AHTD TRC TRANSPORTATION RESEARCH COMMITTEE

TRC8702

Rock Buttress Wall Design for Rural Low-Volume Roads

Thomas J. Parsons

Final Report

1987

ROCK BUTTRESS WALL DESIGN
FOR
RURAL LOW-VOLUME ROADS

BY

THOMAS J. PARSONS

HIGHWAY RESEARCH PROJECT TRC - 8702

CONDUCTED FOR

THE ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

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16. Abstract

The objective of this research project was to investigate the various methods used for rock buttress wall design, adapt them for Arkansas soil conditions and make recommendations on how to implement the procedures. A survey was sent to all fifty states and several federal government agencies. Eighty percent of the states replied, and 14 states responded that they used rock buttress walls. However, none of the states used the walls as a retaining wall. A literature search revealed two design approaches which could be used for rock buttress walls. The first approach, the Indian approach, equates the wall's internal frictional forces to the soil's active forces. This approach is best suited for a uniform soil behind the wall. The second approach, Swedish slice method, uses a circular arc to model the soil failure plane. The soil's tangential, frictional and cohesive forces are equated to the wall's frictional forces to design the rock buttress wall. This approach should be used when the location of failure planes are known or when the soil is layered.

The present geometrical shape of the rock buttress wall used in Arkansas does produce an adequate wall. However, settlements and allowable bearing stresses should be considered in the foundation design along with the bearing stresses within the stones. Also, a new geometrical design should be considered for walls of about six feet or less in height. This design should have side slopes of 1:1 (H:V) to 1:2 and stone should be placed in the backfill area between the wall and soil as the wall is constructed.

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PREFACE

The development of a more comprehensive design procedure for rock buttress walls will be benefical to the AHTD. The present procedure has resulted in some failures. The College of Engineering, Agriculture and Applied Science of Arkansas State University (ASU) under contract to the Arkansas Highway and Transportation Department (AHTD), has performed a research program entitled "Rock Buttress Wall Design". The information contained in this report was collected and developed during this research project to assist engineers and field personnel in designing rock buttress walls. This report provides only the state-of-the-art technology related to rock buttress wall design.

ASU was awarded the AHTD research contract in 1986 to investigate various design methods used by other state highway departments and government agencies. Where possible, design methods were adapted for Arkansas soil conditions, based on field data collected by the AHTD. The engineering department at ASU has conducted this research with major emphasis on data collection and analysis. A relevant literature review and assessment of similar research has been conducted. Indications are that no other state has design procedures which will satisfy the design criteria for rock buttress walls as used in Arkansas. Representative examples of rock buttress wall designs are presented.

FINDINGS AND CONCLUSIONS

A survey was sent to all fifty states and several federal government agencies. Eighty percent of the states responded with 14 stating that they used rock buttress walls. None of the states used the walls as a retaining wall, instead the walls were used for erosion control, slide correction or prevention techniques. A literature search revealed two design approaches which could be used for rock buttress wall designs. The first approach, the Indian approach, equates the wall's internal frictional forces to the soil's active forces. The active forces are determined by the Rankine and Coulomb methods. The second approach, Swedish Slice method, uses a circular arc to model the soil failure plane. The soil's tangential, frictional and cohesive forces are equated to the wall's frictional forces in the design.

Site visits revealed that the quality of the stone used, choking (filling the voids between the stones used to construct the wall with smaller stones) and construction techniques play a major role in the wall's behavior. The stone should be hard to resist weathering and should not be subjected to high bearing stresses. These stresses could be controlled by choking. The visits revealed that when the walls are about six feet or less in height, the slopes tend to be 1:1 (H:V) instead of the 1:2 specified. This results from the construction technique used in the building of the wall (dumping of the stone).

The state survey revealed that Arkansas is the only state that uses the rock buttress wall as a retaining wall. The literature search revealed two design methods for rock retaining walls. The first method comes from India where rock buttress walls have been used extensively with success. Also, the Indian design method appears to give reliable results and is easy to use. A second method, Swedish slice method, has been used in Oklahoma. It should be used when the location of failure planes are known or in layered soil.

The present geometrical shape of the rock buttress wall used in Arkansas does produce an adequate wall. However, settlements and allowable soil bearing stresses should be considered in the foundation design. Also, the bearing stress within the stones should be calculated in order to determine height restrictions. Finally, a new geometrical design should be considered for walls of about six feet or less in height. This design should have side slopes of 1:1 to 1:2 and stone should be placed in the backfill area between the wall and soil as the wall is constructed.

IMPLEMENTATION

The geometrical shape of high rock buttress retaining walls should be maintained. The Indian design procedure should be used when the soil is uniform behind the wall. The Swedish Slice method should be used when the soil is layered or when the location of possible failure planes are known. A shear key should be considered when the resistance to foundation sliding is less than 1.5. In some cases, a subsurface investigation will be required in order to adequately design the rock buttress. The foundation bearing stress, settlement, and allowable soil bearing capacity should be determined. The bearing stress within the stone should be used to develop height restrictions. These design procedures should be used for walls 10 to 25 feet in height. Walls exceeding 15 feet in height should be designed in consultation with the geotechnical engineer. Walls in excess of 25 feet should be designed by the bridge or geotechnical engineer. For walls of six feet or less in height the geometrical slope should be changed. The side slope should be 1:1 to 1:2 and the backfill between the wall and soil should be the same rock as used in the wall. It should be placed as the wall is built. This would help simplify the construction techniques needed. Walls six to 10 feet in height should be constructed by present procedures. The Durability Absorption Ratio (DAR) should be included in the material specifications as a viable alternative for determining stone quality. This would ensure that sound stones are used in the walls.

The specifications should require choking of the walls. This would reduce stone bearing stress and eliminate any tensile stresses in the stone. The gradation of the choking stone should be determined by the AHTD. Maximum size of stone used in the wall should be no more than 1/3 of the wall's width at the level of placement.

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CHAPTER 1

INTRODUCTION

1.1 THE PROBLEM

The Arkansas Highway and Transportation Department has 14,807 miles of rural roads to improve and maintain within the state. A rural road program was recently enacted for the purpose of upgrading these roads. Every effort is being made to provide a safe road at a minimal cost. This is being achieved by minimizing construction costs.

Approximately 45% of the rural roads are in hilly and mountainous areas of the state. This type of terrain imposes right-of-way limitations which increasingly require retaining walls to support cuts and fills. Due to limited traffic volumes, concrete and proprietary walls cannot be justified. One reasonable alternative is the rock buttress wall. However, they have had limited usage because of the absence of a design procedure fully adapted to Arkansas conditions.

1.2 PROJECT OBJECTIVES

The research project consisted of three major objectives:

1. An investigation of the various design methods used by other highway departments and government agencies, such as the forest services, park services, Corps of Engineers, etc. The methods were reviewed for their applicability to the AHTD.

2. The feasible methods identified in objective 1 were evaluated and adapted for Arkansas' soil conditions considering the existing field data collected by the AHTD.

3. Recommendations were made which consisted of one or more design procedures that were made user ready for implementation. Also, any area where future research is needed to refine the design procedures was identified.

1.3 METHODOLOGY

In order to achieve the objectives of this research the following procedure was observed.

Objective 1. An investigation and determination of rock buttress wall design procedures was accomplished in the following manner:

A. A survey questionnaire was sent to all fifty state highway departments and several government agencies. The questionnaire requested information on rock buttress wall designs and design procedures.

B. A literature review was conducted to determine the latest research involving rock buttress wall design.

C. The information obtained from the survey was reviewed for procedures and specification which could be applicable to the AHTD.

Objective 2. Feasible design methods were developed in the following manner:

A. The feasible design methods identified in objective 1 were adapted for Arkansas soil conditions. A design example is presented for each approach identified.

B. Procedures to estimate the stresses within the rocks and resulting height limitations were developed.

Objective 3. Recommendations were developed in the following manner:

A. Recommendations were made with respect to the use of two design approaches to rock buttress walls; the Indian method and the Swedish slice method.

B. Recommendations were made with respect to height limitations and procedures were established to limit wall heights based on the compressive strength of the stones used to construct the wall.

C. Research areas were identified which are needed to refine the design procedures presented.

1.4 LITERATURE REVIEW

1.4.1 Background

Landslides and fills on hill sides along highways have been a major problem for centuries. One of the early means of controlling slides and containing fills was using rock walls. Due to improved construction techniques and materials, other methods of controlling soil movements have been developed, such as concrete retaining walls, cribs, gabions, reinforced earth and others. They have replaced the rock walls but have proven to be expensive and time consuming to build. Due to increased costs in recent years, there has been an effort to find a less expensive means of controlling soil movements along highways and secondary roads. One technique being used is rock buttress walls. The "rock buttress" is a free-draining gravity structure consisting primarily of large blocks of non-degradable sandstone or limestone (1). These walls have been used by several highway departments and federal government agencies. They are less expensive, use natural material and blend into the surrounding landscape.

1.4.2 State of Tennessee

The state of Tennessee has had several landslides along Interstate 40 near Rockwood. One means of controlling these slides is rock buttress walls. The walls are designed to be free-draining gravity structures. The stone used is a non-degradable sandstone or limestone. Fifty percent of the material is greater than 1 cubic foot and no more than 10 percent passing a No. 2 mesh sieve (1). Soil movement is prevented by restraint and since the wall is free-draining, the likelihood of "ponding" and pore pressure buildup is greatly reduced. The greatest disadvantage of a rock buttress is the wide base since it requires more area than other methods for controlling landslides. Also, the wall has to be placed below the colluvium in order to be effective, due to the mass of the structure.

1.4.3 Forest Service

In the article by Carlton Yee, the use of small rock buttress walls by the forest service is described (2). The walls are used to control slope failures by placing them at the toe of prospective slide areas. The wall is designed so the mass or weight of the wall counters the tendency of the soil to slide by providing a resisting moment. Also the rock provides greater shear stability. The wall is built by first digging a trench at the toe of the prospective slide area below the plane of sliding. One rule of thumb for design presented is "on a volume basis, it is often felt that the buttress should equal at least one-fourth or one-third the volume of unstable soil being retained" (2). Another rule is "every cubic yard of soil removed is replaced with 1.5 cubic yards of rock". No other design considerations were presented as to what is meant by small rock buttress walls.

1.4.4 Transportation Research Board

In the Transportation Research Board and National Academy of Science's Transportation Technology support for developing countries "Synthesis 2 - Stage

Construction" the use of "dry stone retaining walls" for supporting roadways in mountainous regions is described (3). The wall should be constructed so that "the stones are in contact with each other and so that their longest dimension is perpendicular to the embankment. Larger stones should be placed at the bottom of the wall. Voids between the larger stones must be filled with small stones". The backfill behind the wall should be compacted and proper drainage provided. The walls are restricted to one meter in height and the suggested side slopes are given in Fig. 1.1. Walls over one meter in height were mortared and stepped as given in Fig. 1.2.

1.4.5 Washington State - Rockeries

The state of Washington permits the use of rock retaining walls as stated in section "342 Retaining Walls" of their specifications. "The rock walls are essentially gravity walls made of stacked large rock, used primarily in cut sections where very good soil exists" (4). They are used to provide erosion protection and limited earth support. The height is limited to 15 feet in cut sections and 10 feet or less in fill sections. Rock walls over five feet high must be designed by the Bridge and Structures Branch. A typical wall is presented in Fig. 1.3.

The report "Uses and Abuses of Rockeries" (5) states that probably the most important reason rockeries are built is because they are less expensive than retaining walls. In Tacoma, Washington they cost about 1/4 the expense of conventional reinforced concrete retaining walls. Also, construction time is much less on rockeries, about one week for a small job. The design of rockeries is based on experience. The report stated that "our general philosophy has been to not recommend the use of rockeries unless we are fairly confident that the slope is stable without one." Table 1.1 presents some general guidelines for design of rockeries based on experience (5).

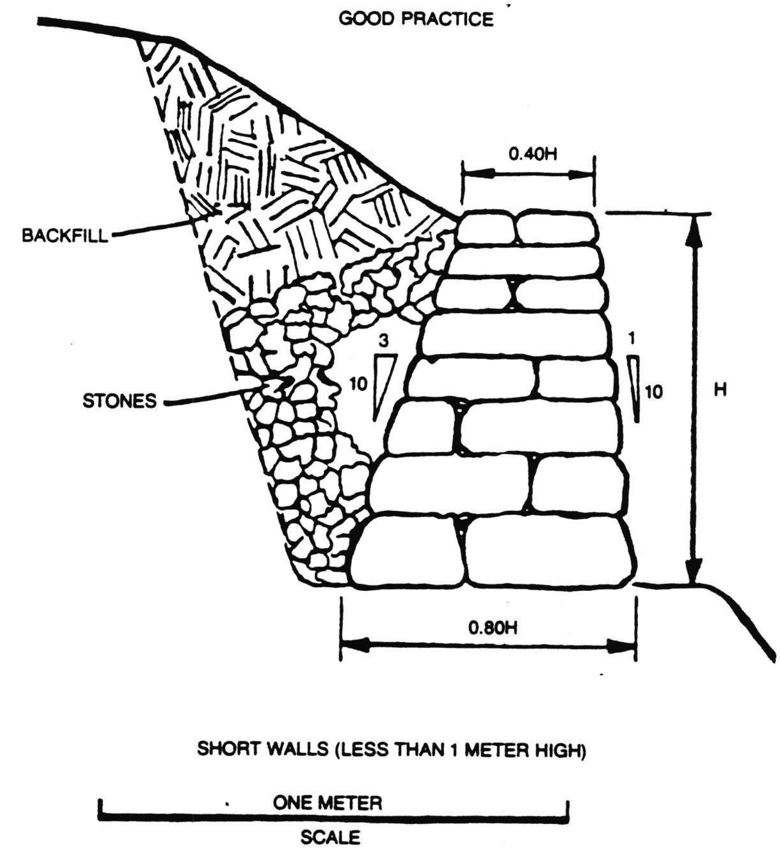


Fig. 1.1 Low Retaining Wall Design

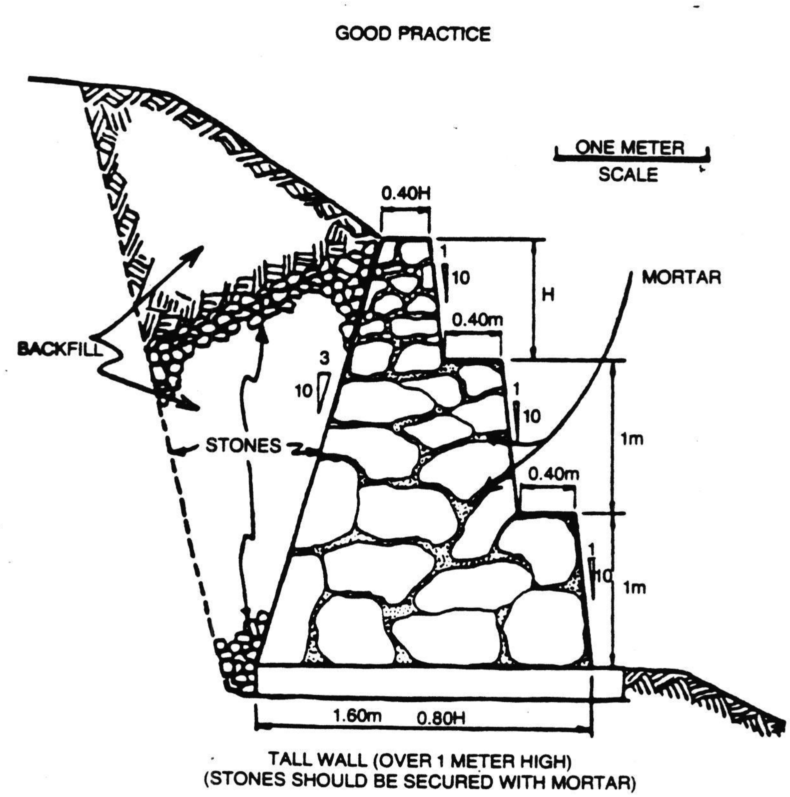


Fig. 1.2 High Retaining Wall Design

		A	B	C	D	E
1.	Source	State	County	County	City	City
2.	Title of Specs.	Rock Walls/Rock Retaining Walls	Rockeries/Rock Retaining Walls	Rock Walls	Rock Retaining Walls	--
3.	Max. Wall Height	--	8 - 12 Ft.	--	~10 Ft.	~12 Ft.
4.	Min. Base Thickness	3 - 4 Ft.	~4 Ft.	--	--	--
5.	Steepest Wall Slope	12V on 2H	B1 12V on 2H B2 12V on 3.5H B3 12V on 4.0H	--	12V on 3H	12V on 2H
6.	Max. Ground Slope Above Top of Wall	--	--	1V on 1.5H	--	1V on 2H
7.	Rock Unit Wgt. (Min.)	155-164 pcf	160 pcf	--	165 pcf	--
8.	Variance from Plane of Slope	0.3-0.5 Ft.	--	--	--	--
9.	Min. Foundation Embedment	FILL: 1+0.8(z)Ft. (z=Ht.-5) CUT: 0.5-1.0 Ft.	0.5-1.0 Ft.	0.3 Ft.	1.0 Ft.	1.0 Ft.
10.	Min. Thickness of Drainage Layer	12 in.	9 in.	--	12 in.	--
11.	Wall Ht./Rock Wgt. Relationship	FILL / CUT				
	Lower 0-3 Ft.	1600 lbs. / 1200 lbs.	4 mr	60%>1 CF	2-3 mr	5-6 mr
	3-6 Ft.	1200 lbs. / 1200 lbs.	4 mr		1-2 mr	5-6 mr
	6-9 Ft.	800 lbs. / 800 lbs.	3 mr			2-4 mr
	9-12 Ft.	400 lbs. / 100 lbs.	2 mr			2-4 mr
	Upper 12-	-- / --				
12.	Rock Size Definitions (mr)		B1 / B2			
	1 man rock	--	400 lbs. / --	--	50-200 lbs.*	400 lbs.
	2	--	800 lbs. / 300-600 lbs.	--	200-600 lbs.*	800 lbs.
	3	--	1200 lbs. / 800-1200 "	--	600-1200 lbs.*	1200 lbs.
	4	--	1600 lbs. / 1500-2200 "	--	1200-2200 lbs.*	1600 lbs.
	5	--	2000 lbs. / --	--	2200-4500 lbs.*	2000 lbs.
	6	--	2400 lbs. / --	--	-- *	2400 lbs.

ABBREVIATIONS:

mr	man rock
lbs.	pounds
CF	cubic feet
Ft.	feet
in.	inches
V	vertical
H	horizontal
pcf	pounds per cubic foot
HT.	wall height

*1 man rock	(.4-1.5 CF)
*2	(1.5-4.6 CF)
*3	(4.6-9.2 CF)
*4	(9.2-16.9 CF)
*5	(16.9-34.6 CF)
*6	--

Table 1.1 Summary of Rockery Design Criteria

[As obtained from Gifford and Kirkland (5)]

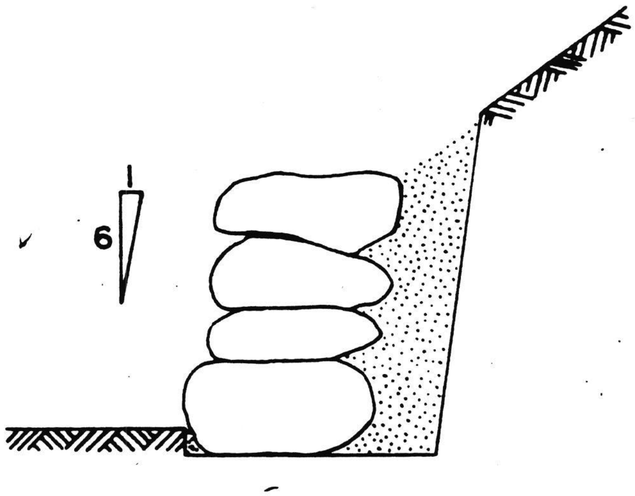


Fig. 1.3 Typical Rockery Wall

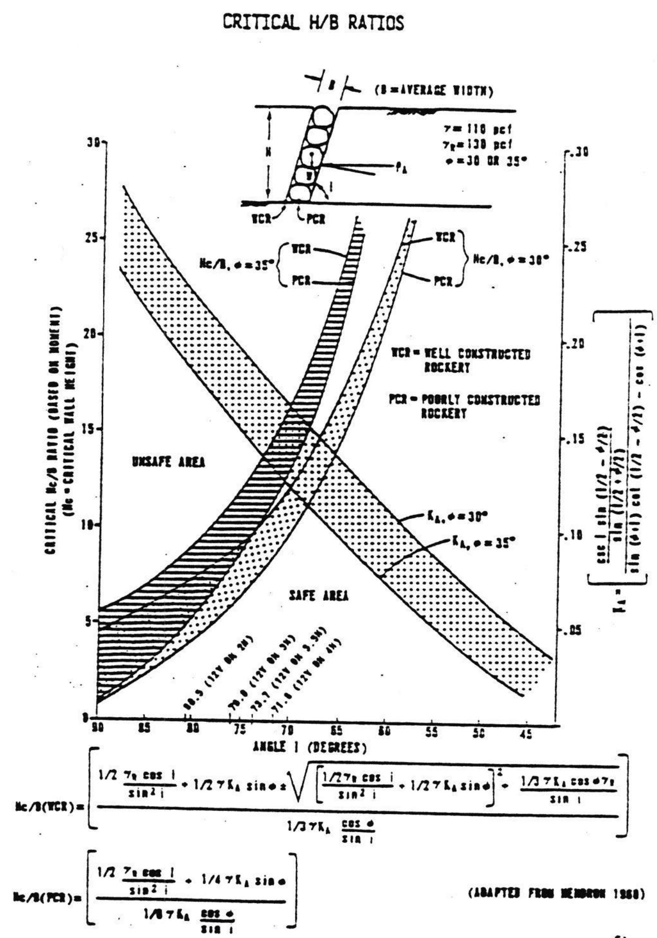


Fig. 1.4 Rockery Design Curves

Also included is a rational approach for rockery design. It states "the theoretical analysis of the stability of a rockery is difficult because so much depends on workmanship of the job" (5). The rational approach uses generalized equations for active earth pressure and it is found that "a slight reduction in the angle of inclination of the wall resulted in a considerable reduction in active earth pressure coefficient and, therefore, the acting earth pressure force" (5). Critical wall height to thickness ratios are determined for various angles of wall inclination based on overturning and sliding type failures. It is found that the moment forces tending to cause overturning result in the critical mode of failure. Critical wall height (Hc) to average thickness (B) curves are developed for different wall inclinations and internal friction angles of backfill or natural soil. The curves are presented in Fig. 1.4. They were developed for a wall with a maximum thickness of four feet and an average thickness of three feet. This would result in a theoretical critical wall height of about 12 to 36 feet depending on the slope of the face. Most walls observed are about 18 to 20 feet high and it was stated that "many rockeries have failed when greater than about 15 feet in height".

Wall thickness is not always specified since it is a function of the size of rock used. The term "man-rock" refers basically to weight of the rock. For example, a two-man-rock is generally the maximum weight of rock that two men can move in place using steel pry bars or about 800 pounds. Common sizes used range from one to six man rocks or 400 to 2400 lbs.

Wall construction consists of the following steps:

1. Foundation preparation.
2. Placement of rocks with a backhoe or small hydraulic crane and steel chairs. The long axis of the rock is placed horizontally or slightly tipped into the slope. The rocks are placed to maximize the amount of tons per square foot of surface area.
3. In most cases a granular filter blanket is placed behind the wall to provide drainage.

Material specifications for the rock state that the rock should be "hard, sound, durable and free of segregation seams, cracks, or other defects tending to destroy its resistance to weathering and cracking". A minimum specific gravity of 155 to 165 pounds per cubic foot (pcf) is often suggested.

The backfilling of the wall includes the chinking (choking) and wedging of openings between larger rocks with aggregate up to six or eight inches in size to control erosion of the rock fill material. Also, a drainage blanket consisting of a well-graded sand and gravel is used. The fill should be solid, tight and free of voids. An example of a suggested rockery detail is given in Fig. 1.5.

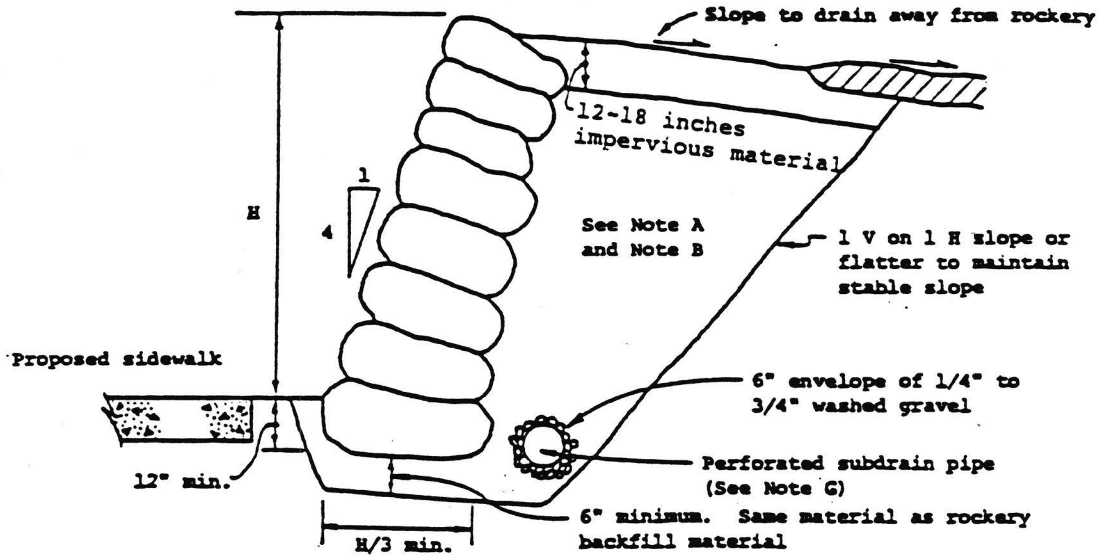
The six basic reasons for rockery failures in decreasing order of frequency are:(5)

1. Too little or no drainage.
2. Poor quality and/or poorly placed backfill.
3. Wall too steep or too high.
4. Poor foundation.
5. Unsound rock.
6. Poor workmanship.

For rockery design, a face slope of 12V to 3H and a maximium height of 15 feet is recommended. If a higher wall is used, the wall should be benched. Also, the overall slope stability should be checked. Settlements should be avoided because the rocks tend to lose contact with each other and shear resistance is reduced. It was emphasized that rockeries afford little or no resistance to slope movements. Therefore, the height should be limited to 15 feet, maximium wall slope should be 12V to 3H, the base width should equal 1/3 the height and good drainage behind the wall should be provided.

1.4.6 Baker and Marshall

A means of controlling landslides is to place restraining structures or a rock buttress at the toe of prospective slides. Empirical relationships are



Not to scale

A. Rockery backfill material to consist of clean, well-graded pit run sand and gravel, 2-inch maximum size with at least 40% gravel (plus No. 4 sieve material). The amount of fines (minus No. 200 mesh size) shall be less than 3%. The fines shall be non-plastic.

B. Backfill material shall be compacted in 6-inch layers with 4 coverages of a hand-operated gasoline-driven tamper.

C. The excavation for the rockery shall be kept free of water and shall be evaluated by an experienced soils engineer prior to placement of the pit run sand and gravel. Rockery base material shall be placed on firm undisturbed ground.

D. If loose or soft materials exist at the base rock location, they shall be removed and replaced with the clean pit run sand and gravel and compacted as recommended in Note B.

E. Large holes between rocks on the backside of the rockery should be filled with gravel or rock spalls to retain the pit run sand and gravel backfill.

F. The base rock shall have a minimum base width of H/3, where H is the height of the rockery. The base rock and other rocks shall also meet the following weight requirements. H shall not exceed 12 feet.

	Minimum Weight of Rock
Lower 6'	2400 lbs.
Upper 6'	1600 lbs.

G. Asphalt parking area should be sloped to drain away from the rockery. Subdrain will be required to collect water that may flow in the base course material beneath pavement if the parking area slopes toward the rockery.

Fig. 1.5 Rockery Design Detail

presented by Baker and Marshall for the design of the restraining devices used to control active slides (6). It is suggested that the rock buttress should be 1/4 to 1/3 the volume of total moving mass to be retained and should extend at least five to 10 feet below the slip plane unless stable bedrock is encountered (6).

1.4.7 Swedish Slice Method

A design method for a rock buttress retaining wall is presented by Baker and Yoder based on the Swedish slice method (7). A circular failure plane is assumed behind the wall. The arc is divided into equal segments and normal and tangential forces are calculated for each segment. The resisting force required by the wall to maintain equilibrium is found by:

$$P_R = F.S. \, (\Sigma T_{soil} + \Sigma T_{wall}) - \Sigma N \tan\phi - C\,L$$

Where

P_R = the resistance required by the wall

ΣT_{soil} = the sum of the tangential forces of the soil mass behind the wall.

ΣT_{wall} = the sum of the tangential forces in the wall.

$\Sigma N \tan\phi$ = the sum of the normal forces times the angle of internal friction of the soil mass.

C L = the cohesion in the natural soil times its length of the arc.

Three modes of failure are anticipated:

1. Friction or shear failure between the buttress and the foundation.
2. Foundation failure beneath the buttress.
3. Shear through the buttress.

The horizontal shear resistance through the buttress is given by:

$$P_R \cos \alpha = \gamma_B A_B \tan \Phi_B$$

Where

α = angle formed by the tangent to the slip-surface and the horizontal at back of buttress. This angle should be at least 10^o

γ = unit weight of the buttress in pcf.

A_B = cross sectional area of the buttress in ft^2 per unit width

ϕ = angle of the internal friction for the rock in the buttress.

