PIT SLOPE MANUAL

supplement 6-1

BUTTRESSES AND RETAINING WALLS

This supplement has been prepared as part of the

PIT SLOPE PROJECT

of the
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THE PIT SLOPE MANUAL

The Pit Slope Manual consists of ten chapters, published separately. Most chapters have supplements, also published separately. The ten chapters are:

1. Summary
2. Structural Geology
3. Mechanical Properties
4. Groundwater
5. Design
6. Mechanical Support
7. Perimeter Blasting
8. Monitoring
9. Waste Embankments
10. Environmental Planning

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SUMMARY

Buttresses and retaining walls stabilize slopes up to 200 ft high by restraining the toe. They can be constructed of a variety of materials but rock fill buttresses and concrete retaining walls are most relevant to mining. It is important to ensure the supported slopes can drain freely, otherwise groundwater pressures may build up and threaten stability. Proper construction procedures must be followed. The slope and stabilizing structure must both be monitored. Case histories of stabilization by a buttress at a Canadian mine and by retaining wall at a South African mine are described.

ACKNOWLEDGEMENTS

Roy Sage was responsible for the chapter and supplement on Mechanical Support. Address enquiries to him at: 555 Booth Street, Ottawa, K1A 0G1.

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Dick Geren, vice-president of the Iron Ore Company of Canada Ltd., kindly gave permission to publish the case history of the buttress at Ruth Lake Mine.

Prime contractors: Dames and Moore.

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The Pit Slope Group has been successively led by D.F. Coates, M. Gyenge and R. Sage; their colleagues have been G. Herget, B. Hoare, G. Larocque, D. Murray and M. Service.

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INTRODUCTION

PURPOSE AND SCOPE

1. Buttresses and retaining walls support slopes by restraining the toe. They are not often used in mining but are common in civil engineering.

2. The purpose of this supplement is to describe how they may be used to support open pit slopes. The scope of the supplement is to describe the various methods of slope toe support, together with procedures for design and construction. Cost estimates and examples of mining applications are included.

TYPES OF RESTRAINING STRUCTURES

3. Five basic types of restraining structure can be installed at the toe of a slope, either to forestall instability or to stabilize a slope already showing signs of unacceptable displacement. As shown in Fig 1, these are: buttresses (b), cribs and gabion walls (d,a), concrete, masonry or sheet pile retaining walls (e), reinforced earth fill (c), and steel piling (f).

4. Each type of structure has definite applications some of which have been summarized by Baker (1). Appropriate parts of this summary are given in Table 1.

5. Buttresses are normally constructed from rock or earth fill and serve three purposes: to provide additional weight at the toe of the slope, to increase shear strength in the toe area above that of the in situ material and to improve drainage. All of these increase resistance to sliding.

6. Low cut slopes which tend to slough or flow may be stabilized by modifying the general buttress principle in which pervious rock material is used both for drainage and as a buttress. A blanket of rock fill or clean gravel is placed over the slope at a flatter angle than the slope. This forms a wedge-shaped buttress of material which allows free drainage of water from the slope and develops toe resistance against sliding.

7. Crib walls are most commonly used in slope engineering as a corrective measure once significant movement of the slope has taken place. Preferably, however, they should be used as a preventive measure before movement has occurred. Cribs may be constructed of logs, cut timber, pre-cast concrete blocks, or metal.

8. The resistance provided by crib structures is limited and depends primarily on the ability of the structure to resist shear action, overturning and sliding on or below its base.

9. Gabion walls, which are heavy free-draining gravity structures, may be considered a type of crib, since they too are constructed in components and are relatively flexible. The structure consists of wire mesh baskets (gabions) filled with non-degradable rock in pieces which range in size from 3 ft by 3 ft by 6 ft to 3 ft by 3 ft by 12 ft and are stacked one upon another to construct the wall.

10. Another method of stabilizing slopes is reinforced earth (2, 3). In this technique, either metal or concrete faces with attached metal strips extend into and reinforce the backfill. The resulting structures are capable of withstanding differential settlement up to 1 per cent of their length without significant distress.

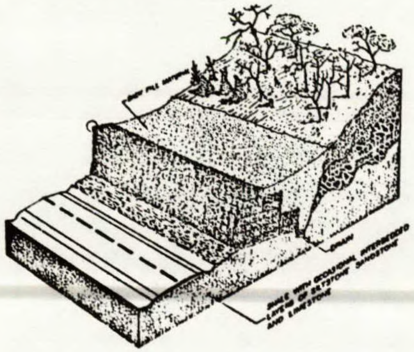
11. Retaining walls of mass concrete, reinforced concrete, pre-cast concrete, masonry or sheet piling are relatively thin structures compared with buttresses and cribs. They rely mainly on their flexural stiffness and strength for support. Some types of mass concrete structures rely primarily on their weight for stability and are more analogous to crib walls. These more rigid structures are less likely than cribs to perform well where differential settlement is significant since resulting cracking would reduce integrity of the structure.

12. Single steel piles linked to each other by tie bars have also been used to stabilize landslides, notably in Japan (4). Bored piles reinforced with steel beams have been used to stabilize slopes in Belgium (5).

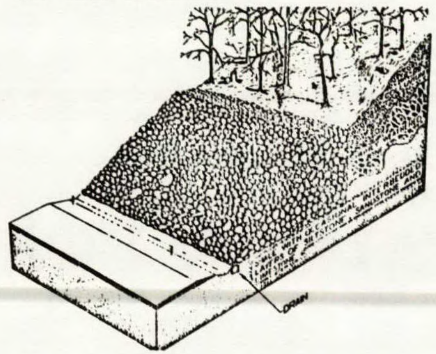
APPLICATION TO OPEN-PIT MINING

13. It has generally been found in highway engineering, where the types of restraining structures described above have widest application,

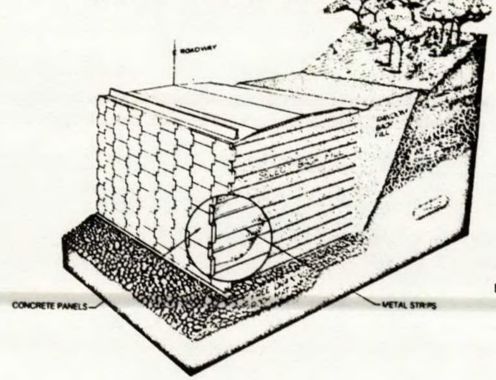
(a) Gabion Wall

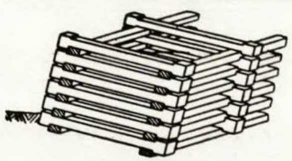


(b) Rock buttress

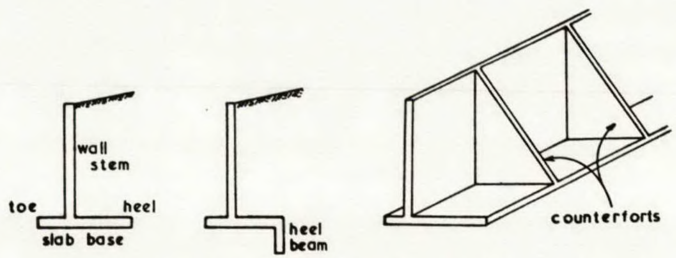


(c) Reinforced earth

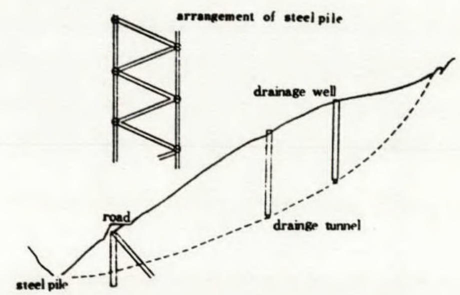




(d) Crib wall



(e) Reinforced concrete retaining walls



(f) Steel piles

Fig 1 - Types of retaining structures.

Table 1: Factors in the design of Corrective Measures

Corrective measures	Best application	Principles involved	Remarks
1. Buttress at toe	Good foundation at toe in shallow or deep soil. Buttress should extend below the slip plane.	Large mass blocks the mass movement and any further movement involves displacement of the buttress. Permanent solution.	An excellent method of correction when the toe conditions permit. Action is that of a retaining device; much less expensive than cribbing or a retaining wall. The buttress should be founded below the slip surface, for, unless the sliding material has an angle of internal friction greater than 15°, the added normal force of a buttress has little influence on the safety factor.
2. Cribbing - timber, concrete, or metal	Shallow soil with good foundations.	A retaining mass, with or without additional lateral restraint, placed in path of the mass movement. Further movement involves displacement of the retaining mass. Permanent solution.	Good method where applicable but relatively costly. Less stable foundation required than for retaining wall as shifting is permissible before cribbing fails. Resistance offered is the weight of the crib unless bedrock is excavated to give resistance to lateral thrust. Recommended for shallow soil profiles. Weight of the crib wall can be compared with weight of material removed for an estimate of the relative degree of stability offered by the cribbing. In slide areas, the resistance required can be estimated by using the formula for the safety factor. Not recommended in creep and flow areas.
3. Retaining wall of stone or concrete	Shallow soil with good foundations.	A retaining mass with lateral restraint, placed in path of the mass movement. Further movement involves displacement of retaining wall. Permanent solution.	The applications are similar to those of cribbing. The cost is a deterrent. Walls are advantageous in urban areas. A wall requires a foundation in bedrock or in good soil below the slip surface. Standard practice is to include weep holes in designing the wall. The formula for the safety factor may be used to estimate required resistance to lateral thrust. Retaining walls in creep and flow movement may take full weight of soil.

that they are best suited to the control or prevention of relatively small-scale slides. Typically, amenable slopes are less than 100 ft in height and less commonly up to about 200 ft. Restraining structures have seldom been totally effective for large landslides. The cost of construction increases with slope height and a point is reached at which other methods of slope stabilization - slope flattening, unloading of crest, drainage - are more economic. Also, the risk of total loss of a costly restraining structure must be taken into account. The greatest potential for these types of structures would be to support small-scale slides in soil-like overburden, stabilize spoil piles, construct truck dump ramps, or to underpin existing structures.

14. Certain restraints are placed on the use of these retaining systems in the open-pit environment.

a. The restraining structure must be flexible enough to adjust to the accompanying slope displacement and maintain adequate support under constantly changing pit geometry.
b. The restraining structure must be able to withstand dynamic loading from blasting.
c. The slope requiring toe support may not be permanent and may be mined out during later stages of mining, so that only temporary or an easily removed support is required. There is also the possibility that what was initially waste rock may later become ore if, for example, the mineral value increases and a lower grade can be mined.
d. The space available for a retaining structure is restricted by the pit slope geometry and operational requirements, as for example, if the structure is to be located along a section of the haul road, such as at the Ruth Lake mine described in the case history below.
e. If mining extends below the restraining structure, the latter will become a surcharge or additional load on the lower slope. This should be kept in mind when designing the overall slope configuration.

15. Because of these limitations, a number of restraining structures used in highway engineering are not generally suitable for use in open-pits. Such in particular are reinforced earth, steel piles, massive structures such as concrete or masonry retaining walls and some bin structures. Some, however, may find application for underpinning permanent slopes beneath such mine facilities as crushers or conveyors. Rock buttresses and crib type restraining devices are more suitable for open pit slope stabilization.

METHODS OF STABILITY ANALYSIS

FIELD INVESTIGATION

16. The basic data for analyzing slope stability and for designing retaining structures is obtained from field investigations. A good summary of the minimum required information has been presented by Baker (1) and is shown in Table 2. This table serves only as a general guideline, and may be expanded or shortened as necessary according to field conditions at a particular site.

GENERAL CONSIDERATIONS

17. The design of cribs and buttresses on the basis of experience alone is a potentially hazardous practice as it offers no information on the magnitude of the forces involved. It is also difficult to compare two different empirical approaches to determine which will be the more economical. For these reasons, quantitative stability analyses must be used. In most cases a relatively simple analysis will be sufficient and, because of the simplifications used, the answers cannot be interpreted as being absolute. The value of such analyses lies in the ability to change the various parameters involved and to assess the relative cost of various designs and the amount of extra stability provided by each measure.

18. The analysis of slopes to be supported by buttresses and retaining walls can be carried out using techniques described in the Design Chapter. The forces necessary to produce the required stability can be determined and the restraining structure built accordingly.

Table 2: Investigations needed for various corrective measures

After Baker (1)

Corrective measure	Topographic survey ground surface	Bedrock profile	Source of water	Undisturbed soil sample	General soil condition
1. Buttress at toe	For stability estimate.	Surface only, for stability estimate.	Not essential.	For stability estimate.	Density and moisture variations for stability estimates.
2. Cribbing - timber, concrete, metal	For computing forces against wall and height determinations.	For depth to foundation, type of foundation, and stability estimate.	Not essential.	To compute forces against wall.	Density and moisture variations for stability estimate.
3. Retaining wall (stone or concrete)	For computing forces against wall and height determinations.	For depth to foundation, type of foundation, and stability estimate.	Not essential.	To compute forces against wall.	Density and moisture variations for stability estimate.

19. In open pit mining, cribs and rock buttresses are most suitable for supporting overburden such as till and alluvium, and highly weathered rock. It can be assumed that the rotational shear mode of instability applies in these cases (Fig 2(a)).

20. The potential surface of sliding may in some cases include the contact between overburden and bedrock (Fig 2(b)). For these cases, the sliding surface may be non-circular and the general sub-surface stability analysis procedures of the Design chapter would apply.

21. Buttresses and cribs may in some cases be used to support faces in which structural discontinuities such as bedding planes, faults and joints are undercut (Fig 2(c)); plane shear instability modes and associated analyses would then apply.

METHODS OF DESIGN

22. In the following pages, various standard designs are presented for cribs, gabions, and buttresses. However, it should be noted and emphasized that these designs are semiempirical and hence are meant only as general guidelines for designing the following designs for stabilizing soil or soil-rock slopes should only be used for a particular case, after a complete analysis has been made using known soil parameters. Cribs, gabions, and buttresses may all be analyzed as gravity structures and should be checked against sliding, overturning, and internal shearing. The recommended factor of safety for mining applications is 1.25, always coupled, however, with sound engineering judgement. Detailed descriptions of stability analysis for returning structures are provided in various text books on soil mechanics, and a brief summary is included in Fig 3. Table 3 shows a range of allowable bearing capacities for gravity structures founded on various types of soil.

