analyzed using the five methods discused, above. Figure 4 shows the results of one such case, with factors of safety being normalized to λ, which is the mathematical function used in the Morgenstern-Price method. This function is zero for the Bishop method and for the Janbu method, it has a finite value which may be back-calculated.

From this figure, we can see that the safety factor varies according to whether it is derived from a force or moment equilibrium. It should be noted that the line representing moment equilibrium appears to be relatively insensitive to the assumed value of λ. Thus we can conclude

TABLE 2. Factors Related to Distributor (after Fredlund, 1978)

(a) Portability	–	STABL is written in standard FORTRAN
(b) Availability	–	Program readily available on 9-track Magnetic Tape
(c) Usability	–	Extensive Documentation with 'problem oriented' language and error-definitions
(d) Adaptability	–	Well-documented program statements for ease of modification
(e) Credibility	–	Verified (Boutrup, 1977)
(f) Maintainability	–	On-going process since 1975
(g) Education	–	STABL literature available
(h) Consultant	–	available at Purdue University

that the Simplified Bishop Method is fairly reliable and not limited by the assumptions used for its derivation. Similarly, the Janbu method, corrected for slope geometry, is also reasonable. However, it is much more sensitive to the values of λ and may not "always" be conservative. It should be noted that the F_m and F_f curves are a function of the slope conditions, with the F_f curve being the most variable of the two. Thus, for circular shapes of potential failure surfaces, the Simplified Bishop Method should be preferred instead of the Simplified Janbu Method. However, for irregular shaped surfaces, the Janbu method is preferred over the more complex Morgenstern-Price method, since it uses only approximately one-sixth of the computer time for a typical analysis (Fredlund and Krahn, 1977). Thus, after preliminary analysis with the Janbu method, a more refined, but limited, analysis may be performed to assess the accuracy of the former.

COMPUTER PROGRAM, STABL

In view of the above considerations and the potential practical limitations of the Spencer and Morgenstern-Price methods, it appears that the Simplified Bishop and Janbu methods are the most useful analyses for general slope stability problems. Also, to eliminate the need for the user to input the potential failure surfaces, various techniques for self generation are also available. These generally use a "grid-type" search and only accommodate circular surfaces.

However, STABL generates both circular and non-circular potential failure surfaces from a point on the ground surface, using a random technique. With a judicious placement of such initiation points, the slope may be explored for the "most-probable" minimum factor of safety using circular or non-circular shapes. Additionally, block or wedge type of potential surfaces can also be readily generated. These generating techniques have been discussed previously (see Siegel, 1975; Boutrup, 1977 and Siegel, et al, 1978), and will not be discussed further in this paper.

The use of limiting boundaries to either deflect potential surface upwards or downwards is also a great advantage. Thus, surfaces may be generated around stabilizing systems, such as retaining walls, and also be prevented from entering unlikely zones such as bedrock or competent material. However, it must be emphasized that surfaces generated in any program, whether by a grid-method or STABL's random technique, do <u>not</u> seek the most critical surface. It is left to the user to investigate a sufficient number of potential failure surfaces and zones within a slope to arrive at the most likely minimum factor of safety.

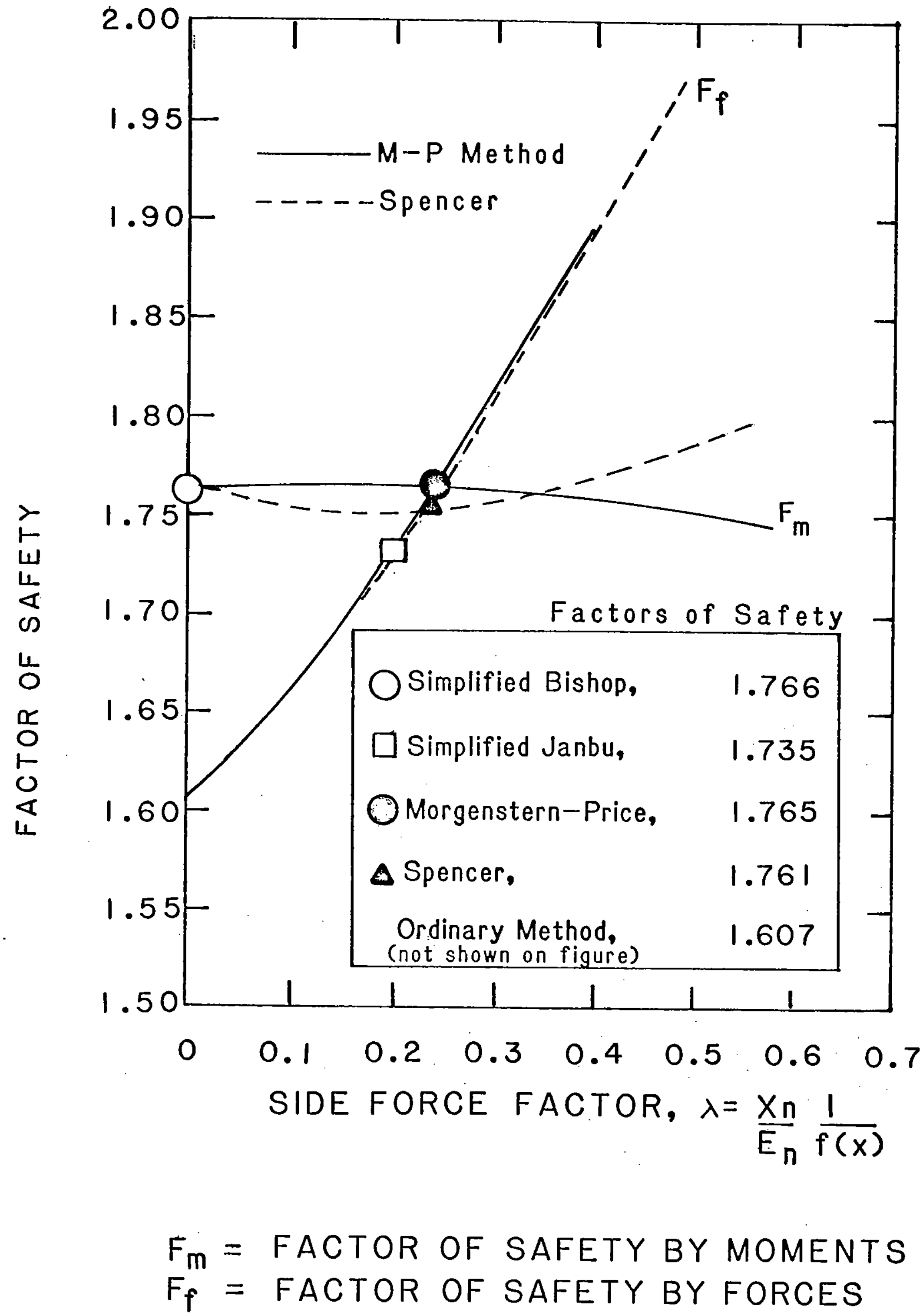


Figure 4, Comparison of Slope Analyses
(after Fredlund and Krahn, 1977)

TABLE 3. Factors Related to Program Capabilities (after Fredlund, 1978)

(a) Type of Analysis	–	Simplified Bishop and Janbu
(b) Shape of Failure Surface	–	Circular or Non-circular
(c) Number of Stratigraphic Units	–	Twenty Soil Types
(d) Partial Submergence	–	achieved with water surface (same for water-filled tension cracks)
(e) Surcharging/Berming	–	available
(f) Boundary Loads	–	available
(g) Earthquake Loading	–	vertical and horizontal accelerations may be input
(h) Pore-water Pressures	–	(1) Up to 10 water table surfaces (2) pore-pressure constants in each layer (3) use of r_u term
(i) Search Routines	–	A random-type search using circular or non-circular (including block) surfaces
(j) Additional options (not included by Fredlund, 1978)	–	(1) Data can be subsequently modified (2) Search may be confined to zones which appear critical based on experience (3) Option to use Simplified Janbu or Bishop (4) Soil strength anisotropy in ten directions (5) Includes Calcomp plotting routines

There are other factors which have also been considered during the development of STABL. These factors, as suggested by Fredlund (1978), are summarized in Tables 2 and 3 being listed under whether they relate to the distributor or the capabilities of the software package. Also, in the same tables, we have shown the extent of compliance which is achieved, by STABL, in meeting these proposed criteria. Most significant of all, considerable research has been expended at Purdue University for performing comparative studies to evaluate the program's potential for computing an accurate factor of safety. This research is on-going with further development leading towards tie-back wall design and three dimensional analysis.