From this, one can find the buttress base area and size the buttress. The shear resistance between the buttress base and soil is checked by the following:

$$P_R \cos \alpha = \gamma_B A_B \tan \phi_s + C_S L_B$$

Where

ϕ_s = angle of internal friction for the foundation soil.

C_S = unit cohesion of the natural soil

L_B = length of the buttress

If needed, the buttress dimensions could be modified.

Once the buttress is sized, the stability against a shear failure through the buttress needs to be checked by:

$$F. S. = (\Sigma N \tan \phi_s + C_S L_B + \Sigma N \tan \phi_R)/(\Sigma T_{soil} + \Sigma T_{wall})$$

1.4.8 Indian Method

Another rock buttress wall design is presented by Arya and Gupta (8). The paper describes the rock buttress design procedure used in India. The following design criteria is given: (8)

(a) There should be no overturning of the wall as a whole or any part of it. According to Indian Standards (IS): 1904, the minimum factor of safety against overturning is specified as 2.0 under normal loads. Under earthquake condition as per IS : 1893 -1975, the factor of safety should be 1.5 or more.

(b) The pressure at the toe should remain less than the safe bearing capacity of foundation soil or rock. The factor of safety with respect to ultimate bearing capacity is kept as 3.0 under normal loads. Under earthquake condition, the allowable bearing pressure may be increased by 25 to 50 percent (IS : 1893).

(c) The sliding or shearing stress should remain less than the safe value of the shear or sliding resistance. A factor of safety of 1.75 under normal loads and 1.33 under earthquake condition is generally adopted, both at the base and for intermediate layers.

A discussion of using mortar bands in walls over 12 feet high is presented, and it was concluded that they provide no additional benefit. It is stated, for a wall to act integrally as one unit, the stones should be roughly rectangular in shape. Also there should be sufficient overlap on each other for interlocking of the stones.

A discussion was conducted on the slope of the foundation with respect to the horizontal. The stones should be sloping toward the soil or backfill. The force to produce sliding for a 1V: 3H foundation slope is two to three times the force needed when the stones are horizontal. This force is reduced as the coeficient of friction between the stones increased. The force to produce sliding is shown in Fig. 1.6 and are expressed as:

$$F \geq u\, W/(\cos\delta - u \sin\delta)$$

for a horizonal foundation, and

$$F \geq W\,(u \cos\theta + \sin\theta)/[\cos(\delta+\theta) - u \sin(\delta+\theta)]$$

for a foundation inclined toward the soil or backfill.

Where:

u = coefficient of stone

W = weight of stone

δ = angle of the force F with respect to the horizonal

θ = angle of the slope

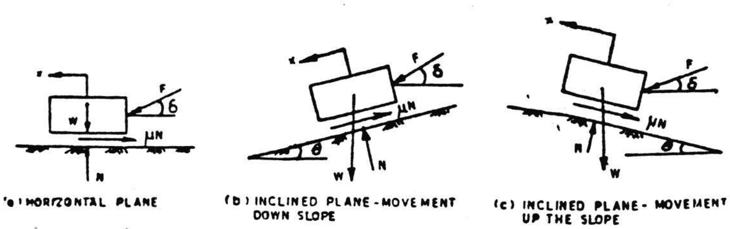


Fig. 1.6 Wall Friction Forces

The back pressure acting on the wall depends on 1) the angle of internal friction, 2) density, and 3) water content of the fill material at the back of the wall. The soil lateral pressure could be determined by Coulomb's theory. Values for internal friction of different soils is given in Table 1.2 (8).

Table 1.2 Values of Angle of Internal Friction

Grain Size	*State of Compaction*	*Value of Ø (degrees)* Rounded Grains, Uniform Gradation	*Value of Ø (degrees)* Angular Grains Well Graded
Sand and Gravel	Loose	34	39
	Moderately Dense	37	41
Blasted rock fragments	as blasted		40 - 50

The angle for clay or expansive soil should be determined from laboratory tests or where the angle of repose at the site of the soil is stable. The angle of wall friction is assumed to be 22.5^{0}. It was stated an angle of $27\text{-}30^{0}$ would be nearer to reality. Illustrations of wall contruction techniques are presented in Fig. 1.7.

Some of the common causes of wall failures are:

(1) Construction of walls just after hill cutting is completed. The slope needs to restablize after the cutting is done. This generally requires one rainy season.

(2) Improper construction of the wall.

(3) Improper backfill. The backfill behind the wall may not be free draining and there could be inadequate quality control.

(4) Improper drainage. The weep holes get clogged or are too small.

(5) Seismic action.

In the construction of the wall it is recommended that:

(1) There be a minimum base slope of 1V in 6H.

(2) Rough flat stones should be used and the size of the stones should be greater than 225 x 110 x 75 mm (weight about 5kg). The largest dimension should be placed across the length of the wall. Also the voids in the wall should be filled.

(3) The backfill should preferably be done by hand-packing to achieve the maximum angle of internal friction. The backfill must be non-cohesive and free draining. The top layer, 300 mm thickness, should be as impervious as possible.

(4) On high walls, a large opening in the wall or scupper should be provided four meters below the road surface and at horizontal spacings of six meters on centers.

(5) If there is falling water, a toe should be provided to prevent erosion at the base of the walls.

(6) Walls up to six meters high may be constructed by round or egg-shaped boulders. The boulders should have a maximum area of contact and a inward slope of bedding planes. The joints and voids between the stones should be filled with a granular material and at least 50 percent of the stones should have a weight greater than 10 kg.

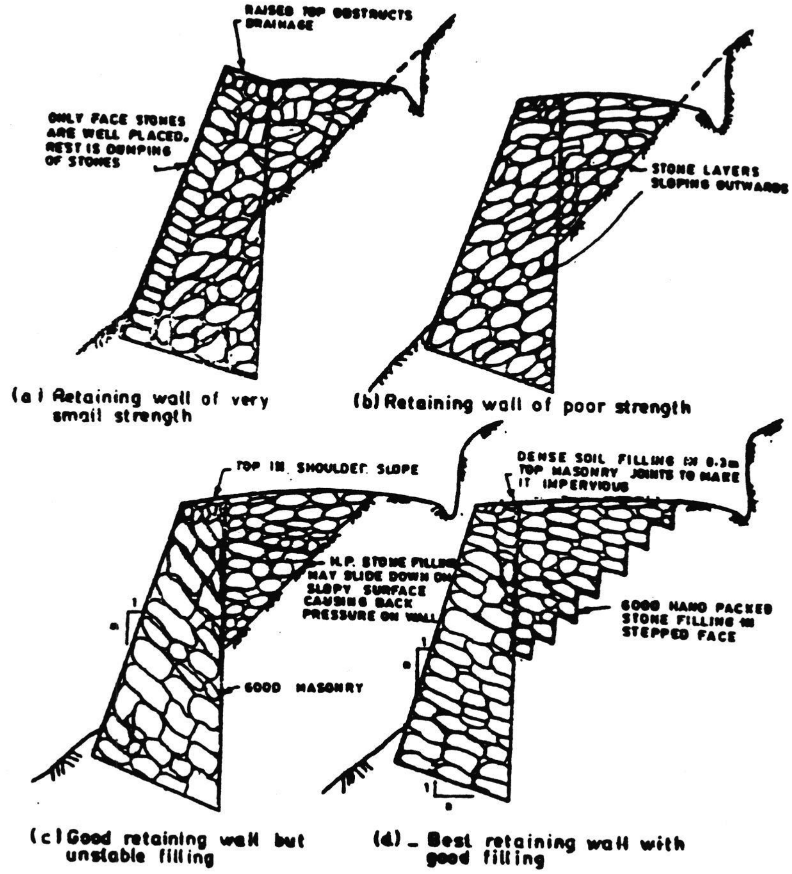


Fig 1.7 Illustrations of Wall Construction

1.4.9 Durability Absorption Ratio

The selection of the rock to be used in the rock buttress is a major factor in the wall's performance. The rock should be durable, hard, sound and free of segregation seam, cracks or other defects. The study "Evaluation of Rock Slope

Protection Material" (9) reviews several of the tests used to evaluate rock and compared these tests to the rock's field performance. The best correlation between the laboratory test and field performance was the durability absorption ratio (DAR). This ratio produced a 97% agreement between the lab and field. The DAR is determined by the following:

DAR = Durability Index/(Percent Absorption + 1)

The following specifications are used:

1. DAR greater than 23, material passes
2. DAR less than 10, material fails
3. DAR of 10 to 23 and (a) durability index 52 or greater material passes; and (b) durability index 51 or less, material fails.

The absorption test used is specified by test method No. Calif. 206-D and the durability index by test method No. Calif. 229-E. The test procedures are presented in Appendix C.

CHAPTER 2

FIELD OBSERVATIONS

2.1 INTRODUCTION

Several sites in Arkansas where rock buttress walls have been constructed were visited. These sites included: (1) a wall failure, Job #R8009, Mt. Levi Rd -Fort Douglas, (2) a wall to support the roads up-hill slope, Job #8894, Ben Hur -Raspberry Mtn.- slide repair, (3) a wall subjected to the test of time, Job #R50024, Hwy 66 S - Sylamore Creek and (4) a new wall supporting a roadway, Job #R90012 Landis - Hwy 66.

2.2 Job #R8009, Mt. Levi Rd - Fort Douglas.

The wall was constructed in the summer and fall of 1985. It was several hundred feet long and 28 feet at the highest point. The side slopes were 1/2:1 and about five feet of overburden was placed on top of the wall to serve as the base for the Mt. Levi Rd. The stone used to construct the wall was sandstone from a local quarry.

In February 1986, the wall failed. The failure was confined to approximately 100 feet of wall at the point of greatest height. There was about five feet of settlement across the top of the wall and a lateral displacement of 5 to 10 feet. The side slope changed to about 1:1 in the failure zone.

Repairs had been made to the wall by at the time of the field visit in August 1986. The following observations were made:

1. The sandstone had weathered to a point where pieces could be turned to sand by mild blows. See Fig. 2.1

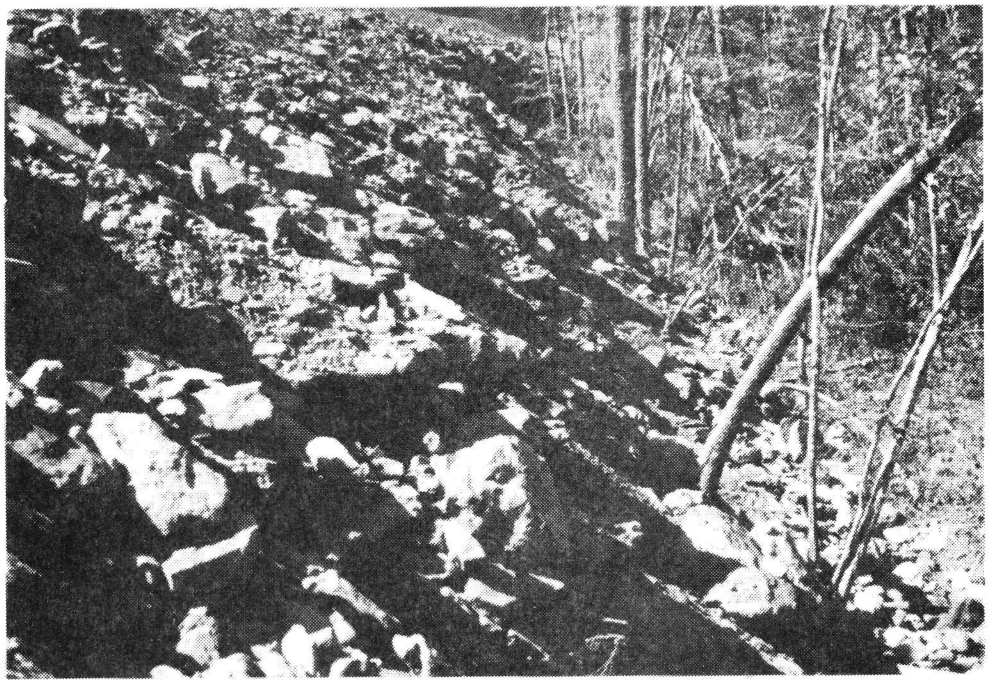


Fig. 2.1 View of Weathered Stones.

2. Some of the stones had cracked due to the imposed loads. See Fig. 2.2

Figure 2.2 Cracked Stone in the Wall.

3. The stones were not choked. The lack of choking changes the load imposed on the stones from a uniform load to a concentrated load. In some cases, the bearing area was about 10 percent of the surface area.

4. Different qualities of sandstone were present in the wall. A stone, brown in color, was present in the upper levels of the wall. This stone appeared to be hard and to resist weathering. A second stone, gray in color, was present in the lower portion of the wall. This stone turned to sand under mild blows. It appeared to be soft and not resist weathering.

2.3 Job #8894, Ben Hur - Raspberry Mtn.: Slide repair.

The rock buttress wall observed at this site was under six feet in height. It was primarily used to support the soil on the uphill side of the road. The observed side slopes were about 1:1. It is believed this slope resulted from the construction technique. The stones were dumped into place, which tends to leave a 1:1 slope. While talking to the personnel at the site, a wall construction problem was disclosed. It was learned that it was difficult to place the backfill once the wall was constructed. This resulted because there was not enough room for heavy equipment between the wall and soil. It was suggested to construct the backfill out of rock and place it as the wall was being constructed. Also, it was suggested to reduce the side slope.

2.4 Job #R50024, Hwy. 66 S - Sylamore Creek.

The rock buttress wall was built some time ago. It was about 15 feet in height and had side slopes of 1:1. The rocks used appeared to be limestone and were round. The wall was overgrown with vegetation and there was no appearance of instability.

2.5 Job #R90012, Landis - Hwy 66.

The rock buttress wall was about 16 feet in height and nearly vertical, see Fig 2.3. The wall was constructed of brown sedementary rock, hard sandstone, which was large and flat. The length of the stone was about three times its thickness. The stones were not choked and the wall appeared to be performing well.

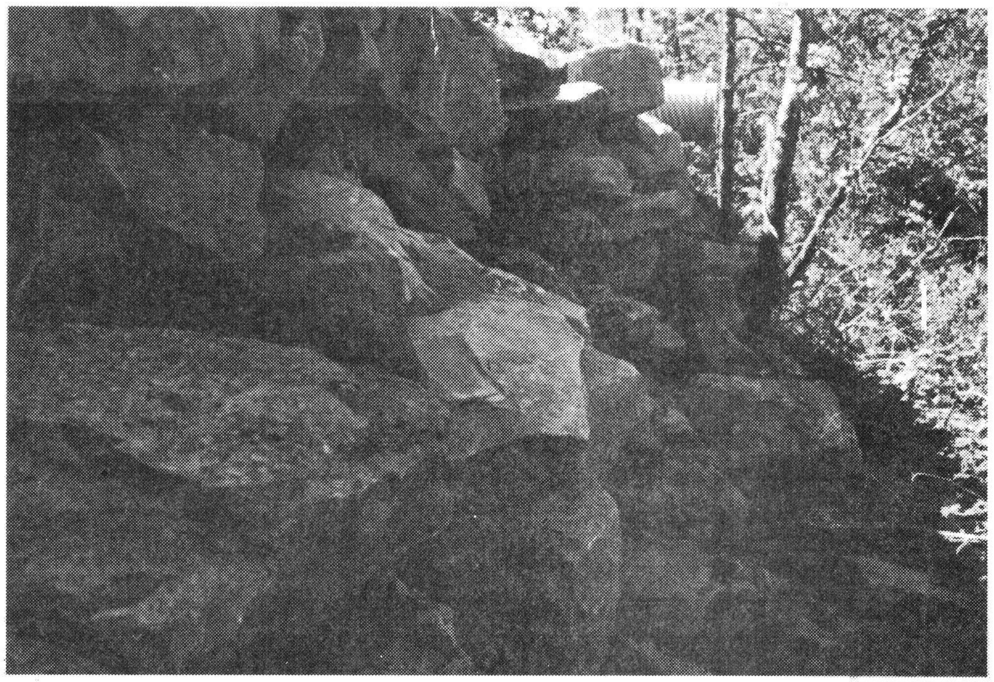


Fig. 2.3 Landis - Hwy 66: Wall

CHAPTER 3

STATE SURVEY

3.1 SURVEY OBJECTIVE

One of the main objectives of this research project was to survey other state highway departments and federal government agencies to:

1. Determine if they use rock buttress walls
2. Obtain information on rock buttress wall design procedures.

3.2 SUMMARY OF RESPONSES

A questionnaire was sent to all fifty state highway departments and several federal government agencies. The information obtained from the states with favorable responses was reviewed and summarized. Table 3.1 summarizes this information.

ALABAMA: Rock buttress walls are used as a slide correction technique. The design procedure consists of a wedge/slice analysis with material properties of:

ϕ, angle of internal friction = 40^{o}

γ, weight of soil = 140 lb/ft^3

c, cohesion = 0.

The walls are designed for active soil pressures and have side slopes of 1:1.

CALIFORNIA: The department does not use rock buttress walls as discussed in this project. They did send information on concreted rock walls used as slope protection. Also, they provided information on stone quality requirements for slope protection as given in section 72-2.02 of their

TABLE 3.1 STATE RESPONSES

Highway Dept.	No Response	Negative Response	Positive Response
ALABAMA			X
ALASKA	X		
ARIZONA	X		
ARKANSAS			X
CALIFORNIA			X
COLORADO		X	
CONNECTICUT		X	
DELAWARE		X	
FLORIDA		X	
GEORGIA			X
HAWAII		X	
IDAHO		X	
ILLINOIS		X	
INDIANA		X	
IOWA			X
KANSAS		X	
KENTUCKY			X
LOUISIANA		X	
MAINE		X	
MARYLAND	X		
MASSACHUSETTS		X	
MICHIGAN			X
MINNESOTA		X	
MISSISSIPPI			X
MISSOURI		X	
MONTANA	X		
NEBRASKA		X	
NEVADA		X	
NEW HAMPSHIRE			X
NEW JERSEY		X	
NEW MEXICO		X	
NEW YORK			X
NORTH CAROLINA		X	
NORTH DAKOTA		X	
OHIO		X	
OKLAHOMA			X
OREGON			X
PENNSYLVANIA		X	
RHODE ISLAND			X
SOUTH CAROLINA	X		
SOUTH DAKOTA		X	
TENNESSEE	X		
TEXAS		X	
UTAH	X		
VIRGINIA	X		
VERMONT		X	
WASHINGTON			X
WEST VIRGINIA	X		
WISCONSIN			X
WYOMING	X		
GUAM		X	
WASHINGTON D.C.		X	

standard specifications. They are as follows:

Apparent Specific Gravity - 2.5 minimum

Absorption - 4.2% maximum

Durability Index - 52 minimum

and if the DAR is more than 24

Absorption may exceed 4.2%

Durability Index may be less than 52

GEORGIA: The department uses rock buttress as a toe treatment for slide protection in rugged terrain or to steepen slopes when rock is readily available. The criteria to steepen slopes is presented in Fig. 3.1.

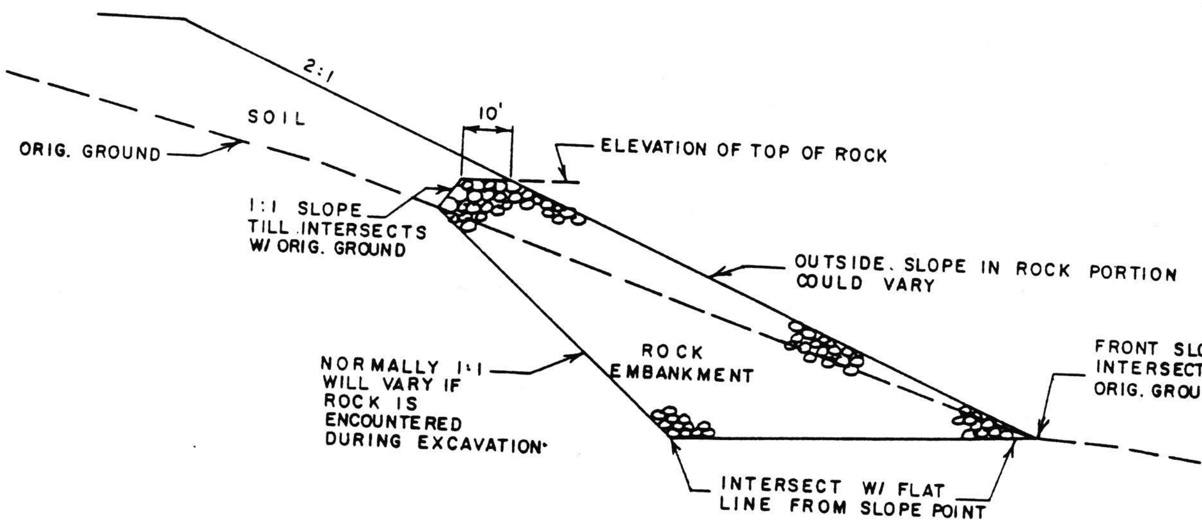


Fig. 3.1 Georgia Rock Embankment Detail

IOWA: The department has used the rock buttress concept to repair landslides for approximately 10 years. Each design is site specific and they have no standard written procedures. The design procedure used is based on reinforcing the existing materials structure. The analysis is performed by utilizing "sliding block" and "bishop" circular analysis programs. Minimum rock facing thickness of 10 feet is required at the most critical indicated failure surface and a minimum five feet on the highest possible failure surface. The rock is well graded with a maximum eight inch top size, and no more than 10% passing the #8 sieve. The exposed slope has a slope of 1-1/2 H to 1 V.

KENTUCKY: The department utilizes rock buttresses extensively. They are constructed as random shot rock fill. The walls may consist of a portion of the embankment toe area or toe berms to effectively flatten the slope with or without shear keys. They are designed by conventional slope stability analysis procedures.

MICHIGAN: The department utilizes rock buttresses as fills, "armored slope", and shoreline protection. The armored slope is a rough triangle of mine rock, which is one to two feet in size with and all faces sharp and broken. The wall is about eight feet high and 10 feet on the leg and is used to support wet sandy slopes. A geotextile is used on the backside. Fills are end dumped mine rock with the outer slope holding at somewhere between 1:1 and a 1:1.5 (H:V) slope. The top is choked with smaller rock, a sand subbase placed and paved. The shoreline protection is similar to the armored slope but with stone sizes varying from over three feet with a one to two foot stone cover of smaller bedding stone. On the backside, a geotextile is placed.

MISSISSIPPI: The department utilities rock buttress as slope protection or corrective measures. The analysis is conducted utilizing LEAST or MIT slope stability analysis. A shear key is placed at the base of the wall and a maximum stone size of 300 lb is placed at the bottom and four inch to 3/4 inch at the top of the wall. The wall has an outer slope of 1:1 to 2:1 (H:V). An internal shear angle of 45^{0} for analysis purposes is used for the rock in the wall.

NEW HAMPSHIRE: The department stated that rock buttress walls do exist in the state, but they have not designed or used a natural stone gravity retaining wall in any modern design for a considerable time. They did provide some information used by the Boston and Maine Corporation. It is enclosed in Appendix B.

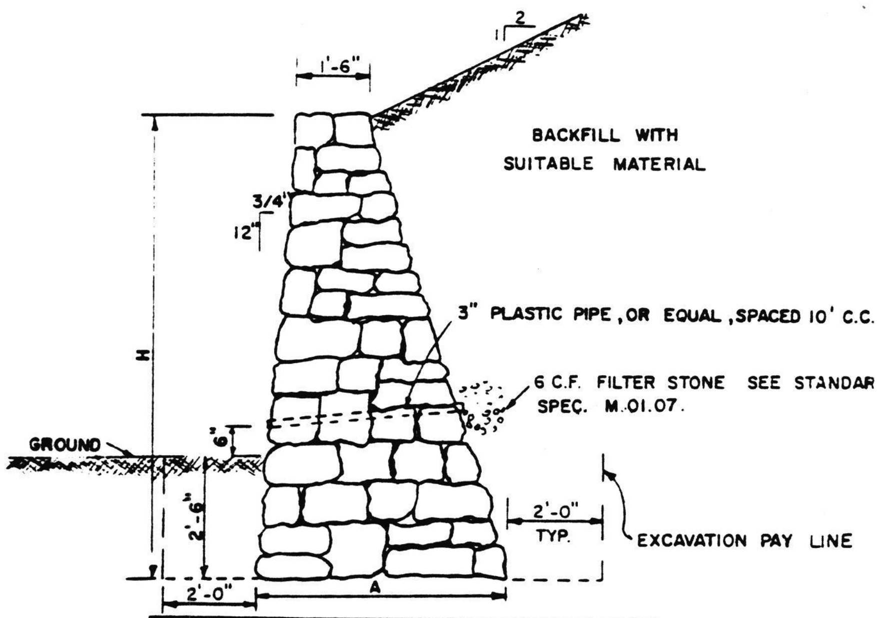
NEW YORK: The state does not presently use rock buttress walls but did provide some specifications used for repair and short extensions on some existing walls. The stones have to be clean, roughly rectangular and sound field or quarry stone. Four-fifths of the stones have to be over one-third cubic feet and face stones had a minimum thickness of two inches and a width of ten inches.

OKLAHOMA: The department has used rock buttresses in the past, primarily in shallow slide restoration contracts. The general analysis procedure used is outlined in the Highway Research Board Special Report 29 for earth buttresses. The material specified for the wall is either native stone or crushed rock. It has to meet their standard rip-rap specifications along with weight and absorption requirements. These tests are determined in accordance with ASTM C97. The minimum weight accepted is 140 pounds per cubic foot and the maximum absorption permitted is six percent. Specific site specifications are given in Table 3.2.

OREGON: The department uses rock buttress walls to stabilize landslides. Typically, the outer slope of the walls are 1.5:1 to 2:1 (H:V). The size of

Table 3.2 Oklahoma Rip-rap Specifications

Riprap Thickness Inches	(cm)	Maximum Pounds	(kg)	Average Size Pounds	(kg)	Not More Than 20 Percent Shall Weigh Less Than	
12	(30.5)	150	(68)	30–50	(14–23)	20	(9)
18	(45.7)	350	(159)	70–125	(32–57)	30	(14)
24	(61.0)	1000	(454)	225–400	(102–181)	40	(18)
30	(76.2)	1000	(454)	225–400	(102–181)	40	(18)



DIMENSIONS AND QUANTITIES		
H	A	CU.FT./LIN.FT. OF WALL
5'-0"	3'-5"	12.30
6'-0"	3'-10"	16.02
7'-0"	4'-3"	20.16
8'-0"	4'-8"	24.72
9'-0"	5'-0"	29.25
10'-0"	5'-6"	35.00
11'-0"	6'-0"	41.25
12'-0"	6'-6"	48.00
13'-0"	7'-0"	55.25
14'-0"	7'-6"	63.00
15'-0"	8'-0"	71.25

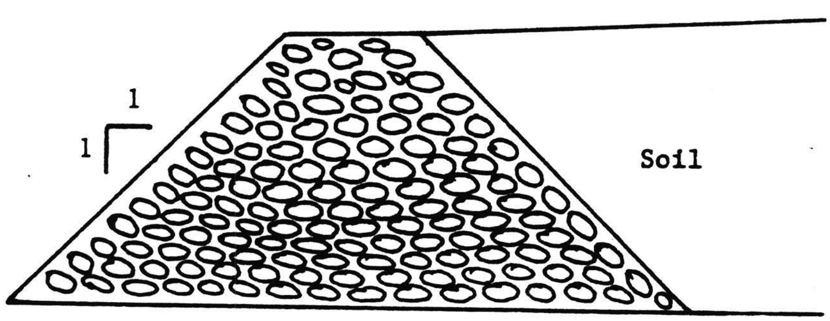
Fig. 3.2 Rhode Island Wet Stone Masonry Retaining Wall

the wall is based on the force needed to stabilize the landslide, and it is determined by a stability analysis.

RHODE ISLAND: The department does not use rock buttress walls, but does use a wet stone masonary retaining wall. A copy of the specifications is given in Fig 3.2.

WASHINGTON: The department does use a form of rock buttress walls called rockeries. The rockeries are used for erosion control or slope potection. They are not designed to resist active soil pressures. Further information is found in the Literature Review.

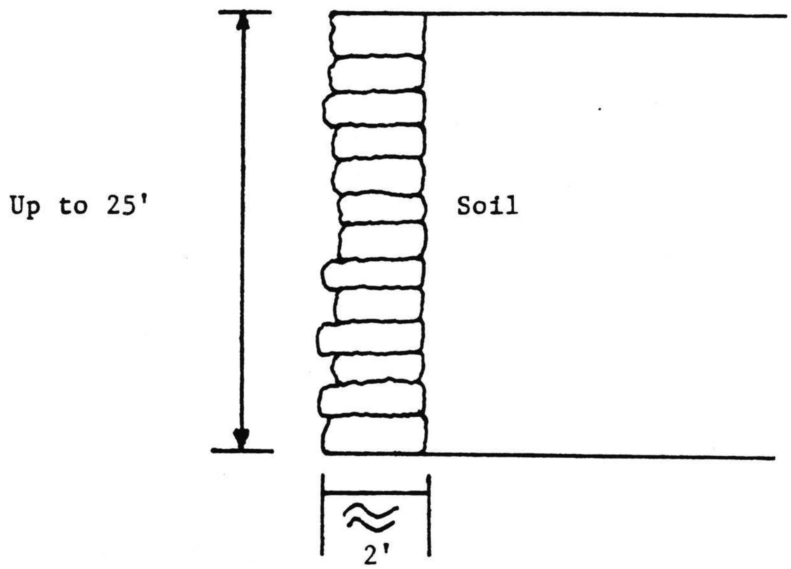
WISCONSIN: The department does use three forms of rock retaining walls as erosion control. No formal analysis procedure is used for the wall designs. The three types of wall are illustrated in Figs. 3.3 to 3.5.



.Largely triangular or trapezoidal in shape.