23. The stability analysis of an existing or potential slide is essential since any type of retaining structure must extend to sufficient depth to prevent a shear surface developing below it. To be effective, it must also be located on the toe or in front of the toe of the slide. Without such a preliminary analysis or investigation, there is always danger that the additional load imposed by the toe support fill may increase the driving force rather than provide added resistance against sliding. Eckel (7) points out that many rock buttresses have failed because they did not extend to sufficient depth, and as a result, moved as part of the slide. It is therefore obvious that any of the following designs will be totally ineffective if the retaining structure is not constructed at a strategic location.

CRIB WALLS

24. The chief advantages of crib walls, as pointed out by Zaruba et al (8), is that they can be built quickly and adjust easily to settlement. They are also effective as soon as they are constructed.

25. The construction sequence for crib type retaining walls is to build individual bins, fill them with material and then backfill against the completed structure. Such a sequence leaves the outside faces of the bin unsupported until adjacent bins have been filled and the backfill placed. The pressures developed are analogous to those exerted by grain on the walls of a bin or silo. Bin pressure equations have been developed by Janssen and are described by Reinsner et al (9). The basic Janssen equation can be modified slightly to include the effects both of soil adhering to the wall and a surcharge pressure. This modified form has been reported by Schuster et al (10) as shown by eq 1:

$$\sigma_x = K\sigma_z = \frac{\gamma A - C_a U}{\mu_1 U}\left[1 - \exp\left(-\mu_1 KUz/A\right)\right] + Kq.\ \exp\left(-\ \mu_1 KU_z/A\right) \qquad \text{eq 1}$$

Where σ_z = vertical pressure inside the crib
σ_x = lateral pressure inside the crib

$K = \frac{\sigma_x}{\sigma_z} = \frac{\cos^2\phi}{2 - \cos^2\phi}$ = Krynine's coefficient of lateral pressure of soil in a cofferdam or crib (11)

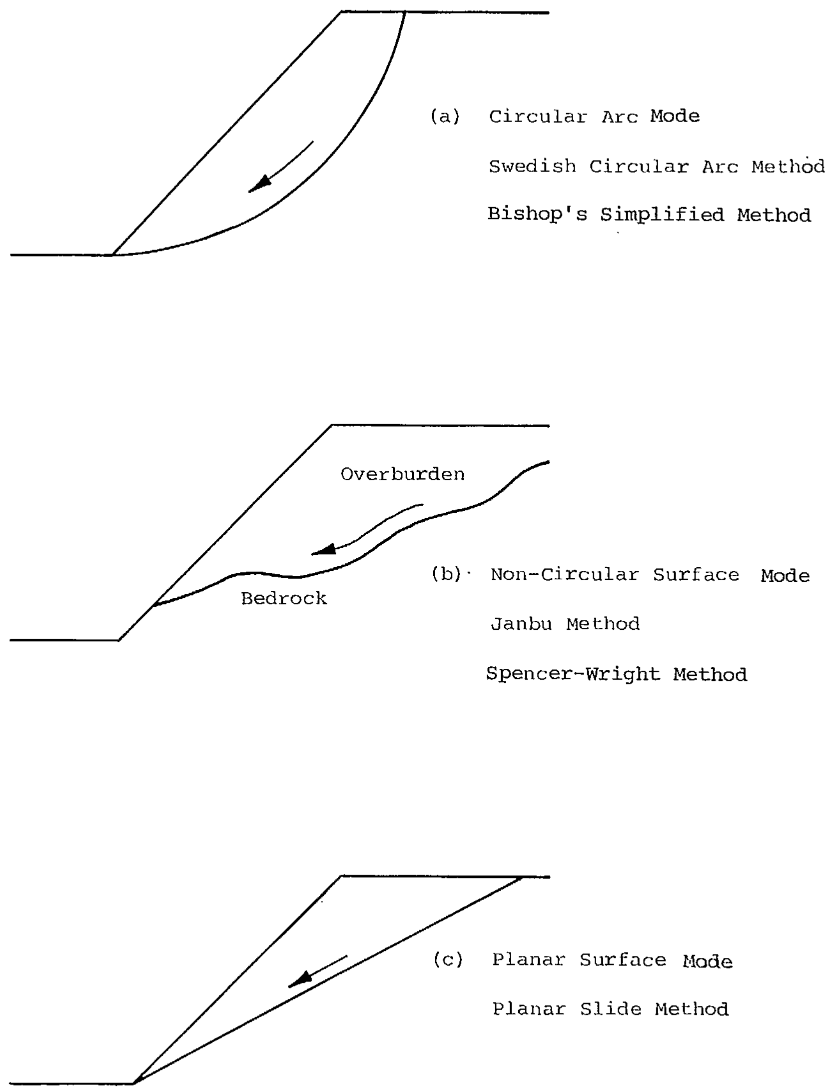
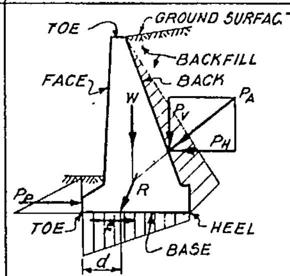
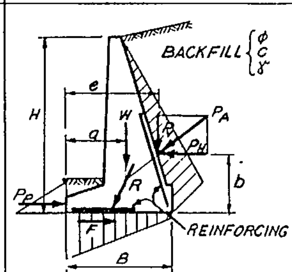


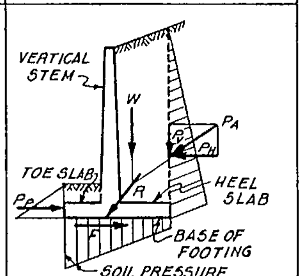
Fig 2 - Different modes of instability and methods of analysis.

<table>
<tr><th>Type of wall</th><th>Load diagram</th><th>Design factors</th></tr>
<tr><td>GRAVITY</td><td>
</td><td>LOCATION OF RESULTANT

Moments about toe:
$$d = \frac{Wa + P_V e - P_H b}{W + P_V}$$
Assuming $P_P = 0$

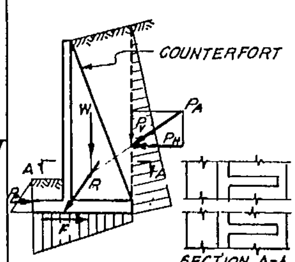
OVERTURNING

Moments about toe:
$$F_S = \frac{Wa}{P_H b - P_V e} \geqq 1.5$$</td></tr>
<tr><td>SEMI-GRAVITY</td><td>
</td><td>Ignore overturning if R is within middle third (soil), middle half (rock). Check R at different horizontal planes for gravity walls.

RESISTANCE AGAINST SLIDING
$$F_S = \frac{(W + P_V) \tan \delta + C_a B}{P_H} \geqq 1.5$$
$$F_S = \frac{(W + P_V) \tan \delta + C_a B + P_P}{P_H} \geqq 2.0$$
$$F = (W + P_V) \tan \delta + C_a B$$</td></tr>
<tr><td>CANTILEVER</td><td>
</td><td>For coefficients of friction between base and soil see Table 10-1.

C_a = adhesion between soil and base

$\tan \delta$ = friction factor between soil and base

W = Includes weight of wall and soil in front for gravity and semi-gravity walls. Includes weight of wall and soil above footing, for cantilever and counterfort walls.</td></tr>
<tr><td>COUNTERFORT</td><td>
</td><td>CONTACT PRESSURE ON FOUNDATION

For allowable bearing pressure for inclined load on strip foundation, see Ch. 11.

For analysis of pile loads beneath strip foundation, see Ch. 13.

OVERALL STABILITY

For analysis of overall stability, see Ch.7.</td></tr>
</table>

Fig 3 - Design criteria for retaining walls (6).

Table 3: Allowable bearing capacities of soils*

Type of bearing material	Consistency in place	Allowable bearing recommended value of allowable bearing capacity (tons per square foot)
Well-graded mixture of fine- and coarse-grained soil; glacial till, hardpan, boulder clay (GW - GC, GC, SC)	Very compact	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)	Very compact Medium to compact Loose	8 6 4
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	4 3 2
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	3 2.5 1.5
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very compact Medium to compact Loose	3 2 1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	4 2 0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium to stiff Soft	3 1.5 0.5

* Adapted from Navfac (6)

γ = unit weight of material in crib

A = horizontal cross-sectional area of crib

U = perimeter of crib

C_a = coefficient of soil-wall adhesion

μ_1 = tan δ = coefficient of friction between fill material in crib and crib wall

z = the depth from the top of the crib at which the pressure is calculated

q = average surcharge pressure at top of the crib

26. Since the following crib designs are only general guidelines it is necessary to check internal stability and stresses in individual members of a chosen crib design for the conditions under which it will function. If the members are inadequate, they may be resized using the procedures outlined by Tschebotarioff (12). A detailed analysis of internal stability is given by Schuster et al. (10).

27. Methods for checking the external stability of a chosen crib are summarized in Fig 3. However, a crib type retaining wall has an irregular back face. The soil resting above this irregular face is assumed to act as part of the wall, not as part of the backfill. As a result, it is assumed that the rear boundary of a crib type retaining wall is a theoretical vertical membrane extending from the surface of the surcharge to the base of the wall, as shown in Fig 4. This vertical membrane is the same as that assumed in the analysis of cantiliver retaining walls (Fig 3). The active earth pressure is therefore assumed to act at this vertical membrane boundary.

28. Crib type retaining walls have been built of precast concrete, cut timber, rough hewn logs, steel and aluminum. There are variations of such structures, some of which are described below.

29. Popular designs of timber cribs have been developed by the American Wood Preservers Institute (13) which has a Canadian affiliate. The AWPI designs presented here are based on a backfill unit weight of 120 pcf in the wall and 130 pcf against the wall. AWPI designs for various conditions are shown in Fig 5-14.

30. In many locations, timber near the site of the proposed crib structure is available at a much lower cost and therefore a log crib structure would be much more economical than a cut timber structure. Three standard designs for log cribs are shown in Fig 15, 16, and 17.

31. Precast concrete members have been used for cribs for many years. These have been designed with various configurations of front wall which include open, closed and flush face type walls. A series of standard wall designs from the U.S. Dept. of Transportation is shown in Fig 18, and details of the structural components are shown in Fig 19. Other types of open or closed walls may have either a fish tail type anchorage or a continuous backwall.

32. Details of various other designs have been reproduced by the Portland Cement Association (PCA). Figure 20 illustrates a fish tail type anchorage with a flush face. This type of construction permits walls to be curved up to about 20° without special components. Figure 21 illustrates a continuous backwall type anchorage with an open face or a flush face. Figure 22(a) is essentially a counterfort wall with a flush face. This type of construction permits considerable wall curvature without special components. Figure 22(b) illustrates a modified bin structure with a discontinuous backwall. Figure 23 illustrates a continuous backwall anchorage with an open face, which may also be constructed with a closed face. This type of wall permits long radius curves to be built without special components.

33. The concrete type walls require a fairly good foundation so as to limit settlement and prevent structural distress. Adverse foundation conditions require a special sill unit or a cast-in-place footing to spread the load. Closed face walls may incorporate small lugs at both ends between the stretchers to provide openings for drainage.

34. Several types of cribs have been constructed of steel and aluminum. The Armco unit is a common steel crib structure and is available in two types. The Armco Type 1 wall uses a U-shaped vertical connector (Fig 24) and is sturdier than