EXAMPLES

In order to illustrate the flexibility of STABL, three examples of slope stability analyses are presented.

The first problem relates to the assessment of foundation strength for a proposed highway embankment. Since the subsoils are soft clays, it is desirable to know the minimum factor of safety against a deep seated foundation shear failure. Figures 5 and 6 show the subsurface profile and the ten most critical circular surfaces, respectively. These ten were selected from 100 surfaces investigated from 5 different initiation points extending from 5 to 15 feet from the toe of the embankment. From Figure 6, we can see that the minimum factor of safety is 1.555. However, it is left to the designer's judgment whether a more critical, potential failure surface can be found to reduce the factor of safety determined from these initial 100 circular surfaces. Also, non-circular surfaces may be investigated.

Another case is shown in Figures 7 and 8, where the overall stability analysis of a "reinforced earth" type slope constructed in clayey soils is examined. Here, the limitation boundaries were imposed around the "reinforced earth" area and the surface of the bedrock to prevent generation of potential surfaces through these zones. Thus, the program effectively generates only surfaces which do not pass through the reinforced earth wall or the underlying bedrock. By simply not generating such surfaces, which would only lead to high factors of safety, the program allocates its computational time more efficiently to the potential surfaces which are significant. For this case, 100 irregular surfaces were generated from 5 initiation points which are within 20 feet of the toe area. The minimum factor of safety for the most critical surface was calculated to be 1.219, as shown on Figure 8. Again, this should not be treated as the <u>minimum</u> factor of safety for the slope