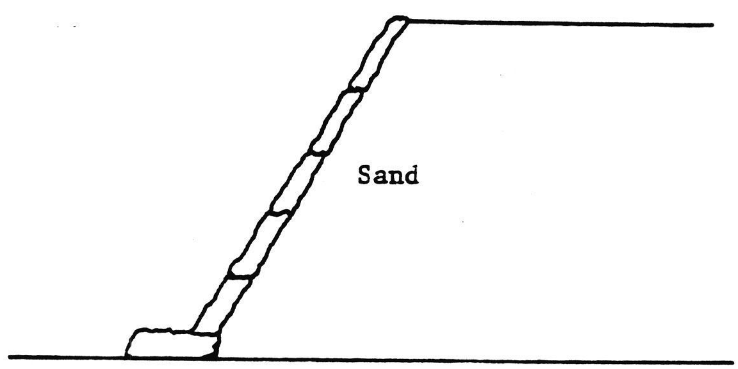
.Used mainly as an "anchor" at toes of fills or in landslide repairs.

Fig. 3.3 Shot-run Rock Bulkhead



.Used as a facing in heavily preconsolidated tills where the benefits are largely erosion control.

Fig. 3.4 Erosion Prevention Wall



.Used flat stones, possibly split concrete block.

.Used largely as an erosion protection in sands.

Fig. 3.5 Erosion Protection in Sand

Several federal government agencies were contacted concerning their design procedure for rock buttress walls. The results of the discussions are as follows:

U.S. Army Corps of Engineers - The corps does not use rock buttress walls.

Tennessee Valley Authority - The TVA does not use rock buttress walls.

Bureau of Reclamation - The bureau does not use rock buttress walls.

National Park Service - The park service does not design gravity retaining walls of unmortared stone.

U.S. Forest Service - No response.

CHAPTER 4

DESIGN APPROACHES

4.1 INTRODUCTION

The literature review and state survey revealed that there are four basic approaches to rock buttress wall design. The first approach is the rule of thumb or no specified analytical design. These walls generally are used for erosion control and tend to be small. The second approach is to use slope stability programs to design the walls. The walls are sized so that their mass will prevent soil movements. This approach is generally associated with repair work. The third approach is the Indian method. In this method, the internal friction force between the stones resist the active soil pressures behind the wall. The method is designed for granular soils and is modified for cohesive soils. The last approach is the Swedish slice method. A circular failure arc is assumed and the soil forces (sliding, cohesion and friction) are resisted by the wall's internal friction force. The wall is sized so its internal friction force balances the soil forces.

The Indian and Swedish approaches are illustrated by designing a 28 foot rock buttress wall. The wall has a side slope of 1:2 and is supporting a silty-clay soil. A five foot overburden is placed on the wall which supports the roadway.

4.1 INDIAN METHOD

The following approach is used for the design of rock buttress walls by the Indian Method:

1. Calculate the force needed to cause sliding in the wall.
   a. From Table 4.1, find sliding force in terms of the wall weight.

      ϕ = internal friction angle of the stone.

      W = weight of wall

      F = the force to cause sliding.

b. From Table 4.2, find the weight of the wall in terms of the density of the stone used.

H = height of the wall

γ stone = density of the stone used to construct the wall.

c. F = (value from Table 4.1) x (value from Table 4.2) x (γstone).

2. Calculate force imposed on the wall by the soil

a. Granular soil (Coulomb's active earth pressure)

$P_a = 1/2\ K_a\ \gamma\ H^2$

γ = density of the soil

K_a = Coulomb active earth pressure coefficient found in Table 4.3

ϕ = internal friction angle of the soil

y = H/3 distance from base of wall to centroid of active force.

b. Clay soil (Rankine Active Earth Pressure)

$P_a = 1/2\ (H-Z_c)(8\ H\ K_a - 2\ c\sqrt{K_a})$

$K_a = \mathrm{Tan}^2\ (45^o - \phi/2)$

$Z_c = 2\ C/(\gamma\sqrt{K_a})$ = length of tension crack from top of wall along the face of the wall.

C = cohesive strength of the soil.

$y = (H-Zc)/3$

c. Effects of overburden.(10)

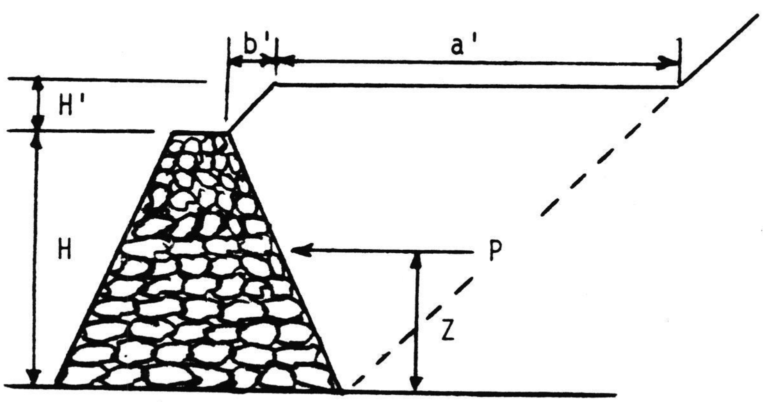


Fig. 4.1 Overburden Configuration

Table 4.1 Stone Wall Resisting Force Ratio F/W for $\delta = 22.5^{o}$

ϕ	Base slope of the wall: 0	6H:1V	3H:1V
20	.464	.798	1.278
25	.626	1.038	1.686
30	.821	1.352	2.291
35	1.068	1.790	3.305
40	1.392	2.454	5.408
45	1.848	3.607	12.617

Table 4.2 Weight of Wall Ratio W/γ_{stone}

H	1H:1V	1H:1.5 V	1H:2V
3	18	15	13.5
4	28	22.7	20
5	40	31.7	27.5
6	54	42.	36
7	70	53.7	45.5
8	88	66.7	56
9	108	81	67.5
10	130	96.7	80
12	180	132	108
14	238	172.7	140
16	304	218.7	176
18	378	270	216
20	460	326.7	260
22	550	388.7	308
24	648	456	360
26	754	528.7	416
28	868	606.7	476
30	990	690	540

Table 4.3 Coulomb's Active Pressure Coefficient

δ - wall friction = 22.5^{o}

Angle of incline of Backfill

ϕ	0	5	10	15	20	25	30
				Vertical Wall			
26	.343	.369	.401	.446	.517	.691	-
28	.319	.341	.369	.406	.462	.567	-
30	.296	.316	.340	.371	.415	.490	.812
32	.275	.292	.312	.339	.375	.431	.551
				1H:2V side slopes			
26	.625	.690	.771	.881	1.053	1.466	-
28	.598	.658	.731	.826	.965	1.23	-
30	.571	.625	.691	.774	.890	1.081	1.896
32	.548	.598	.657	.731	.830	.981	1.296
				1H:1.5V side slopes			
26	.810	.912	1.039	1.211	1.480	2.146	-
28	.724	.806	.907	1.038	1.230	1.592	-
30	.697	.773	.864	.980	1.142	1.409	2.286
32	.672	.742	.825	.928	1.067	1.278	1.726
				1H:1V side slopes			
26	1.058	1.220	1.424	1.702	2.147	3.296	-
28	1.027	1.178	1.364	1.609	1.973	2.681	-
30	.998	1.139	1.309	1.527	1.835	2.356	4.876
32	.969	1.101	1.258	1.454	1.720	2.133	3.042

$P = (q/90)\ [H(\theta_2 - \theta_1)]$

q = overburden stress - psf

$\theta_1 = \tan^{-1}\ (b'/H)$

$\theta_2 = \tan^{-1}\ [(a' + b')/H]$

a' = width of overburden

b' = distance from backface of the wall to overburden

$Z = H - [H^2\ (\theta_2 - \theta_1) + (R+Q) - 57.30\ a'H]/[2H(\theta_2 - \theta_1)]$

$R = (a'+b')^2\ (90 - \theta_2)$

$Q = (b')^2\ (90 - \theta_1)$

d. Safety factor against failure of sliding of stones in wall.

$S.F. = (P_a + P)/F$

3. Calculate stress under wall.

a. Sum the moments about the outside toe of wall.

b. Find location of resultant with respect to toe of wall.

$x = \Sigma M / \Sigma V$

c. Find $e = b/2 - x$

d. Soil stress under wall is equal to:

$q = (\Sigma V/B)(1 \pm 6e/B)$

B = width of wall at base.

4. Estimate stone bearing stress

Bearing stress = q_{max}/percent of surface area in bearing)

If bearing stress exceeds allowable for stone, redesign the wall.

Example 1 - Indian Method.

The length of the wall is 28 feet and a five foot overburden is in place. The wall has side slopes of 1:2.

Properties:

silt-clay soil	sandstone rock	wall
C=600 psf		H = 28 ft (wall height)
$\phi = 5^o$	$\phi = 35^o$	H'= 5ft (overburden height)
γ= 120 pcf	γ= 135 pcf	Base is horizontal
		1:2 (H:V) side slopes

1. Calculate force to cause sliding in wall:
   a. from Table 1 $\quad F = 1.068\ W$
   b. from Table 2 $\quad W = 476\gamma$
   c. $F = 1.068\ (476)\ 135 = 86{,}630$ lb/ft width of wall.
2. Calculate force imposed on wall by slit-clay soil:
   b. $P_a = 1/2\ (H-Zc)\ (\gamma H\ K_a\ -2C\sqrt{K_a})$

   $K_a = \tan^2(45^o - \phi/2) = \tan^2\ (45 - 5/2) = 0.840$

   $Z_c = 2\ C/(8\sqrt{Ka}) = 2\ (600)/[125\ (0.916)] = 10.48$ ft

   $P_a = 1/2\ (28-10.48)(120 \times 28 \times .840 - 2 \times 600 \times .917)$ (active force)

   $P_a = 15{,}085$ lb.

   $y = (28 - 10.48)/3 = 5.84$ ft (centroid of active force).

   c. Calculate effects of overburden:

   $a'+b'$ = effective width of failure zone = $H/[\tan(45+\phi/2)] = 25.7$ ft.

   $P = (q/90)\ [H\ (\theta_2-\theta_1)]$

   $\theta_1 = \tan^{-1}\ (b/H) = 5.1^o$

   $\theta_2 = \tan^{-1}\ [(a' + b')/H] = 42.5^o$

   $Z' = H - [H^2\ (\theta_2 - \theta_1) + (R-Q) - 57.30\ a'H]/[2H(\theta_2 - \theta_1)]$

   $R = (a'+b')^2(90-\theta_2)$

   $Q = (b')^2\ (90-\theta_1)$

   $q = \gamma soil\ H'= 120(5) = 600 lb/ft^2$

$b' = H(1/2) = 2.5$ ft.

$P = (600/90)[28(42.5-5.1)] = 6981$ lb

$R = 25.7^2\ (90-42.5) = 31373.3$

$Q = 2.5^2\ (90-5.1) = 530.6$

$Z = 28 - [28^2\ (42.5-5.1) + (31373.3 - 530.6) - 57.30\ (23.2)\ 28]$

$/[2\ (28)\ (42.5 - 5.1)]$

$Z = 17.0$ ft from base.

S.F. $= 68630/(15085-6981) = 3.1$

3. Calculate stresses under wall:

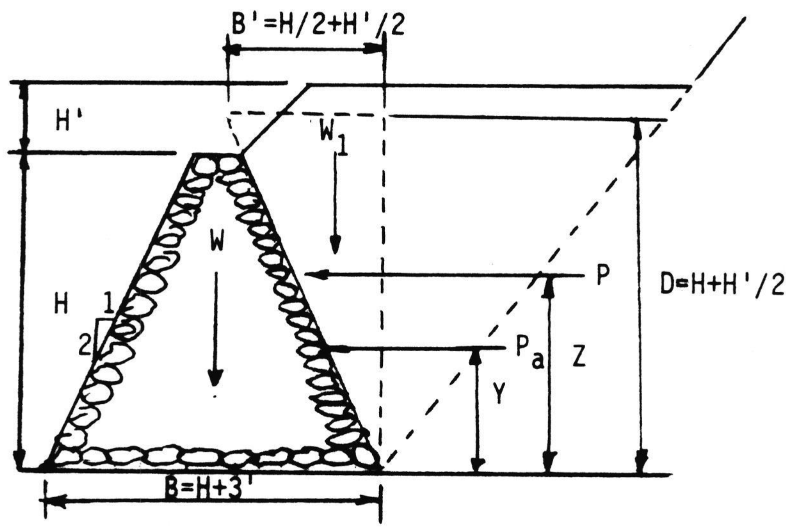


Fig 4.2 Wall Free Body Diagram

$\Sigma M_A = W(B/2) + W_1\ (B-B'/3) - P\ (Z) - P_a\ Y$

$= 64260\ (31/2) + (30.5/2)(15.25)\ 120\ (31-(15.25/3) - 6981(17) -$

$15085\ (5.84)$

$= 1512526$ ft-lb

$\Sigma V = 64260 + (30.5/2)(15.25)(120) = 92167.5$ lb

$X = 1512526/92167.5 = 16.4$ ft

$e = b/2 - X = 31/2 - 16.4 = -0.9$

Bearing on soil.

$q_{max} = (\Sigma V/B)\ (1 \pm 6e/b) = (92167.5/31)(1+ 6(.9)/31) = 3491\ lb/ft^2$

$q_{min} = 2455\ lb/ft^2$

4. Estimated compressive strength forces on stone:

   $q\ max = 3491\ lb/ft^2 = 24.2$ pounds per square inch (psi)

   if bearing area of stone is 10 to 20%, (not choked)

   effective stress = (24.2/0.1) = 242 pounds per square inch (psi)

A minimum 242 psi would be required to resist crushing of the stone. Also, the shear strength of the stone needs to be checked.

4.3 SWEDISH SLICE METHOD

The following approach is used for the design of rock buttress walls by the Swedish slice method.

1. Draw the proposed wall and supported soil to scale.
2. On the inside surface of the wall at the expected shear plane location, about one or two feet from the base, pass a line through the wall. The line

   should have about a 10^{o} slope with respect to the horizontal. At the point where the line meets the interior edge of the wall, draw a perpendicular. The center of rotation is on this perpendicular line.

3. The radius of the arc is arbitrarily chosen. Several trials are required in order to obtain the optimium solution. Draw the circular failure arc and divide the arc into segments of equal width and number the segments. The accuracy of the approach is increased as the segments are made smaller. The number of segments is determined by experience.

4. Mark the midpoint of each segment on the arc and measure the height and width of each segment at the midpoint. Draw a line from the midpoint to the center of rotation and measure the angle made with respect to the horizontal -θ .

5. Set up a table composed of segment area, tangent area and normal area.

Tangent area = Area x $\cos\Theta$

Normal area = Area x $\sin\Theta$

6. Determine safety factor against the rock sliding in the wall.

$$S.F. = [\Sigma N \tan\phi_s + C\ L + \Sigma N \tan\phi_r]/[\Sigma T_{soil} + \Sigma T_{rock}]$$

$\Sigma N \tan \phi_s$ = sum of the normal areas composed of soil times the tangent of the internal friction angle of the soil and density of the soil.

$\Sigma N \tan \phi_r$ = sum of the normal areas composed of rock times the tangent of the internal friction of the rock and density of the rock.

C L = cohesive strength of the soil times the length of the arc in the soil zone.

ΣT_{soil} = sum of the tangent areas in the soil zone times the density of the soil.

ΣT_{rock} = sum of the tangent areas in the rock zone times the density of the rock. This value could be assumed to be zero for design of the wall.

7. To determine the size of wall needed, pick a safety factor (1.5 or more) and determine the force needed to support the soil - F.

$$F = S.F.\ \Sigma T_{soil} - \Sigma N \tan\phi_s - C\ L$$

The force needed in the wall is

$$F_{wall} = F \cos\alpha = W \tan\phi_s$$

$$W = F \cos\alpha / \tan\phi_s$$

Determine height of wall from Table 4.2.

8. Determine safety factor against sliding of the wall at foundation.

Given

$$P_R = S.F. \Sigma T_{soil} - \Sigma N \tan\phi_s - C\ L$$

and

$$P_R \text{ Cos}\alpha \leq W \tan\phi_s + C\ B$$

where

B = width of base

W = weight of wall = areas on rock zone times density of rock.

results in

$$S.F. = [W \tan\phi_s + C\ B + \Sigma N \tan\phi_s \cos\alpha + C\ L \cos\alpha]/[\Sigma T_{soil} \cos\alpha]$$

9. Repeat procedure to find worst case or critical failure plane.

Example 2 - Swedish Slice Method

Trial 1 - Center of rotation above ground. See Fig 4.1

Segment	1	2	3	4	5	6	7
Area	117.0	207.0	256.5	285.5	292.5	270.0	184.5
Normal	58.5	153.6	232.5	281.2	292.1	256.8	154.7
Tangential	101.3	133.1	108.4	49.6	-15.3	-83.4	-100.6

1. Calculate factor of safety against sliding of rock:

$$F.S. = (\Sigma N \tan\phi_s + C\ L + \Sigma N \tan\phi_r)/(\Sigma T_{soil} + \Sigma T_{rock})$$

Soil Properties

$\gamma_s = 120$ pcf

$\phi_s = 5^o$

$C = 600$ psf.

Rock Properties

$\gamma_r = 135$ pcf

$\phi_r = 35^o$

$\Sigma N \tan\phi_s = 1022.9\ (120) \tan 5^o = 10739$ lb

$\Sigma N \tan\phi_r = [256.8(125) + 154.7(135)] \tan 35^o = 37100$

$C\ L = 600(62) = 37200$ lb.

$\Sigma T_{soil} = 377.1(120) = 45252$

$\Sigma T_{rock} = 0$ or $-83.4(125) - 100.5(135) = -23993$

$F.S. = (10739 + 37200 + 37100)/45252 = 1.88$ with $\Sigma T_{rock} = 0$

$F.S. = 4.0$ with $\Sigma T_{rock} = -23993$

2. Calculate factor of safety against foundation sliding:

$B = 31$ ft

$W = 64260 + (28/2)(1/2)(120) = 87780$ lb.

$$S.F. = (W \tan\phi_s + C\ B + \Sigma N \tan\phi_s \cos\alpha + C\ L \cos\alpha)/(T_{soil} \cos\alpha)$$

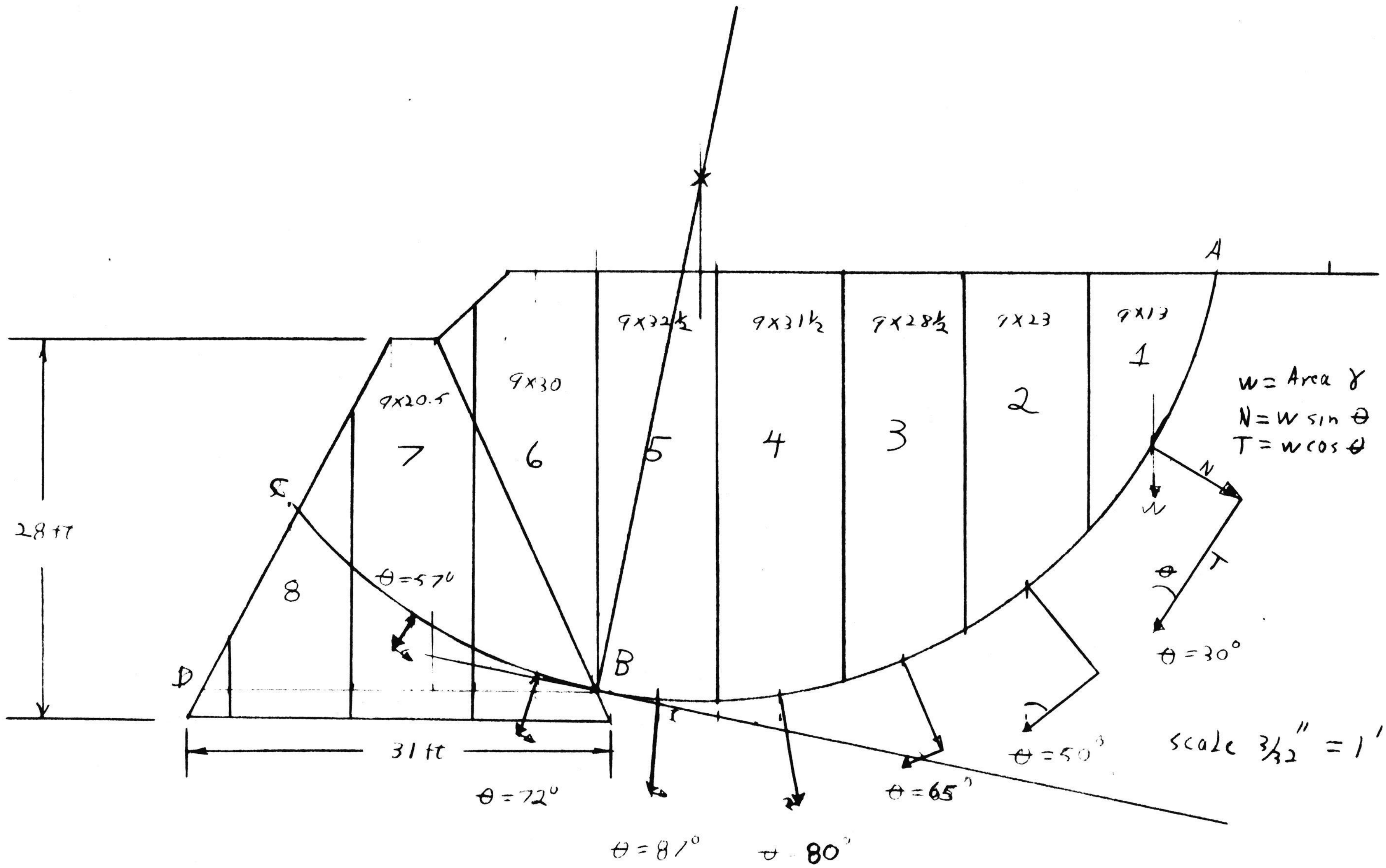


Fig. 4.3 Swedish Slice - Trial 1

$S.F. = (87{,}780 \tan 5^o + 600(31) + 10739 \cos 10^o + 37200 \cos 10^o)/$

$(45252 \cos 10^o)$

S.F. = 1.65 (whole wall moving)

Trial 2 - Radius approximately equal to H tan(45 + /2) See Fig. 4.4

Segment	1	2	3	4	5	6
Area	153.0	225.0	261.0	279.0	261.0	180.0
Normal	78.8	184.3	250.0	278.8	245.3	135.8
Tangential	131.1	129.1	71.9	- 9.7	-89.3	-118.1

1. Calculate factor of safety against sliding of rock:

$F.S. = (\Sigma N \tan\phi_s + C\ L + \Sigma N \tan\phi_r)/(\Sigma T_{soil} + \Sigma T_{rock})$

$\Sigma N \tan\phi_s = 791.9(120) \tan 5^o = 8314$ lb

$\Sigma N \tan\phi_r = [245.3(125) + 135.8(135)] \tan 35^o = 34307$ lb.

C L = 600(54) = 32400 lb

$\Sigma T_{soil} = 332.4(120) = 39880$

$\Sigma T_{rock} = 0$ or $-89.3(125) - 118.1(135) = -27106$ lb.

F.S. = (8314 + 32400 + 34270)/39880 = 1.88 with $\Sigma T_{rock} = 0$

F.S. = 5.9 with $\Sigma T_{rock} = -27106$ lb.

2. Calculate factor of safety against foundation sliding:

B = 31 ft.

W = 87,780 lb

$S.F. = (W \tan\phi_s + C\ B + \Sigma N \tan\phi_s \cos\alpha + C\ L \cos\alpha)/(\Sigma T_{soil} \cos\alpha)$

$S.F. = (87780 \tan 5^o + 31(600) + 8314 \cos(10^o) + 32400 \cos(10^o))/$

$(39880 \cos 10^o)$

S.F. = 1.69

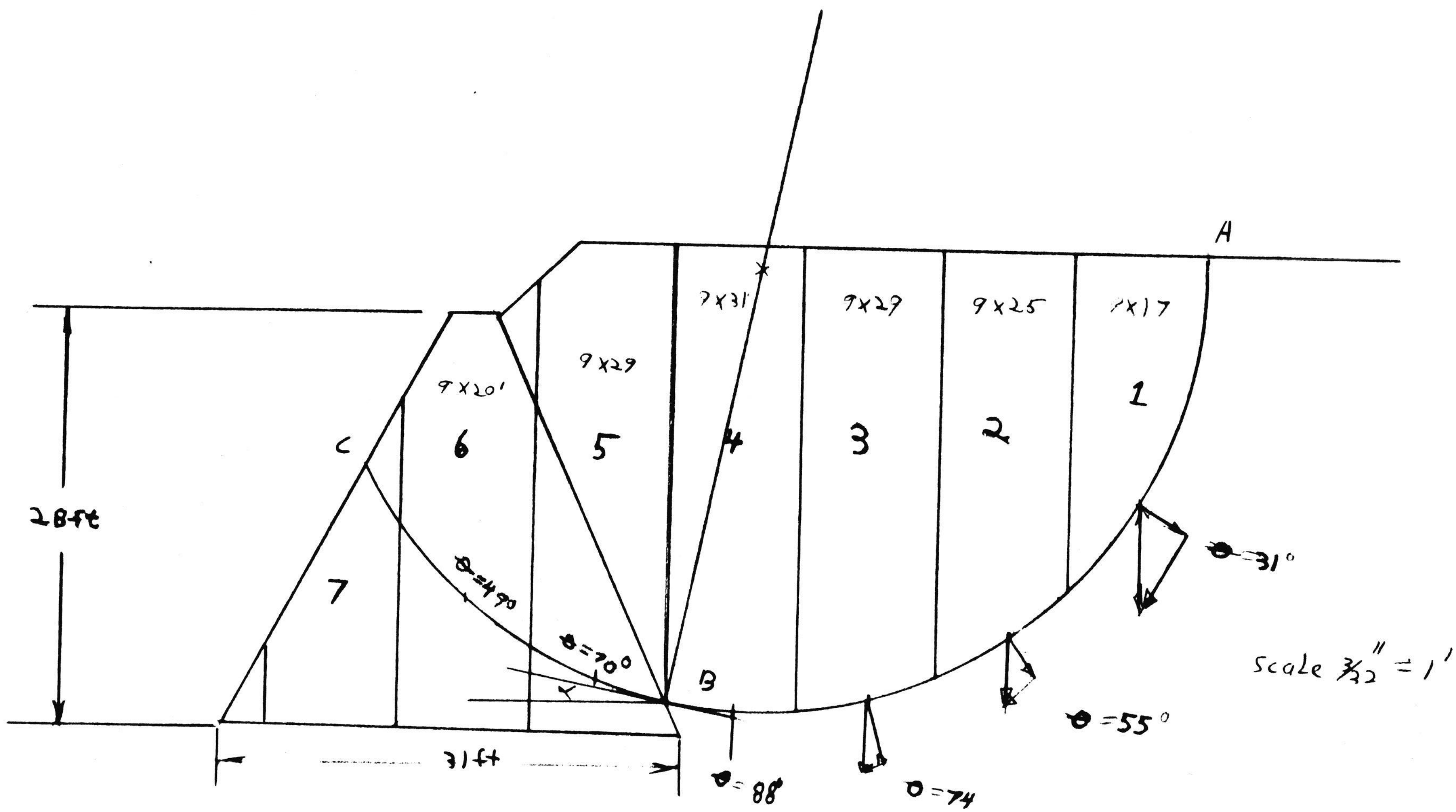


Fig. 4.4 Swedish Slice - Trial 2

CHAPTER 5

DISCUSSION OF RESULTS

5.1 SURVEY RESULTS

There was a good response from the state highway departments surveyed. Eighty percent of the states responded to the survey, with fourteen states giving a positive response. That is, they used some form of a rock buttress wall. A review of the responses showed that the rock buttress wall is primarily used for erosion control or slope stability. None of the states presently use the wall as a retaining wall. The wall designs were either a specified shape to be used with no set design procedure or designs produced by slope stability analysis. These walls were used as either slide correction or prevention measures.

Specified shapes were used by Georgia, Michigan, Wisconsin and Washington. Michigan, Wisconsin and Washington used the walls primarily as erosion control. Georgia used the wall as a toe treatment for slide protection or to steepen the slope.

Alabama, Iowa, Kentucky, Mississippi, and Oregon used conventional slope stability analysis to design the walls. The walls were used as toe protection in slide correction or prevention techniques.

The specification for rocks to be used in the walls varied widely. They ranged from eight inch maximum size to one to six man-rock-size (400 to 2400 lb). There was no general accepted size for the stones to be used. However, the tendency was: the steeper the side slope, the larger the stones. Oklahoma and California were the only state that listed specific tests to judge the quality of the rock. The tests were for weight, absorption, and durability.

Oklahoma used a design approach very similar to the Swedish slice method. They used an earth buttress wall instead of a rock buttress. Both approaches are the same except for the material to be used in the wall.

5.2 FIELD OBSERVATIONS

Visits were made to four sites where rock buttress were used. As a result of these visits, five basic observations were made. First, the quality of the stone has a major impact on the wall. At the Mt. Levi Road site, the wall had failed. It was observed that some of the sandstone used to construct the wall was of poor quality. It could be crumbled by mild blows. This stone was used in the construction of the base of the wall. A visit to the local quarry showed the stone to be sound. However, after a season of weathering, the quality of stone was greatly reduced. This wall was analyzed in the design examples. The calculations revealed that there was ample strength against the shearing of the wall. But when the stone's bearing stress were calculated, it was estimated to be at 250 psi. This exceeds the estimated bearing stress of the weathered stone. Therefore, the stone used in the wall should be of good quality.