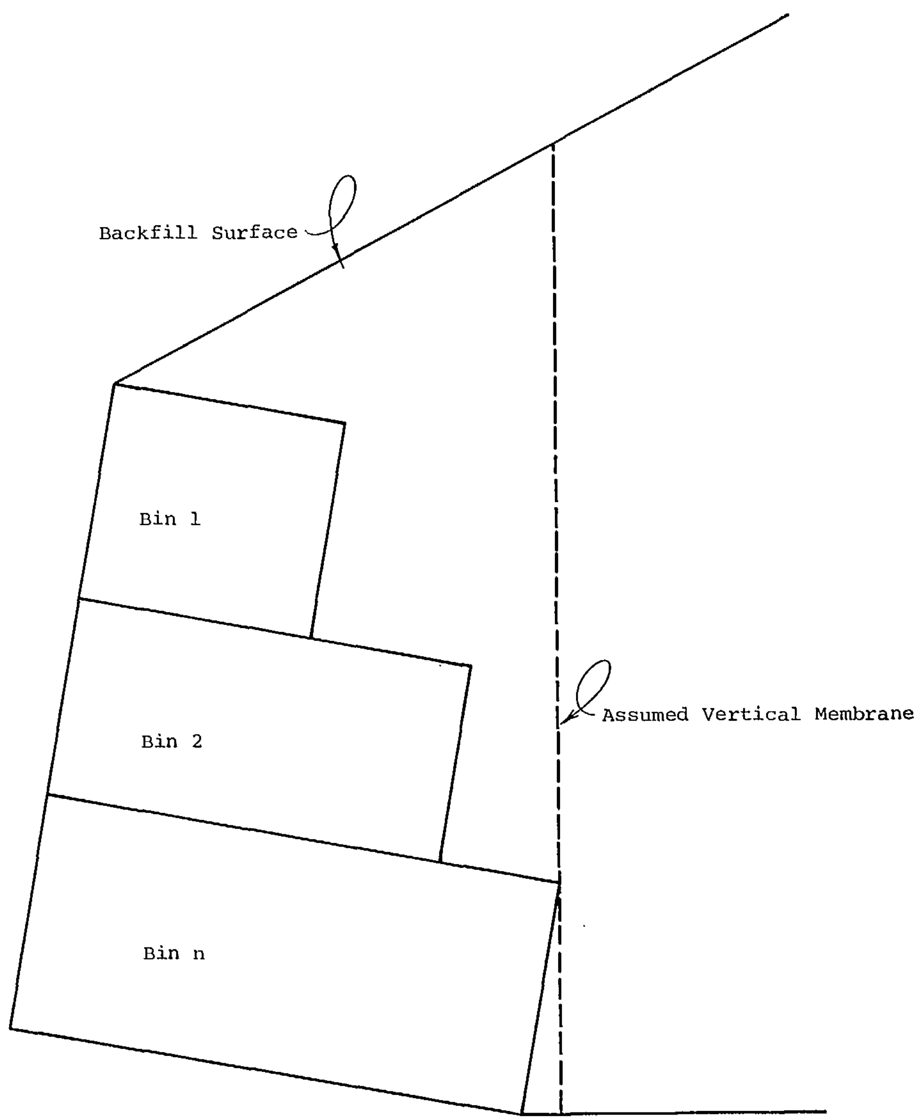
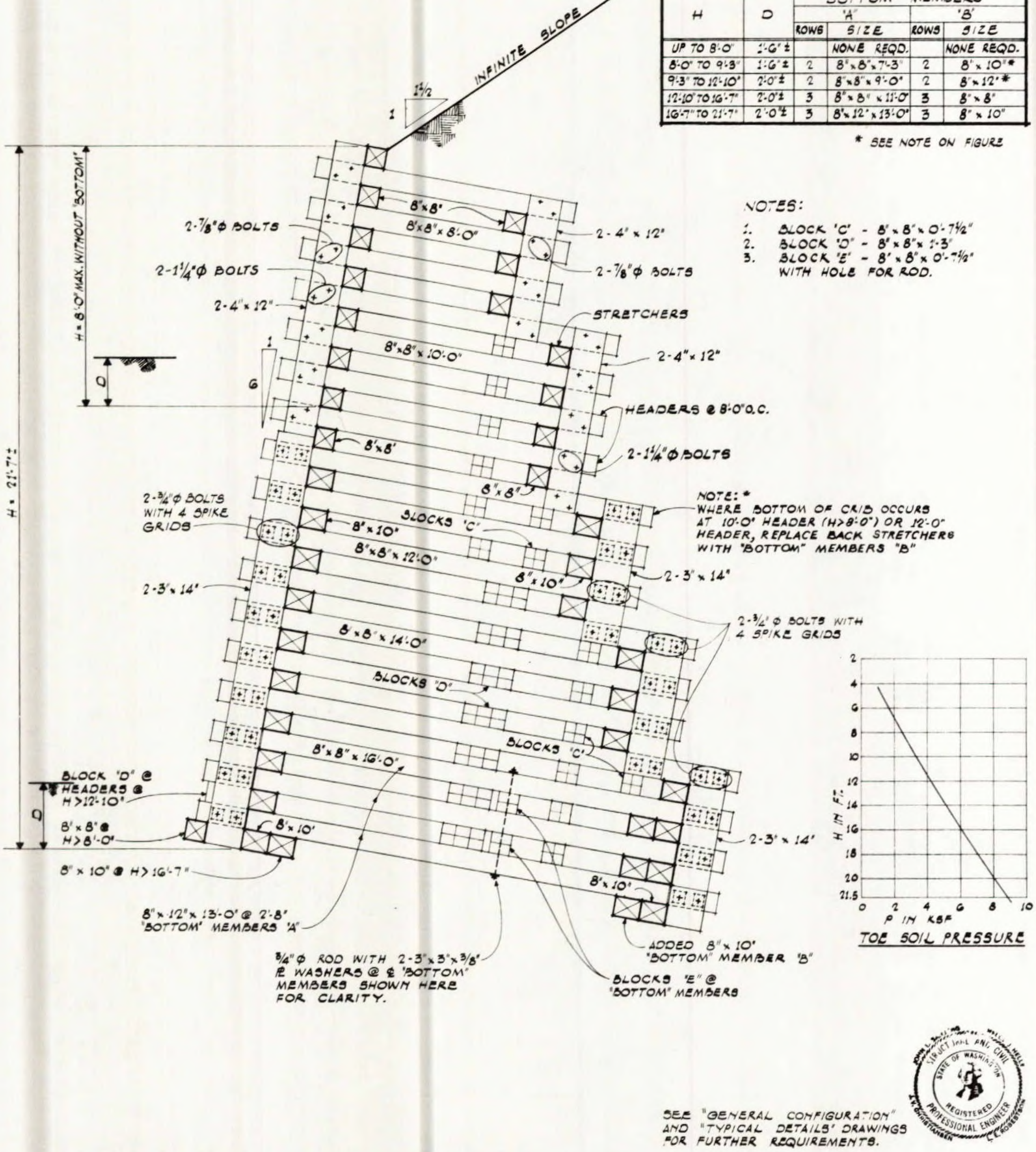


Fig 4 - Crib-type retaining wall and back-fill geometry (10).



SCHEDULE

H	D	"BOTTOM" MEMBERS			
		'A'		'B'	
		ROWS	SIZE	ROWS	SIZE
UP TO 8'-0"	1'-6"±		NONE REQD.		NONE REQD.
8'-0" TO 9'-3"	1'-6"±	2	8"x8"x7'-3"	2	8"x 10"*
9'-3" TO 12'-10"	2'-0"±	2	8"x8"x 9'-0"	2	8"x 12"*
12'-10" TO 16'-7"	2'-0"±	3	8"x 8" x 11'-0"	3	8"x 8"
16'-7" TO 21'-7"	2'-0"±	3	8"x 12" x 13'-0"	3	8" x 10"

* SEE NOTE ON FIGURE

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE A - 1½ TO 1 SURCHARGE

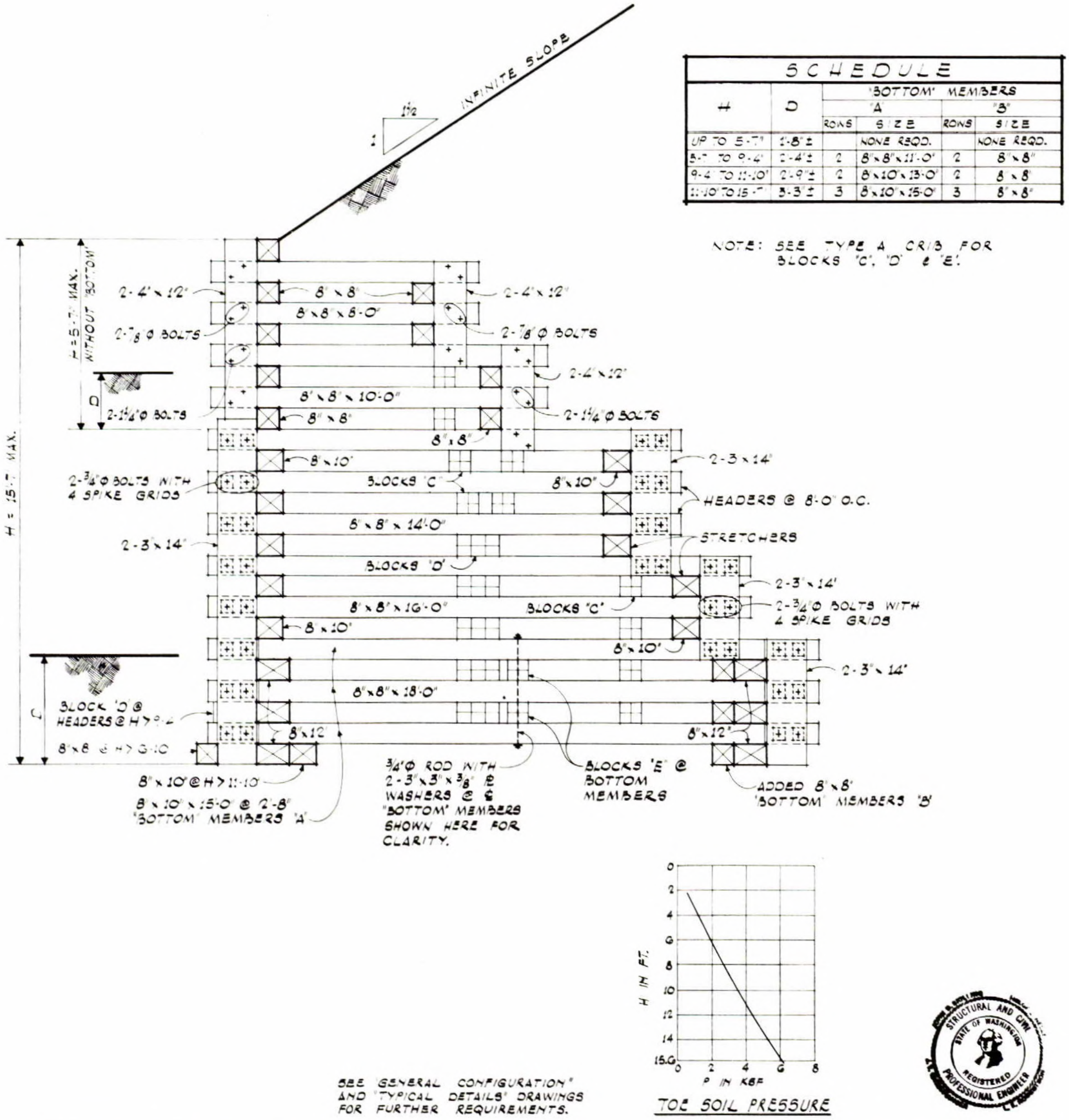
AMERICAN WOOD PRESERVERS INSTITUTE
Washington, D.C.
Portland, Oregon — San Francisco, California

DATE: JAN. 1969 — GRAPHIC SCALE: 0 1 2 3 4 5 10

SKILLING, HELLE, CHRISTIANSEN, ROBERTSON
SEATTLE, WASHINGTON
STRUCTURAL ENGINEERS

SHANNON AND WILSON
SEATTLE, WASHINGTON
SOILS ENGINEERS

Fig 5 - Timber cribbing design - Type A.



SCHEDULE

H	D	'BOTTOM' MEMBERS 'A' ROWS	'A' SIZE	'B' ROWS	'B' SIZE
UP TO 5'-7"	1'-8" ±		NONE REQD.		NONE REQD.
5'-7" TO 9'-4"	2'-4" ±	2	8" x 8" x 11'-0"	2	8" x 8"
9'-4" TO 11'-10"	2'-9" ±	2	8" x 10" x 13'-0"	2	8" x 8"
11'-10" TO 15'-7"	3'-3" ±	3	8" x 10" x 15'-0"	3	8" x 8"

NOTE: SEE TYPE A CRIB FOR BLOCKS 'C', 'D' & 'E'.

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE B - 1½ TO 1 SURCHARGE

AMERICAN WOOD PRESERVERS INSTITUTE
Washington, D.C.
Portland, Oregon — San Francisco, California

SKILLING, HELLE, CHRISTIANSEN, ROBERTSON
SEATTLE, WASHINGTON
STRUCTURAL ENGINEERS

SHANNON AND WILSON
SEATTLE, WASHINGTON
SOILS ENGINEERS

DATE: JAN. 1969

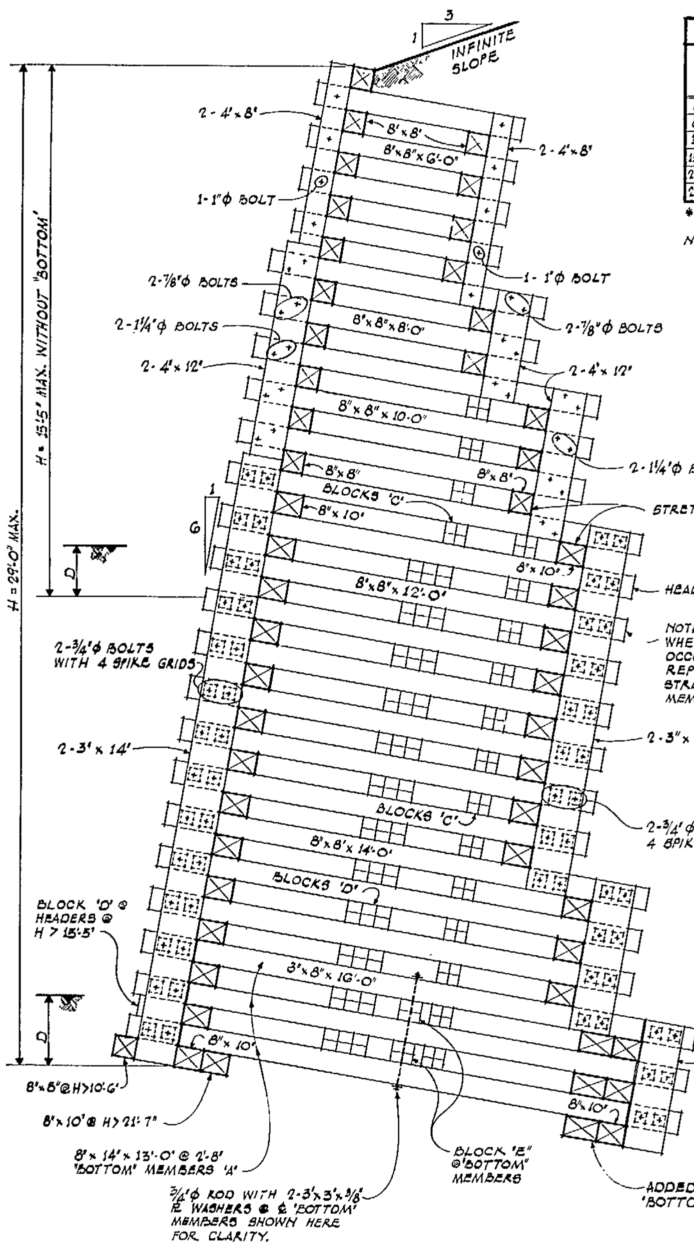
GRAPHIC SCALE: 0 1 2 3 4 5 10

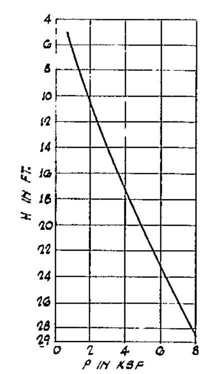
Fig 6 - Timber cribbing design - Type B.