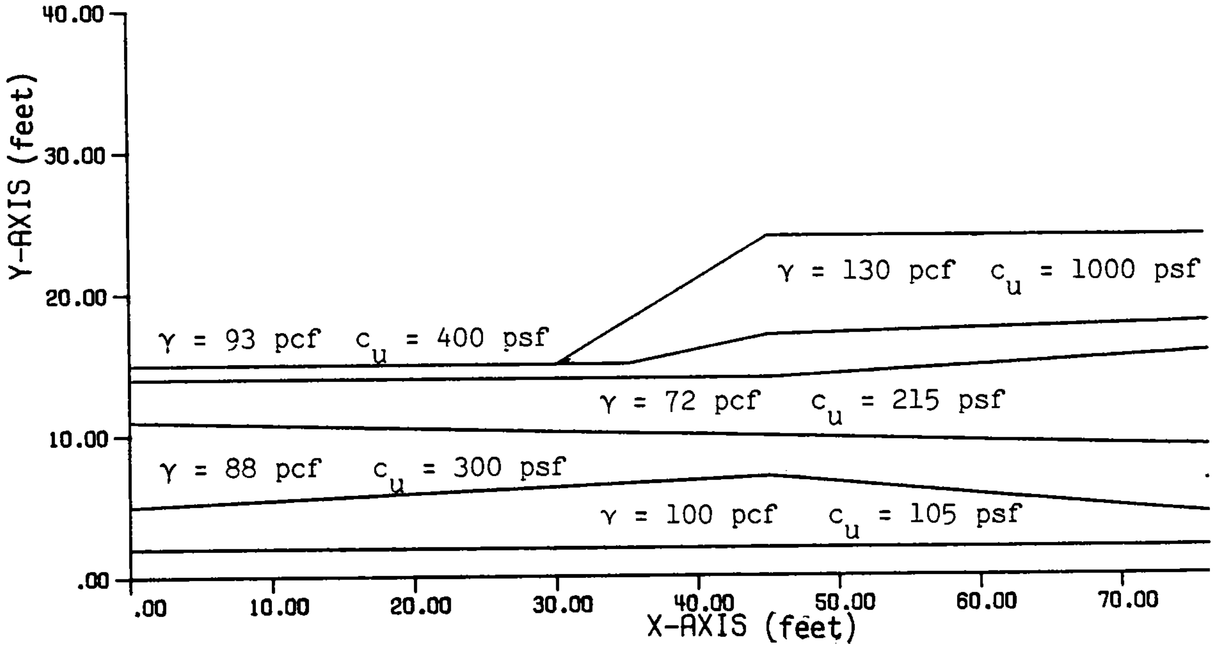


Figure 5, Subsurface Profile below Embankment

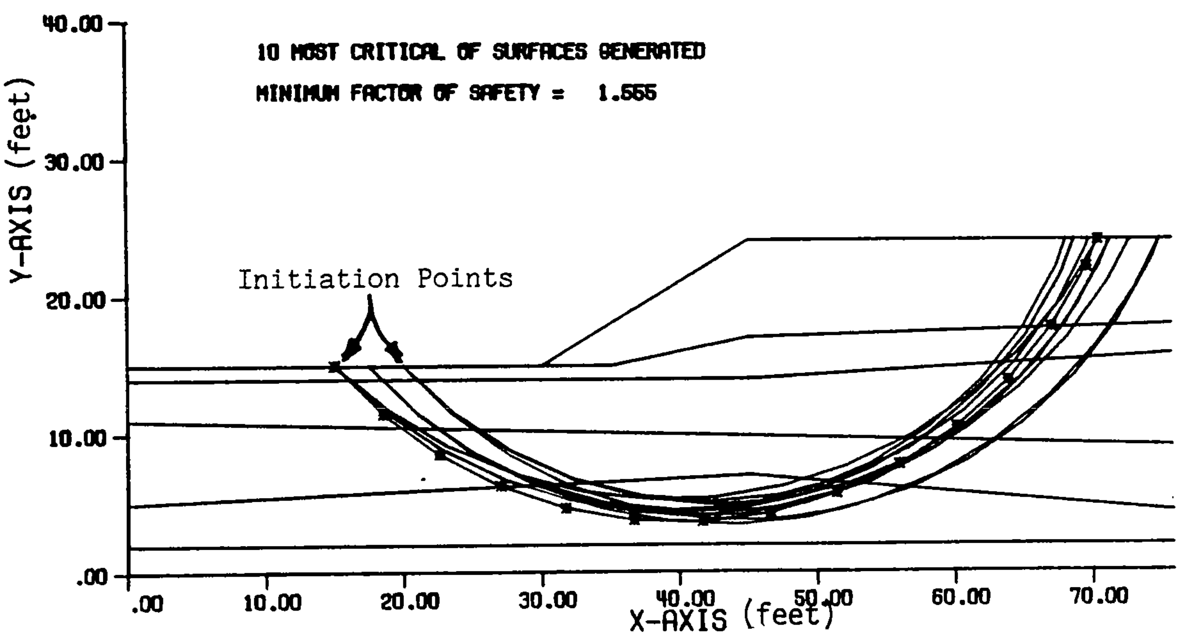


Figure 6, Stability Analysis of Embankment Foundation - Total Stress Analysis

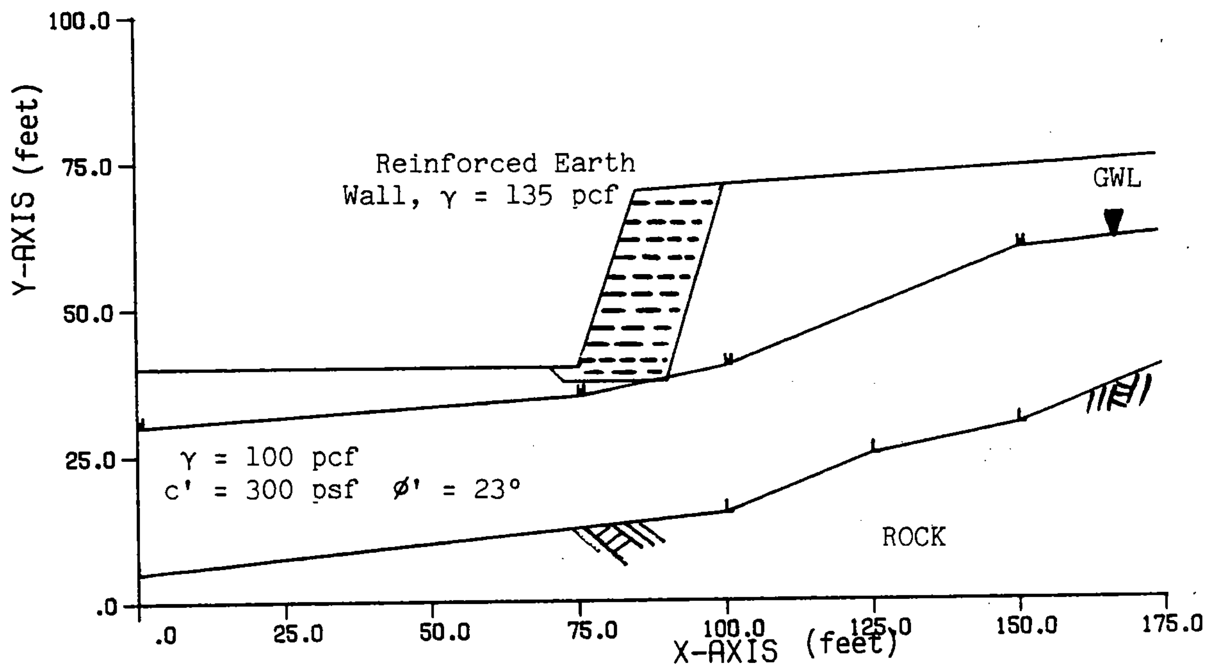


Figure 7, Subsurface Profile, Reinforced-Earth Wall

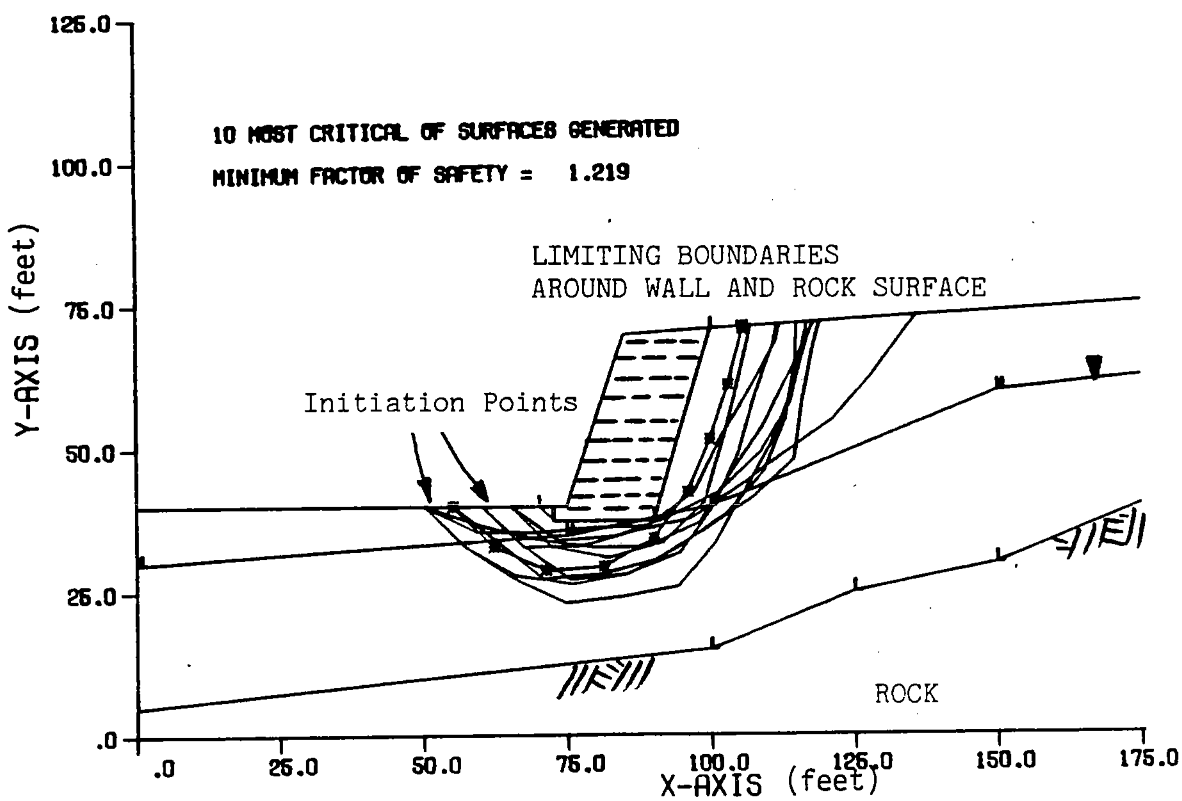


Figure 8, Overall Stability - Drained Analysis since other, more critical surfaces of either circular or non-circular shapes, may exist.

In order not to limit the examples to highway construction, the final example shown in Figure 9 displays a cross-section of a dam embankment. For this case, the subsurface properties are not included due to the complexity of the cross-section. The block or wedge type of analysis is shown in Figure 9. Since a weak seam of low strength material was encountered during soil investigation, it was anticipated that a potential block failure surface may provide the most critical factor of safety. For such a surface, a major portion of the potential failure surface would be located in this weak seam. In order to generate such surfaces, "initiation boxes" are specified for the central portion of the surface and the program randomly selects a surface of exit at the toe and crest. Although, only two boxes were specified for this case, up to ten may be specified to "force" surface generation through potential weakness zones. This is useful in fissured and/or jointed materials. A minimum factor of safety of 1.046 was computed for this case.

CONCLUSIONS

If a designer proceeds to use slope analysis programs without being aware of their analytical characteristics, it is likely that some of his or her designs may lead to failures. Thus we have tried to summarize the most common methods of slope analysis in order to make comparisons of their effectiveness. Many more programs are being developed for research purposes. However, these are only for individual projects and are thus not always maintained after the completion of the original research. Also, the documentation is usually very sparse and does not always indicate subsequent corrections of the source files. In concluding this paper, we offer the following suggestions for performing slope analysis with current methods :

1. Must determine the type of analysis used in the program
2. Use the most accurate analysis available, unless cost constraints force the use of less accurate methods
3. Preference should be given to analytical methods which use the moment equilibrium equations for the derivation of the factor of safety
4. Use of programs should be restricted to those which have on-going maintenance, credibility and documentation procedures
5. Users should establish personal requirements for software, similar to the one presented in Tables 2 and 3, for assessing newly available programs