Second, the walls were not choked. There were several large voids in the walls, which could lead to large stress concentration. In some cases it was estimated that about 10 percent of the stone's surface area was in bearing. By choking the walls, these stress concentrations would be reduced. Also, further evidence of the presence of tensile forces was indicated by the splitting of several stones. It appeared that the stones were subjected to concentrated loads and behaved as beams. Choking would help to reduce tensile stress and stress concentrations.

Third, the wall side slopes were not always 1:2. Walls under 10 feet in height tend to have a 1:1 side slopes. It is felt that this resulted from the construction technique used, dumping the stones. This would tend to produce a 1:1 side slope. In the higher walls, the stones were placed which resulted in approximately 1:2 slopes.

Fourth, it was hard to construct the wall according to the specifications when they were under six feet in height. Once the wall was built, there was not enough room behind the wall for the equipment to properly place the backfill. It would be better to place stone in the backfill area as the wall is constructed and design the walls with 1:1 to 1:2 side slopes.

Fifth, the walls with apparent greater stability were constructed with the larger stone in the bottom layers. The stone's length was about three times its thickness. It was impossible to determine the average width due to their placement in the wall.

5.3 DESIGN APPROACHES

Two design approaches were presented, the Indian method and the Swedish slice approach. Both approaches have their advantages. The Indian approach is easy to use and is composed of a relatively simple set of computations. The approach should be used when the soil behind the wall is uniform. The technique is greatly aided by the use of tables. The method provides a means of computing the wall's shear resistance, foundation stresses and stone bearing stresses. These values are needed for the foundation design and determining wall height restrictions. The foundation stresses could be compared to the soil's allowable bearing capacity to determine if the soil is overloaded. Also, tolerable settlements could be determined. By calculating the stone's bearing stresses, height limitations could be checked. The wall's height could be limited by restricting the bearing stresses imposed by the wall's weight to one-half or less of the stones ultimate bearing capacity. Also the stones shear stress needs to be checked. The main disadvantage of this design method is the cumbersome treatment of layered soil.

The Swedish approach requires the wall to be drawn to scale and forces determined by a graphic procedure. This approach is useful if the location of slip or failure planes are known. The arc could pass along these planes and the size of the wall needed to resist the soil's active forces could be determined.

The approach could be used for layered soil and to calculate the factor of safety against the foundation sliding. The main disadvantage of the approach is that several trials are needed before the critical failure arc is determined.

Each approach has its advantages and disadvantages. They do provide a good means of estimating the soil's active forces and designing the retaining wall. They are simple to use and could lead to a wall design in a relatively short time.

CHAPTER 6

SUMMARY and CONCLUSION

6.1 SUMMARY

The objective of this research project was to investigate the various methods used for rock buttress wall design, adapt them for Arkansas soil conditions and make recommendations on how to implement the procedures. A survey was sent to all fifty states and several federal government agencies. Eighty percent of the states replied, and 14 states responded that they used rock buttress walls. However, none of the states used the walls as a retaining wall. Instead, they used them for erosion control, slide correction or prevention techniques. A literature search revealed two design approaches could be used for rock buttress walls. The first approach, the Indian approach, equates the wall's internal frictional forces to the soil's active forces. The active forces are determined by the Rankine and Coulomb methods. The second approach, Swedish slice method, uses a circular arc to model the soil failure plane. The soil's tangential, frictional and cohesive forces are equated to the wall's frictional forces to design the rock buttress wall.

Site visits revealed that the quality of the stone used, choking and construction techniques play a major role in the wall's behavior. The stone should be hard and resist weathering. It is believed that poor quality stone played a major role in the failure of the wall on the Mt. Levi-Fort Douglas Road. It is also felt that high bearing stresses could be avoided in the wall by choking of the wall. These stresses could lead to the failure of the stone in the wall. The visits revealed that when the walls are about six feet or less in height, the side slopes tend to be 1:1 instead of the 1:2 specified. This results from the construction technique used in the building of the wall (dumping of the stone).

6.2 CONCLUSION

The states survey reveals that presently Arkansas is the only state that uses the rock buttress wall as a retaining wall. In other countries, such as India, these walls have been used extensively with success. Their design approach, the Indian method, appears to give reliable results and is easy to use. It is best suited for a uniform soil behind the wall. A second method, Swedish slice method, has been used in Oklahoma and should be used when soil conditions merit its use. This would be used when the location of failure planes are known or when the soil is layered.

The present geometrical shape of the rock buttress wall used in Arkansas does produce an adequate wall. However, settlements and allowable bearing stresses should be considered in the foundation design. Also, the bearing stress within the stones should be calculated in order to determine wall height restrictions. Finally, a new geometrical design should be considered for walls of about six feet or less in height. This design should have side slopes of 1:1 to 1:2 and stone should be placed in the backfill area between the wall and soil as the wall is constructed.

CHAPTER 7

RECOMMENDATIONS

The present geometrical shape of the rock buttress wall used in Arkansas does appear to produce a reliable retaining wall for Arkansas soil conditions. However, the following recommendations are suggested in order to improve the present techniques.

1. Walls 10 to 25 feet in height should be designed by the Indian or Swedish slice methods. The Indian method should be used when the soil is uniform behind the wall. The Swedish slice method should be used when the soil is layered or the location of failure planes are known.

2. Walls should be choked to reduce bearing and tensile stresses. By choking the wall, a greater percentage of the stone surface area will be in bearing, thus, reducing stress concretrations. Also by choking the wall, the stones would no longer behave as beams. This would eliminate any tensile stresses in the stones. The choking would result in the stone being continuously supported.

3. A test for stone quality such as the DAR, which is used in California, should be implemented. This would insure that good quality stone is used in the walls. It has been shown that weathering could greatly reduce the stone's strength.

4. A minimum stone compressive strength should be established. A strength of 1000 psi should be adequate for walls up to 25 feet in height.

5. Special care should be given to the walls foundation design:

   A. The wall should be keyed to prevent slippage when the resistance to the foundation sliding is less than 1:5.

   B. Settlements should be calculated and differential settlements prevented.

   C. If possible, high walls should be placed on bedrock.

   D. The foundation bearing stress should be checked to ensure that it does not exceed the soil's allowable bearing capacity.

6. The geometrical slope of the walls with heights of six feet or less should be changed. The side slope should be 1:1 to 1:2 and the backfill area between the wall and soil should be the same rock as used in the wall. This rock should be placed as the wall is built.

7. Wall heights should be limited to 25 feet.

8. Maximum size of stone used in the wall should be no more than 1/3 of the wall's width at the level of placement.

9. The minimum factor of safety should be 1:5 for design evaluation.

The following research is suggested in order to refine the design processes presented.

1. The DAR selection criteria presented should be verified for the native stones found in Arkansas.

2. The native stones found in Arkansas should be cored and their compressive strength and tensile stregths determined.

3. The design procedures presented should be verified by the STABLE and SOIL TEST slope stability computer programs.

4. The effects of saturated soils should be investigated to determine reductions in safety factors. This would simulate wall conditions during an abnormally wet spring.

CHAPTER 8

IMPLEMENTATION OF PROCEDURE AND BENEFITS

The present geometrical shape of high rock buttress retaining walls should be maintained. These walls should be designed by the Indian or Swedish design approaches. The Indian design procedure should be used when the soil is uniform behind the wall. The Swedish slice method should be used when the soil is layered or the location of possible failure planes are known. In these designs, shear keys should be used when the resistance to the foundation sliding is less than 1.5. In some cases, a subsurface investigation will be required in order to adequately design the rock buttress. That is, foundation bearing stresses, settlements and allowable soil bearing compacities should be determined. The bearing stresses within the stones should be used to determine height restrictions. These design procedures should be used for walls 10 to 25 feet in height. Walls exceeding 15 feet in height should be designed in consultation with the geotechnical engineer. Walls in excess of 25 feet should be designed by the bridge or geotechnical engineer. For walls of six feet or less in height, the geometrical slope should be changed. The side slope should be 1:1 to 1:2 and the backfill between the wall and soil should be the same rock used in the wall. It should be placed as the wall is built. This would help to simplify the construction techniques used. Walls six to 10 feet in height should be constructed by present procedures. The Durability Absorption Ration (DAR) should be included in the material specifications as a viable alternative for determining stone quality. This would ensure that sound stones are used in the walls.

The wall specifications should require choking of the walls. This would reduce stone bearing stress and eliminate any tensile strength in the stone. The gradation of the choking stone should be determined by the AHTD. The maximum size of stone used in the wall should be no more than 1/3 of the wall's width at the level of placement.

Several benefits would be achieved by the implementation of the proceeding recommedations.

1. A simple engineering procedure would be used to check wall designs for any possible failure conditions. If a failure condition is encountered, changes can be made in the design before the wall is built in the field.

2. The change of geometry for walls with heights of six feet or less in height would make it easier to construct them in the field.

3. The DAR will provide a means of judging the quality of stones to be used in the wall which will give the field inspector a basis for accepting or rejecting stones.

4. These recommendations could be implemented with minimal increase in costs. The walls would be engineered and the resulting likelihood of a failure would be greatly reduced. Also, better quality control would be achieved in the field which would help reduce maintance costs.

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APPENDIX A

SELECTED STATE SURVEY RESPONSES

ALABAMA HIGHWAY DEPARTMENT

Arkansas State University
COLLEGE OF ENGINEERING, AGRICULTURE
AND APPLIED SCIENCES

DEPARTMENT OF ENGINEERING
P.O. DRAWER 1080
STATE UNIVERSITY, ARKANSAS 72467-1080
TELEPHONE 501/972-2088 JONESBORO

July 31, 1986

Mr. Larry Lockett
Geotechnical Engineering
Alabama Highway Department
1409 Coliseum Blvd.
Montgomery, Al 36130

Dear Sir:

I am working with the Arkansas Highway and Transportation Department on a research project designed to investigate rock buttress wall design procedures. We define them as mortar free, natural stone, gravity retaining wall. I would like to know, (1) if your department uses this type of retaining wall, (2) if so, would you please send a copy of the procedures or inform me who I could contact in order to obtain a copy of the procedure.

Thank you for your cooperation.

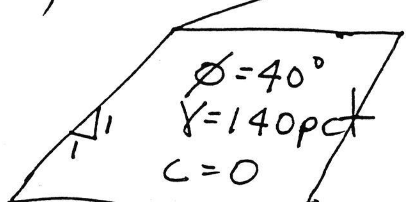
Sincerely yours,

Thomas J. Parsons
Assistant Professor of Civil
Engineering

bm

1) YES, EXTENSIVELY, but USUALLY AS A SLIDE CORRECTION TECHNIQUE – OTHER RETAINING STRUCTURES with STEEPER – OR VERTICAL front FACES Are USUALLY better suited FOR RETAINING walls with only Active soil pressures Acting on them, i.e. not slide forces.

2)



USE wedge/slice ANALYSES with MATERIAL properties shown – check with LEASE OR STABAN

LOCKETT

CALIFORNIA DEPARTMENT OF TRANSPORTATION

STATE OF CALIFORNIA—BUSINESS, TRANSPORTATION AND HOUSING AGENCY GEORGE DEUKMEJIAN, *Governor*

DEPARTMENT OF TRANSPORTATION
20 N STREET
ACRAMENTO, CA 95814
TDD (916) 323-7665
(916) 445-6519

August 14, 1986

File: 900.05

Thomas J. Parsons
Assistant Professor of Civil Engineering
Arkansas State University
Department of Engineering
P. O. Box Drawer 1080
State University, AR 72467-1080

Dear Mr. Parsons:

In response to your letter of July 31, 1986 we are sending you Section 72 - Slope Protection out of our Standard Specifications and Slope Protection Details No. 1 and No. 2 from our Standard Plans. We do not use mortar free, natural stone, gravity retaining walls and these procedures for rock slope protection are the best that we can provide.

You may find that Mr. Robert K. Barrett of the Colorado Division of Highways could be of assistance. The phone number we have has been discontinued but you should be able to contact him through Mr. J. B. Gilmore who is the Chief Engineering Geologist for the Colorado Division of Highways, at 4201 East Arkansas Avenue, Denver, Colorado 80222. Mr. Gilmore, whose telephone number is (303) 757-9275, may also be able to assist you.

I hope that you find this information to be of help and wish you success in your research project for the Arkansas Highway and Transportation Department.

Sincerely,

James E Roberts

JAMES E. ROBERTS, Chief
Office of Structure Design

Attachments

C W Purkiss

Standard Specifications

STATE OF CALIFORNIA

DEPARTMENT OF TRANSPORTATION

JULY, 1984

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SECTION 72

SLOPE PROTECTION

72-1 GENERAL

72-1.01 Description.—Slope protection consists of rock, sacked concrete, concrete, concreted-rock or slope paving. The type of slope protection to be used will be designated in the special provisions or shown on the plans. The slope protection shall be placed in conformance with these specifications, the special provisions, and the details and dimensions shown on the plans or directed by the Engineer.

72-2 ROCK SLOPE PROTECTION

72-2.01 Description.—This work shall consist of placing revetment type rock courses on the slopes.

The size of the individual pieces of rock shall be as indicated in the table in Section 72-2.02, "Materials," or as specified in the special provisions. The classes of rock slope protection are indicated by the average size of the individual piece to be used and will be designated in the Engineer's Estimate as 8-ton, 4-ton, 2-ton, 1-ton, ½-ton, ¼-ton, Light, Facing, and No. 1, No. 2 or No. 3 Backing.

72-2.02 Materials.—The individual classes of rocks used in rock slope protection shall conform to the following, unless otherwise specified in the special provisions, or as shown on the plans.

PERCENTAGE LARGER THAN *

	Method A Placement					Method B Placement							
	Classes					Classes							
											Backing		
Rock Sizes	8 Ton	4 Ton	2 Ton	1 Ton	½ Ton	1 Ton	½ Ton	¼ Ton	Light	Facing	No. 1	No. 2	No. 3
16-Ton	0–5												
8-Ton	50–100	0–5											
4-Ton	95–100	50–100	0–5										
2-Ton		95–100	50–100	0–5		0–5							
1-Ton			95–100	50–100	0–5	50–100	0–5						
½-Ton				95–100	50–100	------	50–100	0–5					
¼-Ton					95–100	95–100	------	50–100	0–5				
200-Lb.							95–100	------	50–100	0–5	0–5		
75-Lb.								95–100	------	50–100	50–100	0–5	
25-Lb.									95–100	90–100	90–100	25–75	0–5
5-Lb.												90–100	25–75
1-Lb.													90–100

* The amount of material smaller than the smallest size listed in the table for any class of rock slope protection shall not exceed the percentage limit listed in the table determined on a weight basis.
Compliance with the percentage limit shown in the table for all other sizes of the individual pieces of any class of rock slope protection shall be determined by the ratio of the number of individual pieces larger than the specified size compared to the total number of individual pieces larger than the smallest size listed in the table for that class.

The material shall also conform to the following quality requirements:

Tests	Test Method No. Calif.	Requirements
Apparent Specific Gravity	206	2.5 Min.
Absorption	206	4.2% Max.*
Durability Index	229	52 Min.*

$$\frac{\text{Coarse Durability Index}}{\text{\% Absorption} + 1} = \text{Durability Absorption Ratio (DAR)}$$

* Based on the formula contained herein, absorption may exceed 4.2 percent if DAR is greater than 10. Durability Index may be less than 52 if DAR is greater than 24.

Rocks, when conforming to the provisions in this Section 72-2.02, may be obtained from rock excavation of the roadway prism or other excavation being performed under the provisions of the contract, in accordance with the provisions in Section 4-1.05, "Use of Materials Found on the Work."

Rocks shall be of such shape as to form a stable protection structure of the required section. Rounded boulders or cobbles shall not be used on prepared ground surfaces having slopes steeper than 2 to one. Angular shapes may be used on any planned slope. Flat or needle shapes will not be accepted unless the thickness of the individual pieces is greater than ⅓ the length.

72-2.03 Placing.—Rock slope protection shall be placed in accordance with one of the following methods as designated in the Engineer's Estimate.

Method A Placement

A footing trench shall be excavated along the toe of slope as shown on the plans.

The larger rocks shall be placed in the footing trench.

Rocks shall be placed with their longitudinal axis normal to the embankment face and arranged so that each rock above the foundation course has a 3-point bearing on the underlying rocks. Foundation course is the course placed on the slope in contact with the ground surface. Bearing on smaller rocks which may be used for chinking voids will not be acceptable. Placing of rocks by dumping will not be permitted.

Local surface irregularities of the slope protection shall not vary from the planned slope by more than one foot measured at right angles to the slope.

Method B Placement

A footing trench shall be excavated along the toe of the slope as shown on the plans.

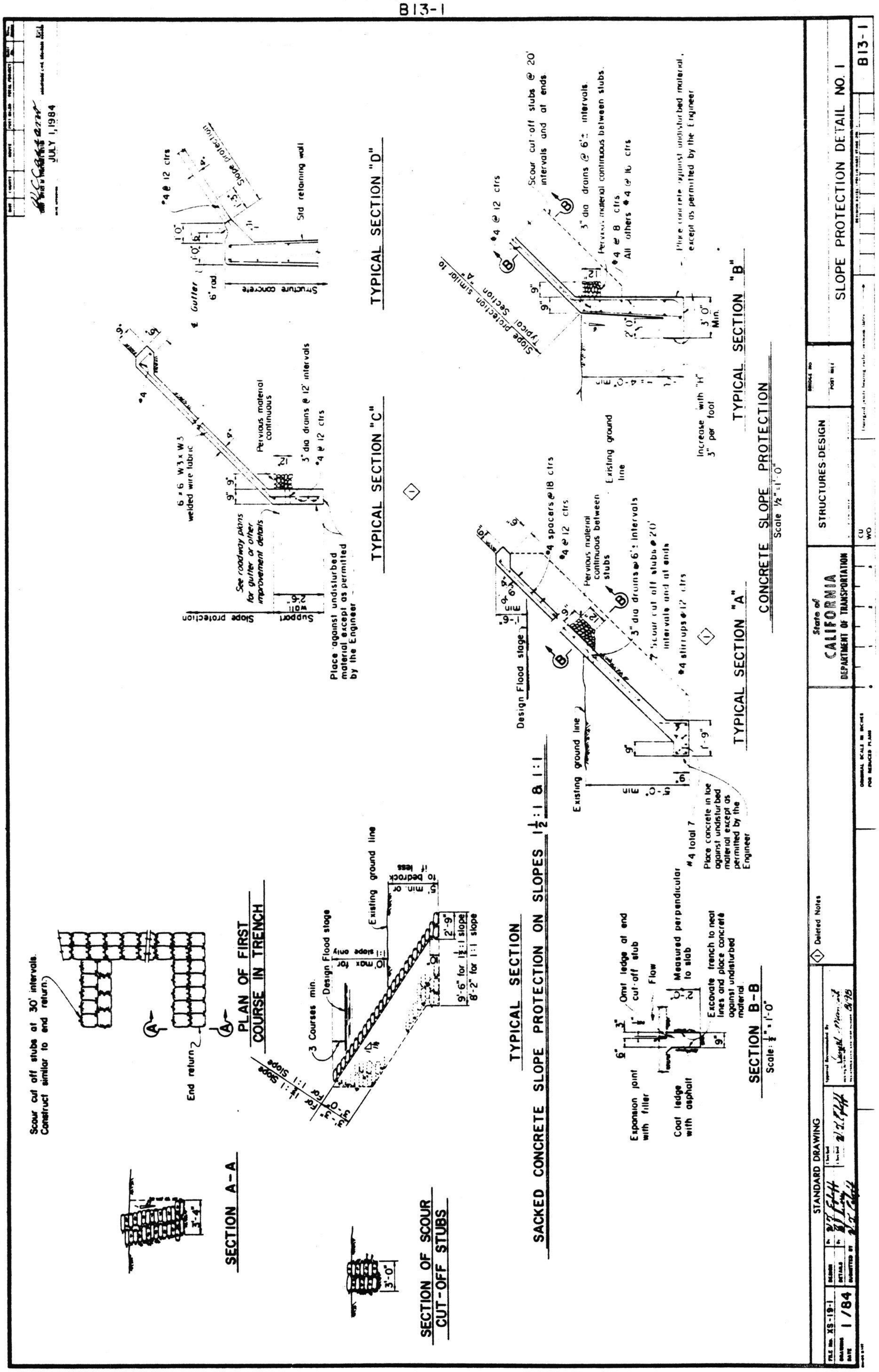
Rocks shall be so placed as to provide a minimum of voids and the larger rocks shall be placed in the toe course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other suitable equipment.

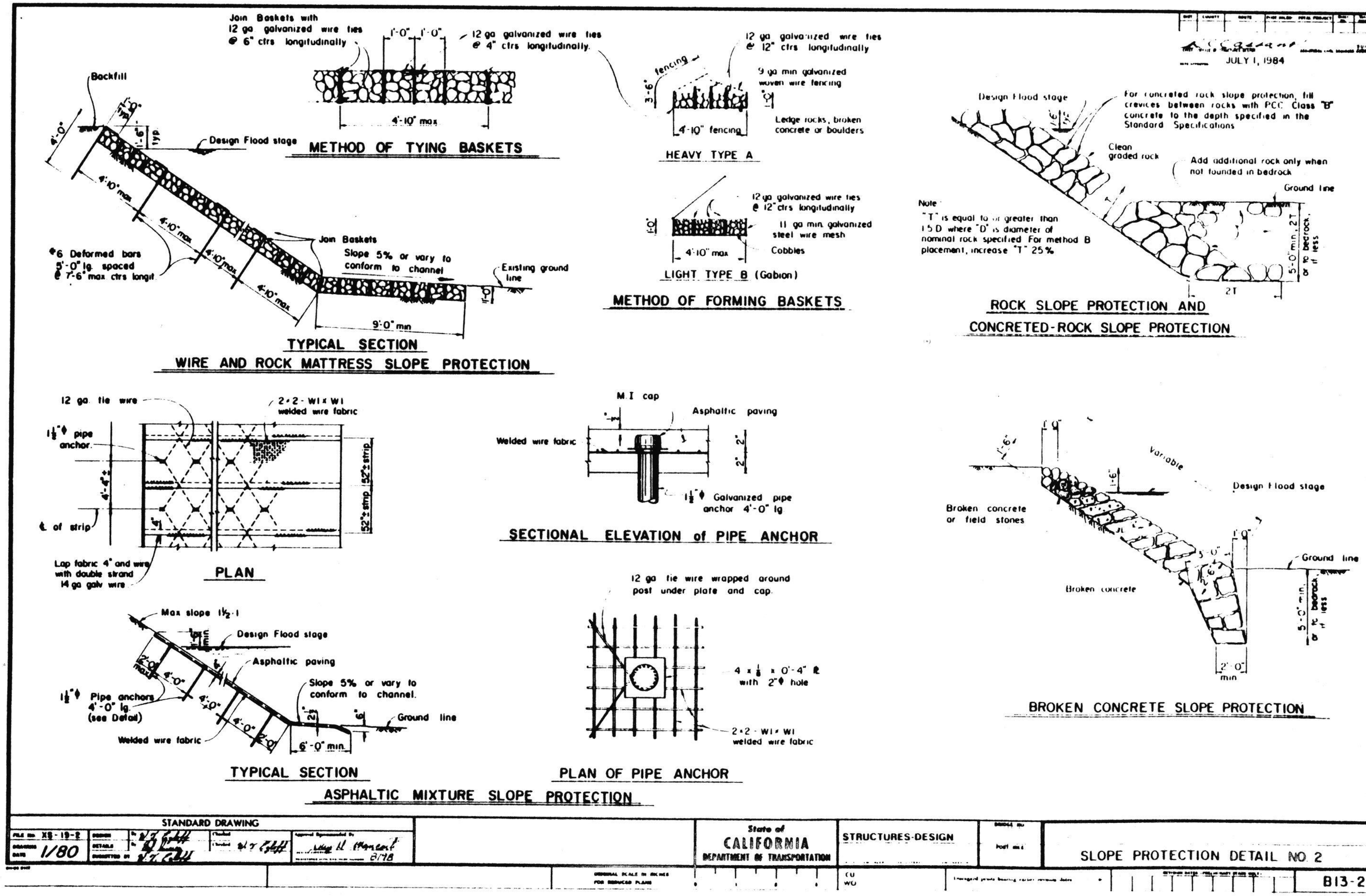
Local surface irregularities of the slope protection shall not vary from the planned slopes by more than one foot measured at right angles to the slope.

At the completion of slope protection work, the footing trench shall be filled with excavated material and compaction will not be required.

72-2.04 Measurement.—Rock slope protection will be paid for either by the ton or cubic yard as designated in the Engineer's Estimate.

Quantities of rock slope protection to be paid for by the cubic yard will be determined from the dimensions shown on the plans or the dimensions directed by the Engineer and rock slope protection placed in excess of these dimensions will not be paid for.


SECTION A-A
SECTION OF SCOUR CUT-OFF STUBS
Scour cut off stubs at 30' intervals. Construct similar to end return.
End return
PLAN OF FIRST COURSE IN TRENCH
3 Courses min.
Design Flood stage
Existing ground line
10' max for 1:1 slope only
5' min or to bedrock if less
9'-6" for 1½:1 slope
8'-2" for 1:1 slope
TYPICAL SECTION
SACKED CONCRETE SLOPE PROTECTION ON SLOPES 1½:1 & 1:1
Expansion joint with filler
Coat ledge with asphalt
Omit ledge at end cut-off stub
Flow
Measured perpendicular to slab
Excavate trench to neat lines and place concrete against undisturbed material.
SECTION B-B
Scale: ¾" = 1'-0"
Deleted Notes
#4 total 7
Place concrete in toe against undisturbed material except as permitted by the Engineer
Existing ground line
Design Flood stage
#4 spacers @ 18 ctrs
#4 @ 12 ctrs
Pervious material continuous between stubs
3" dia drains @ 6'± intervals
Scour cut off stubs @ 20' intervals and at ends
#4 stirrups @ 12 ctrs
Existing ground line
TYPICAL SECTION "A"
CONCRETE SLOPE PROTECTION
Scale ½" = 1'-0"
Slope protection
Support wall
See roadway plans for gutter or other improvement details
6 x 6 W3 x W3 welded wire fabric
Pervious material continuous
3" dia drains @ 12' intervals
#4 @ 12 ctrs
Place against undisturbed material except as permitted by the Engineer
TYPICAL SECTION "C"
Structure concrete
Gutter
6" rad
#4 @ 12 ctrs
Slope protection
Std retaining wall
TYPICAL SECTION "D"
Slope protection similar to Typical Section "A"
#4 @ 12 ctrs
Scour cut-off stubs @ 20' intervals and at ends
3" dia drains @ 6'± intervals
Pervious material continuous between stubs
#4 @ 8 ctrs
All others #4 @ 16 ctrs
Place concrete against undisturbed material except as permitted by the Engineer
Increase with "H" 3" per foot
TYPICAL SECTION "B"
JULY 1, 1984
STANDARD DRAWING
State of CALIFORNIA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN
SLOPE PROTECTION DETAIL NO. 1
B13-1
XS-19-1
1/84



Join Baskets with 12 ga galvanized wire ties @ 6" ctrs longitudinally
1'-0" 1'-0"
12 ga galvanized wire ties @ 4" ctrs longitudinally
4'-10" max
METHOD OF TYING BASKETS
Backfill
Design Flood stage
4'-0"
4'-10" max
#6 Deformed bars 5'-0" lg spaced @ 7'-6" max ctrs longit
Join Baskets
Slope 5% or vary to conform to channel
Existing ground line
9'-0" min
TYPICAL SECTION
WIRE AND ROCK MATTRESS SLOPE PROTECTION
12 ga galvanized wire ties @ 12" ctrs longitudinally
fencing
3'-6"
9 ga min galvanized woven wire fencing
4'-10" fencing
Ledge rocks, broken concrete or boulders
HEAVY TYPE A
12 ga galvanized wire ties @ 12" ctrs longitudinally
11 ga min galvanized steel wire mesh
4'-10" max
Cobbles
LIGHT TYPE B (Gabion)
METHOD OF FORMING BASKETS
JULY 1, 1984
Design Flood stage
For concreted rock slope protection, fill crevices between rocks with PCC Class "B" concrete to the depth specified in the Standard Specifications
Clean graded rock
Add additional rock only when not founded in bedrock
Ground line
Note:
"T" is equal to or greater than 1.5 D where "D" is diameter of nominal rock specified. For method B placement, increase "T" 25%
5'-0" min, 2T or to bedrock, if less
2T
ROCK SLOPE PROTECTION AND
CONCRETED-ROCK SLOPE PROTECTION
12 ga tie wire
2×2 - W1×W1 welded wire fabric
1½"Φ pipe anchor
℄ of strip
52"± strip
Lap fabric 4" and wire with double strand 14 ga galv wire
PLAN
M.I cap
Asphaltic paving
Welded wire fabric
1½"Φ Galvanized pipe anchor 4'-0" lg
SECTIONAL ELEVATION of PIPE ANCHOR
12 ga tie wire wrapped around post under plate and cap.
4 × ⅛ × 0'-4" ℞ with 2"Φ hole
2×2 - W1×W1 welded wire fabric
PLAN OF PIPE ANCHOR
Max slope 1½:1
Design Flood stage
Asphaltic paving
1½"Φ Pipe anchors 4'-0" lg (see Detail)
Slope 5% or vary to conform to channel.
Ground line
Welded wire fabric
6'-0" min
TYPICAL SECTION
ASPHALTIC MIXTURE SLOPE PROTECTION
Variable
Design Flood stage
Broken concrete or field stones
Broken concrete
Ground line
5'-0" min or to bedrock, if less
2'-0" min
BROKEN CONCRETE SLOPE PROTECTION
STANDARD DRAWING
XS-19-2
1/80
State of CALIFORNIA DEPARTMENT OF TRANSPORTATION
STRUCTURES-DESIGN
SLOPE PROTECTION DETAIL NO 2
B13-2

GEORGIA DEPARTMENT OF TRANSPORTATION

Department of Transportation
State of Georgia
Office of Materials and Research
15 Kennedy Drive
Forest Park, Georgia 30050

August 12, 1986

Mr. Thomas J. Parsons
Assistant Professor of Civil Engineering
Arkansas State University
Department Of Engineering
P.O. Drawer 1080
State Unversity, Arkansas 72467-1080

Dear Mr. Parsons:

Enclosed are two examples of our use of rock buttresses. One is a rock buttress used in construction of a roadway through rugged terrain. The other is the use of rock buttresses for landslide corrections. We commonly use rock buttresses to steepen slopes when rock is readily available.