SCHEDULE					
H	D	'BOTTOM' MEMBERS			
		'A'		'B'	
		ROWS	SIZE	ROWS	SIZE
UP TO 8'-0"	1'-6" ±		NONE REQD.		NONE REQD.
8'-0" TO 11'-9"	1'-6" ±		DO		DO
11'-9" TO 15'-5"	1'-9" ±		DO		DO
15'-5" TO 21'-7"	2'-0" ±	3	8"x 8"x 9'-0"	3	8"x 12"*
21'-7" TO 25'-4"	2'-0" ±	3	8"x 10"x 11'-0"	3	8"x 8"
25'-4" TO 29'-0"	2'-0" ±	3	8 x 14" x 13'-0"	3	8"x 12"

* SEE NOTE IN FIGURE

NOTE: SEE TYPE A CRIB FOR BLOCKS "C", "D" & "E".





TOE SOIL PRESSURE

SEE 'GENERAL CONFIGURATION' AND 'TYPICAL DETAILS' DRAWINGS FOR FURTHER REQUIREMENTS.

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE C - 3 TO 1 SURCHARGE

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DATE: JAN. 1969

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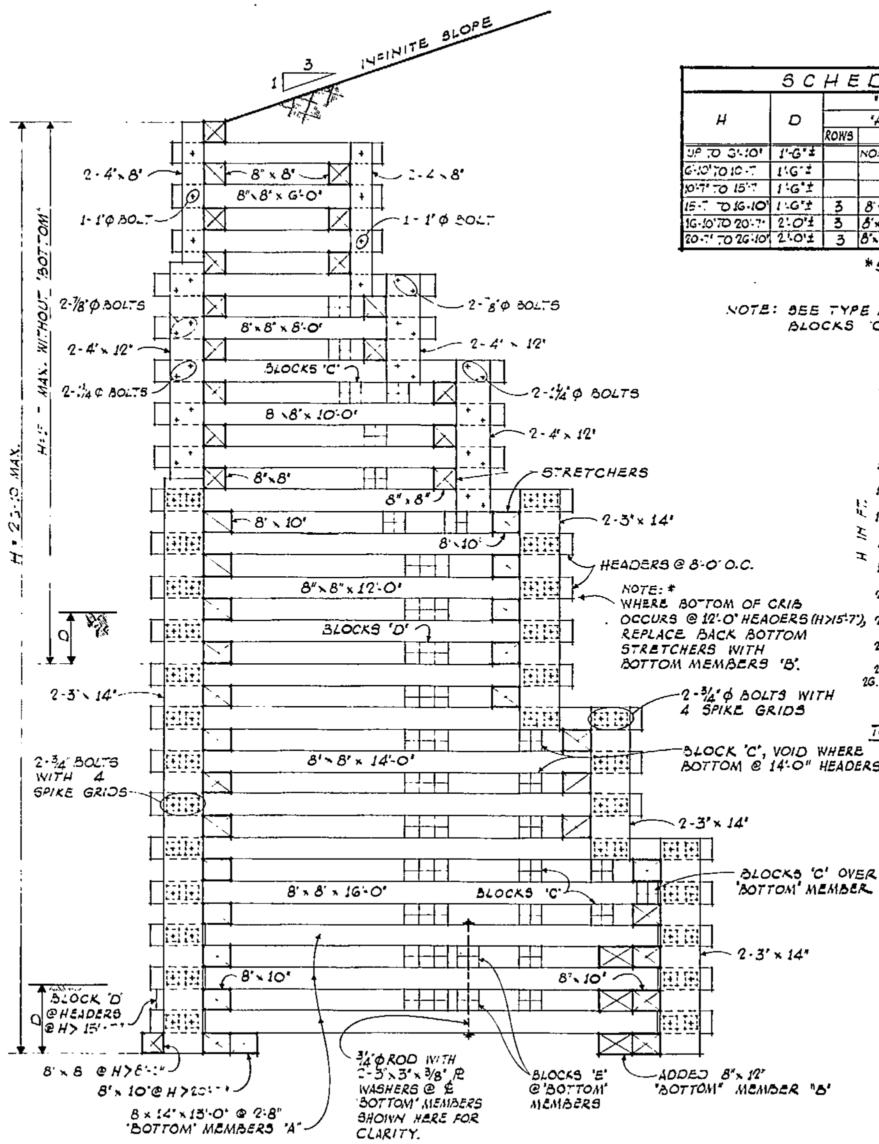
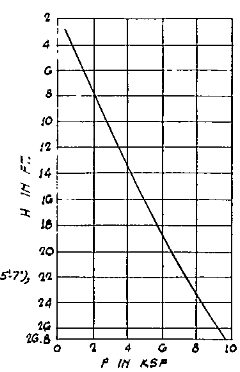
Fig 7 - Timber cribbing design - Type C.



SCHEDULE					
H	D	"BOTTOM" MEMBERS			
		'A'		'B'	
		ROWS	SIZE	ROWS	SIZE
UP TO 6'-10"	1'-6"±		NONE REQD.		NONE REQD.
6'-10" TO 10'-7"	1'-6"±		DO		DO
10'-7" TO 15'-7"	1'-6"±		DO		DO
15'-7" TO 16'-10"	1'-6"±	3	8" x 8" x 9'-0"	3	8" x 12" *
16'-10" TO 20'-7"	2'-0"±	3	8" x 10" x 11'-0"	3	8" x 8"
20'-7" TO 26'-10"	2'-0"±	3	8" x 14" x 13'-0"	3	8" x 12"

* SEE NOTE ON FIGURE

NOTE: SEE TYPE A CRIB FOR BLOCKS 'C', 'D' & 'E'.



TOE SOIL PRESSURE

SEE "GENERAL CONFIGURATION" AND "TYPICAL DETAILS" DRAWINGS FOR FURTHER REQUIREMENTS.

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE D - 3 TO 1 SURCHARGE

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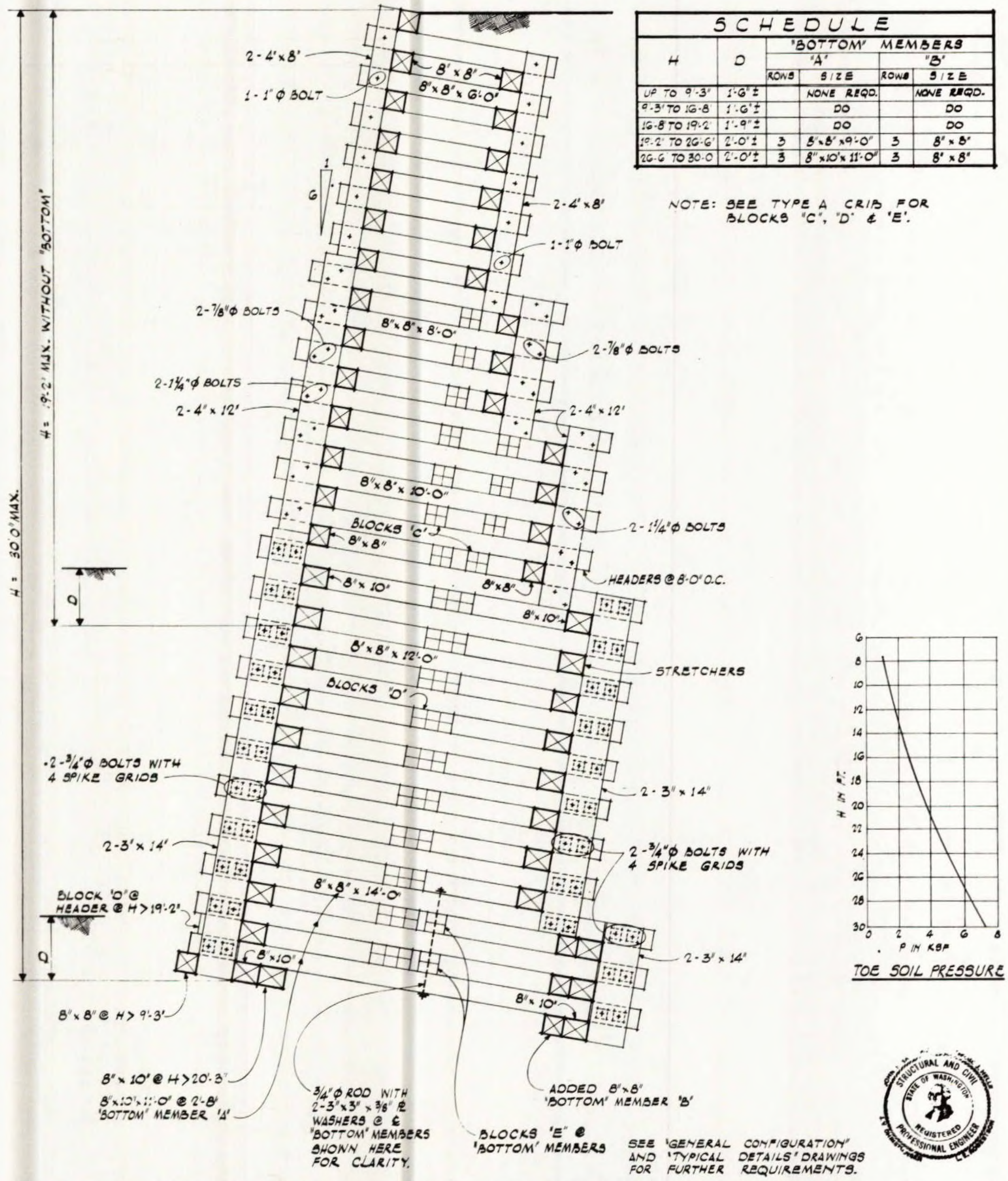
Fig 8 - Timber cribbing design - Type D.



SCHEDULE

H	D	"BOTTOM" MEMBERS			
		"A"		"B"	
		ROWS	SIZE	ROWS	SIZE
UP TO 9'-3"	1'-6" ±		NONE REQD.		NONE REQD.
9'-3' TO 16'-8'	1'-6" ±		DO		DO
16'-8 TO 19'-2'	1'-9" ±		DO		DO
19'-2' TO 26'-6'	2'-0" ±	3	8" x 8" x 9'-0"	3	8" x 8"
26'-6 TO 30'-0	2'-0" ±	3	8" x 10" x 11'-0"	3	8" x 8"

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE E - LEVEL SURCHARGE

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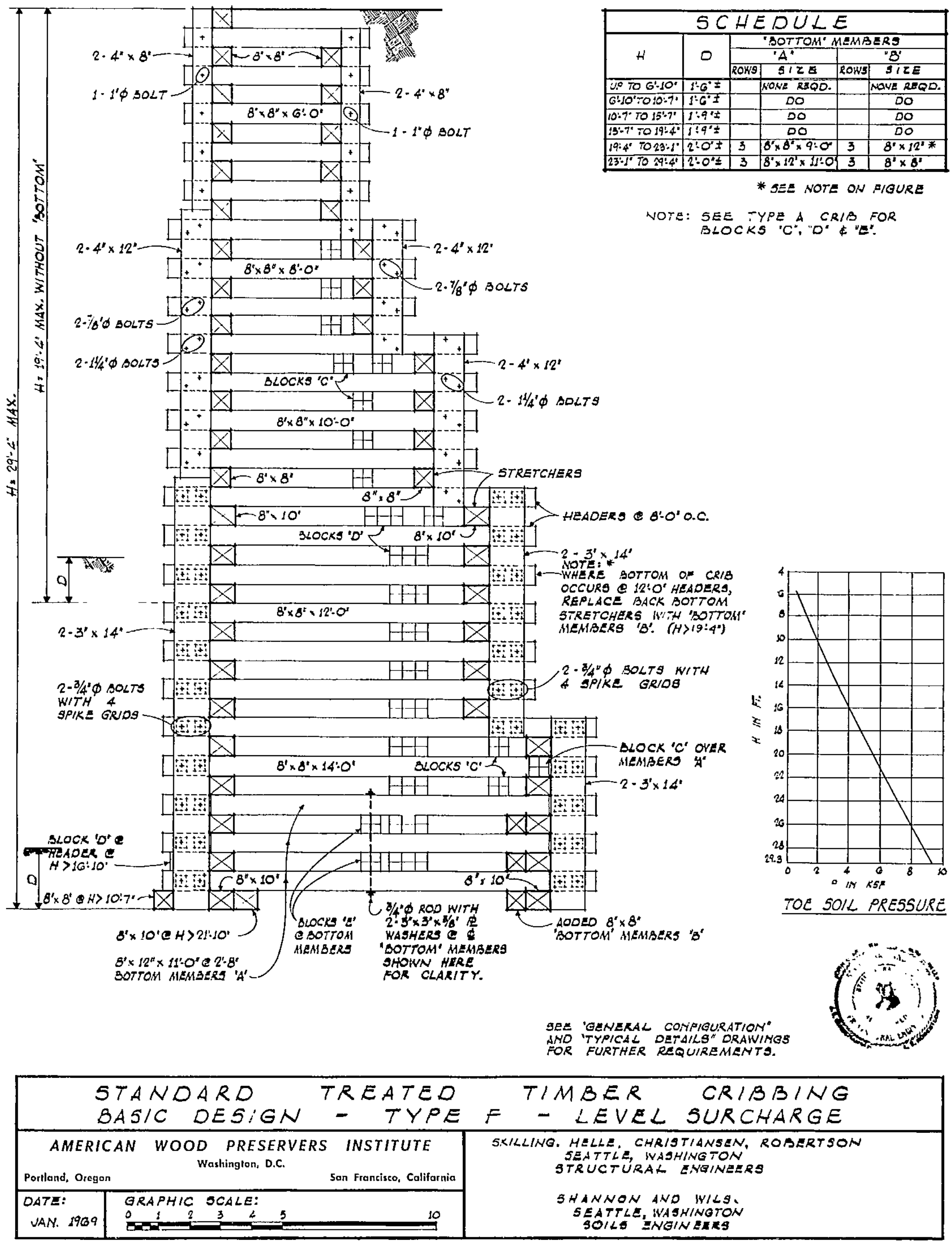
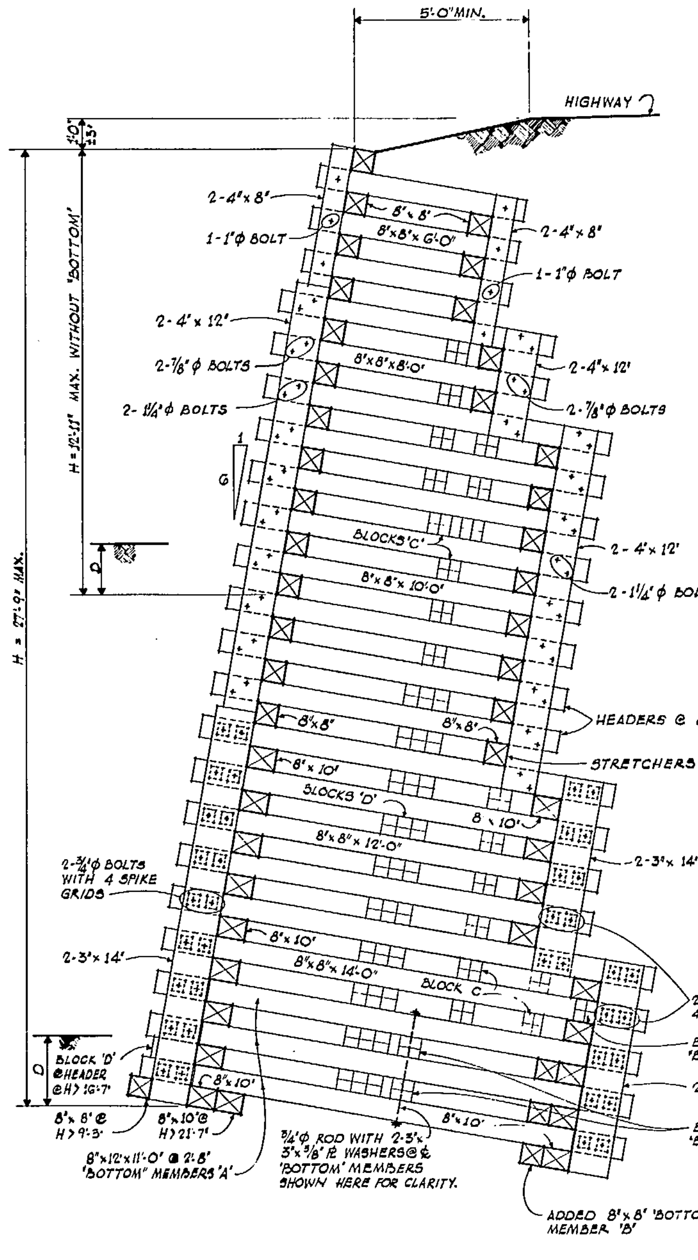
Fig 9 - Timber cribbing design - Type E.