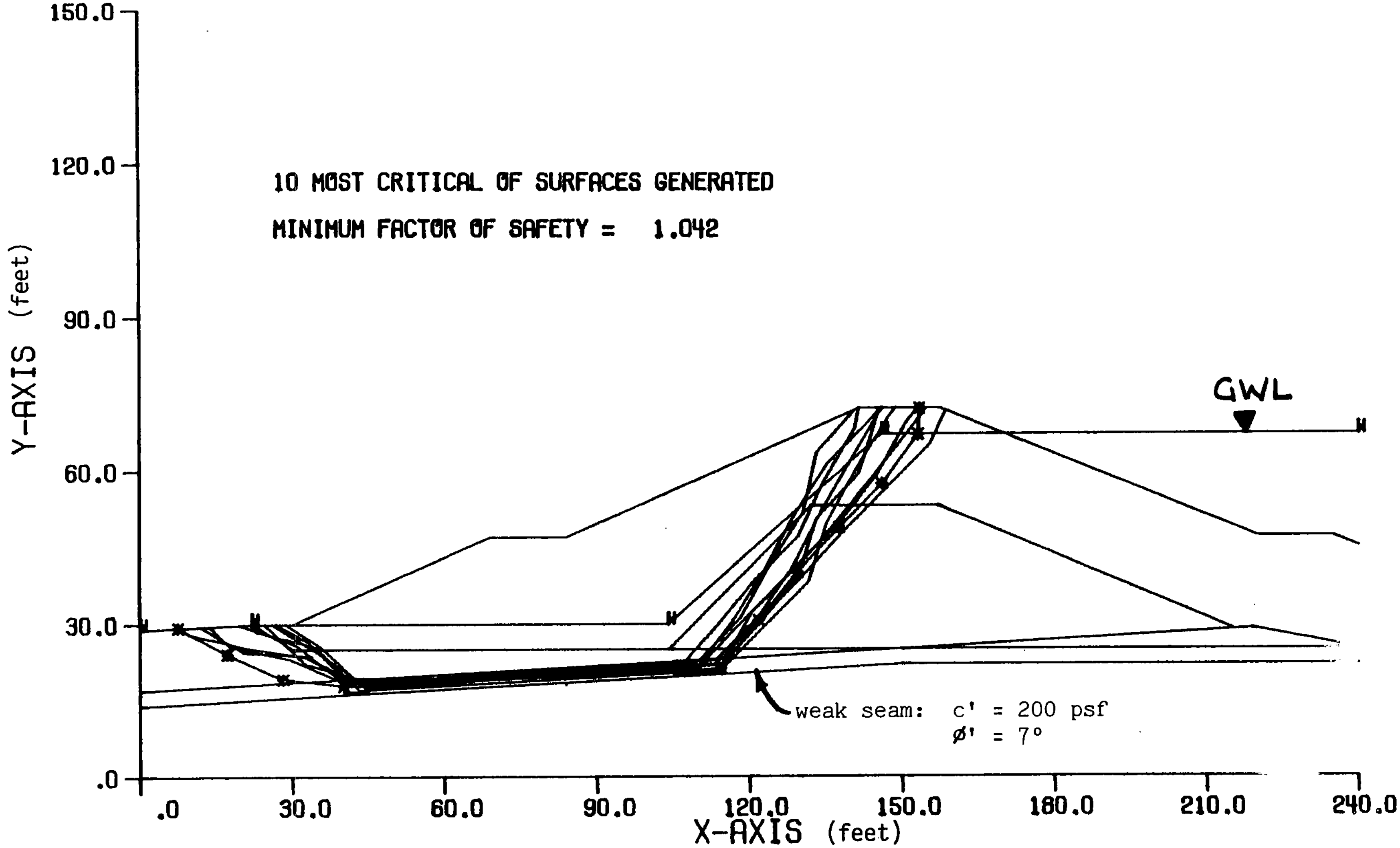


Figure 9, Example of Block Analysis for Dam Embankments

6. Users should check prospective programs for credibility on their "own" system and preferably against their current program(s) which are slated for replacement

Providing the above criteria are considered, and similar attention is paid to the soil sampling and testing of the subsoils in the slope areas, all the designer then requires is "EXPERIENCE" for economical design. However, we hope that the selection of the best analytical tools has been made easier by this paper.

ACKNOWLEDGMENTS

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GEOTECHNICAL DATA COLLECTION
FOR DESIGN OF THE CUMBERLAND GAP
PILOT BORE

by
Gary H. Collison, Principal
Golder Associates

ABSTRACT

The Cumberland Gap Pilot Bore is proposed as the primary exploration tool to provide data for design of twin, dual lane highway tunnels through Cumberland Gap, Tennessee. No traditional test boring program along the full length of the alignment has been performed, nor was one believed necessary for the pilot bore design. The geotechnical data collection consisted of three primary phases: background data compilation, field observation and mapping, and data interpretation.

A good deal of published geologic literature for the Cumberland Gap area was available because of it sgeologic and historic significance. Aerial photographs and limited boring data and seismic refraction study data obtained at the proposed main tunnel portal locations were also available. Use of these sources to assess major geologic trends and the structural attitudes and disposition of the various lithologies within the study area are discussed.

Field observation and mapping were done to verify existing geologic maps and published geologic interpretations of the area as well as collecting geotechnical data, hydrogeologic data, and geologic samples. A description of the field mapping and reconnaissance program is provided.

The background information and site reconnaissance data were used to define the stratigraphic ascension and formation thickness along the alignment. Contact relationships between lithologic units as well as structural discontinuities and associated trends are presented. Physical characteristics of dominant lithologies, such as weathering, solutioning, cementation, joint pattern, and shear zones, were assessed. Use of these data to estimate geotechnical engineering parameters of materials expected to be encountered by the pilot bore are discussed.

1.0 INTRODUCTION

On behalf of the National Park Service the Federal Highway Administration is embarking on a program to relocate U.S. 25E beneath Cumberland Mountain between Cumberland Gap, Tennessee and Middlesboro, Kentucky. The relocation will include construction of two dual-lane highway tunnels through Cumberland Mountain. The general location of this project is shown on Figure 1. The initial phase of this effort is the construction of an exploratory pilot bore within the cross section of the proposed southbound main tunnel. The pilot bore will be the primary exploration tool for investigating subsurface conditions for design of the main tunnels.

The design of the pilot bore has been performed by the Federal Highway Administration. To assist in this work, Golder Associates was retained to provide geological, hydrogeological, and geotechnical engineering services. These services included collection of data and evaluation of the site geology and hydrology with respect to construction of the pilot bore and main tunnels. The data sources were published literature, an extensive field mapping program by Golder Associates, and seismic survey data and logs of borings drilled in the portal areas as part of an exploration program being carried out by another consultant for the highway approaches to the tunnel.

The overall objective of the studies for this project was to collect information from which a geotechnical model could be established. This model of the ground expected to be encountered by the proposed tunnels is used to define tunneling classification and rock categories for engineering design purposes. This task was accomplished without an extensive subsurface exploration program along the alignment.

This paper presents a review of the data gathering and data evaluation for design of the pilot bore, a general description of the geologic conditions expected to be encountered by the pilot bore, and proposed support requirements for the pilot bore.

2.0 DATA GATHERING AND EVALUTION

In assessing the overall geologic character in the vicinity of the proposed highway tunnel alignment, the following primary sources of data were considered:

- Published literature.
- History of the nearby L&N Railroad tunnel.
- Seismic survey data in the vicinity of the proposed main tunnel portals provided by the Federal Highway Administration.

- Logs of the test borings drilled in the vicinity of the main tunnel portals (performed by others for the Federal Highway Administration).

- Review of the aerial photographs provided by the Federal Highway Administration.

- A detailed field reconnaissance and mapping program performed by Golder Associates.

Because of the historical significance and geologic interest of the Cumberland Gap area, a great deal of published literature was available. This literature was thoroughly reviewed and integrated to provide a general understanding of the geology of the area. Preliminary geologic maps were prepared for guidance during field reconnaissance.

The L&N Railroad is present owner of a single track tunnel, approximately 3,700 ft. long, which was driven beneath Cumberland Gap in 1889. This tunnel is approximately parallel to the proposed highway tunnel alignment and intersects many of the formations expected to be encountered by the highway tunnels. However, only about 20% of this tunnel is unlined and available for inspection.

A geologic reconnaissance was conducted to confirm the overall validity of published information and locate prospective sites for detailed mapping. Rock exposures were mapped along U.S. 25E road cuts and the unlined portion of the L&N Railroad tunnel. Although good outcrops were available in these areas, most of them are separated from the proposed highway tunnels by the Rocky Face Fault and some distortion of the structure has resulted. Also, only the stronger rock units are exposed in the L&N Railroad tunnel. Rock outcrops over the proposed pilot bore and in the general vicinity of the proposed pilot bore were also mapped.