Also, there are two other people that you might contact with possibly more experience in the use of rock buttresses. Mr. Larry Lockett and Mr. William D. Trolinger of the Alabama Highway Department and the Tennessee Department Of Transportation, respectively. I have attached their addresses and phone numbers for your use.

I hope this information will be of some use to you. If we can be of further assistance, please let us know.

Peggy E. McGee
Engineering Design Unit

PEMc:cdj

Mr. Larry Lockett
Geotechnical Engineer
Alabama Highway Department
11 South Union Street
Montgemery, Alabama 36130
Phone: 205-832-5506

Mr. William D. Trolinger
Assistant Chief of Soil Engineering
Tennessee Department of Transportation
2200 Charlotte Avenue
Nashville, Tennessee 37203
Phone: 615-741-4775

D.O.T. 66

DEPARTMENT OF TRANSPORTATION
STATE OF GEORGIA

INTERDEPARTMENT CORRESPONDENCE

FILE Slope Stability on State Route 136
Taylors Ridge - Walker County

OFFICE Materials & Research
Forest Park, Georgia

DATE December 21, 1984

FROM David A. Mitchell, Chief, Geotechnical Engineering Bureau

TO Steve Parks, District Maintenance Engineer

Landslide County

SUBJECT Slope Stability

We have evaluated the repair of the slope stability problem at the above mentioned location. The attached sketch shows our recommendations for installing the shear key. The following are our recommendations for reconstructing the slope. (See drawing for details).

1. The bottom of the shear key trench should be scarified as much as possible to provide good interlock with the rock key.
2. As much of the loose soils within shear zone should be removed before placing new fill.
3. The new fill should be benched into the existing slope. A benching detail sheet is attached.
4. We recommend that a good quality soil be used as replacement fill. Shales and shaley clays should be avoided.

It does not appear that the excavation of the shear key has endangered the two remaining lanes of traffic. The benching of the slope during fill placement will however possibly take out the remainder of the passing lane. Any springs or water flow encountered during the repair should be drained with underdrains. Any drainage should be carried to a point outside the slope area.

We will continue to work with you on this project. Please let us know if you have any questions or encounter problems during reconstruction.

Warren F. Bailey

Warren F. Bailey, P.E.
Engineering Design Unit

WFB:gt

Attachments

February 7, 1983

DEPARTMENT OF TRANSPORTATION
State of Georgia

SPECIAL PROVISION

PROJECT: APD-056-2(6) Pickens-Gilmer Counties

ROCK EMBANKMENT

The Contractor shall place rock embankment and/or "Rock Buttress" at the locations shown on the Plans and/or at similar locations as directed by the Engineer. The Contractor shall place the rock removed from the adjacent cuts into fill sections where either indicated on the Plans or the Engineer has determined that slope instability will occur.

The Contractor will stockpile rock for use as rock embankment at sites of removal and/or sites that will require construction of rock embankments. If rock embankment material is not available on the Project in the immediate vicinity of required embankments/buttress, it may be obtained from other areas on the Project. All available suitable material on the Project is to be exhausted before the Contractor will be permitted to obtain rock embankment material from other sources.

All unclassified excavation to be used in rock embankment and/or rock butress shall meet the requirements of Section 811 Rock Embankment and/or to be approved by the Engineer.

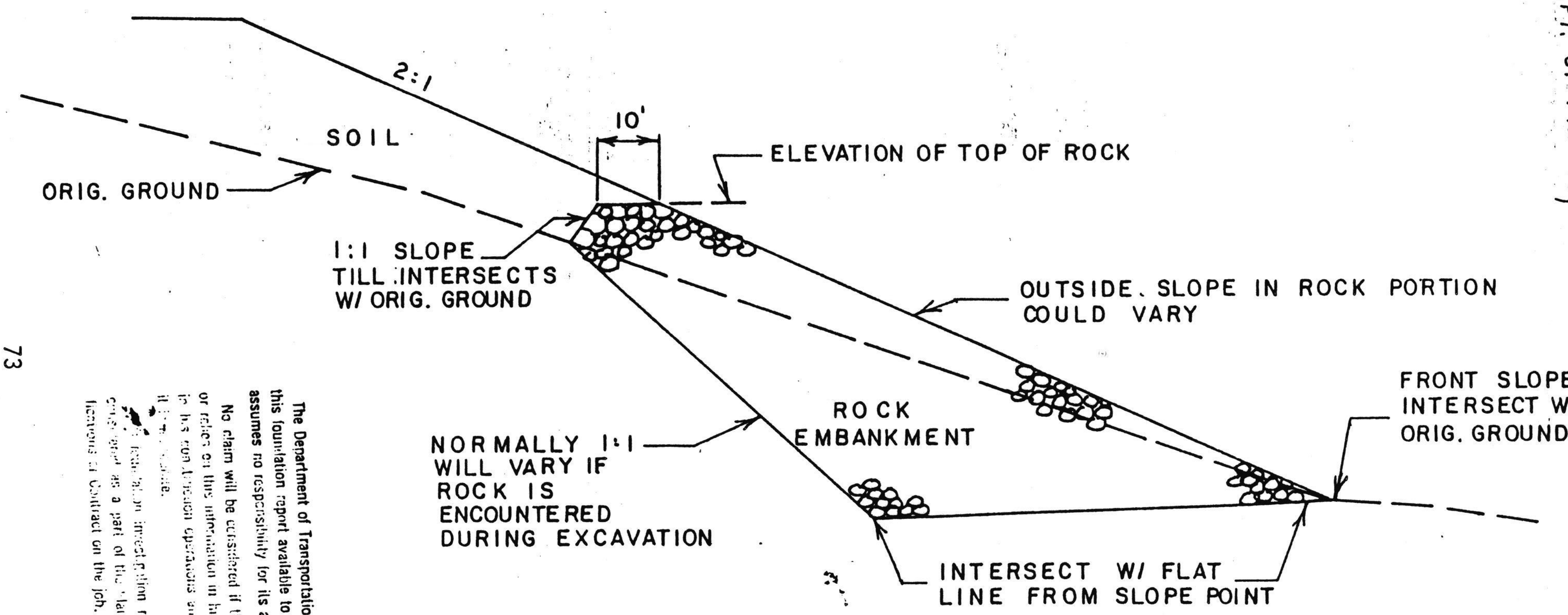
The rock embankment and/or rock butress shall be built as detailed in the Plans or as modified by the Engineer. The rock embankment work shall not commence on any site until the site is inspected and approval given by the Engineer. Any exceptions to the sites shown on the Plans or additional sites shall require the approval of the Engineer.

MEASUREMENT: Measurement for rock embankment and/or rock buttress will be in accordance with Section 208.05 of the Standard Specifications.

PAYMENT: Payment will be included at the contract unit price per cubic yard for rock embankment and will be full compensation for furnishing suitable material, hauling, placing, compacting, finishing and dressing in accordance with the Plans, Specifications or as directed by the Engineer.

Item No. 208 Rock Embankment------------------------------cu. yds.

GEORGIA (DEPT. OF TRANS., OFFICE OF MATERIALS & RESEARCH)



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.

[illegible] foundation investigation report is not considered as a part of the Plans and Specifications of Contract on the job.

NOTE: ROCK EMBANKMENT TOE TREATMENT THAT WAS USED SATISFACTORILY IN RUGGED TERRAIN.

ROCK EMBANKMENT DETAIL

IOWA DEPARTMENT OF TRANSPORTATION

Iowa Department of Transportation
800 Lincoln Way, Ames, Iowa 50010 (515)239-1008

August 13, 1986 REF: 570

Thomas J. Parsons
Arkansas State University
Department of Engineering
P.O. Drawer 1080
State University, Arkansas 72467-1080

Dear Mr. Parsons:

The Iowa Department of Transportation has used the rock buttress concept to repair landslides for approximately 10 years. Each design is site specific and we have no standard written procedures.

By intent, these designs are based on reinforcing the existing materials' structure. We use standard geotechnical design procedures utilizing "sliding block" and "Bishops" circular analysis programs. The dimensions of the rock mass are determined on the total strength required for a stable slope (increasing friction angle). Our rock is not confined and typically has a lesser bulk weight than the retained material.

We have established a minimum facing thickness of 10' at the most critical indicated failure surface and a minimum 5' load on the highest possible failure surface. This load and cover for the exposed rock face generally utilizes material available from construction excavation. We require that the buttress rock be well graded with a maximum 6" top size, and no more than 10% of the material passes the #8 sieve. We generally maintain an exposed slope angle of no more than 1½ horizontal to 1. vertical.

We have had good success with these designs. They are easily constructed and appear to be very forgiving. If problems do arise, they are easily repaired or modified. For this reason, we use a normal soil design safety tractor of 1.3.

If further information is required, please contact Kermit L. Dirks, Iowa Department of Transportation, 515/239-1476.

Sincerely,

R. L. Humphrey
Highway Division Director
Chief Engineer

RES:rc
cc: K. L. Dirks

KENTUCKY TRANSPORTATION CABINET

COMMONWEALTH OF KENTUCKY
TRANSPORTATION CABINET
FRANKFORT, KENTUCKY 40622

C. LESLIE DAWSON
SECRETARY

MARTHA LAYNE COLLINS
GOVERNOR

September 15, 1986

Mr. Thomas J. Parsons
Assistant Professor of Civil Engineering
Arkansas State University
P. O. Drawer 1080
State University, Arkansas 72467-1080

Dear Sir:

The Kentucky Department of Highways utilized rock buttresses extensively. They are constructed as randum shot rock fill, and may consist of a portion of the embankment toe area or toe berms to effectively flatten the slope, with or without shear keys. They are designed by conventional slope stability analysis procedures.

As such, I do not think our rock buttresses fit your definition of rock buttress walls. Anyway, I trust this answers your question. If you have additional questions, please call me at 502-564-3161.

Very truly yours,

DIVISION OF MATERIALS

Gordon Scott

Gordon Scott, Trans. Engr. II
KYDOH, Geotechnical Branch

GS:ks

MICHIGAN DEPARTMENT OF TRANSPORTATION

STATE OF MICHIGAN

JAMES J. BLANCHARD, GOVERNOR

DEPARTMENT OF TRANSPORTATION

TRANSPORTATION BUILDING, 425 WEST OTTAWA PHONE 517-373-2090
POST OFFICE BOX 30050, LANSING, MICHIGAN 48909

JAMES P. PITZ, DIRECTOR

August 14, 1986

Mr. Thomas J. Parsons
Assistant Professor of Civil
Engineering
Department of Engineering
P.O. Drawer 1080
State University, Arkansas 72467-1080

Dear Mr. Parsons:

Mr. W. J. MacCreery has asked that I reply to your inquiry of July 31, 1986, concerning rock buttress. The Michigan Department of Transportation has had only limited experience with rock buttresses or fills. In our Upper Peninsula we have constructed several fills of waste mine rock from the copper or iron workings and one "armored slope" which could be considered a buttress. We have also used them for Great Lakes shoreline protection.

The armored slope was simply a rough triangle of mine rock (mine rock is somewhere around 1' to 2' in size with all faces sharp and broken) about 8' high and 10' on the leg to support a wet sandy slope. A geotextile was used on the backside and the mine rock placed and tamped. There has not be any problems in 6-8 years.

Fills are end dumped of the same material with the outer slope holding at somewhere between a 1 on 1 and a 1 on 1.5 slope. The top is choked with smaller rock, sand subbase placed and paved. There is nothing special about it.

Shoreline protection is about the same as the armor slope but placed to protect from wave action. Several size stones are used from 3'+ armor thru 1-2' cover stone and a smaller bedding stone, all on geotextile, of course.

Hope this can be of some assistance. We haven't had much call for them.

Sincerely,

Thomas A. Coleman

Thomas A. Coleman
Construction Staff Engineer
Telephone: (517) 373-2301

NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION

The State of New Hampshire
Department of Transportation
John O. Morton Building

Wallace E. Stickney, P.E.
Commissioner

Hazen Drive
P.O. Box 483
Concord, N.H. 03301-0483

August 12, 1986

Thomas J. Parsons
Assistant Professor of
Civil Engineering
Arkansas State University
P.O. Drawer 1080
State University, Arkansas 72467-1080

Dear Sir:

In response to your letter of July 31, 1986, the State of New Hampshire, Department of Transportation has not designed or used a natural stone gravity retaining wall in any modern design for a considerable time. We have in past years been forced to tie into existing rock gravity dry stone retaining walls and we have checked the stability of these walls.

I know for certain there exists a design criteria for large cut dry stone gravity walls which were designed by the Boston & Maine Railroad Corp. Their address is:

Boston & Maine Corporation
Iron Horse Park
N. Billerica, MA 01862-1688

Tel: (617) 663-1112

Very truly yours,

Duncan S. Pearson

Duncan S. Pearson
Administrator
Bureau of Highway Design

DSP:HAS:s

NEW YORK DEPARTMENT OF TRANSPORTATION

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
ALBANY, N.Y. 12232

FRANKLIN E. WHITE
COMMISSIONER

August 20, 1986

Mr. Thomas J. Parsons
Assistant Professor of Civil Engineering
Department of Civil Engineering
Arkansas State University
State University, Arkansas 72467-1080

Dear Sir:

This is in reply to your letter of July 31, 1986, to the Chief Engineer, New York State Department of Transportation, requesting any procedures or copies of procedures that the Department might have for Rock Buttress Wall designs.

We find the Department does not presently have any procedures for design or use of this type retaining wall. Several years ago, the Department repaired and made short extensions to some existing walls of this type. We have attached a copy of the specification for these operations. We have no further information on this subject.

Very truly yours,

R. H. EDWARDS
Deputy Chief Engineer
Facilities Design Division

cc: E. A. Fernau, Soil Mechanics Bureau, 7-105

82

ITEM 08560.25 - STONE WALL RESTORATION (DRY)

Description:

Under this item, the Contractor shall furnish and build dry stone masonry walls at those locations shown on the plans or ordered by the Engineer.

Material:

Dry stone masonry walls shall be built of clean, roughly, rectangular, sound, field or quarry stone. At least four-fifths of the stone shall be over one-third cubic foot. Face stones shall have a minimum thickness of two inches and a minimum width of 10 inches. Selected stone, roughly squared and pitched to line shall be used at all angles and ends of walls.

Construction Details:

All portions of the existing stone wall which have been disturbed or are unstable shall be removed and sufficient material behind the wall will be excavated to provide ample work area for reconstruction.

The restored wall shall then be brought up to grade shown on typical sections.

The restored wall shall be laid to a line which as nearly as possible approaches the original line of construction.

Method of Measurement:

The quantity of stone masonry to be paid for under this item shall be the number of cubic yards measured in the completed work, and the limits shall not exceed those shown upon the plans or fixed by the Engineer.

Basis of Payment:

The unit price bid shall cover all labor, materials and incidental expenses necessary to satisfactorily complete the work including any excavation and backfill necessary.

Item 16560.1016 Remove and Reset-Stone Masonry (Dry)

Description

The Contractor shall carefully remove the existing stone masonry wall from the original position, clean the stones if necessary, and reset the stone masonry wall as shown on the plans, in accordance with the specifications, or as ordered by the Engineer.

Materials

Stone masonry used under this item is existing material.

Construction Details

The Contractor shall carefully remove the existing stone masonry wall and clean the stones as ordered by the Engineer. The stones shall be carefully stored at a location approved by the Engineer. When the south west abutment is completed, the Contractor shall reset the stone masonry wall using qualified personnel. The reset stone masonry wall shall have the appearance of the original stone masonry wall. The wall shall be re-established as shown on the plans.

Care shall be taken so as not to damage any of the stone masonry during the removal, storage, or resetting operations of the work. Any damage to the stone masonry, caused by the Contractor's carelessness, shall be repaired by the Contractor at no additional expense to the State.

Method of Measurement

The quantity of stone masonry removed and reset will be the number of square feet of reset wall measured along the top of the re-established stone masonry wall.

Basis of Payment

The unit price bid per square foot shall include the cost of furnishing all labor, materials, and equipment necessary to complete the work. This work includes the cleaning and storage of the stone masonry.

OKLAHOMA DEPARTMENT OF TRANSPORTATION

STATE OF OKLAHOMA
DEPARTMENT OF
TRANSPORTATION

200 N. E. 21st Street
Oklahoma City, OK 73105-3204

November 12, 1986

Refer: Our File No. 1-6-2-3

Mr. Thomas J. Parsons
Assistant Professor
Department of Engineering
P. O. Drawer 1080
State University, Arkansas 72467-1080

Dear Professor Parsons:

The Department of Transportation has in the past used rock buttress retaining walls primarily in shallow slide restoration contracts. The general analysis procedure is that outlined in Highway Research Board Special Report 29 for earth buttresses. The material specified for rock buttress walls is either native stone or crushed rock meeting our standard rip-rap specification underlain by standard bedding material (see enclosed standards).

If there are any further questions, please advise.

Sincerely,

J. D. Telford, P. E.
Materials Engineer

By:

James B. Nevels, Jr.

James B. Nevels, Jr., P. E.
Soils & Foundations Engineer

a

Enclosure

cc: Materials File
Soils & Foundations Branch

readings, will not be less than 20 mils nor more than 30 mils (0.51 – 0.76 mm).

Removability. Striping tape shall be removable by following the manufacturer's recommendations so long as the material is substantially intact. Removal shall not require sandblast, solvent or grinding methods and shall not result in objectionable staining of the pavement surface.

Durability and Wear Resistance. The striping material applied in accordance with manufacturer's recommended procedures shall be weather resistant and show no appreciable fading, lifting or shrinkage during the useful life of the line. Samples of material applied to standard specimen plates and tested in accordance with Federal Test Method Standard No. 141, Method 6192 using a CS-17 wheel; and 1000 gram load shall not wear through to the metallic surface after 2000 cycles.

The striping material shall be packaged in standard commercial containers so constructed as to insure acceptance by the carrier and prevent damage during shipment and storage.

The striping material as supplied shall be capable of being stored at temperatures up to 100° F (37° C) for periods up to one year without deterioration.

SECTION 713
STONE FOR MASONRY AND RIPRAP

713.01. MATERIALS COVERED. This Section covers stone for Ashlar Masonry, Mortar Rubble Masonry, Dry Rubble Masonry, Plain Riprap, Laid-Up Riprap or Grouted Riprap, precast concrete blocks for Laid-Up Riprap or Grouted Riprap, stone for Special Plain Riprap and material for Filter Blanket.

713.02. ASHLAR STONE. The stone shall be tough, dense, sound, and durable, resistant to weathering action and shall be free from seams, cracks, or other structural defects. Preferably, stone shall be from a quarry the product of which is known to be of satisfactory quality. Stone shall be of such character that it can be wrought to such lines and surface, whether curved or plain, as may be required. Any stone having defects which have been repaired with cement or other material shall be rejected.

Size. The individual stones shall be large and well proportioned. They shall not be less than 12 or more than 30 inches (31-76 cm) in thickness.

713.03. RUBBLE STONE. Stone for mortar rubble or dry rubble masonry shall be of approved quality, sound and durable, free from segregations, seams, cracks, and other structural defects or imperfections tending to destroy its resistance to the weather. Stone for mortar rubble shall be reasonably free from rounded, worn or weathered surfaces and weathered stone shall be rejected. Selected stones with flat faces as nearly parallel as practicable shall be used.

Size. Individual stones shall have a thickness of not less than 4 inches (10.2 cm) and a width of not less than 1.5 times the thickness. No stones, except headers, shall have a length less than 1.5 times their width.

713.04. RIPRAP STONE. General. Stone for riprap shall be hard, sound and durable and shall be approved by the Engineer prior to use. Samples of the stone to be used shall be submitted to and approved by the Materials Engineer before any stone is used.

Tests for weight and absorption will be determined in accordance with ASTM C97. The minimum weight shall be 140 pounds per cubic foot (2 243 kg/cu M) and the maximum absorption shall be 6 percent.

The size of stone for the various kinds of riprap shall be as follows:

(a) Stone for Plain Riprap.

Riprap Thickness		Maximum		Average Size		Not More Than 20 Percent Shall Weigh Less Than	
Inches	(cm)	Pounds	(kg)	Pounds	(kg)		
12	(30.5)	150	(68)	30–50	(14–23)	20	(9)
18	(45.7)	350	(159)	70–125	(32–57)	30	(14)
24	(61.0)	1000	(454)	225–400	(102–181)	40	(18)
30	(76.2)	1000	(454)	225–400	(102–181)	40	(18)

When placed on the embankment the smaller stones shall be well distributed throughout the mass. Neither the breadth or the thickness of any piece of riprap shall be less than 1/3 of its length.

(b) Stone for Laid Up or Grouted Riprap.

12 inch (30.5 cm) thick riprap size of stone 50 to 250 lbs. (23-113 kg) with at least 60 percent weighing 100 lbs. (45.4 kg) or more. 18 inch (45.7 cm) thick riprap size of stone 50 to 500 lbs. (23-227 kg) with at least 60 percent weighing 150 lbs. (68 kg) or more.

Slabs or sliver will be rejected. Spalls shall be well graded, of a suitable size for the work.

(c) Stone for Special Plain Riprap.

40 percent to 60 percent – 5 c.f. to 12 c.f. in volume
20 percent to 30 percent – 2 c.f. to 5 c.f. in volume
10 percent to 20 percent – 0.25 to 2 c.f. in volume
5 percent to 15 percent – may be less than 0.25 c.f. in volume

713.05. PRE-CAST CONCRETE BLOCKS. Pre-cast concrete blocks for laid-up riprap or grouted riprap shall have a vertical dimension of 6 to 8 inches, (15.2-20.3 cm) a horizontal dimension of from 8 to 16 inches, (15.2-40.6 cm) and a dimension perpendicular to the slope of the wall of 9 inches (22.9 cm).

Concrete blocks shall be made of Class C concrete or equivalent.

Cement-sand blocks will be permitted when composed of one part of cement to three parts of approved clean sand by volume.

713.06. FILTER BLANKET MATERIAL. Material for a filter blanket shall consist of sand, gravel, crushed stone, or other approved materials processed, blended, or naturally combined. It shall be reasonably free from lumps or balls of clay, organic matter, objectionable coatings, or other foreign materials, and shall be durable and sound. Blanket material shall be reasonably free from flat and/or elongated particles in an amount exceeding 20 percent. A flat or elongated piece is one the length is greater than 5 times the average thickness. The backing material in place shall be reasonably well graded within the following limits:

SINGLE COURSE BACKING (FILTER BLANKET)

Sieve Designation U.S. Standard Square Mesh		Percent by Weight Passing
Inch	(mm)	
4	(100)	100
2	(50)	60–90
1	(25)	40–70
3/8	(9.5)	15–40
No. 4	(4.75)	0–15

TWO COURSE BACKING (FILTER BLANKET)

Sieve Square Mesh		Percent by Weight Passing	
Inch	(mm)	Lower Course of Two Layers	Upper Course of Two Layers
6	(150)	–	100
4	(100)	–	90–100
2	(50)	–	65–85
1	(25)	–	40–70
3/8	(9.5)	100	15–35
No. 4	(4.75)	95–100	0–10
No. 8	(2.36)	80–90	–
No. 16	(1.18)	55–75	–
No. 30	(0.60)	30–60	–
No. 50	(0.30)	12–30	–
No. 100	(0.150)	0–10	–

OREGON DEPARTMENT OF TRANSPORTATION

VICTOR ATIYEH
GOVERNOR

Department of Transportation

HIGHWAY DIVISION

TRANSPORTATION BUILDING, SALEM, OREGON 97310

August 14, 1986

In Reply Refer to
File No.:

DES

Thomas J. Parsons
Assistant Professor
Arkansas State University
Department of Engineering
P.O. Drawer 1080
State University, AR 72467-1080

The Oregon State Highway Division does not design mortar-free, natural stone, gravity retaining walls (rockery walls).

We do design rock buttresses to stabilize landslides. Typically, the outer slope of the rock buttresses are 1.5:1 to 2:1. The size of each buttress is based on the resisting force required to stabilize the landslide, which is determined by a stability analysis.

If you desire further information, please contact George Machan, Geotechnical Supervisor, 1178 Chemeketa Street, Salem, OR 97310 or phone (503) 373-7994.

E S Hunter

E. S. Hunter
Assistant State Highway Engineer

Form 734-3122

RHODE ISLAND DEPARTMENT OF TRANSPORTATION

STATE OF RHODE ISLAND AND PROVIDENCE PLANTATIONS

Department of Transportation
DIVISION OF PUBLIC WORKS
State Office Building
Providence, R.I. 02903

August 14, 1986

Mr. Thomas J. Parsons
Assistant Professor of Civil Engineering
Arkansas State University
P.O. Drawer 1080
State University, Arkansas 72467-1080

Dear Mr. Parsons:

This is in response to your letter of July 31, 1986 regarding rock buttress wall designs.

The Rhode Island Department of Transportation does not use this type of wall, as described. The closest we have is a wet stone masonry wall, which is a gravity wall using natural stone. A copy of this detail is attached for your information.

Very truly yours;

Richard B. Kalunian
Bridge Design Engineer

RBK/fmv

REVISION	
BY	DATE

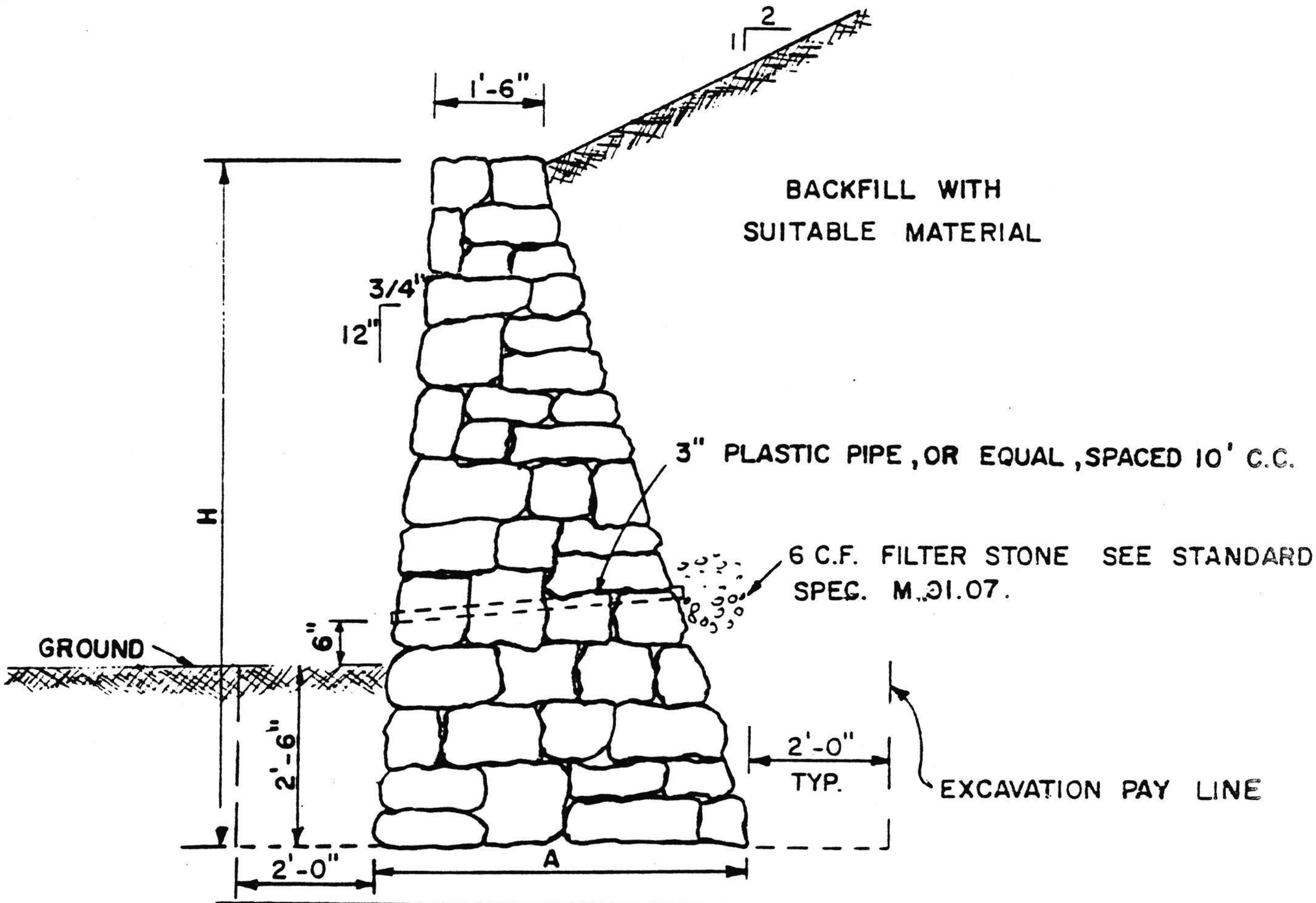
RHODE ISLAND DEPARTMENT OF TRANSPORTATION

DIVISION OF PUBLIC WORKS

WET STONE MASONRY

RETAINING WALL

R.I. STANDARD 10.1



DIMENSIONS AND QUANTITIES		
H	A	CU.FT./LIN.FT. OF WALL
5'-0"	3'-5"	12.30
6'-0"	3'-10"	16.02
7'-0"	4'-3"	20.16
8'-0"	4'-8"	24.72
9'-0"	5'-0"	29.25
10'-0"	5'-6"	35.00
11'-0"	6'-0"	41.25
12'-0"	6'-6"	48.00
13'-0"	7'-0"	55.25
14'-0"	7'-6"	63.00
15'-0"	8'-0"	71.25

Supervising Civil Eng.
Road Design

Chief Design Eng.
Public Works

Chief Engineer
Dept. of Transportation

WASHINGTON DEPARTMENT OF TRANSPORTATION

BEAM GUARD RAIL CONNECTION AT TRAFFIC BARRIER

Where shown in the plans, the Contractor shall connect beam guard rail to the precast concrete traffic barrier per Standard Plans C-3, Type 2 and C-5, Type A.