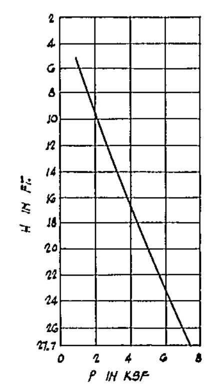
H	D	'BOTTOM' MEMBERS 'A' ROWS	'A' SIZE	'B' ROWS	'B' SIZE
UP TO 6'-10"	1'-6"±		NONE REQD.		NONE REQD.
6'-10" TO 10'-7"	1'-6"±		DO		DO
10'-7" TO 15'-7"	1'-9"±		DO		DO
15'-7" TO 19'-4"	1'-9"±		DO		DO
19'-4" TO 23'-1"	2'-0"±	3	8"x 8"x 9'-0"	3	8"x 12" *
23'-1" TO 29'-4"	2'-0"±	3	8"x 12"x 11'-0"	3	8"x 8"

Fig 10 - Timber cribbing design - Type F.



SCHEDULE					
H	D	'BOTTOM' MEMBERS			
		'A'		'B'	
		ROWS	SIZE	ROWS	SIZE
UP TO 6'-9"	1'-6"±		NONE REQD.		NONE REQD.
6'-9" TO 12'-11"	1'-6"±		DO		DO
12'-11" TO 16'-7"	1'-9"±	2	8"x8" x 7'-3"	2	8" x 8"
16'-7" TO 21'-7"	2'-0"±	3	8"x8" x 9'-0"	3	8" x 8"
21'-7" TO 27'-9"	2'-0"±	3	8"x12"x11'-0"	3	8" x 8"

NOTE: SEE TYPE A CRIB FOR BLOCKS 'C', "D" & 'E'.



TOE SOIL PRESSURE

SEE 'GENERAL CONFIGURATION' AND 'TYPICAL DETAILS' DRAWINGS FOR FURTHER REQUIREMENTS.

STANDARD TREATED TIMBER CRIBBING
BASIC DESIGN - TYPE G - HIGHWAY SURCHARGE

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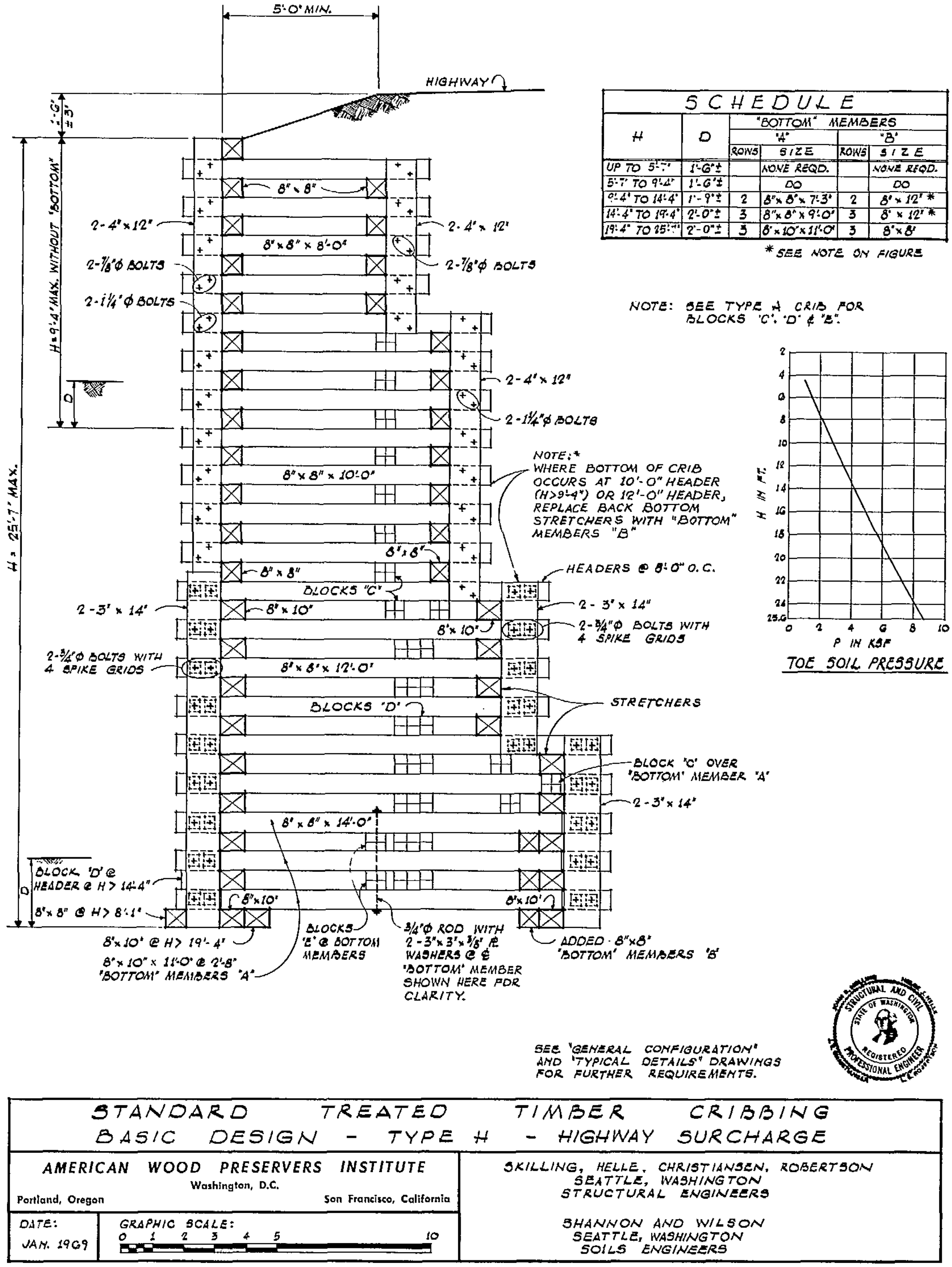
Fig 11 - Timber cribbing design - Type G.

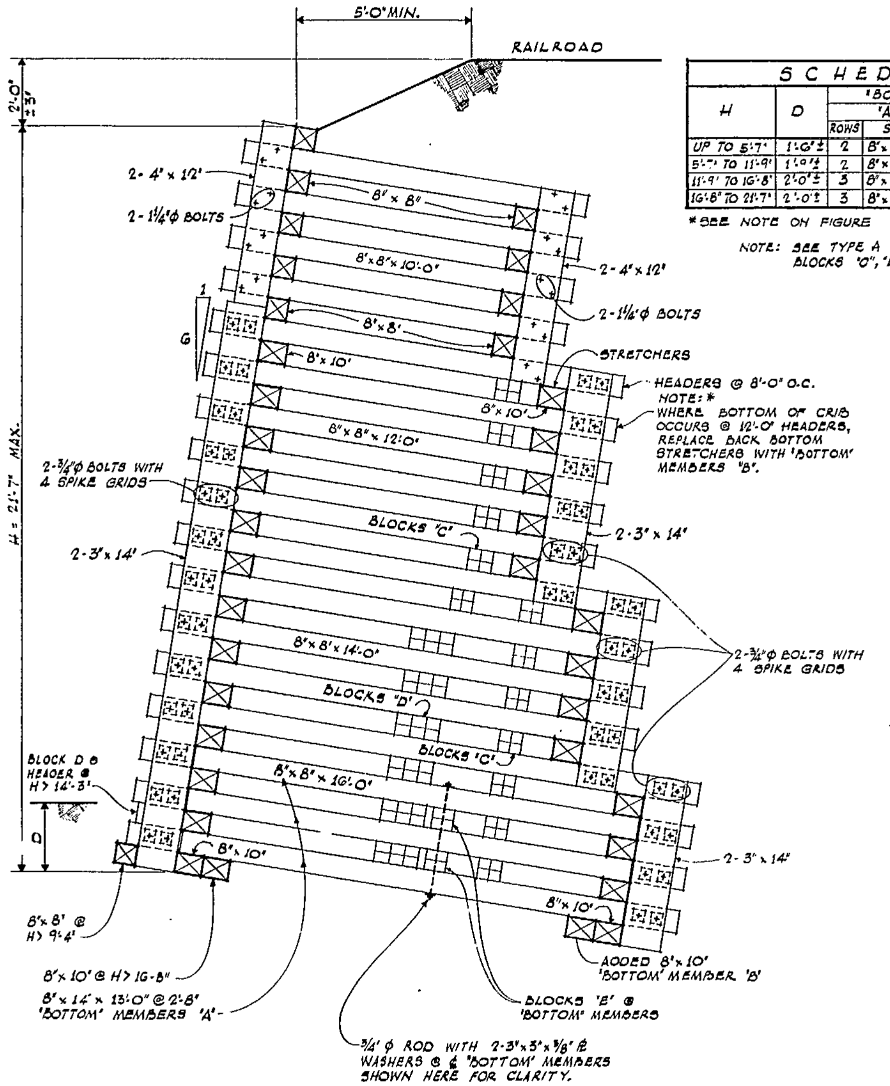


SCHEDULE

H	D	"BOTTOM" MEMBERS "A" ROWS	"A" SIZE	"B" ROWS	"B" SIZE
UP TO 5'-7"	1'-6"±		NONE REQD.		NONE REQD.
5'-7" TO 9'-4"	1'-6"±		DO		DO
9'-4" TO 14'-4"	1'-9"±	2	8"x 8"x 7'-3"	2	8"x 12" *
14'-4" TO 19'-4"	2'-0"±	3	8"x 8"x 9'-0"	3	8"x 12" *
19'-4" TO 25'-7"	2'-0"±	3	8"x 10"x 11'-0"	3	8"x 8"

*SEE NOTE ON FIGURE

Fig 12 - Timber cribbing design - Type H.

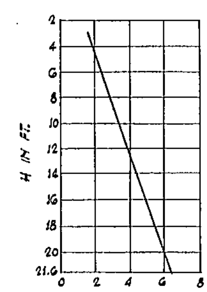


H	D	'BOTTOM' MEMBERS			
		"A"		"B"	
		ROWS	SIZE	ROWS	SIZE
UP TO 5'-7"	1'-6"±	2	8"x 8"x 7'-3"		NONE REQD.
5'-7" TO 11'-9"	1'-9"±	2	8"x 8"x 9'-0"	2	8"x 12" *
11'-9" TO 16'-8"	2'-0"±	3	8"x 10"x 11'-0"	3	8"x 8
16'-8" TO 21'-7"	2'-0"±	3	8"x 14"x 13'-0"	3	8"x 10"

SCHEDULE

* SEE NOTE ON FIGURE

NOTE: SEE TYPE A CRIB FOR BLOCKS "C", "D" & "E".



TOE SOIL PRESSURE

SEE 'GENERAL CONFIGURATION' AND 'TYPICAL DETAILS' DRAWINGS FOR FURTHER REQUIREMENTS.