The detailed mapping included lithologic description and an assessment of the overall rock mass character of the formations. Window surveys were performed at most of the major outcrop sites. Window surveys consisted of scaling an approximate one meter square area on a rock face and taking precise measurements of the attitudes of the discontinuities in the scaled area. At the same locality details were collected of the whole rock mass thus putting the window data sample into a larger context. For convenience of data collection and evaluation, geotechnical "units" were defined within the stratigraphic sequence. Field measurements and observations are noted in a specific format for ease in data compilation and management by computer.

For each "window" or data station, the major joint sets were surveyed. Readings of dip angle and dip direction (azimuth) were taken for each joint, its total length or persistence was measured, the width of the aperture or "openness" of the joint and its infilling were recorded, and the roughness of the discontinuity was estimated. Evidence of water was noted.

The structural data resulting from the survey for each unit was plotted on an equal area lower hemisphere polar projection. These scatter diagrams, plotted by computer, were contoured to obtain point maxima for definition of the major discontinuity sets.

The rock mass survey provided a petrographic description of the rock material, its state of weathering and a field estimate of the uniaxial compressive strength of the rock. The number of major discontinuity sets dividing the rock were estimated, the average size and shape of the joint blocks assessed, and the spacing of the discontinuities measured. In this way, the geotechnical character of the rock mass was broadly described; detailed data collection on the discontinuities then supplemented the rock mass survey. Laboratory work on samples collected extended the descriptions wherever possible by providing petrographic analysis, rock strength, susceptibility to weathering, and clay mineralogy.

3.0 STRATIGRAPHY AND STRUCTURE

The proposed pilot bore will be driven through Cumberland Mountain which is an overthrust block near the junction of the Valley and Ridge and Appalachian Plateau provinces. The pilot bore is about one-half mile from Cumberland Gap which represents a pass created by preferential erosion through Cumberland Mountain along the Rocky Face Fault zone. A surface geologic map of the project area showing the various formations and fault locations is presented on Figure 2.

The rocks expected to be traversed by the proposed pilot pore range from the Silurian age Rockwood Formation to the Hensley member of the Pennsylvanian age Lee Formation. The rock types range from uniform shales and limestones to interbedded sandstones, shales and coals. The geologic structure along the proposed pilot bore alignment is relatively simple. It consists of sedimentary beds dipping moderately (30^{o} to 40^{o}) to the northwest. The anticipated geology at the pilot bore level as projected and extrapolated from outcrops is shown on Figure 3.

Three prominant and regular joint directions were inferred from the aerial photographs. They are characterized by very fine (thin) linear features such as microdrainage, tone alignments, etc. The evidence suggests that these are steeply dipping; dips greater than 60^{o} and most being nearly vertical. One of the prominant directions has a strike direction nearly parallel to the bedding strike and the other nearly normal to the bedding strike. A third prominant set appears to be sub-parallel to the bedding dip direction, sometimes wandering to a mid-way between the other two primary sets. A fourth apparent direction of jointing may exist, but such features were fairly discontinuous.

The field measurements of exposures in the vicinity of the proposed pilot bore generally agree with the aerial photographic interpretation. Bedding was noted to dip to the northwest in the 30° to 40° range. Most joints are considered to be related to the folding; they dip steeply, being almost normal to the bedding. The two major joint sets strike parallel and normal to the strike of the bedding. Joint spacing varies widely, on the order of inches to several feet, depending on the lithology; generally being spaced more closely in the thinly bedded formations. Shear zones were noted more commonly in the weaker shales and were usually parallel to bedding.

Evidence of extensive karstification is abundant in the Mississippian limestones of Cumberland Mountain. The Cudjo Cave system developed in the Lower Newman Member can reputedly be traced for two miles underground and exhibits five different levels. Small karstic cavities have been mapped in the limestone sequence along the proposed tunnel alignment. Solutioning of the limestone and the development of small cavities is also apparent in the limestones exposed in a quarry near the southeast portal of the L&N Railroad and in the limestones exposed along the highway. Sink holes have not been observed at the ground surface in the area but could well be masked by thick colluvium on the slopes above the limestone outcrops.

It is very likely that the limestone through which the proposed tunnels will be driven will contain karstic cavities. Whether the development will be as extensive as those in Cudjo Cave is difficult to predict; that cave system may be extensive because of its proximity to three major faults.

4.0 GEOTECHNICAL EVALUATION

Due to the complexities involved in underground construction, analytical methods are not often applicable in determining excavation stability and support requirements. Empirical methods utilizing substantial experience are employed instead to assess support requirements for the expected ground conditions. This approach generally consists of "classifying" the expected ground conditions with respect to a standard set of conditions and applying the collective experience representative of each class. This classification approach takes a variety of forms. Bieniawski[2] suggested a classification system for jointed rock masses should:

- divide the rock mass into groups of similar behavior;

- provide a good basis for understanding the characteristics of the rock mass;

- facilitate the planning and the design of structures in rock by yielding quantitative data required for the solution of real engineering problems; and

- provide a common basis for effective communication among all persons concerned with a geomechanics problem.

These aims should be fulfilled by ensuring that the adopted classification is simple and meaningful in terms, and based on measurable parameters which can be determined quickly and cheaply in the field.

The most widely used classification systems are:

- Terzaghi's[4] rock load

- Deer's[3] Rock Quality Designation (RQD)

- Bieniawski's[2] South African Council for Scientific and Industrial Research (CSIR) Geomechanics Rock Mass Rating (RMR)

- Barton, Lien and Lunde[1] of the Norwegian Geotechnical Institute (NGI)

Of these four systems, the CSIR and NGI systems were employed. These systems provide a rational basis for systematically considering the strength of the intact material and the frequency and nature of the discontinuities and relating the resulting rock mass categories to tunnel support designs derived from precedent practice. The qualitative system proposed by Terzaghi was intended for use in estimating the loads to be supported by timber arches in tunnels. As will be discussed later in this paper, the recommended system of support rock bolts and shotcrete, so the Terzaghi system is not dealt with here. The RQD system was not employed because of the absence of core data for most of the units.

All of the geotechnical units along the alignment were grouped into one of three composite Rock Categories. These Rock Categories are shown superimposed on the projected geologic section on Figure 3. The relative grouping of the classifications of the various rock units into the three main Rock Categories is shown in the plot of the RMR (rock mass rating) values of the CSIR system and the Q (tunneling quality index) values of the NGI system in Figure 4.

The stability of a jointed rock mass is derived from the interlocking of individual blocks. The retention of this interlocking is an important consideration in the design of a support system to control structural failure in a tunnel. The only effective method of retaining this interlocking is to inhibit deformation of the rock mass by installing support very close to the tunnel face. The availability of reliable rock bolting and shotcreting methods has made it possible to achieve this rapid support installation without causing major disruption to the tunneling cycle and these methods are now widely used in rock tunneling. Steel sets, which only become operative when a significant amount of deformation has occurred in the rock surrounding the tunnel, are not very effective in retaining the interlocking of a jointed rock mass and should not be used unless the intact rock strength is too low to allow effective anchoring of rockbolts or the depth of cover is too small to allow arching to develop.

In the case of Cumberland Gap tunnel, consideration of the geological data, topographic data, and evidence from the existing railroad tunnel suggests that stress-induced instability will be a minor problem and that the dominant failure mode will involve block release by intersecting discontinuities. This type of failure will take the form of wedges formed by the rock discontinuity systems and the pilot bore excavation outline. A wedge may be large enough to span the tunnel, but smaller wedges are more likely in this dimension. Theoretically, there is no limit to the size of such wedges in the dimension along the tunnel alignment. In fact, not all joint surfaces are continuous and there is a tendency for arching to occur at some stage.

It should be noted that none of the rock mass classification systems deal adequately with this specific type of failure. Consequently, the NGI and CSIR systems have been used to classify and group the formations and to provide a means of estimating the size of individual blocks to be supported. The support system has been chosen on the basis of this block size rather than by reference to the support recommendations given by Bieniawski and/or Barton, et. al.

5.0 PILOT BORE SUPPORT SYSTEM

The primary function of ground support is to ensure that the rock mass preserves most of its strength by preventing major displacement of the mass. This is best accomplished by reinforcing the rock with rockbolts and/or shotcrete. These forms of reinforcement are active support measures and are preferable to purely passive measures, such as installation of steel sets, which have traditionally been used in tunneling.