All costs for completing the connection as specified shall be incidental to and included in the unit contract price per linear foot for "Beam Guard Rail Type 1".

8 INCH QUARRY ROCK

Eight inch quarry rock shall consist of crushed quarry rock and shall meet the following requirements for gradation:

Passing 8 inch screen	100%
Passing 3 inch screen	20% maximum
Passing 3/4 inch screen	10% maximum
All percentages by weight.	

The stone shall be hard, sound and durable. It shall be free from segregation seams, cracks and other defects tending to destroy its resistance to weather.

Eight inch quarry rock will be measured by the ton of rock placed.

The unit contract price per ton for "8 Inch Quarry Rock" shall be full compensation for furnishing and placing the rock as shown in the plans and specified herein.

ROCK RETAINING WALL

This work shall consist of constructing rock retaining walls in accordance with the details shown in the plans and these special provisions.

The rock retaining wall shall be constructed of rock ranging in size from 400 pounds to 1,600 pounds and shall have a uniform range of size in between. The minimum rock weight shall increase from top to bottom with the top 3 feet consisting of rock having a minimum weight of 400 pounds in a cut and 800 pounds in a fill and with a minimum rock weight for each succeeding 3 foot zone increasing by an additional 400 pounds over the zone above.

The rock shall be hard, sound, and durable. It shall be free from segregation, seams, cracks, and other defects tending to destroy its resisistance to weather. Rock used shall have a density of at least 155 pounds per cubic foot. Each horizontal row of rocks shall be seated and bedded by placing and tamping backfill for rock wall material behind the rock to provide a stable condition for the entire wall. In addition, each rock shall be keyed into adjacent rocks by utilizing the natural irregular shapes of the rocks. Any large voids existing between each course of rock as it is placed shall be filled by wedging smaller rock of the same quality into the voids until the maximum remaining void is 1 inch.

The rock retaining wall shall be constructed one 3-foot zone course at a time. Rock selection and placement shall be such that at least 80 percent of the exposed face of the wall is rock.

A 6-inch tolerance will be allowed for the exterior slope plane and grade in the finished surface of the wall.

Measurement of rock retaining wall will be by the ton.

The unit contract price per ton for "Rock Retaining Wall" shall be full compensation for furnishing all labor, tools, material, and equipment necessary to construct the rock retaining wall as specified.

BACKFILL FOR ROCK WALLS

Backfill for the rock wall shall be made from crushed quarry rock of the same hardness and durability as the rock used for rock wall construction and shall meet the following gradation requirements:

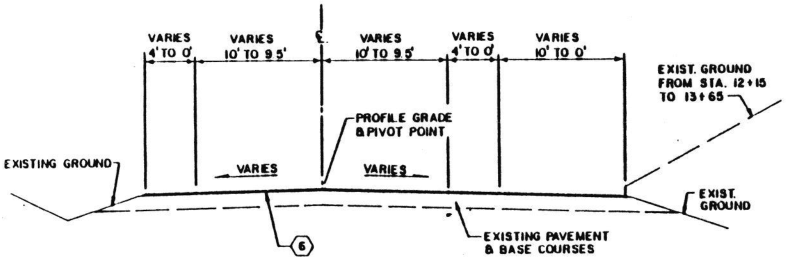
Passing 2-1/2 inch screen	90-100%
Passing 1-1/2 inch screen	50-80%
Passing 5/8 inch screen	0-20%

Backfill for rock retaining walls will be measured by the ton.

The unit contract price per ton for "Backfill For Rock Retaining Wall" shall be full compensation for furnishing all labor, materials, tools, and equipment and all other costs and expense for loading, hauling, and tamping the specified material.

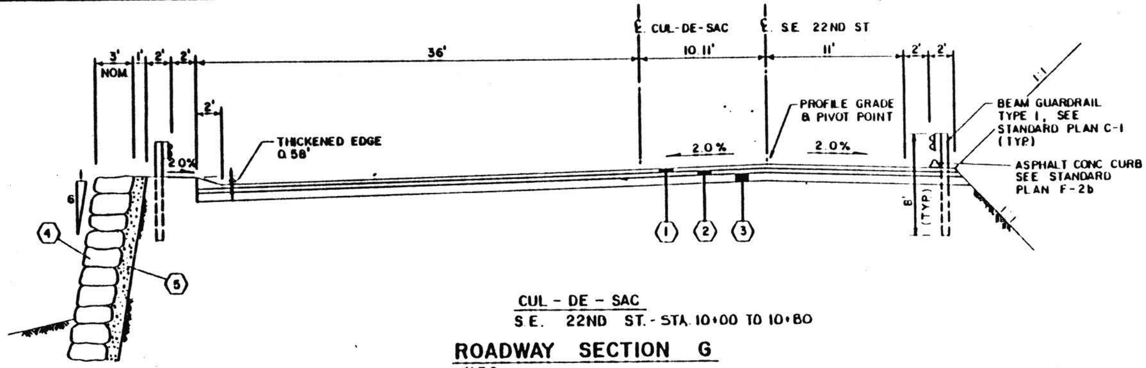
ILLUMINATION, TRAFFIC SIGNAL SYSTEMS, AND ELECTRICAL

Section 8-20 of the standard specifications is supplemented by the following:



REC. 60TH AVE. S.E. STA. 10+00 TO 16+87

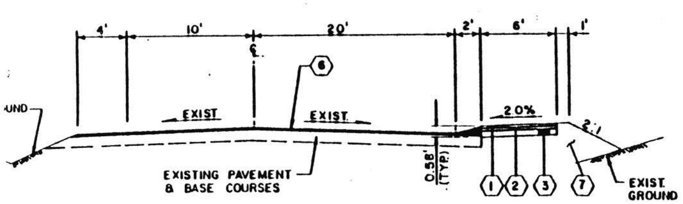
ROADWAY SECTION F
N.T.S.



CUL - DE - SAC
S.E. 22ND ST. - STA. 10+00 TO 10+80

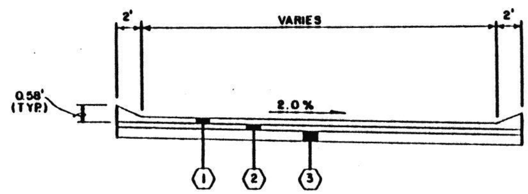
ROADWAY SECTION G
N.T.S.

NOTE: SEE MICRO TERRACING DETAIL SHEET C-35



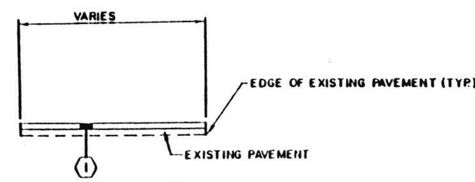
REC. S.E. 24TH ST. - STA. 21+40 TO 21+97

ROADWAY SECTION I
N.T.S.



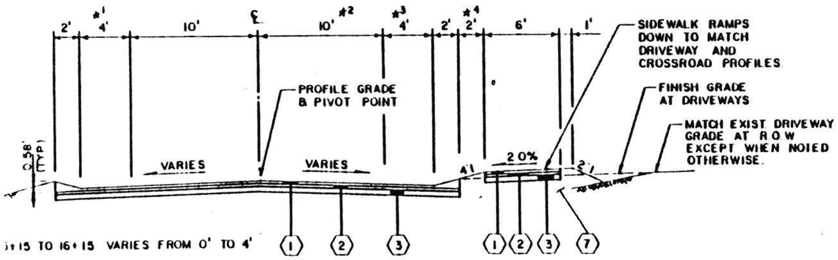
CUL - DE - SAC
WEST MERCER WAY - SEE SHEET C-31A

ROADWAY SECTION H
N.T.S.



WEST MERCER WAY - SEE LIMITS SHEET C-31A

ROADWAY SECTION P
N.T.S.



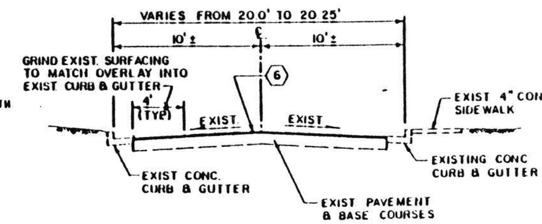
…5+15 TO 16+15 VARIES FROM 0' TO 4'
…1+08 TO 21+40 VARIES FROM 10' TO 20'
…1+08 TO 21+40 VARIES FROM 2' TO 0'
…5+00 TO 17+20 EQUALS 5'

REC. S.E. 24TH ST. - STA. 15+15 ± TO 21+40

ROADWAY SECTION J
N.T.S.

LEGEND:

1. 0.17' ACP PAVEMENT CLASS B COMP. DEPTH
2. 0.17' CRUSHED SURFACING TOP COURSE COMP. DEPTH
3. 0.33' GRAVEL BASE COMP. DEPTH
4. ROCK RETAINING WALL COMP. DEPTH
5. GRAVEL BACKFILL FOR WALL COMP. DEPTH
6. 0.10' OVERLAY ACP CLASS B COMP. DEPTH
7. GRAVEL BORROW



REC. S.E. 24TH ST. - STA. 10+00 TO 15+15

ROADWAY SECTION K
N.T.S.

WISCONSIN DEPARTMENT OF TRANSPORTATION

State of Wisconsin \ DEPARTMENT OF TRANSPORTATION

DIVISION OF HIGHWAYS AND
TRANSPORTATION FACILITIES
SOILS SECTION
3502 Kinsman Blvd.
Madison, WI 53704

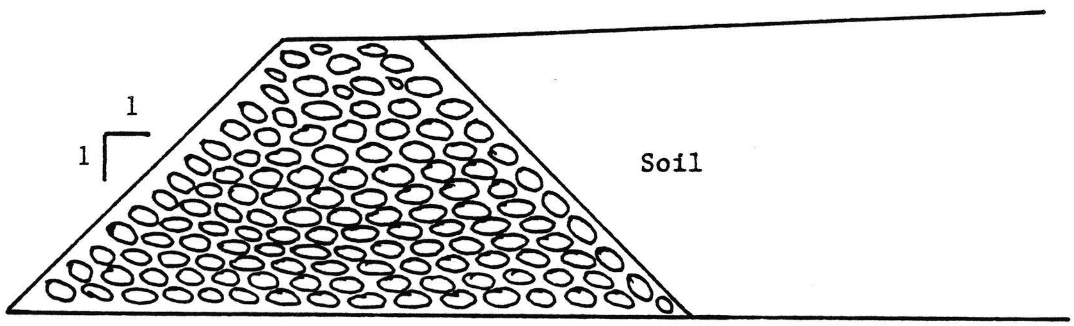
August 12, 1986

Professor Thomas J. Parsons
Assistant Professor of Civil Engineering
College of Engineering, Agriculture and
Applied Science
P.O. Drawer 1080
State University, AR 72467-1080

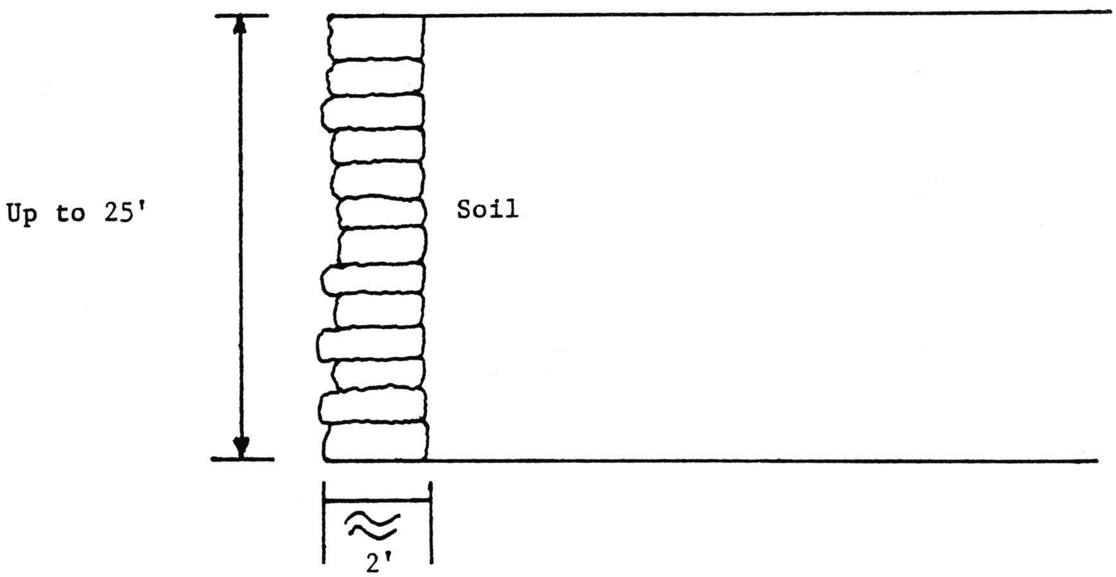
Dear Professor Parsons:

I am responding to your letter of July 31, 1986. I will attempt to answer although terminology may make a difference in our understanding. But first let me say we do not have a standard drawing or specifications for what we have done. I believe our basic usages with some hybrid have been:

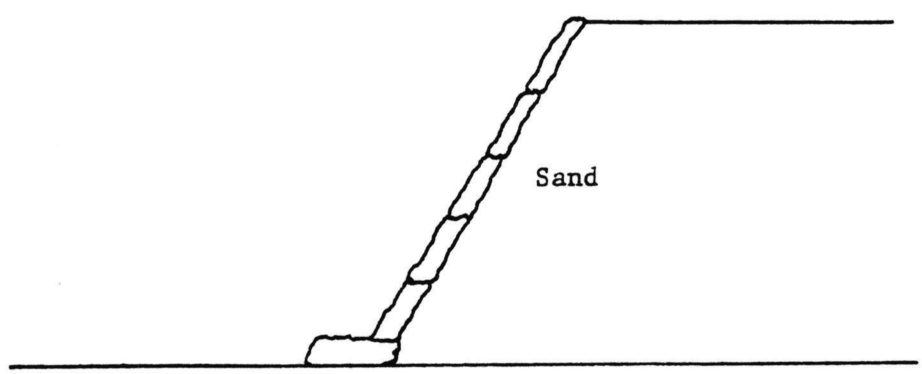
1. A large shot-run rock bulkhead, largely triangular or trapezoidal in shape. This had been used predominantly as an "anchor" at toes of fills or in landslide repair. The shape is usually:



2. Another is a facing in heavily preconsolidated tills where the benefits are largely erosion preventive. We make no effort at analyses but merely stock blocks along the face. We often feel this is a situation where a detailed study would prove they would not work - analogous to the bumblebee not flying.



3. A third type is in sand, the use of flat stones, possibly split concrete block, as largely an erosion protection in sands although in usage we do go to steeper slopes than allowed for an exposed face.



I trust this has been of some use to you. If we can assist further, please let us know. Ordinarily these have been predominantly expedient measures with little analyses, rather spur-of-moment direction to contractor (specification if you wish) and yet the success has been so good that we suspect that we are over conservative. Gabions are not considered in this class.

Sincerely

Clyde N. Laughter

Clyde N. Laughter
Chief Soils Engineer

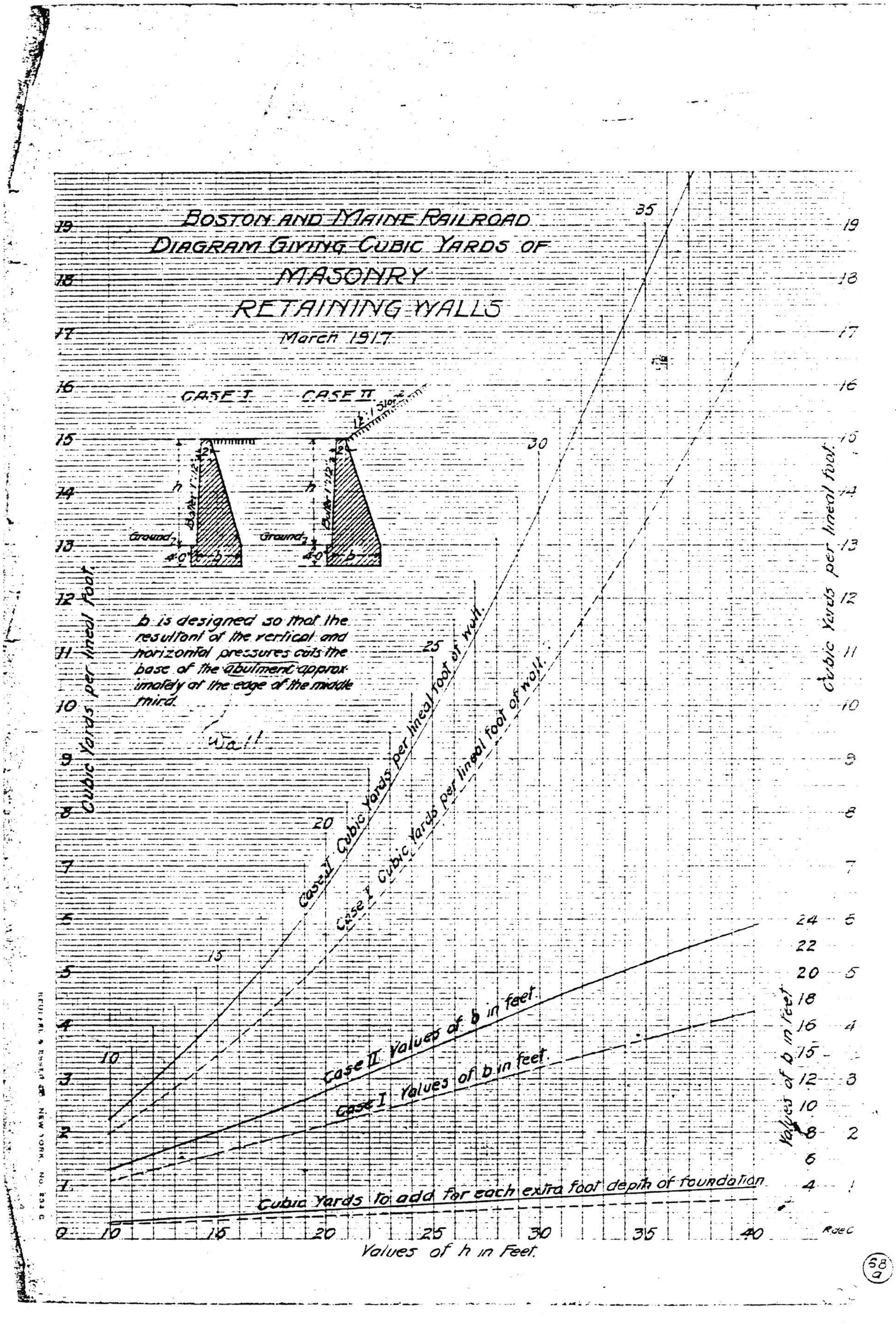
CNL:lcr

cc: GHZ
File

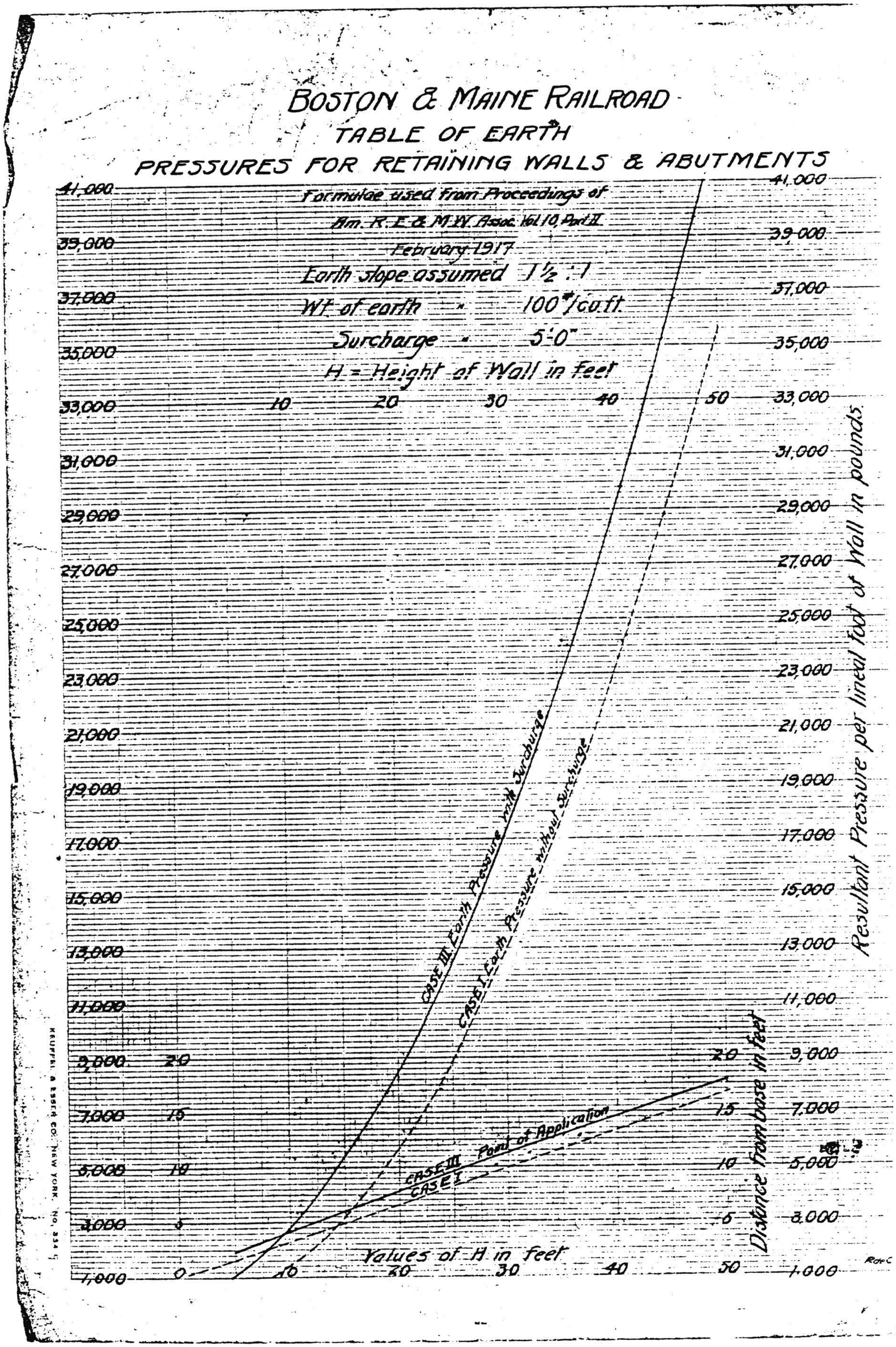
APPENDIX B

BOSTON - MAINE CORPORATION

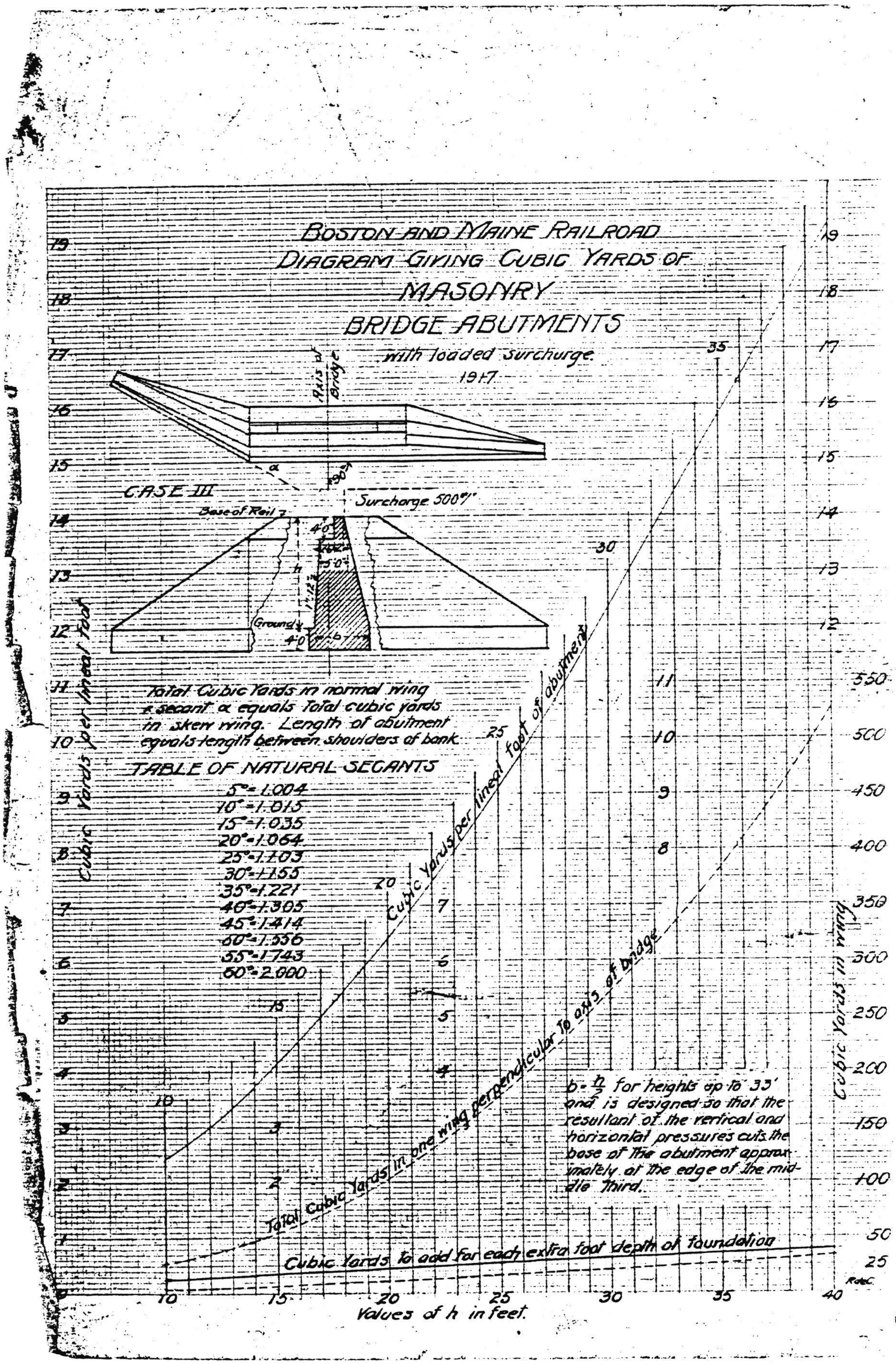
WALL DESIGN APPROACH



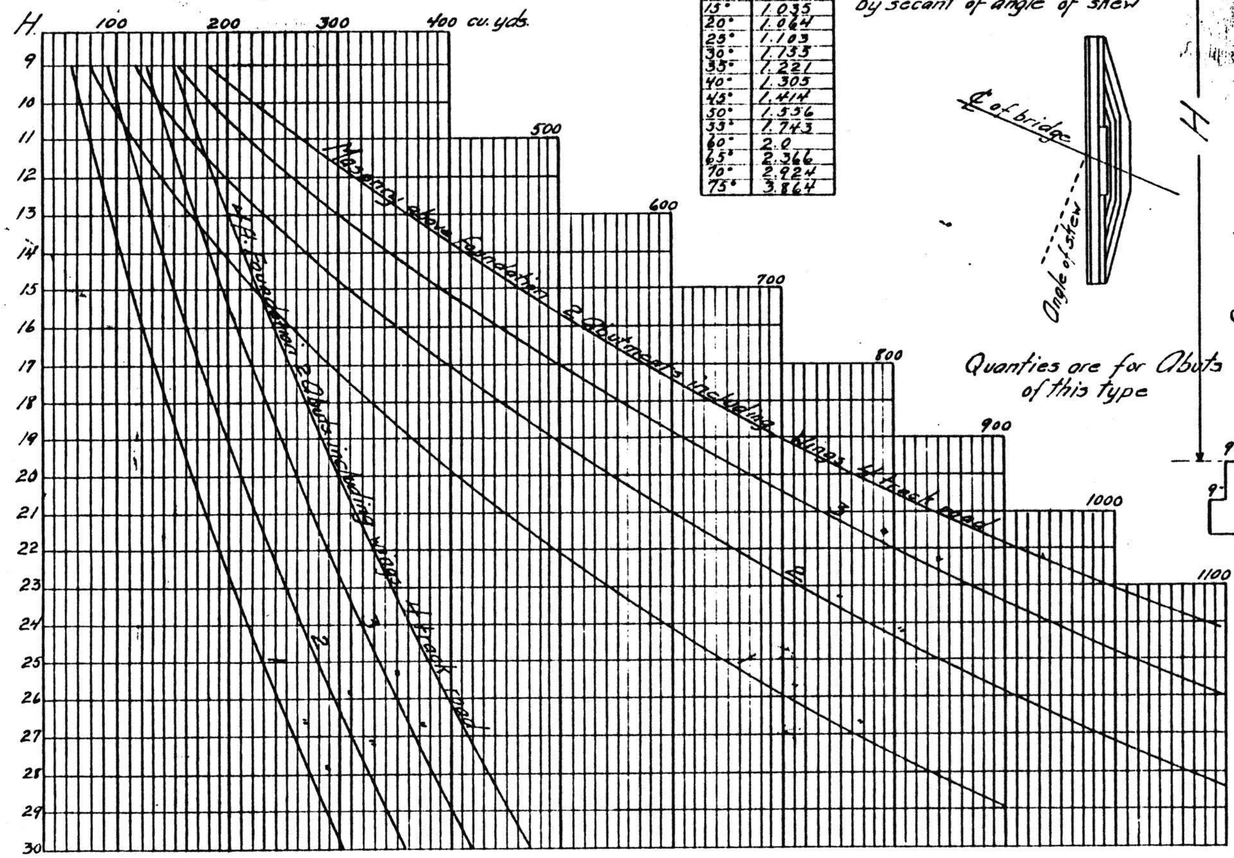
BOSTON AND MAINE RAILROAD
DIAGRAM GIVING CUBIC YARDS OF
MASONRY
RETAINING WALLS
March 1917
CASE I
CASE II
Ground
b is designed so that the resultant of the vertical and horizontal pressures cuts the base of the abutment approximately at the edge of the middle third.
Case II Cubic Yards per lineal foot of wall.
Case I Cubic Yards per lineal foot of wall.
Case II Values of b in feet.
Case I Values of b in feet
Cubic Yards to add for each extra foot depth of foundation
Cubic Yards per lineal foot.
Values of b in feet
Values of h in Feet.