STANDARD TREATED TIMBER CRIBBING BASIC DESIGN - TYPE J - RAILROAD SURCHARGE	
AMERICAN WOOD PRESERVERS INSTITUTE Washington, D.C. Portland, Oregon — San Francisco, California	SKILLING, HELLE, CHRISTIANSEN, ROBERTSON SEATTLE, WASHINGTON STRUCTURAL ENGINEERS
DATE: JAN. 1969 — GRAPHIC SCALE: 0 1 2 3 4 5 10	SHANNON AND WILSON SEATTLE, WASHINGTON SOILS ENGINEERS

Fig 13 - Timber cribbing design - Type J.

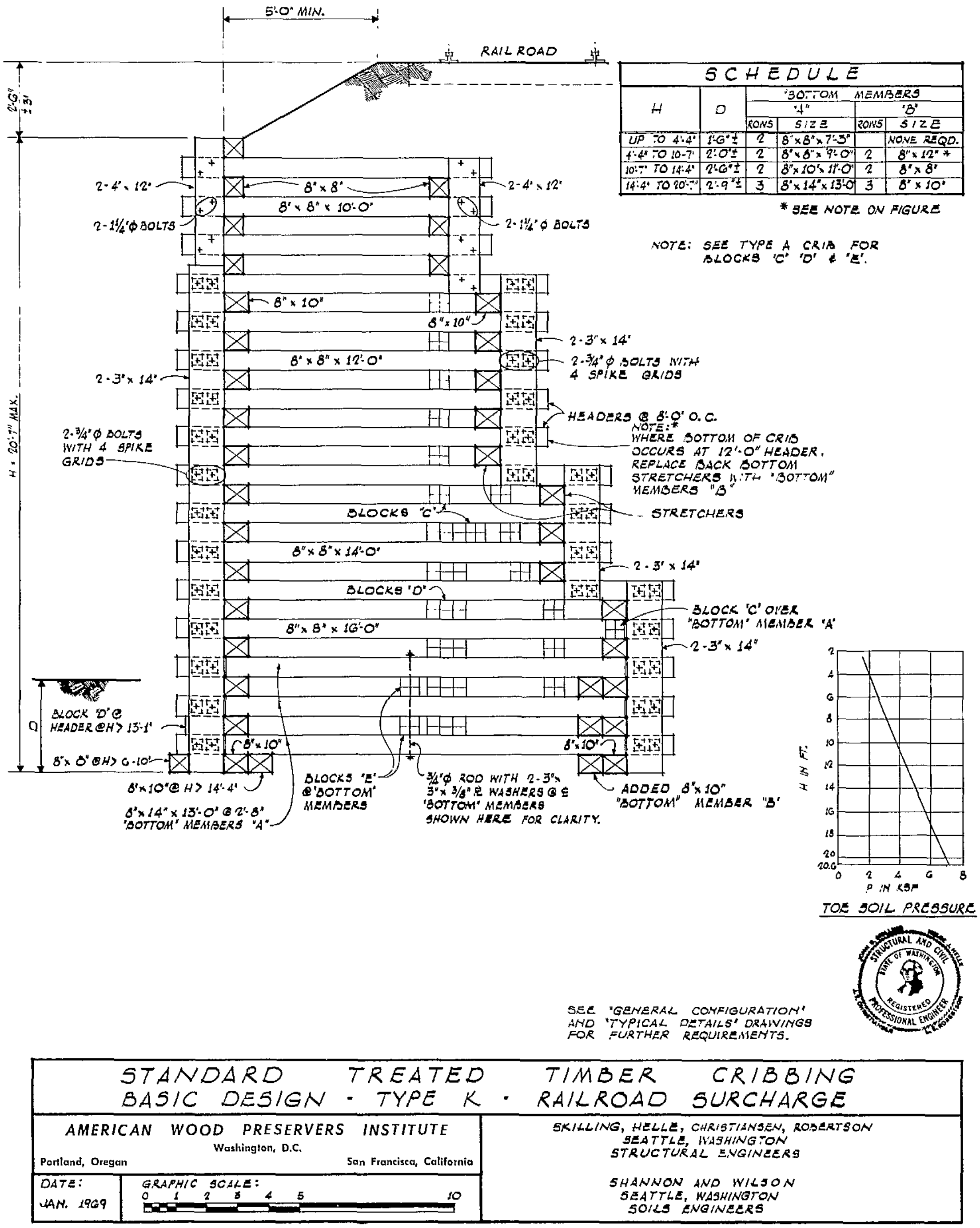


Fig 14 - Timber cribbing design - Type K.

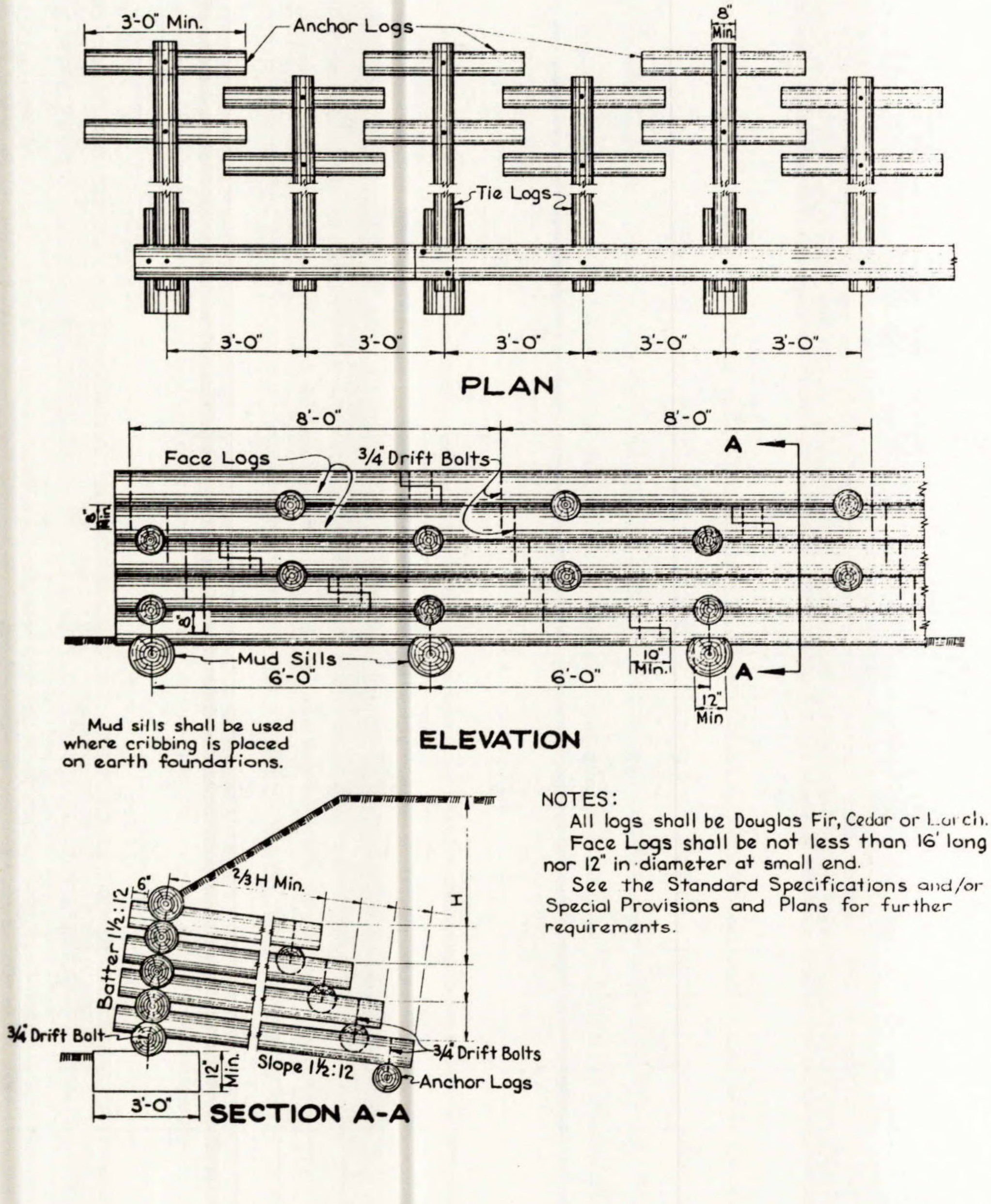
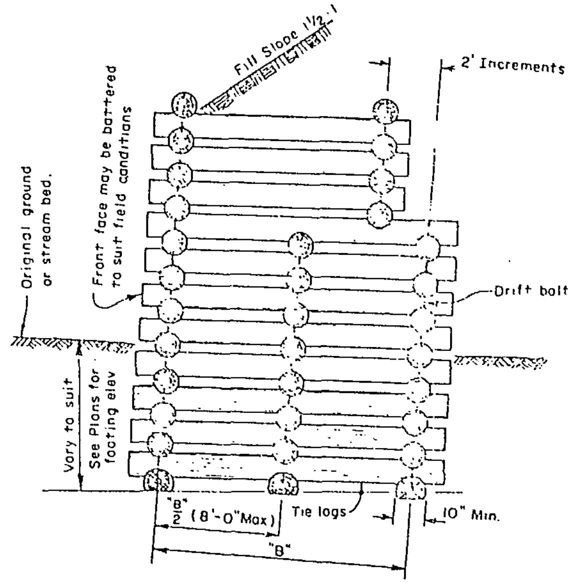
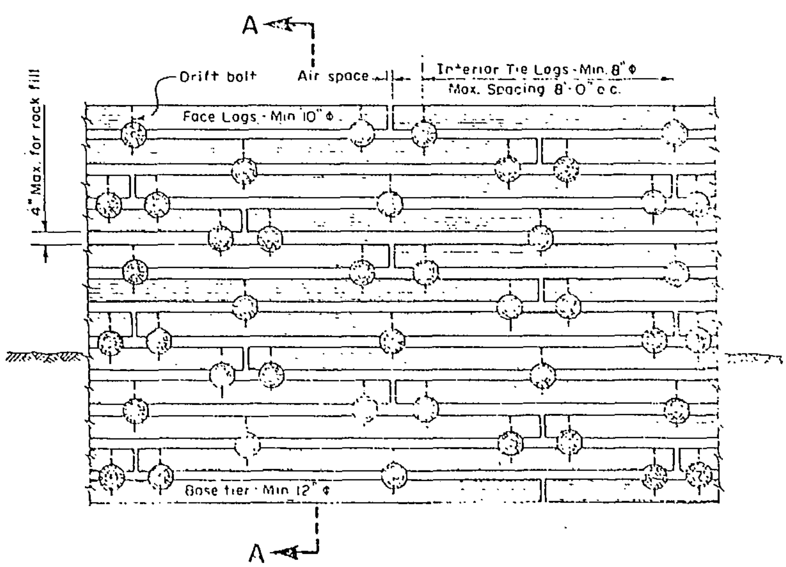


Fig 15 - Standard plans for log cribbing - Washington State Department of Highways (14).



SECTION A-A
(TYPICAL)



TYPICAL FRONT ELEVATION

Fig 16 - Typical front elevation and cross-section for standard log cribbing - U.S. Bureau of Public Roads (10).

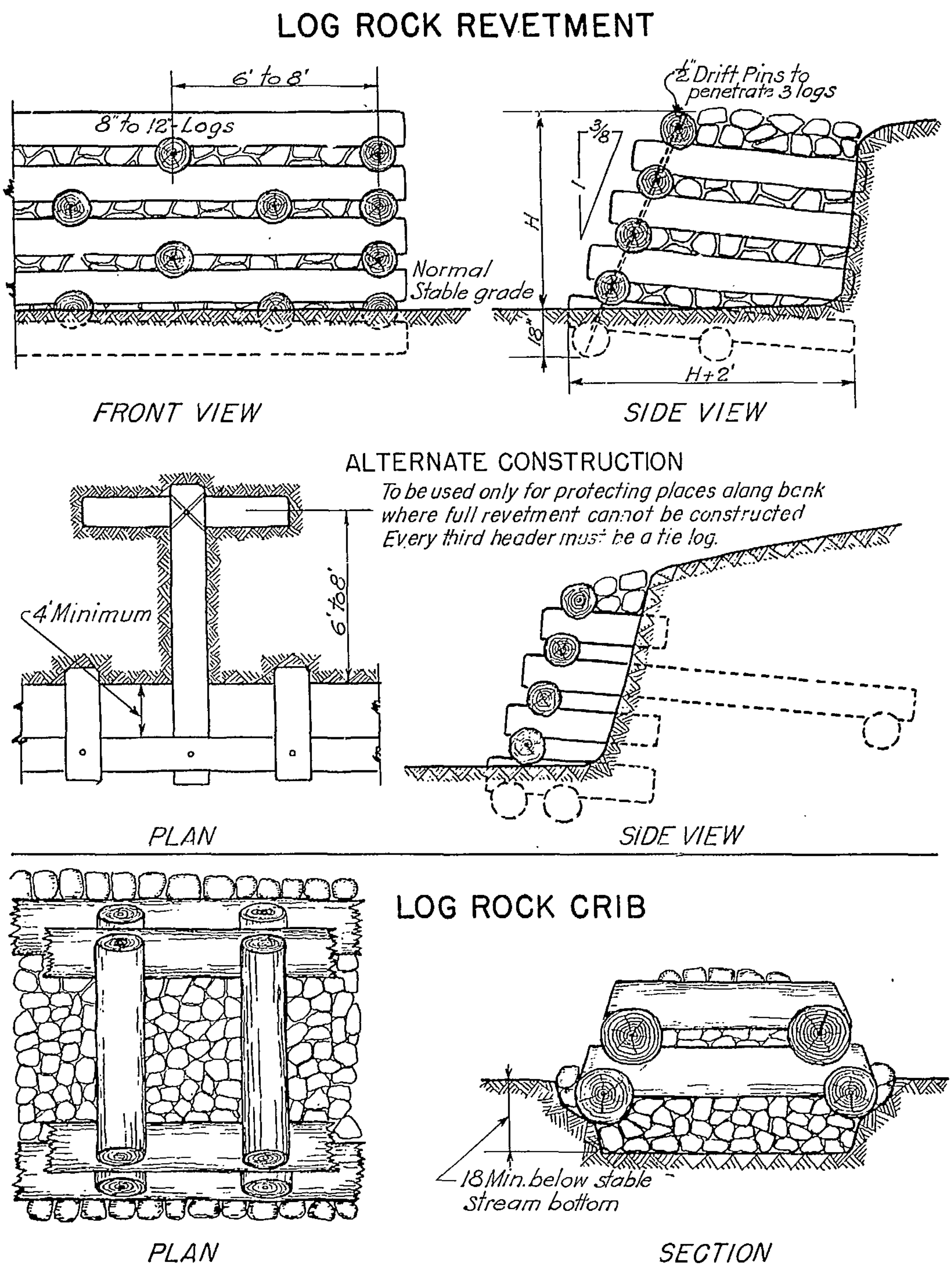


Fig 17 - Typical log-rock retaining structures (15).