Based on the assessment of the quality of the rock at the proposed tunnel site and the advantages associated with active support systems, it has been proposed that the pilot tunnel be excavated by drill and blast or roadheader equipment and be primarily supported with a rockbolt and shotcrete system. Three patterns of rockbolts with varying treatments of shotcrete and wire mesh are intended for the three Rock Categories expected to be encountered in the pilot bore. These patterns are shown on Figure 5. The choice of rockbolt pattern was dependent on consideration of the smallest wedges which could potentially fall out between the rockbolts. Smoothwall blasting of the crown is also proposed since the pilot bore will be at the crown of the main tunnel. Tensioning of the rockbolts should be unnecessary provided that they are installed within one tunnel diameter of the face because deformation of the rock mass will tension the rockbolts. In both pilot bore portal areas, light steel sets may need to be installed due to the weathered nature of the rock expected at, and above, the crown.

For the pilot bore alone, six foot long rockbolts would be adequate for support. However, for the main tunnels, which will be about three times wider than the pilot bore, the crown rockbolts would need to be about eight feet long. Thus, it is planned that eight foot long rockbolts will be used in the pilot bore.

It is planned that all rockbolts will be one inch diameter and be fully grouted and anchored with a quick setting (30 to 60 seconds) resin-type grout. The resin grout installation is much faster than other methods and thus reduces the cycle time at the tunnel face. Even though the unit cost for resin anchored bolts may be greater than for mechanically anchored bolts, the time saved usually dominates in the total cost analysis.

As an exploration and modeling tool, certain aspects of the pilot bore will be approached differently than for the main tunnels. Wherever possible, the pilot bore will be left unlined so that it is available for inspection during the design period, especially by potential main tunnel contractors. Where shotcrete is recommended, windows about three feet square will be cut in the shotcrete to permit inspection. These windows are planned to be placed on 25 foot centers, both sides, with changes in spacing or extra windows cut to expose lithologic contacts or other notable geologic features. The pilot bore will permit an excellent assessment of the geology expected in the main tunnels.

6.0 SUMMARY

The geotechnical investigation program for the Cumberland Gap pilot bore is an example of the use of available literature and field reconnaissance studies to provide data for rock tunnel design. A model of the anticipated ground conditions was developed to define tunneling classification and rock categories for engineering purposes. This approach, which has often been used in the mining community and for other civil engineering tunnels, can be successfully applied to highway tunnels.

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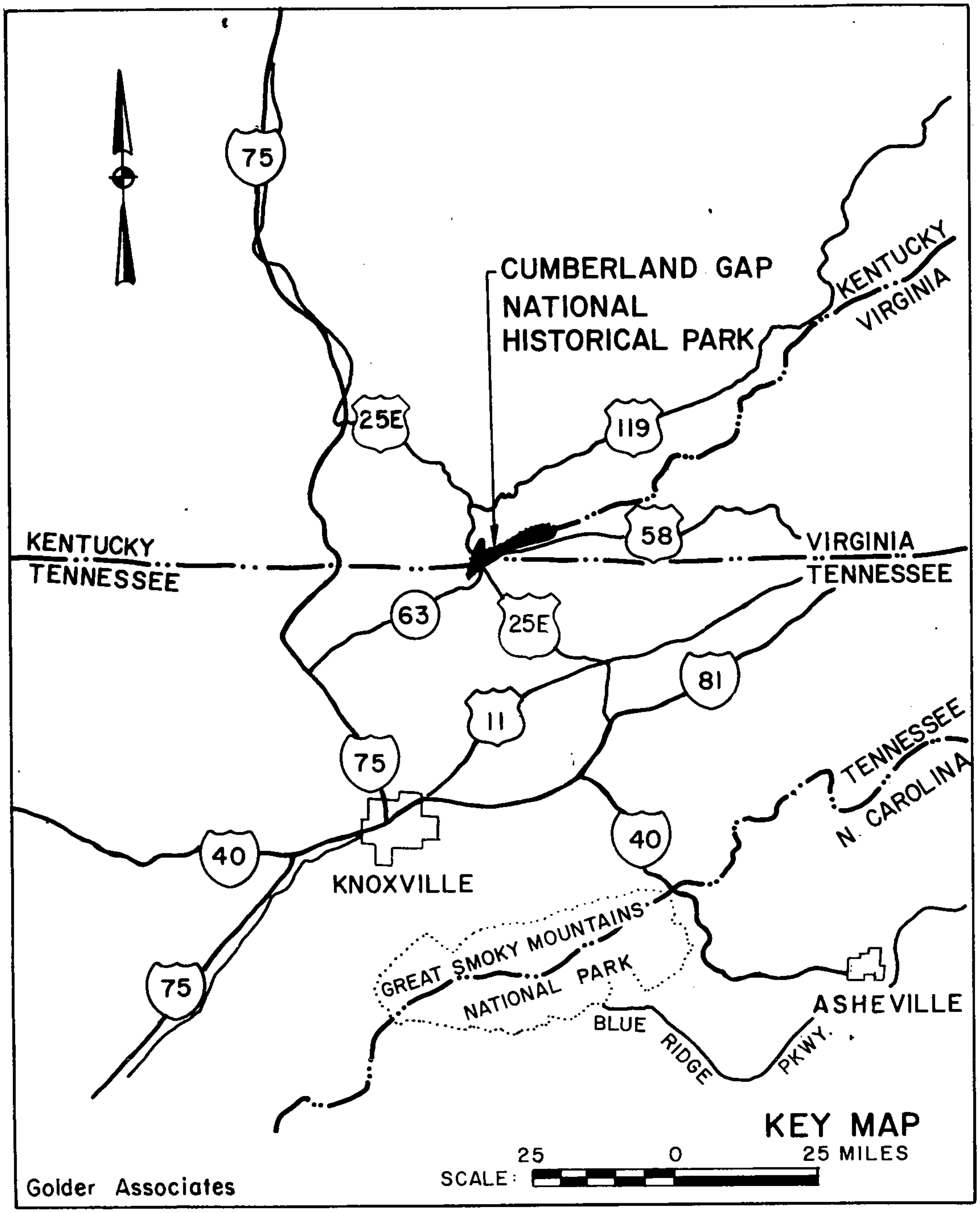
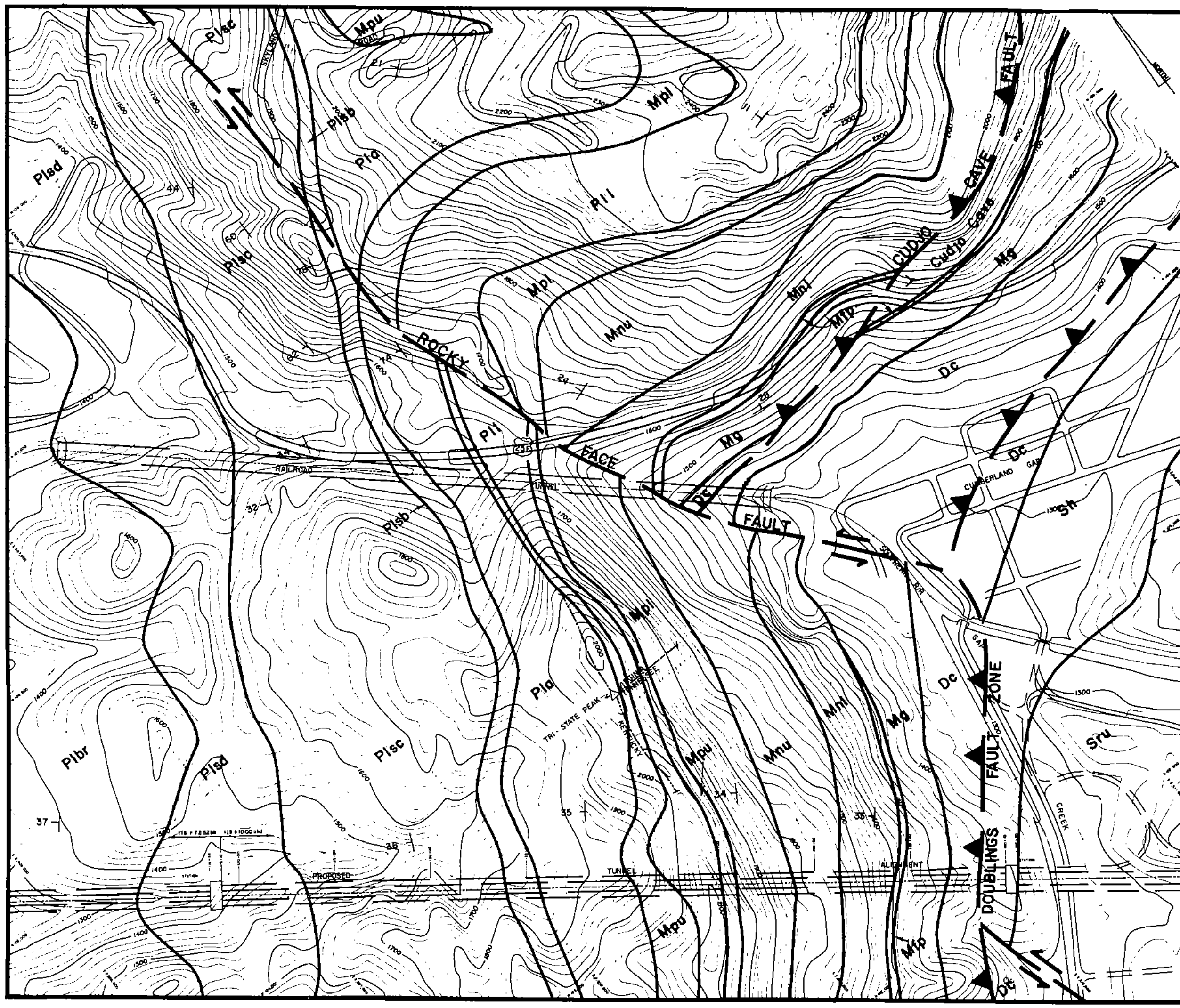