BOSTON & MAINE RAILROAD
TABLE OF EARTH
PRESSURES FOR RETAINING WALLS & ABUTMENTS
Formulae used from Proceedings of
Am. R. E. & M. W. Assoc. Vol 10, Part II
February 1917
Earth slope assumed 1½ : 1
Wt. of earth = 100#/cu.ft.
Surcharge = 5'-0"
H = Height of Wall in feet
10 20 30 40 50
CASE III Earth Pressure with Surcharge
CASE I Earth Pressure without Surcharge
CASE III Point of Application
CASE I
Values of H in feet
0 10 20 30 40 50
Resultant Pressure per lineal foot of Wall in pounds
Distance from base in feet
41,000
39,000
37,000
35,000
33,000
31,000
29,000
27,000
25,000
23,000
21,000
19,000
17,000
15,000
13,000
11,000
9,000
7,000
5,000
3,000
1,000
20
15
10
5
0



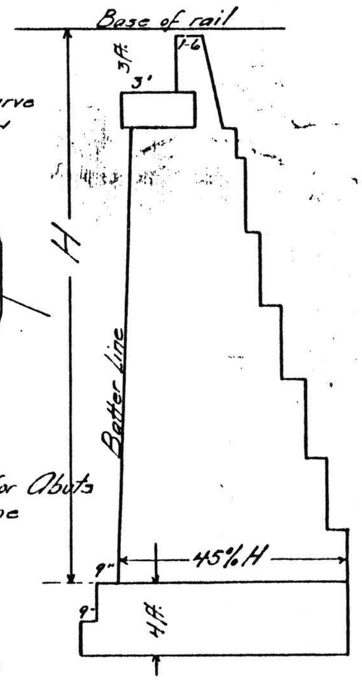
BOSTON AND MAINE RAILROAD
DIAGRAM GIVING CUBIC YARDS OF
MASONRY
BRIDGE ABUTMENTS
with loaded surcharge
1917
Axis of Bridge
CASE III
Surcharge 500#/'
Base of Rail
Ground
Total Cubic Yards in normal wing x secant α equals Total cubic yards in skew wing. Length of abutment equals length between shoulders of bank.
TABLE OF NATURAL SECANTS
5° = 1.004
10° = 1.015
15° = 1.035
20° = 1.064
25° = 1.103
30° = 1.155
35° = 1.221
40° = 1.305
45° = 1.414
50° = 1.556
55° = 1.743
60° = 2.000
Cubic Yards per lineal foot
Cubic Yards per lineal foot of abutment
Total Cubic Yards in one wing perpendicular to axis of bridge
Cubic Yards to add for each extra foot depth of foundation
b = h/2 for heights up to 33' and is designed so that the resultant of the vertical and horizontal pressures cuts the base of the abutment approximately at the edge of the middle third.
Cubic Yards in wing
Values of h in feet
68



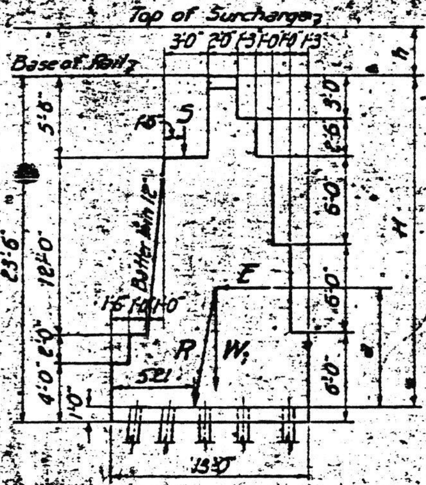
Angle	Secant
5°	1.004
10°	1.015
15°	1.035
20°	1.064
25°	1.103
30°	1.155
35°	1.221
40°	1.305
45°	1.414
50°	1.556
55°	1.743
60°	2.0
65°	2.366
70°	2.924
75°	3.864

For skewed abutments multiply quantity from curve by secant of angle of skew

Quantities are for Abuts of this type



Assumed Dead and Live Load = 650,000# Span Load.
Assume Cen. to Cen. of Girders = 16'-9"



SECTION THRU ABUTMENT

—DESIGN—

Surcharge distribution = 8.5 + H = 8.5 + 22.5 = 31'
" height (h) = 14,000 ÷ 31 × 100 = 4.5'
$E = .285\,H(H+2h)\frac{100}{2} = 10{,}190$#
$a = \frac{H^2+3Hh}{3(H+2h)} = 8.57'$
Overturning Moment = 8.57 × 10,190 = 86,500'#
Superstructure distribution = ℄ to ℄ of girders + 17.0 = 34'
" Dead and Live Load = 650,000 ÷ 34 = 19,100 #/ft.

Resisting Moment about Toe at Top of Piles.

	Weight		Arm		Moment
Concrete =	28,230#	×	7.07	=	199,800#
Earth Fill =	6,180	×	11.28	=	69,700
Super D&L =	19,100	×	6.0	=	95,500
Total	53,510				365,000#
Overturning Moment				=	−86,500
Stability					278,500#

Distance of R from Toe = 278,500 ÷ 53,500 = 5.21'
Middle third = 13.0 × ⅓ = 4.33 ∴ Resultant strikes within middle third.

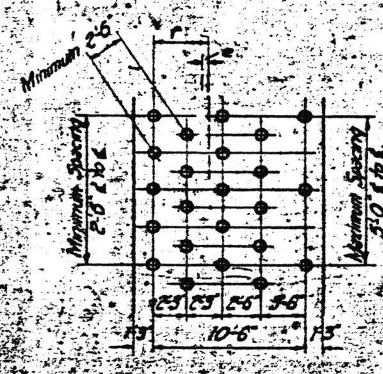
Resisting Moment Neglecting Super Live Load.

	Weight		Arm		Moment
Total	53,500#				278,500'#
Super Live Load	−14,300	×	5.0	=	−71,500
	39,200#				207,000'#

Distance of R from Toe = 207,000 ÷ 39,200 = 5.28'

Distance of Pile Group C.G. from front row = r = 4.22'
" " Resultant (R) " " " = 3.96'
Excentricity (e) = 4.22 − 3.96 = .26'
Moment = 53,500 × 0.26 = 13,900 '# = M.
Average Pile Load = 53,500 ÷ 18 = 29,700 # = P
Total " " = $P + \frac{Mc}{I}$
Load on front row = $29{,}700 + \frac{13{,}900}{19.69} \times 4.22 = 32{,}700$#
" " rear " " " " × 6.28 = 25,300#

NOTE:
Above design based on load of 15 tons per pile. Use 17.5 tons per pile unless specially instructed otherwise.



PILING PLAN

C.G. of Pile Group	I of Pile Group
$\frac{1}{2.5}$ − 0.4 × 0.0 = 0.0	$\frac{1}{2.5}$ × 4.22^2 = 7.12
" 4 × 2.25 = 0.9	" × 1.97^2 = 1.55
" 4 × 4.5 = 1.8	" × $.28^2$ = .03
" 4 × 7.0 = 2.8	" × 2.78^2 = 3.09
$\frac{1}{3.0}$ − 2 × 10.5 = 2.1	$\frac{1}{3.0}$ × 6.28^2 = 2.90
18 = 7.6	19.69
Dist. (r) = 7.6 ÷ 1.8 = 4.22	
" 5.21 − 1.25 = 3.96	
e = .26	

Boston and Maine Railroad
TYPICAL DESIGN OF ABUTMENT WITH PILES
Scale 1 in. = 10 ft.
From Engineering Department − Boston, Mass.

Issue		Drawn by H.R.D.	
A	10-1-30	Traced by J.F.L.	Approved:—
		Checked J.R.P.	Engineer of Structures
		Correct	

Earlier Issues Obsolete

Sheet 2 of 3 Sheets

9 × 13

PRESSURE ON FOUNDATION:-

The maximum pressures on the foundation shall be determined from the formulas and the footing designed so that these shall not be excessive. Where a pile foundation is used the pressures for each pile can be figured by taking the moment of inertia of the pile tops about an axis through the center of gravity and parallel with the axis of the pier under investigation, neglecting the moment of inertia about the gravity axis of the individual pile tops. The intensity of pressure at the corner of pier can be obtained by taking the sum of the pressures caused by the horizontal forces from both directions plus the pressure due to the vertical force.

GENERAL DESIGN:-

The top of the pier shall be a rectangle whose width (under coping) extends 3" beyond the edges of the base plate and whose length is not less than the distance out to out of superstructure bearings plus the width of pier. The dimensions should never be less than required for stability of the pier.

The coping shall be from 1½ to 2½ feet thick with an offset.

The shaft shall have a batter of ½" to 1" in 12"

Cut waters should begin not less than 2 feet above high water, and have a section and rake determined by local conditions.

STABILITY AGAINST OVERTURNING. (cont.)

If water can find its way under the foundation in hydrostatic condition. the weight of the part of the pier immersed will be diminished by $62\frac{1}{2}$ lbs. per. cu. ft., and the stability of the pier should be investigated for the different cases considering buoyancy.

When line of action of vertical forces coincides with the gravity axis of the pier the condition that the pier is stable against overturning is that e_1 and e_2 shall be $\leq \frac{1}{6}$ of the length of the base of the section investigated. This eccentricity should be determined for each of the above combinations of forces. The resultant eccentricity $= \sqrt{e_1^2 + e_2^2}$ and should be computed for the different cases to determine the maximum. If this is $\leq \frac{1}{6}$ of the diagonal of the section of the pier in question the pier is safe against overturning about that section.

If line of action of vertical forces does not coincide with gravity axis of pier the condition of stability is that $\frac{R}{M_1}$ or $\frac{R}{M_2} \geq 3$.

STABILITY AGAINST SLIDING:-

Sliding shall be investigated for the maximum conditions determined from the combinations of forces, using a coefficient of friction in accord with prevailing conditions at the site. The resultant tendency to slide is equal to the square root of the sum of the squares of the longitudinal and of the transverse forces tending to produce sliding.

STABILITY AGAINST CRUSHING:-

Stability against crushing shall be determined from the formulas, at base of shaft.

DESIGN:-

All piers shall be designed so as to be stable against-

(a) Overturning about base or any section above.

(b) Sliding on base or any section above.

(c) Crushing

at the base of the shaft and the bottom of the footing.

The pressure on the foundation shall not be excessive.

STABILITY AGAINST OVERTURNING:-

The following combinations of forces shall be used to determine the maximum tendency to overturning.

CASE I. Assume a wind pressure perpendicular to bridge tangent, 50#/□' and no train on the bridge.

Horizontal forces acting parallel to bridge tangent = E, N.

" " " perp. " " " = A, B, F, G, J, L.

Vertical forces = P_1, P_2, P_3, P_4, W_1

CASE II. Assume a wind pressure of 50#/□' parallel to bridge tangent and no train on bridge.

Horizontal forces acting parallel to bridge tangent = E, O, K, N.

" " " perp. " " " = F, G, L.

Vertical forces = $P_1, P_2, P_3, P_4, W_1, \ldots$

CASE III Assume a wind pressure of 30#/□' perpendicular to bridge tangent with train on bridge.

Horizontal forces acting parallel to bridge tangent = E, H, N.

" " " perp. " " " = A, B, C, D, F, G, J, L.

Vertical forces = $P_1, P_2, P_3, P_4, P_1', P_2', P_3', P_4', W_1$.

CASE IV. Assume a wind pressure of 30#/□' parallel to bridge tangent with train on bridge.

Horizontal forces acting parallel to bridge tangent = E, O, H, K, N.

" " " perp. " " " = C, F, G, L.

Vertical forces = $P_1, P_2, P_3, P_4, P_1', P_2', P_3', P_4', W_1$.

81 (4)

FORMULAS:-

$A = c \times 1.8 \times$ exposed surface of one truss applied half way between top and bottom chords.

$B = c \times$ area of floor system applied at center of side elevation.

C - Centrifugal force computed from Standard B.&M. curves.

D - 400 lbs × length of train in ft. applied 7'-0" above rail.

$E - 0.2 \times$ Dead load applied at top of shoes.

F = 400#/□" × area of pier covered by ice applied at high water mark.

$G = m \times V^2 \times a$ applied one third of the depth down from surface of water. $m = 1.24$ for square piers, 0.62 for circular piers, 0.46 for piers five or six times as long as broad, with cut waters the faces of which make an angle of 30°, and 1.29 for piers three times as long as broad with flat ends.

$H - 0.2 \times$ live load on spans applied at the rail.

$J = c \times$ area of end of pier above water applied half way between top of pier and water.

$K - c \times$ area of side of pier above water applied half way between top of pier and water.

$N - 14.25 \times h^2 \times l$ applied $\frac{h}{3}$ above section investigated.

$L - 14.25 \times h^2 \times b$ applied $\frac{h}{3}$ above section investigated.

$O = 0.7(A+B)$ applied at top of shoes.

$e_1 = \frac{M_1}{T}$

$e_2 = \frac{M_2}{T}$

$f = Tj \div \int$ of horizontal forces applicable.

$p = \frac{T}{lb} + \frac{6M_1}{bl^2} + \frac{6Tn}{bl^2}$

$p' = \frac{T}{lb} - \frac{6M_1}{bl^2} - \frac{6Tn}{bl^2}$

81 ③

NOMENCLATURE (cont.)

a = Area of pier exposed to current in square feet.

b = Width of pier at section investigated in feet.

c = Pressure exerted by wind in pounds per square foot.

d = Distance between middle of top of pier and loads from superstructure.

e_1 = Eccentricity of the resultant of the horizontal and vertical forces acting parallel to bridge tangent on the base of the section investigated.

e_2 = Eccentricity of the resultant of the horizontal and vertical forces acting perpendicular to bridge tangent on base of section investigated.

f = Factor of safety against sliding at section investigated.

h = Depth of embedment of pier below river bottom.

j = Coefficient of friction at section investigated.

l = Length of pier at section investigated in feet.

M_1 = Algebraic sum of moments of horizontal forces acting parallel to bridge tangent about section investigated.

M_2 = Algebraic sum of moments of horizontal forces acting perpendicular to bridge tangent about section investigated.

n = Eccentricity of line of action of dead and live loads from center of section, positive on leeward side and negative on windward side, n is dependent on conditions of loading from superstructure:- Example. n may be $= \frac{d[(P_3+P_3'-P_4)]}{W_1+(P_3+P_3')+P_4}$

p = Resultant pressure on leeward edge of section investigated.

p' = " " " windward " " " "

R = Moment of vertical forces taken about leeward edge of section investigated.

V = Velocity of stream in feet per second.

m = Value of $\frac{kw}{2g}$ in the formula $G = \frac{kw}{2g} V^2 a$

81 (2)

APPENDIX C

CALIFORNIA TEST SPECIFICATIONS 206 AND 229

1. California Test 206
   Method of Test for Specific Gravity and Absorption of Coarse Aggregate

2. California Test 229
   Method of Test for Durability Index

DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
Office of Transportation Laboratory
P. O. Box 19128
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California Test 206
1978

METHOD OF TEST FOR SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE

A. SCOPE

This test method, which is a modification of AASHTO Designation: T 85, specifies procedures for the determination of the bulk and apparent specific gravities and absorption of coarse aggregate.

B. APPARATUS

1. A balance having a capacity of at least 5,500 g. sensitive to 1 g. or less.
2. A wire mesh basket made of No. 8 mesh, and of sufficient capacity for samples weighing up to 5,500 grams.
3. Immersion tank of sufficient size to allow the wire mesh basket to be completely immersed. The immersion tank and balance shall be arranged in a manner that will allow weighing the wire mesh basket and test sample while immersed.
4. Corrosion resistant containers with a capacity of approximately 2-gallons.

C. PREPARATION OF SAMPLE

1. Rock Slope Protection: Crush the submitted sample to pass the 1½ inch sieve. Then sieve the crushed material over the 1½ inch, 1 inch and ¾ inch sieves. Prepare a test specimen weighing 5000 ± 500 g. by combining equal weights of the 1½ inch x 1 inch and 1 inch x ¾ inch sieve size fractions of material.
2. All Other Materials: Prepare a representative 5,000 ± 500 g. portion of the retained No. 4 sieve size material for testing.

D. TEST PROCEDURE

1. Place sample in 2-gallon container, cover with water at a temperature of 59° to 77° F., and soak for a minimum period of 15 hours.
2. Transfer the sample to the wire basket and rinse clean with fresh water.
3. Suspend the wire basket from the balance immersing the basket and sample completely in water and weigh to the nearest gram.
   a. Record the weight as "Weight of Sample in Water".
4. Transfer the sample onto a large absorbent cloth and remove all visible films of water.
   a. Surface water can be removed by rolling the sample in the cloth or by blotting with a towel.
   b. Large aggregate particles may be individually wiped with a cloth towel.
5. Weigh the sample to the nearest gram.
   a. Record the weight as "Weight of saturated surface-dry sample in air".
   b. Avoid loss of absorbed water by drying the sample to surface dry condition as rapidly as possible and then weighing immediately.
6. Transfer the sample to a suitable container and dry to constant weight at 230° ± 9° F. (110° ± 5° C.).
7. Cool to room temperature and weigh to nearest gram.
   a. Record the weight as "Oven-dry weight".

E. CALCULATIONS

1. Description of factor:
   A = weight in grams of sample in oven-dry condition.
   B = weight in grams of sample in saturated surface-dry condition, and
   C = in grams of saturated sample immersed in water.
2. Bulk specific gravity (oven-dry basis).
   a. Use this procedure for bituminous mix aggregates, aggregate base and cement treated base aggregate.
   b. Specific Gravity $= A/(B-C)$
3. Bulk specific gravity (saturated surface-dry basis)
   a. Use this procedure for portland cement concrete aggregates.
   b. Specific Gravity $= B/(B-C)$
4. Bulk specific gravity (apparent).
   a. Use this procedure for rock slope protection.
   b. Specific Gravity $= A/(A-C)$
5. Absorption.
   a. Percent Absorption $= [(B-A)/A] \times 100.$

F. PRECAUTIONS

When tare weights are used to compensate the weight of the basket and/or apparatus used to suspend the basket from the balance, be certain the correct tare weight is used.

G. REPORTING OF RESULTS

Report specific gravities to the nearest hundredth (2.65, 2.52, etc.), and absorptions to the nearest tenth (1.4, 2.3, etc.).

REFERENCES
AASHTO Designation: T85
End of Text (2 pgs) on Calif. 206

DEPARTMENT OF TRANSPORTATION
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California Test 229
1978

METHOD OF TEST FOR DURABILITY INDEX

A. SCOPE

The durability index test provides a measure of the relative resistance of an aggregate to producing clay-sized fines when subjected to prescribed methods of interparticle abrasion in the presence of water. Four procedures are provided for use with materials with various nominal sizes and specific gravities.

Procedure	*Designation*	*Type of Material*	*Section*
A	Dc	Retained No. 4 sieve	F ~~E~~-1
B........................	Dc "modified"	Lightweight or porous, retained No. 4 sieve	F ~~E~~-2
C	Df	Passing No. 4 sieve	F ~~E~~-3
D	Df "modified"	No. 4 x No. 16 sieve (pea gravel, chips)	F ~~E~~-4

B. APPARATUS.

The following equipment is required to perform this test. Detailed descriptions and specifications are included as necessary to assure standardization. Items bearing an Office of Business Management (OBM) catalog number are available to California State Agencies from the Department of Transportation, Office of Business Management. Detailed plans are available for those items bearing a Transportation Laboratory (TL) drawing number.

1. Agitator (Figure 1). A mechanical device designed to hold the wash vessel in an upright position while subjecting it to a lateral reciprocating motion at a rate of 285 ± 10 complete cycles per minute. The reciprocating motion shall be produced by means of an eccentric located in the base of the carrier and the length of the stroke shall be 1.75 ± .025 inches. The clearance between the cam and follower of the eccentric shall be .001 to .004 inches.

The combination sieve shaker-agitator, OBM catalog number 6635-0940-6, meets these requirements when in the agitation mode.

The Tyler portable sieve shaker meets these requirements when modified according to TL drawing number D-536.

2. Mechanical Sand Equivalent Shaker (Figure 2)

a. A mechanical device designed to hold a graduated plastic cylinder in a horizontal position while subjecting it to a reciprocating motion parallel to its length. The motion shall provide a stroke length of 8 ± 0.04 inches. The device shall operate at a speed of 175 ± 2 complete cycles per minute. Prior to use, the shaker shall be fastened securely to a firm and level mount.

b. OBM catalog number 6635-0930-5.

c. TL drawing number D-256.

3. Sand Equivalent Test Apparatus (Figure 3)

a. A graduated plastic cylinder, rubber stopper, irrigator tube, weighted foot assembly and siphon assembly, all conforming to the specifications and dimensions shown in TL drawing number C-218 (Figure 4).

A one gallon minimum size glass or plastic container with cover and fitted with the siphon assembly or a discharge tube near the bottom shall be used to disperse the working calcium chloride solution. The container shall be placed on a shelf or suspended above the work area in such a manner that the level of the solution is maintained between 36 and 46 inches above the work surface.

b. OBM catalog number 6635-0610-7.

4. Measuring Tin. A 3 ounce tin approximately 2¼ inches in diameter having a capacity of 85 ± 5 ml.

5. Wash Vessel. A flat bottomed, straight sided cylindrical vessel equipped with a watertight removable lid and conforming to the dimensions and tolerances shown in Figure 4.

The "Stainless Steel Pot", OBM catalog number 7330-0130-1, meets these requirements.

6. Collection Pot. A round pan or container having vertical or nearly vertical sides and equipped as necessary to hold the wire mesh of an 8-inch diameter sieve at least 3 inches above the bottom. An adaptor which will not allow loss of fines or wash water may be used to nest the sieve with the container, or the sieve may be nested with a blank sieve frame resting in the bottom of the pan.

7. Graduated Cylinder. A graduated cylinder having a capacity of 1000 mls.

8. Rubber Stopper. A stopper to fit the plastic cylinder.

9. Funnels

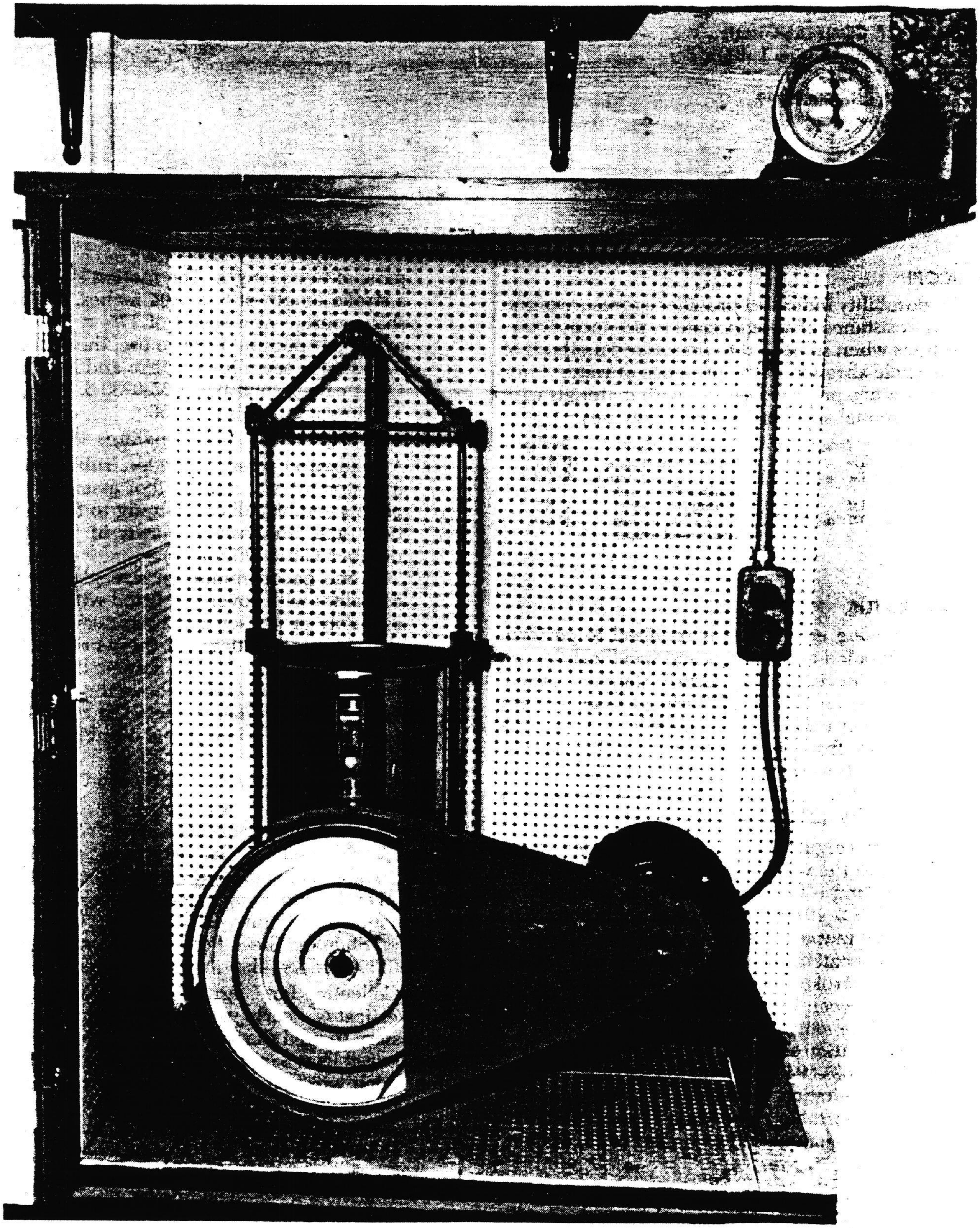

FIGURE 1

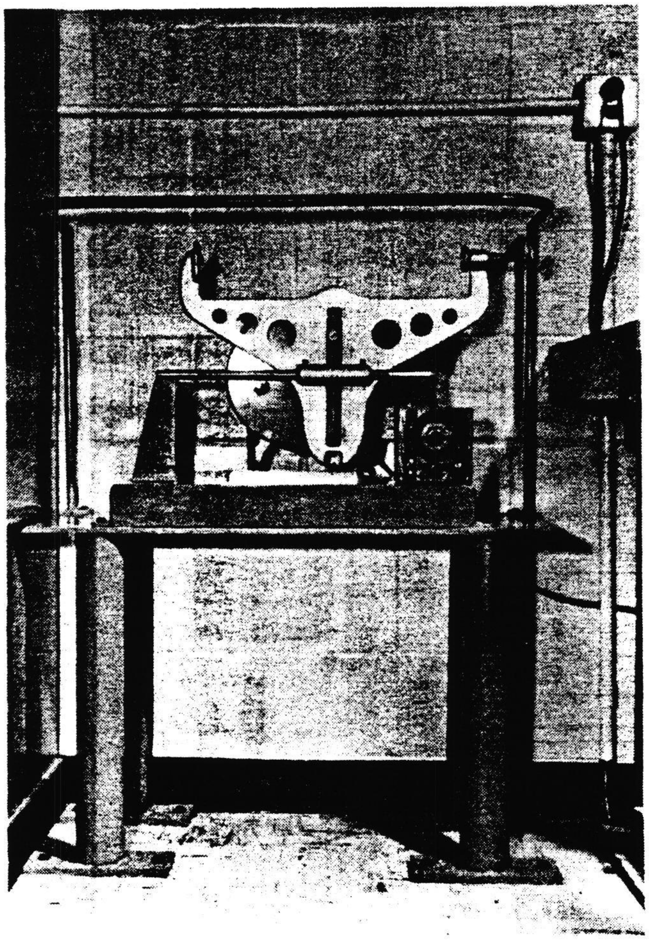

FIGURE 2

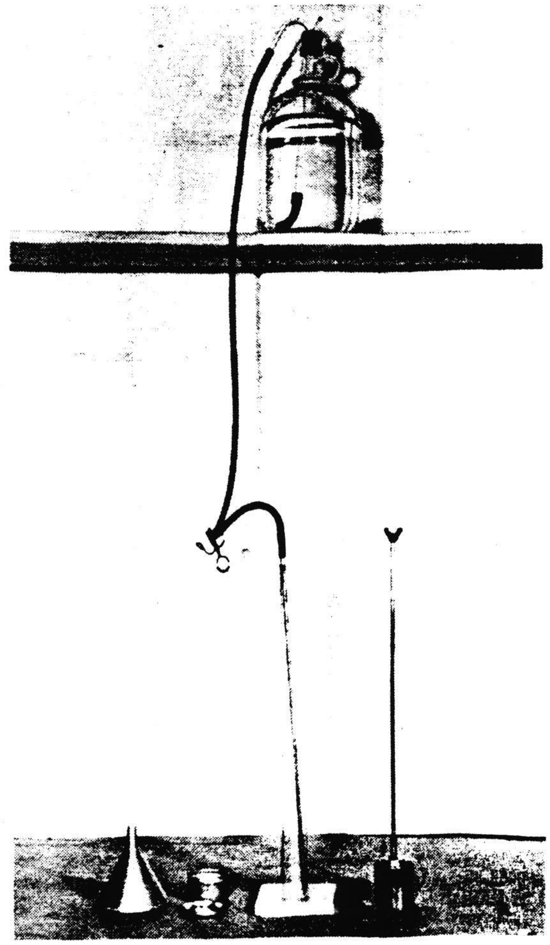

FIGURE 3

a. A wide mouth funnel suitable for directing water or aggregate into the plastic cylinder.
b. A wide mouth funnel large enough to hold an 8-inch diameter sieve while directing water into the plastic cylinder.