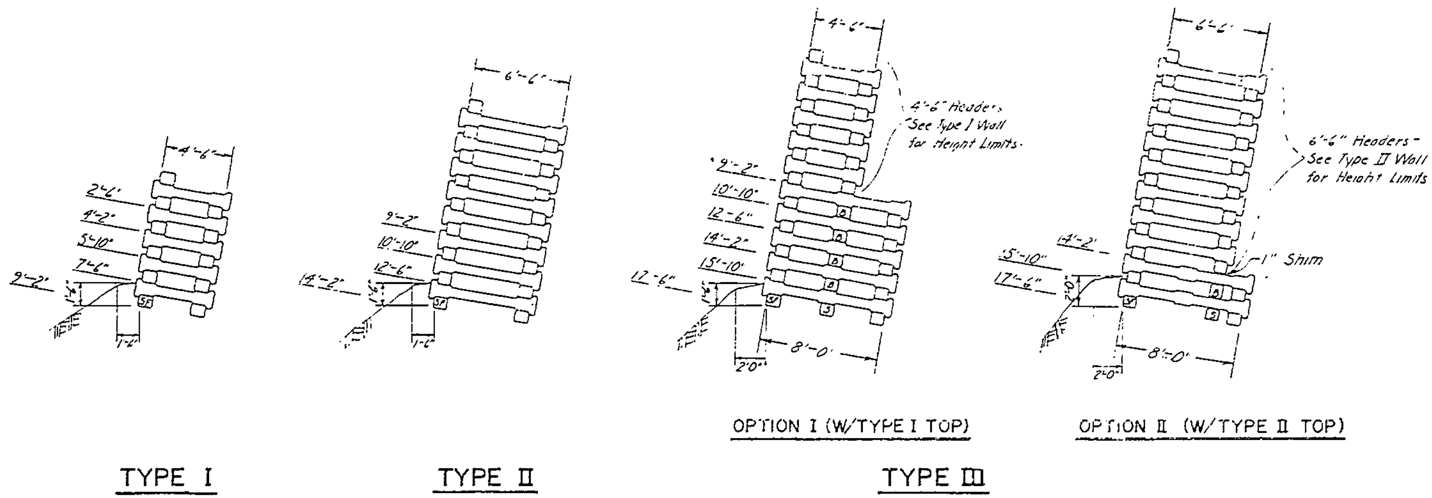


Fig 18 - Standard concrete crib wall designs (16).

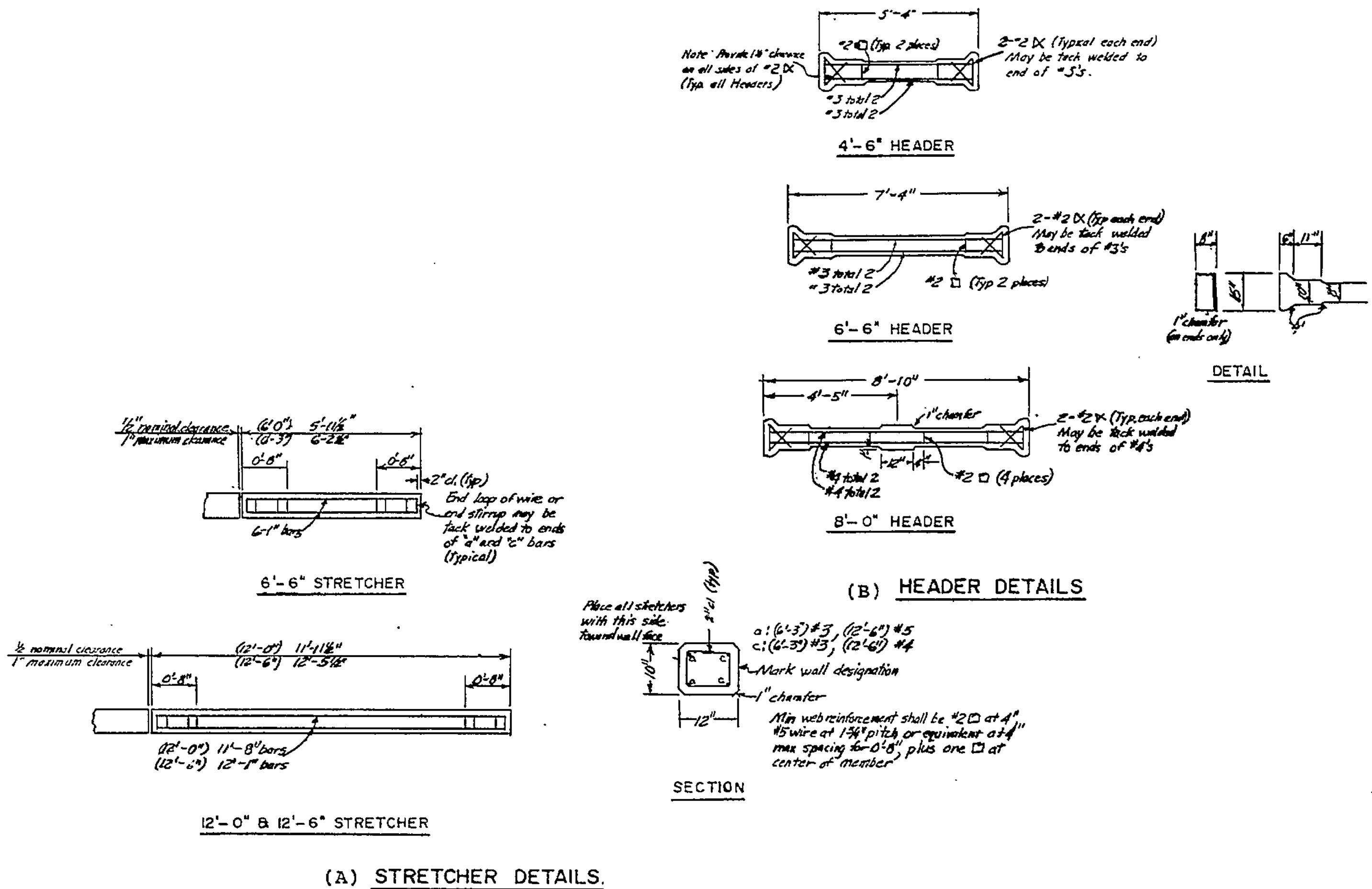


Fig 19 - Detail of concrete crib members - U.S. Department of Transport (16).

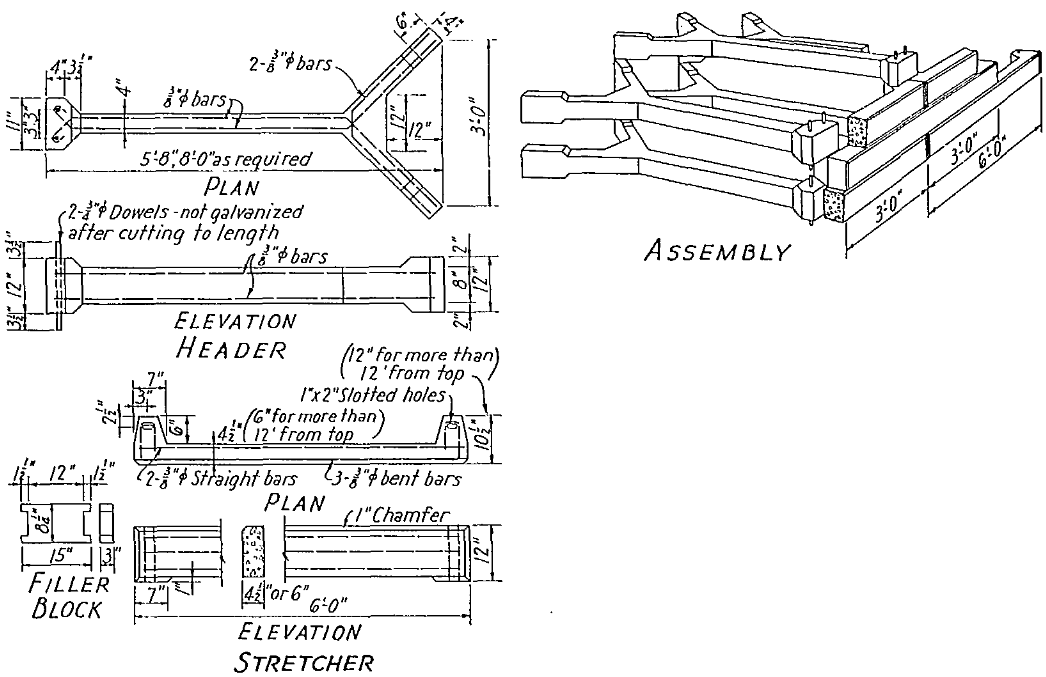
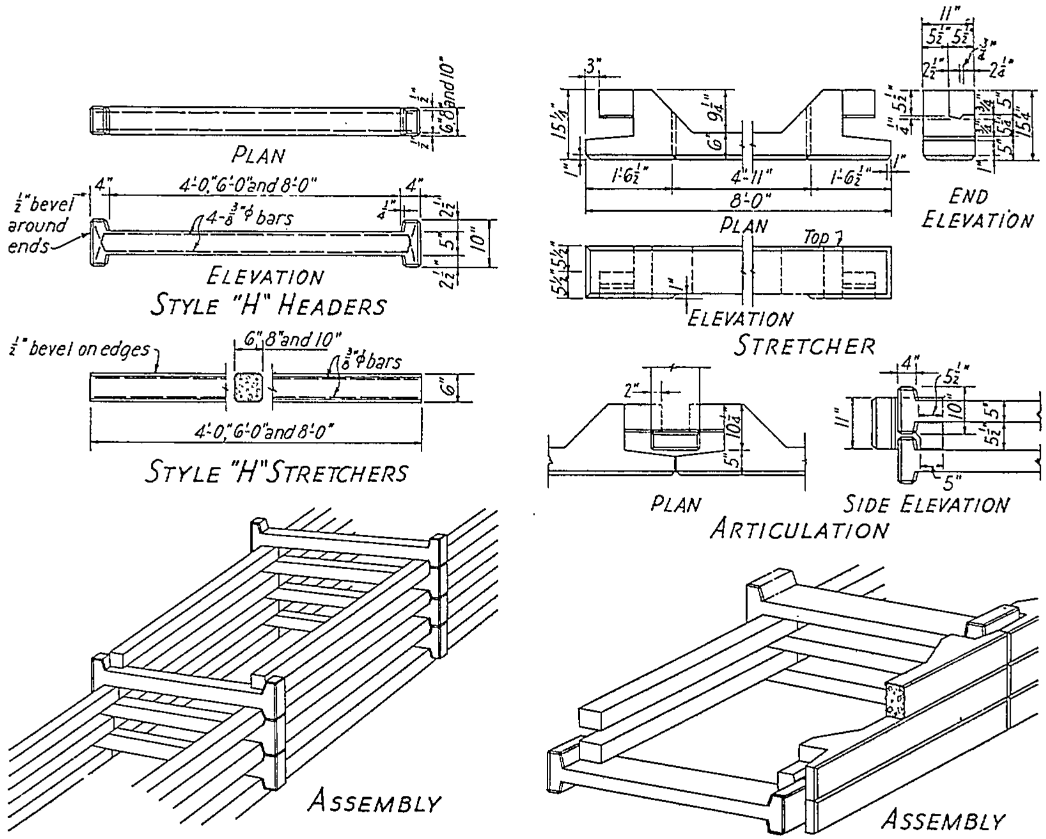


Fig 20 - Fishtail anchorage with a flush face wall (17).



(a) Open Face (b) Closed Face

Fig 21 - Continuous backwall anchorage (17).