Figure I. CUMBERLAND GAP PILOT BORE SITE LOCATION



LEGEND

Plbr	—	Bee Rock Sandstone
Plsd	—	Hensley Sandstone and Shale
Plsc	—	Middlesboro Sandstone and Shale
Plsb	—	Dark Ridge Shale
Pla	—	Chadwell Sandstone
Pll	—	Pinnacle Overlook Sandstone
Mpu	—	Upper Pennington Shale
Mpl	—	Lower Pennington Sandstone and Shale
Mnu	—	Upper Newman Limestone and Shale
Mnl	—	Lower Newman Limestone
Mfp	—	Ft. Payne Chert
Mg	—	Grainger Shale
Dc	—	Chattanooga Shale
Sh	—	Hancock Dolomite
Sru	—	Upper Rockwood Shale

Approximate location of strike slip fault; arrows show relative movement.

Approximate location of thrust fault; sawteeth on upthrown block.

24 — Strike and Dip of bedding.

Cave entrance at point.

NOTES

1.) Topographic base map from U.S.G.S. and FHWA.
2.) Geologic mapping from U.S.G.S. Middlesboro South Geologic Quadrangle Map. Formational boundaries in vicinity of the proposed tunnel alignment based on field survey by Golder Associates.
3.) Tunnel alignment taken from plans provided by FHWA.

0 100 200 400 600 800
SCALE IN FEET

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Figure 2. SURFACE GEOLOGY MAP OF CUMBERLAND GAP AREA