10. Balance. A balance or scale accurate to 0.2 percent of the weight of the sample to be tested.

11. Oven. A drying oven set to operate at 230° ± 9°F (110° ± 5°C).

12. Timer. A clock or watch reading in minutes and seconds.

13. Sieves. U.S. Standard Sieves, ¾ inch (19.0 mm), ½ inch (12.5 mm), ⅜ inch (9.5 mm), No. 4 (4.75 mm), No. 8 (2.36 mm) and No. 200 (0.075 mm). The No. 8 and No. 200 sieves shall be in standard 8-inch diameter frames.

14. Flexible Hose

C. MATERIALS

1. Stock Calcium Chloride Solution
   a. "Sand Equivalent Stock Solution", OBM catalog number 6810-0100-6.
   b. Solution may be prepared from the following:
      454 g (1 lb) tech. anhydrous calcium chloride
      2,050 g (1,640 ml) USP glycerine.
      47 g (45 ml) formaldehyde (40 percent by volume) solution.

Dissolve the calcium chloride in ½ gal of distilled or demineralized water. Cool the solution to room temperature, then filter it through Whatman No. 12 or equivalent filter paper. Add the glycerine and formaldehyde to the filtered solution, mix well, and dilute to 1 gal with distilled or demineralized water

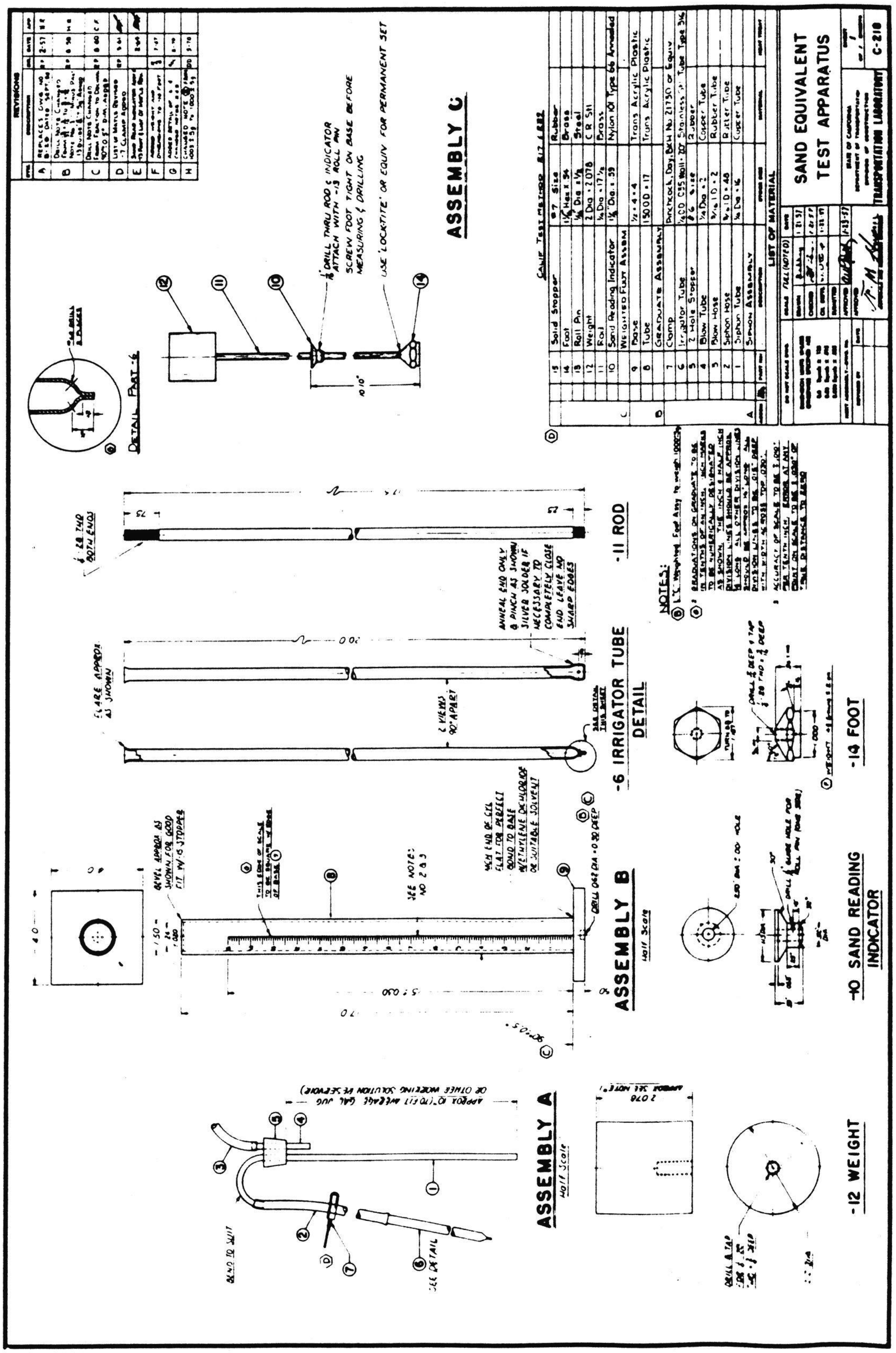
ASSEMBLY A
ASSEMBLY B
ASSEMBLY C
-12 WEIGHT
-10 SAND READING INDICATOR
-6 IRRIGATOR TUBE
DETAIL
-14 FOOT
-11 ROD
LIST OF MATERIAL
SAND EQUIVALENT
TEST APPARATUS
TRANSPORTATION LABORATORY
C-218

FIGURE 4

2. Working Calcium Chloride Solution. Prepare the working calcium chloride solution by diluting 85 ± 5 ml of the stock calcium chloride solution with water to obtain 1 gal of working solution.

3. Water. Use distilled or demineralized water for the normal performance of this test, including the preparation of the working calcium chloride solution. If it is determined, however, that the local tap water is of such quality that it does not affect the test results, it is permissible to use it in lieu of distilled or demineralized water.

D. CONTROL

The temperature of all solutions and water should be maintained at 72° ± 5°F during the performance of this test. Individual test results which meet the minimum durability index value when the temperature is below the recommended range are acceptable.

E. SAMPLE PROCESSING

1. Obtain a representative sample of the material to be tested.

2. Process the sample and separate on the No. 4 sieve according to the procedures in California Test 201. The material passing the No. 4 sieve is tested independently from the material retained on the No. 4 sieve. If either of these primary size portions amounts to less than 15% of the total sample, that portion should not be tested. The durability index of the tested portion will represent the entire sample.

3. Separate the retained No. 4 material on the ¾ inch, ½ inch and ⅜ inch sieves.

4. Calculate the size distribution of the ¾ inch x No. 4 portion of the material. Do not include the material retained on the ¾ inch sieve or the material passing the No. 4 sieve in this calculation.

5. Materials with a minimum nominal size larger than ¾ inch shall be crushed to pass the ¾ inch sieve and then processed as described below. The portion of the crushed material which passes the No. 4 sieve shall not be tested for durability index.

F. TEST PROCEDURES

1. Procedure A, Coarse Durability (D_c) for material retained on a No. 4 sieve.

a. Process the material to be tested as described in Section ~~D~~ E "Sample Processing".

b. Prepare a test specimen having an air-dry weight of 2550 ± 25 grams by combining the graded fractions as specified below.

(1) For materials which have a minimum of 10 percent in each of the specified fractions, prepare the test specimen according to the weights listed in Table No. 1.

Table No. 1

Basic Test Specimen Grading

Aggregate Passing	*Sieve Size Retained*	*Air-Dry Weight in grams*
¾ inch	½ inch	1070 ± 10
½ inch	⅜ inch	570 ± 10
⅜ inch	No. 4	910 ± 5
Total Test Specimen Weight		2550 ± 25

(2) For materials with less than 10 percent in any of the fractions specified in Table No. 1, prepare the test specimen using the actual calculated percentage for the deficient fraction and proportionally increase the weights of the remaining fractions to obtain the 2550 gram test specimen.

Example 1—Less than 10% of ¾ in. x ½ in. aggregate.

Aggregate Sieve Size	*Percent Each Size*	*Calculations*	*Air-Dry Weight Grams*
¾ in. x ½ in.	6	.06 x 2550	153 ± 10
½ in. x ⅜ in.	26	$\frac{570\ (2550 - 153)}{570 + 910}$	923 ± 10
⅜ in. x No. 4	68	$\frac{910\ (2550 - 153)}{570 + 910}$	1474 ± 5
Totals	100		2550 ± 25

Example 2—Less than 10% of ¾ in. x ½ in. and ½ in. x ⅜ in. aggregate.

Aggregate Sieve Size	*Percent Each Size*	*Calculations*	*Air-Dry Weight Grams*
¾ in. x ½ in.	4	.04 x 2550	102 ± 10
½ in. x ⅜ in.	7	.07 x 2550	179 ± 10
⅜ in. x No. 4	89	2550 − (102 + 179)	2269 ± 5
Totals	100		2550 ± 25

c. Wash the test specimen using the following procedure.

(1) Place the test specimen in the wash vessel.

(2) Add 1000 ± 5 ml water, clamp the lid in place and secure the vessel in the agitator.

(3) At 1 minute ± 10 seconds after adding the water to the specimen, start the agitator and shake the vessel for 2 minutes ± 5 seconds.

(4) Pour the contents of the vessel into a No. 4 sieve and rinse with fresh water until the water passing through the sieve is clear

d. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.

e. Abrade the test specimen using the following procedure.

(1) Place the washed and dried test specimen in the wash vessel.

(2) Add 1000 ± 5 ml water, clamp the lid in place and secure the vessel in the agitator.

(3) At 1 minute ± 10 seconds after adding the water to the specimen, start the agitator and shake the vessel for 10 minutes ± 15 seconds.

f. Separate the aggregate and water on the No. 200 sieve.

(1) Remove the lid from the wash vessel and bring the fines into suspension by holding the vessel in an upright position and moving it vigorously in a horizontal circular motion 5 or 6 times causing the contents to swirl inside.

(2) Immediately pour the contents of the vessel into the No. 8 and No. 200 sieves nested over the collection pot.

(3) Tilt the No. 8 sieve to promote drainage, then discard the material retained on the No. 8 sieve.

(4) Collect all of the wash water and minus No. 200 sieve material in the collection pot. To assure that all material finer than the No. 200 sieve is washed through the sieve, use the following procedure:

(a) As the wash water is draining through the No. 200 sieve, apply a jarring action to the sieve by lightly bumping the side of the sieve frame with the heel of the hand.

(b) When a concentration of material is retained on the No. 200 sieve, rerinse this fine material by pouring the wash water through the sieve again, using the following procedure:

(1) Allow the wash water to stand undisturbed in the collection pot for a few moments to permit the heavier particles to settle to the bottom.

(2) Set the No. 200 sieve aside and pour the upper portion of the wash water into a separate container.

(3) Place the No. 200 sieve back on the collection pot and pour the water back through the material on the No. 200 sieve. (If two collection pots are available the specimen may be rinsed by alternately placing the sieve on one and then the other while pouring the wash water through the material on the sieve. Before each rinsing allow the heavier particles to settle to the bottom and pour only the upper portion of the water through the material.)

(4) Repeat this procedure as necessary until all of the minus No. 200 material has been washed through the sieve. When the material has been rinsed sufficiently the material on the sieve will be free of visible streaks of clay and the wash water will flow freely through the sieve and accumulated material.

g. Pour all of the wash water and passing No. 200 sieve material into a graduated cylinder. Use fresh water as necessary to flush all the fines from the collection pot and adjust the volume to 1000 ± 5 mls.

h. Return the wash water to the collection pot taking care to include all water and fines.

i. Fill the graduated plastic cylinder to the 0.3 inch mark with stock calcium chloride solution and place the funnel on the cylinder.

j. Stir the wash water vigorously with one hand to bring all the fines into suspension. Use a circular motion allowing the fingers to rub the sides and bottom of the collection pot.

k. Immediately fill the graduated plastic cylinder to the 15-inch mark with the turbulent wash water.

l. Stopper the cylinder and thoroughly mix the wash water and calcium chloride solution by inverting the cylinder 20 times in approximately 35 seconds. Allow the air bubble to completely transverse the length of the cylinder each time.

m. Immediately place the cylinder on a work bench or table free of vibrations, remove the stopper, and allow the cylinder to stand undisturbed for 20 minutes ± 15 seconds.

n. Immediately read the top of the sediment column to the nearest 0.1 inch.

o. Determine the coarse durability index (d_c) from Table No. 2.

TABLE NO. 2
DURABILITY INDEX OF COARSE AGGREGATE AND CHIPS

Sediment height (inches)	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*	*Sediment height (inches)*	*Durability index*
0.0	100	3.0	53	6.0	39	9.0	29	12.0	18
0.1	96	3.1	52	6.1	38	9.1	29	12.1	18
0.2	93	3.2	52	6.2	38	9.2	28	12.2	18
0.3	90	3.3	51	6.3	38	9.3	28	12.3	17
0.4	87	3.4	51	6.4	37	9.4	28	12.4	17
0.5	85	3.5	50	6.5	37	9.5	27	12.5	16
0.6	82	3.6	49	6.6	37	9.6	27	12.6	16
0.7	80	3.7	49	6.7	36	9.7	27	12.7	15
0.8	78	3.8	48	6.8	36	9.8	26	12.8	15
0.9	76	3.9	48	6.9	36	9.9	26	12.9	14
1.0	74	4.0	47	7.0	35	10.0	26	13.0	14
1.1	73	4.1	47	7.1	35	10.1	25	13.1	13
1.2	71	4.2	46	7.2	35	10.2	25	13.2	13
1.3	70	4.3	46	7.3	34	10.3	25	13.3	12
1.4	68	4.4	45	7.4	34	10.4	24	13.4	12
1.5	67	4.5	45	7.5	34	10.5	24	13.5	11
1.6	66	4.6	44	7.6	33	10.6	24	13.6	11
1.7	65	4.7	44	7.7	33	10.7	23	13.7	10
1.8	63	4.8	43	7.8	33	10.8	23	13.8	9
1.9	62	4.9	43	7.9	32	10.9	23	13.9	9
2.0	61	5.0	43	8.0	32	11.0	22	14.0	8
2.1	60	5.1	42	8.1	32	11.1	22	14.1	7
2.2	59	5.2	42	8.2	31	11.2	22	14.2	7
2.3	59	5.3	41	8.3	31	11.3	21	14.3	6
2.4	58	5.4	41	8.4	31	11.4	21	14.4	5
2.5	57	5.5	40	8.5	30	11.5	20	14.5	4
2.6	56	5.6	40	8.6	30	11.6	20	14.6	4
2.7	55	5.7	40	8.7	30	11.7	20	14.7	3
2.8	54	5.8	39	8.8	29	11.8	19	14.8	2
2.9	54	5.9	39	8.9	29	11.9	19	14.9	1
								15.0	0

2. Procedure B, Coarse Durability (D_c) "Modified" (for lightweight or porous aggregates)

Because of the low specific gravity and/or high absorption rate of some aggregates, the proportions of aggregate to wash water are too great to permit the intended interparticle abrasion. Testing of these materials will require adjustment of the test specimen weight and volume of test water. All materials which are not completely inundated, when 1000 mls of water are added to a ~~2500~~ 2550 gram test specimen, shall be tested according to Method A with the following modifications.

a. Determine the bulk, oven-dry specific gravity and the percentage of absorption of the aggregate in accordance with California Test 206.

b. Adjust the total weight of the test specimen specified in E-1-b using the formula:
Adjusted Specimen Wt. (grams) = ((Specific Gravity of Aggregate)/2.65) x ~~2500~~ 2550

c. Adjust the weight of material in each size fraction proportionally to the weights specified in E-1-b.

d. Adjust the volume of test water specified in E-1-c and E-1-e using the formula except that the volume of water shall always be at least 1000 mls.
Adjust Water = 1000 + (A x W) − 50
Where: A = Absorption of Aggregate (%)
W = Weight of Test Specimen

3. Procedure C, Fine Durability (D_f) for material passing a No. 4 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Split or quarter 500 ± 25 grams of material from the passing No. 4 portion of the sample.
(1) See step 3-f for optional preparation procedure.

c. Dry to constant weight at 230° ± 9°F and cool to room temperature.

d. Wash the dried material by the following procedure:
(1) Place the material in the wash vessel.
(2) Add 1000 ± 5 mls of water, clamp the lid in place and secure the vessel in the agitator.
(3) At 10 minute ± 30 seconds after adding water to the material start the agitator and shake the vessel for 2 minutes ± 5 seconds.
(4) Pour the contents of the vessel into a No. 200 sieve and rinse with fresh water until the water passing through the sieve is clear. Use a flexible hose attached to a faucet to direct water onto the material.

e. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.
(1) Use water from the flexible hose as necessary to rinse the material from the sieve into the pan.
(2) Free water can be removed by tilting the pan and then, after the fines have settled, carefully pouring off the clear water.

f. A 500 gram fine sieve analysis test specimen which has been tested in accordance with California Test 202, may be utilized in lieu of the material prepared according to steps b. through e. above. If the fine sieve analysis test specimen is used, all of the material separated during sieving including that portion retained in the sieve pan shall be thoroughly recombined before proceeding to step g. below.

g. ~~Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full.~~
~~(1) When filling the measuring tin, consolidate the material in the tin by tapping the bottom edge on a hard object such as the work bench.~~
~~(2) Fill the measuring tin to slightly rounded above the brim and then strike off to level full using a straightedge.~~

See Below

h. Fill the graduated plastic cylinder to 4 ± 0.1 inches with working calcium chloride solution.

i. Pour the prepared test specimen into the plastic cylinder.
(1) Use the funnel to avoid spillage.
(2) Release air bubbles and promote thorough wetting by bumping the base of the cylinder against a firm object while the test specimen is being poured into the cylinder or by tapping the cylinder sharply on the heel of the hand several times after the test specimen has been poured in.

j. Allow the wetted material to stand undisturbed for 10 ± 1 minutes.

k. Abrade the test specimen by the following procedure:
(1) At the end of the 10 minute soaking period, stopper the cylinder, then loosen the material from the bottom by shaking the cylinder while holding it in a partially inverted position.
(2) Secure the cylinder in the mechanical sand equivalent shaker.
(3) Start the shaker and allow it to operate for 10 minutes ± 15 seconds.

Insert "Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full. Pre-determine the exact amount of material to be split using the following procedure.

(1) Fill the measuring tin to overflowing with the prepared material.

(2) Consolidate the material in the tin by tapping

Because of the low specific gravity and/or high absorption rate of some aggregates, the proportions of aggregate to wash water are too great to permit the intended interparticle abrasion. Testing of these materials will require adjustment of the test specimen weight and volume of test water. All materials which are not completely inundated, when 1000 mls of water are added to a ~~2500~~ 2550 gram test specimen, shall be tested according to Method A with the following modifications.

a. Determine the bulk, oven-dry specific gravity and the percentage of absorption of the aggregate in accordance with California Test 206.

b. Adjust the total weight of the test specimen specified in E-1-b using the formula:
Adjusted Specimen Wt. (grams) = ((Specific Gravity of Aggregate)/2.65) x ~~2500~~ 2550

c. Adjust the weight of material in each size fraction proportionally to the weights specified in E-1-b.

d. Adjust the volume of test water specified in E-1-c and E-1-e using the formula except that the volume of water shall always be at least 1000 mls.

Adjust Water = 1000 + (A x W) − 50
Where: A = Absorption of Aggregate (%)
W = Weight of Test Specimen

3. Procedure C, Fine Durability (D_f) for material passing a No. 4 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Split or quarter 500 ± 25 grams of material from the passing No. 4 portion of the sample.
(1) See step 3-f for optional preparation procedure.

c. Dry to constant weight at 230° ± 9°F and cool to room temperature.

d. Wash the dried material by the following procedure:
(1) Place the material in the wash vessel.
(2) Add 1000 ± 5 mls of water, clamp the lid in place and secure the vessel in the agitator.
(3) At 10 minute ± 30 seconds after adding water to the material start the agitator and shake the vessel for 2 minutes ± 5 seconds.
(4) Pour the contents of the vessel into a No. 200 sieve and rinse with fresh water until the water passing through the sieve is clear. Use a flexible hose attached to a faucet to direct water onto the material.

e. Return the washed material to the pan and dry to constant weight at 230° ± 9°F and cool to room temperature.
(1) Use water from the flexible hose as necessary to rinse the material from the sieve into the pan.
(2) Free water can be removed by tilting the pan and then, after the fines have settled, carefully pouring off the clear water.

f. A 500 gram fine sieve analysis test specimen which has been tested in accordance with California Test 202, may be utilized in lieu of the material prepared according to steps b. through e. above. If the fine sieve analysis test specimen is used, all of the material separated during sieving including that portion retained in the sieve pan shall be thoroughly recombined before proceeding to step g. below.

g. ~~Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full.~~
~~(1) When filling the measuring tin, consolidate the material in the tin by tapping the bottom edge on a hard object such as the work bench.~~
~~(2) Fill the measuring tin to slightly rounded above the brim and then strike off to level full using a straightedge.~~
See Below

h. Fill the graduated plastic cylinder to 4 ± 0.1 inches with working calcium chloride solution.

i. Pour the prepared test specimen into the plastic cylinder.
(1) Use the funnel to avoid spillage.
(2) Release air bubbles and promote thorough wetting by bumping the base of the cylinder against a firm object while the test specimen is being poured into the cylinder or by tapping the cylinder sharply on the heel of the hand several times after the test specimen has been poured in.

j. Allow the wetted material to stand undisturbed for 10 ± 1 minutes.

k. Abrade the test specimen by the following procedure:
(1) At the end of the 10 minute soaking period, stopper the cylinder, then loosen the material from the bottom by shaking the cylinder while holding it in a partially inverted position.
(2) Secure the cylinder in the mechanical sand equivalent shaker.
(3) Start the shaker and allow it to operate for 10 minutes ± 15 seconds.

Insert "Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full. Pre-determine the exact amount of material to be split using the following procedure.

(1) Fill the measuring tin to overflowing with the prepared material.

(2) Consolidate the material in the tin by tapping the bottom edge with a hard object.

(3) Strike off to level full using a straight edge and determine the weight of the material."

l. Irrigate the test specimen to flush the abraded fines from the sand using the following procedure:
   (1) At the end of the shaking period remove the cylinder from the shaker and set it upright on the work bench. Insert the irrigator tube in the cylinder, start the flow of working calcium chloride solution, and rinse the material from the sides of the cylinder as the irrigator is lowered.
   (2) With the cylinder remaining in an upright position and the solution flowing from the tip, apply a twisting action to the irrigator and force it to the bottom of the cylinder. The flow of solution will flush the clay size particles upward and into suspension. Withdraw the irrigator from the sand as necessary to change position and again force it to the bottom. The most effective technique for penetrating the test sample with the irrigator is to hold the irrigator between the palms of both hands and rotate it by rubbing the hands back and forth while applying a downward pressure.
   (3) Continue twisting and forcing the irrigator to the bottom of the cylinder until the fines have been flushed from all areas of the sample. Rotate the cylinder with each penetration of the irrigator and visually inspect the test specimen for pockets of fine material.
   (4) When the solution reaches the 15-inch mark in the cylinder, slowly withdraw the irrigator without shutting off the flow so that the liquid level is maintained at about 15 inches. Regulate the flow just before the irrigator is entirely withdrawn and adjust the find level to 15 inches.

m. Immediately place the cylinder on a work bench or table free of vibrations and allow the cylinder and contents to stand undisturbed for 20 minutes ± 15 seconds from the time the irrigation is completed.

n. Determine the "clay reading".
   (1) At the end of the 20-minute period read and record the level of the top of the sediment column. This is the clay read.
   (2) When the clay reading falls between 0.1-inch graduations, record the level of the higher graduation.
   (3) If a clearly defined line of demarcation does not form between the sediment and the liquid above it in the specified 20-minute period, allow the cylinder to stand undisturbed until the clear demarcation line does form. Then immediately read and record the time and the height of the column. If tap water was used retest an untested portion of the sample using distilled or demineralized water.
   (4) If the liquid immediately above the line of demarcation is still darkly clouded at the end of 20 minutes, and the demarcation line, although distinct, appears to be in the sediment column itself, read and record the level of this line at the end of the specified 20-minute period. If tap water was used, retest an untested portion of the sample using distilled or demineralized water.

o. Determine the "sand reading".
   (1) After the clay reading has been taken gently lower the weighted foot assembly into the cylinder until it comes to rest on the sand. Do not allow the indicator to hit the mouth of the cylinder as the assembly is being lowered.
   (2) As the weighted foot comes to rest on the sand, tip the assembly toward the graduation on the cylinder so that the position of the indicator is visible. Take care not to press down on the assembly.
   (3) Read the level of the top edge of the indicator.
   (4) Subtract 10 inches from the observed reading. This is the sand reading.
   (5) When the sand reading falls between 0.1-inch graduations, record the level of the higher graduation.

p. Calculate the fine durability index (D_f) using the formula:

D_f = (Sand Reading/Clay Reading) x 100
   (1) If the calculated durability index is not a whole number, report it as the next higher whole number.

4. Procedure D, Fine Durability (D_f) "Modified", for pea gravel or chips having a nominal minimum size no smaller than a No. 16 sieve.

a. Process the material to be tested as described in Section D "Sample Processing".

b. Split or quarter out 500 ± 25 grams of material from the passing No. 4 portion of the sample

c. Wash the test specimen by the following procedure.
   (1) Place the material in the wash vessel.
   (2) Add 1000 ± 5 mls of water, clamp the lid in place and secure the vessel in the agitator
   (3) At 10 minutes ± 30 seconds after adding water to the material, start the agitator and shake the vessel for 2 minutes ± 5 seconds
   (4) Pour the contents of the vessel into a No

200 sieve and rinse with fresh water until the water passing through the sieve is clear. Use a flexible hose attached to a faucet to direct water onto the material.

d. Transfer the material to a pan, dry to constant weight at 230° ± 9°F, and cool to room temperature.
   (1) Use water from the flexible hose as necessary to rinse the material from the sieve into the pan.
   (2) Free water can be removed by tilting the pan and then, after the fines have settled, carefully pouring off the clear water.

e. Split or quarter the washed and dried material to provide a test specimen of sufficient size to fill the measuring tin to level full.
   (1) When filling the measuring tin, consolidate the material in the tin by tapping the bottom edge on a hard object such as the work bench.
   (2) Fill the measuring tin to slightly rounded above the brim and then strike off to level full using a straightedge.

f. Fill the graduated plastic cylinder to 4 ± 0.1 inches with water.

g. Pour the prepared test specimen into the plastic cylinder.
   (1) Use the funnel to avoid spillage.
   (2) Release air bubbles and promote thorough wetting by bumping the base of the cylinder against a firm object while the test specimen is being poured into the cylinder or by tapping the cylinder sharply on the heel of the hand several times after the test specimen has been poured.

h. Allow the wetted material to stand undisturbed for 10 ± 1 minutes.

i. Abrade the test specimen by the following procedure:
   (1) At the end of the 10-minute soaking period, stopper the cylinder, then loosen the material from the bottom by shaking the cylinder while holding it in a partially inverted position.
   (2) Secure the cylinder in the mechanical sand equivalent shaker.
   (3) Start the shaker and allow it to operate for 30 ± 1 minutes.

j. Transfer the water and passing No. 200 sieve size material to a second graduated plastic cylinder.
   (1) Fill an empty graduated plastic cylinder to the 0.3 inch mark with stock calcium chloride solution.
   (2) Place a No. 200 sieve into a funnel that empties into the cylinder containing the calcium chloride solution.
   (3) Tip the stoppered cylinder containing the test specimen upside down and shake to loosen the material from the bottom.
   (4) Hold the mouth of the inverted cylinder over the sieve and remove the stopper, allowing the test specimen and water to pour onto the sieve.
   (5) Collect the water and passing No. 200 material in the second cylinder.
   (a) Rinse the remaining fines from the first cylinder onto the sieve with a small amount of fresh water.
   (b) Rinse the material retained on the sieve with additional fresh water to assure that the minus No. 200 portion passes through the sieve. Take care not to fill the cylinder above the 15-inch mark.
   (c) Adjust the level of the liquid to the 15-inch mark with fresh water.

k. Stopper the cylinder and thoroughly mix the wash water and calcium chloride solution by inverting the cylinder 20 times in approximately 35 seconds. Allow the air bubble to completely traverse the length of the cylinder each time.

l. Place the cylinder on a work bench or table free of vibrations, remove the stopper and allow to stand undisturbed for 20 minutes ± 15 seconds.

m. Immediately read the top of the sediment column to the nearest 0.1 inch.

n. Determine the Fine Durability index (D_f) "modified" from Table No. 2.

End of Text (10 pgs) on Calif. 229

Page 10. Add Section "G. Reporting. When both Dc and Df are determined for a material, report the lowest of the two values. In no case shall the Dc and Df be averaged or otherwise combined".