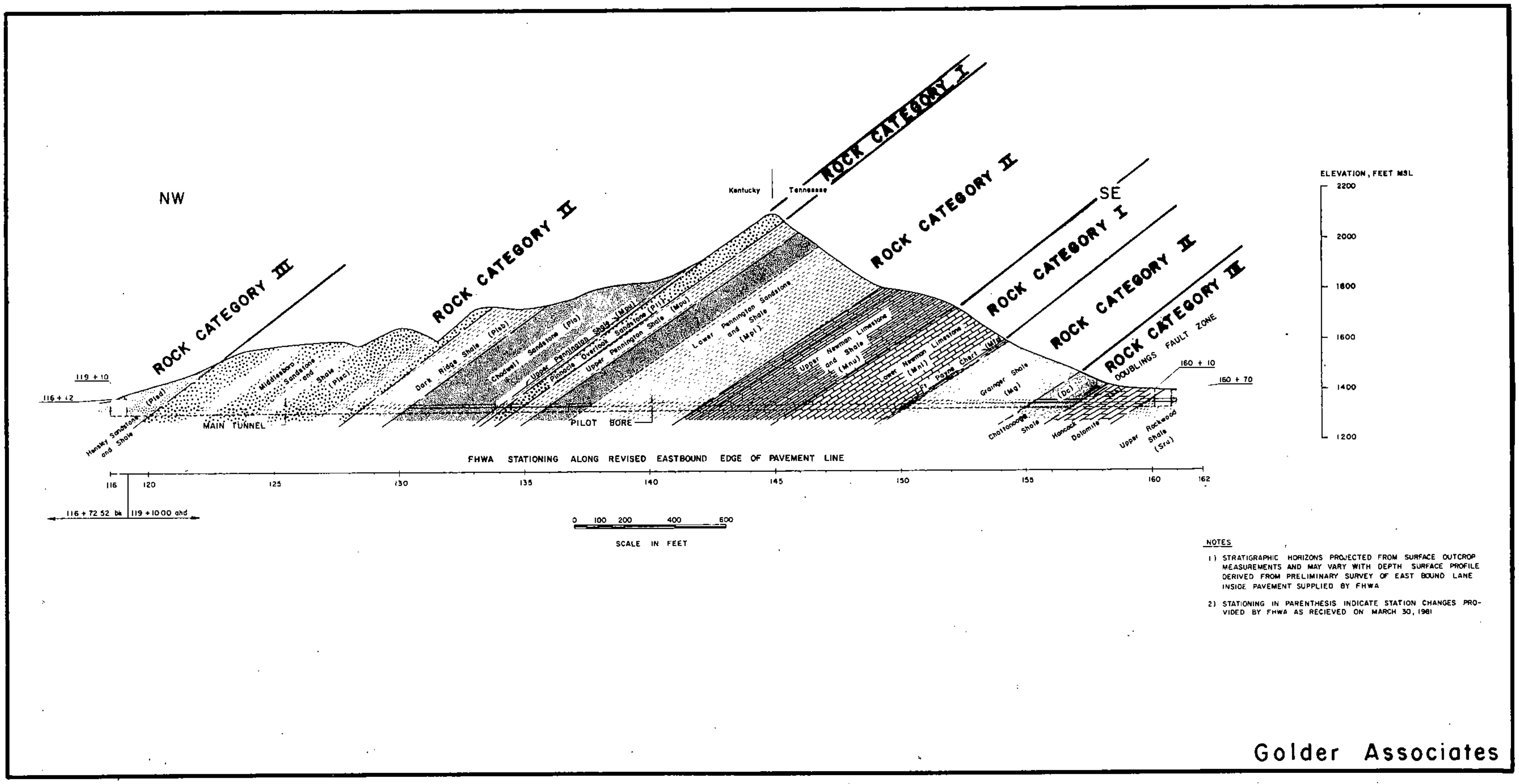

Figure 3. PROJECTED GEOLOGY OF TUNNEL LEVEL AND ROCK CATEGORIES FOR DESIGN

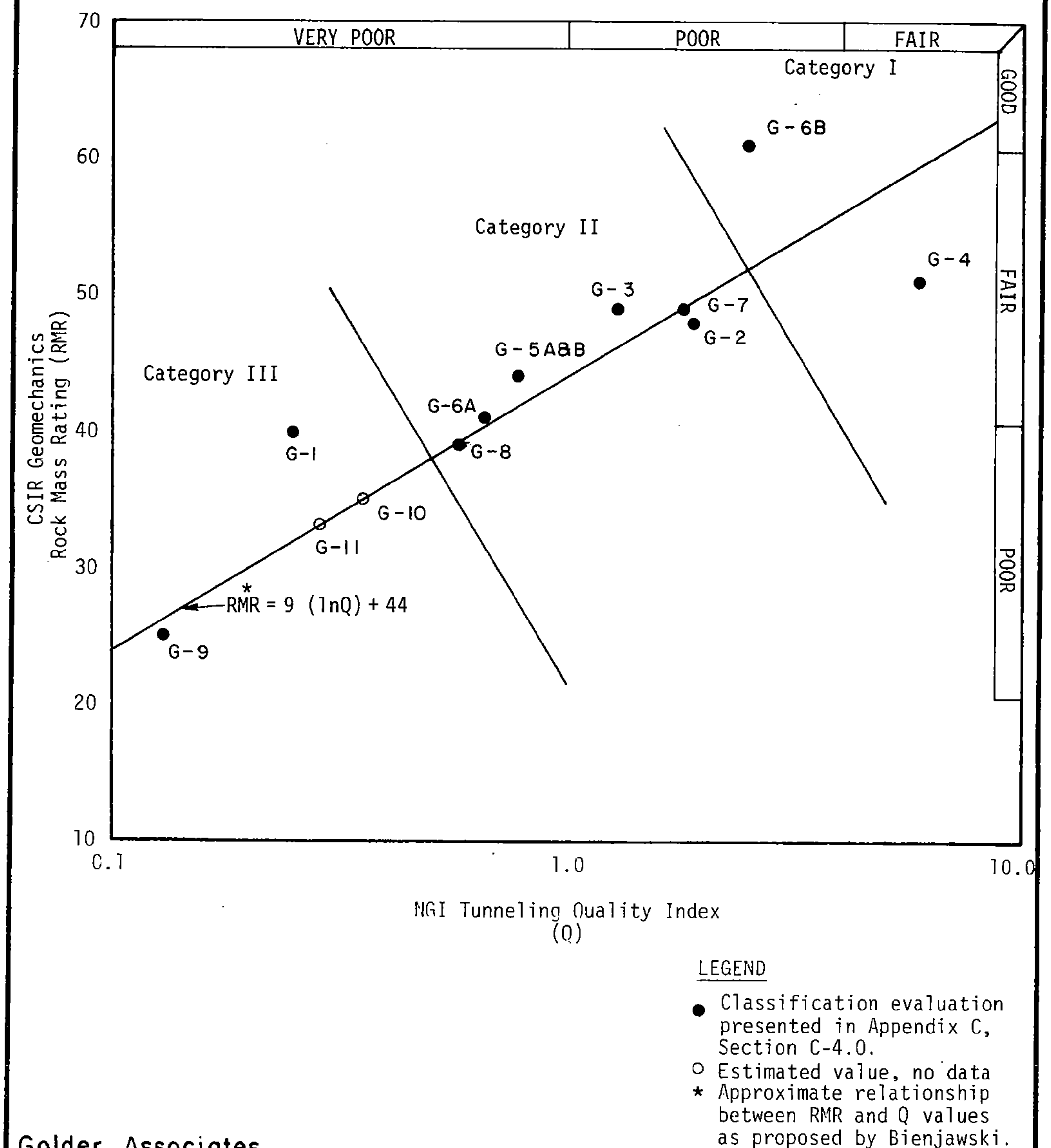

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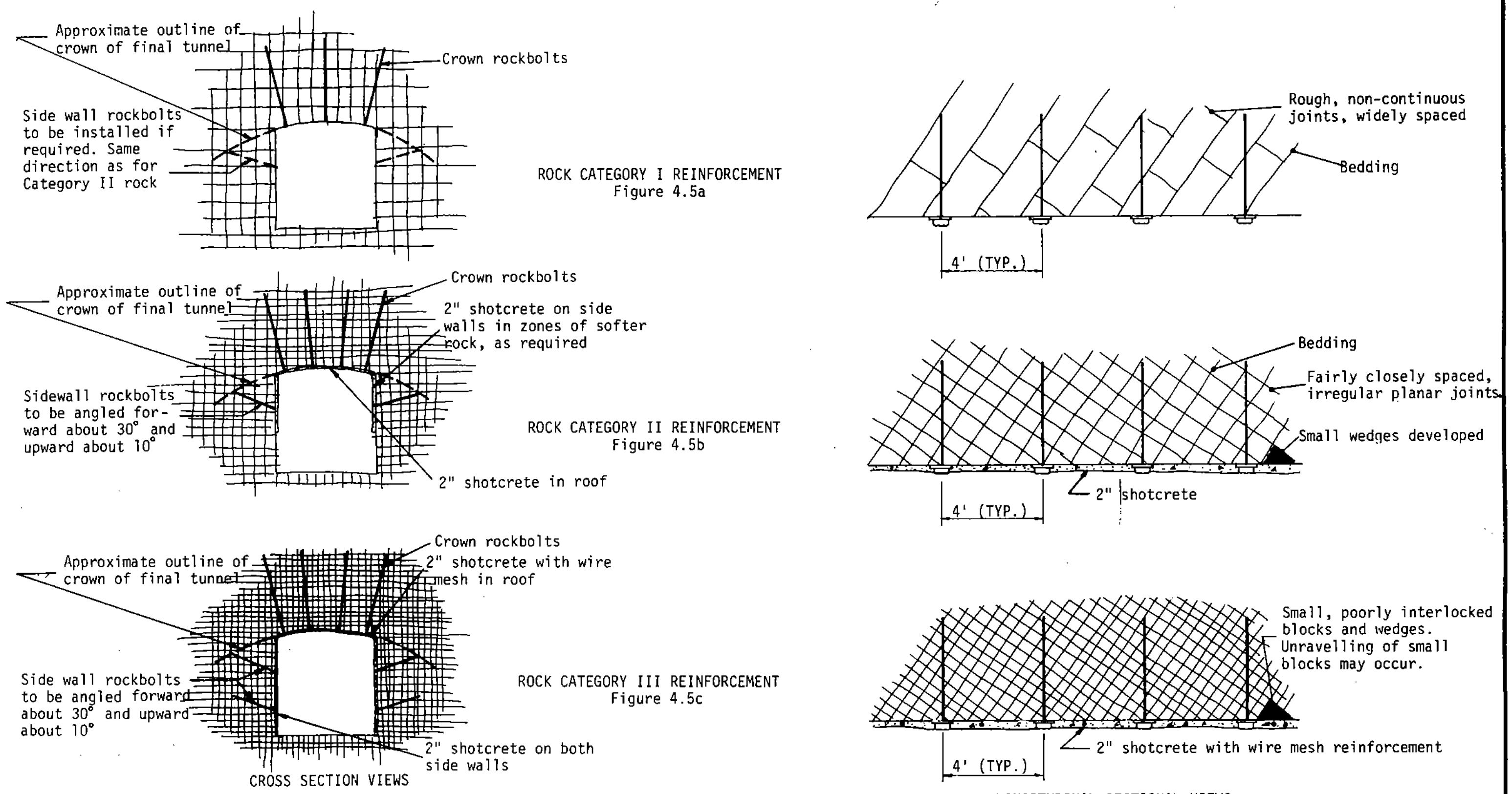
Figure 4. RELATIONSHIP OF NGI AND CSIR CLASSIFICATIONS



GENERAL NOTES:

1. All rockbolts to be one inch diameter, untensioned, fully resin grouted.
2. All rockbolts to be 8 feet long. Side wall bolts should not intersect outline of final tunnel crown.
3. Light steel sets may be required in addition to Rock Category III. reinforcement in vicinity of pilot bore portals.
4. Windows, nominal 3 feet by 3 feet, can be cut in shotcrete to expose rock for inspection at a later date.

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Figure 5. PILOT BORE ROCK REINFORCEMENT SCHEMATIC