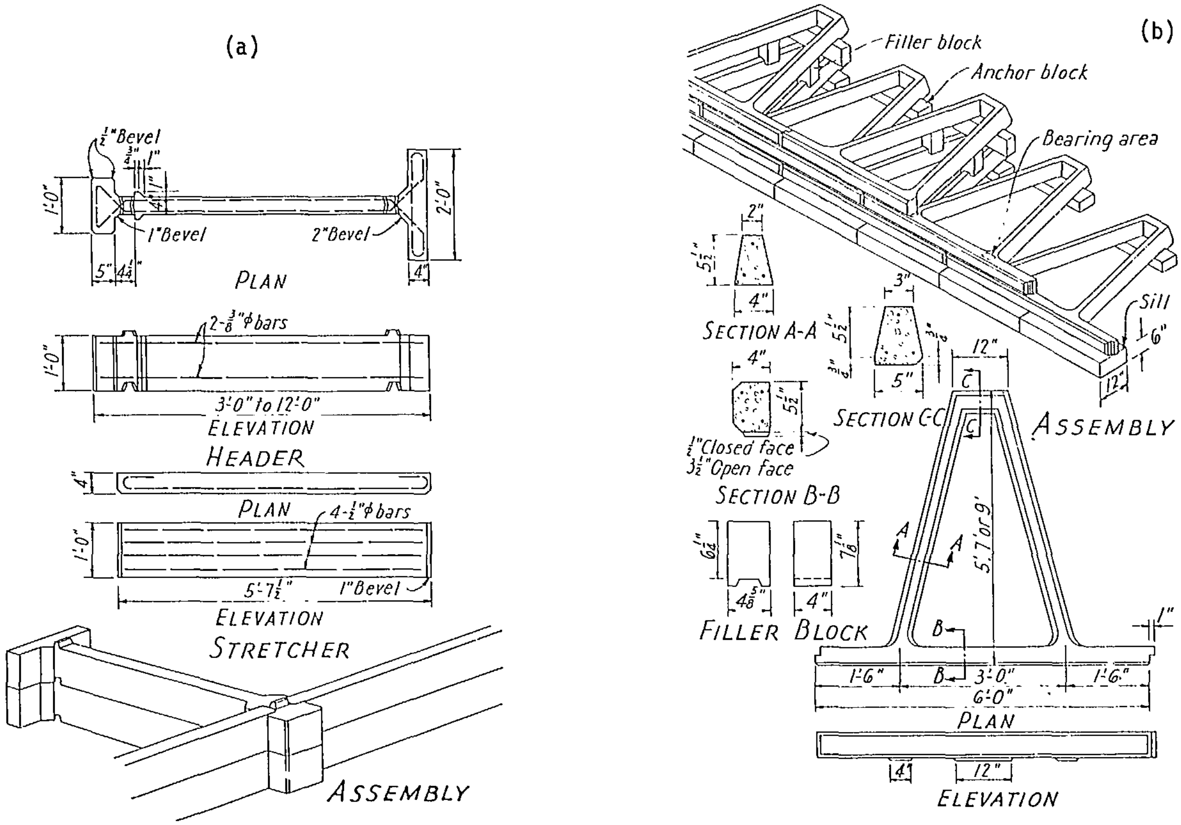


Fig 22 - Pre-cast concrete crib walls; a) counterfort-type wall with a flush face. b) Modified bin structure with a discontinuous back wall (17).

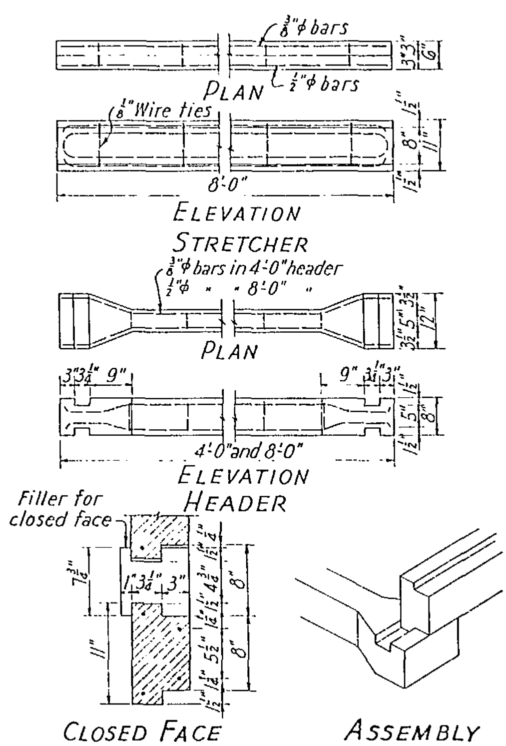
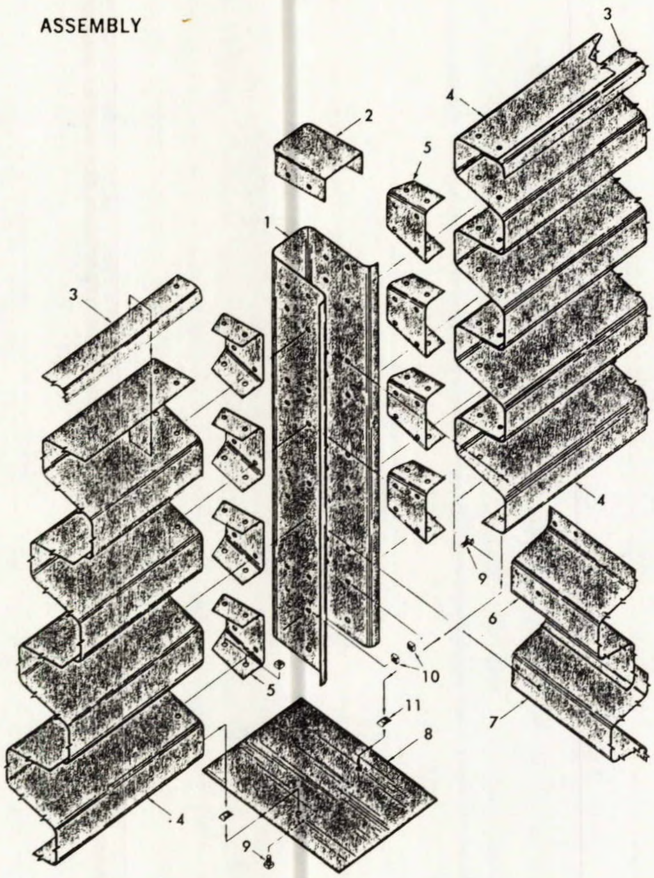
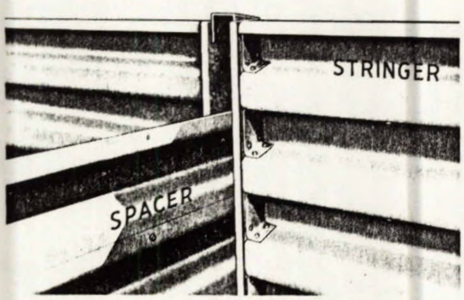


Fig 23 - Continuous back wall anchorage with an open or closed face (17).

ASSEMBLY





View from back of bin to inside of face. Note how the stringers and spacers are securely joined to the vertical connectors to make a completely closed bin.

LIST AND DESCRIPTION OF UNITS

	NAME	DESCRIPTION
1	Vertical connector	Vertical member connecting all other units
2	Vertical connector cap	Cover for front vertical connector
3	Stringer stiffener	Top flange protector
4	Stringer	Horizontal longitudinal members in front and rear walls
5	Connecting channel	Connector for attaching stringers to vertical connectors
6	Spacer	Transverse members that separate the front and rear vertical connectors
7	Bottom spacer	Special bottom transverse member
8	Grade plate	Installation plate on which the vertical connector rests
9	1¼" x ⅝" bolts	
10	⅝" nuts	
11	⅝" spring nuts	
Not shown	Split vertical connector	Used where bins of different thicknesses are joined

Fig 24 - Armco bin-type retaining wall (18).

Armco Type 2 wall which uses a T-shaped vertical connector (Fig 25). As with other cribs, the Armco unit is designed as a gravity structure. An empirical design chart is shown in Fig 26 for the various loading conditions shown in Fig 27. A particular wall design is selected from Fig 26 based on the given loading condition and required wall height. The six different designs of the Type 1 wall are illustrated in Fig 28.

35. A similar design sequence is carried out for Armco Type 2 crib walls. The basic design chart is shown in Fig 29 and the same loading conditions are used as for the Type 1 (Fig 27). A particular wall design is selected from this design sequence. The six different designs of the Type 2 wall are illustrated in Fig 30.

36. Both Types 1 and 2 Armco walls can be built in a curved configuration with the radius of curvature depending on the type of construction technique. For relatively large curvatures, special components are used in the wall construction. These walls may be curved either to the inside or the outside.

37. Standard Armco wall units are fabricated of galvanized steel. However, if extremely corrosive service conditions are expected, the units are available in a special asbestos-bonded metal for additional corrosion protection. Specifications for construction and backfill are described later in this supplement.

38. Kaiser cellular type aluminum walls are designed for the five basic loading conditions shown in Table 4. They are composed of cylindrical cells placed adjacent to one another and rigidly attached to form an integral wall.

39. Structural components to satisfy various loading conditions for various wall heights are available in different metal thicknesses. They can be used to fabricate cellular walls of variable diameter (thickness). The available choices are shown in Table 5.

40. For any required wall height and particular loading condition, an adequate structure type may be chosen from Fig 31 and the required burial depth at the toe can be found from Table 7.

41. Table 6 summarizes the expected bearing pressure to be applied to the soil for any expected load configuration and wall height. Table 8 shows the maximum permissible wall height as being governed by internal cell pressure (based on 5/8 in. dia bolts, and equivalent fluid pressure of 45 pcf).

42. Once a design for a crib wall has been chosen, it is advisable to check the stability of the structure for shearing through and beneath the structure (Fig 32). Resistance to sliding and overturning should already have been checked in the design process.

43. Shear failure through the wall may occur along arc EH or line JH (or along any other critical slip surface). Foundation failure may occur beneath the wall, along arc FG.

44. In the stability analysis, the resisting forces are compared with the driving forces to arrive at a factor of safety. Crib walls or any retaining walls are more stable if placed on a rock foundation (where possible or practicable) as foundation failures through rock are unlikely.

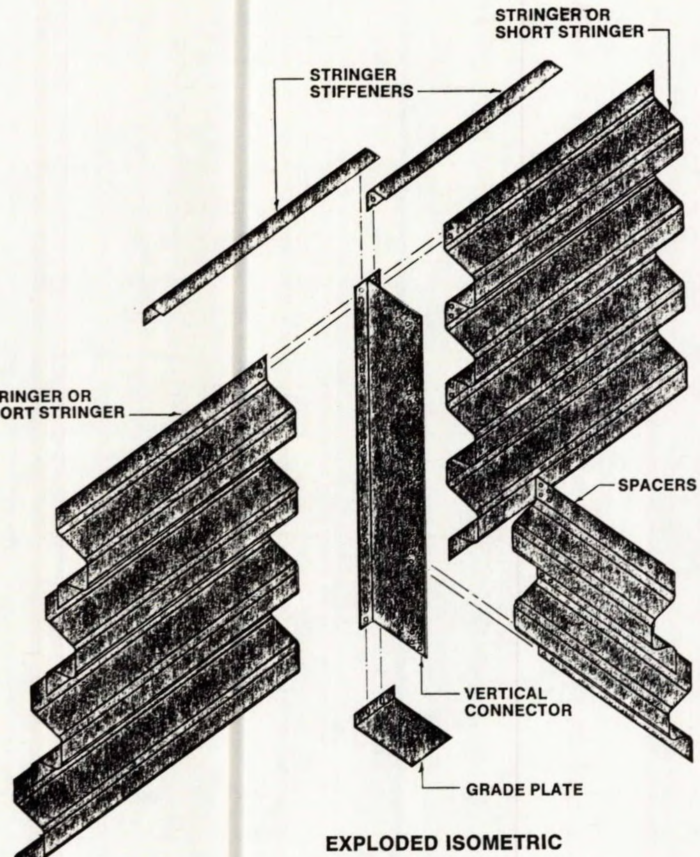
GABIONS

45. Gabions are gravity structures and their design follows standard engineering practice for retaining walls, as illustrated in Fig 3. The gabion itself is a woven wire basket filled with rock and stacked one upon another to form a gravity type retaining wall. The final structure may have an irregular backwall and hence must be analyzed using the vertical membrane analogy as used for analyzing crib structures.

46. Some advantages of gabion walls are (22, 23):

a. they can be erected quickly,
b. they afford good drainage,
c. they are flexible and can withstand reasonable, differential settling without fracture and need not be founded below the frost line,
d. they require a minimum of lateral space, and
e. if excessive lateral movement is observed after construction, another course of gabion baskets can be added to the face of the wall to provide additional gravity load (provided space permits) with minimal disturbance of the existing structure.

47. Gabions are available from a number of



PARTS LIST

PART	FUNCTION
Vertical Connector	Connects stringers and spacers
Corner Vertical Connector	Connects stringers and spacers
Stringer Stiffener	Front face top trim
Stringer	Forms front and rear faces (panel sections)
Spacer	Connects front and rear face (transverse sections)
Grade Plate	Base for vertical connectors
⅝" Bolts and Nuts	Fasteners
Split Vertical Connector	Used where bins of different thicknesses meet
Spacer Closure	Retains fill at ends of walls
Stringer Closure	Retains fill at special corners

Fig 25 - Armco bin wall, Type 2 (19).

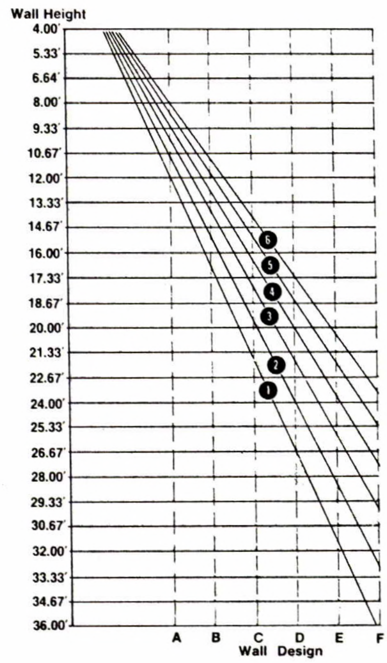


Fig 26 - Design chart for Armco, Type 1 crib wall (18).

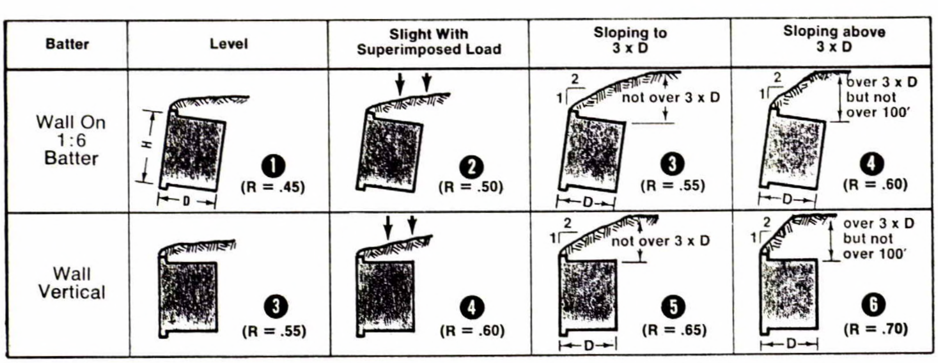


Fig 27 - Loading conditions for Armco, Type 1 crib wall: R = Depth D/Height H (18).

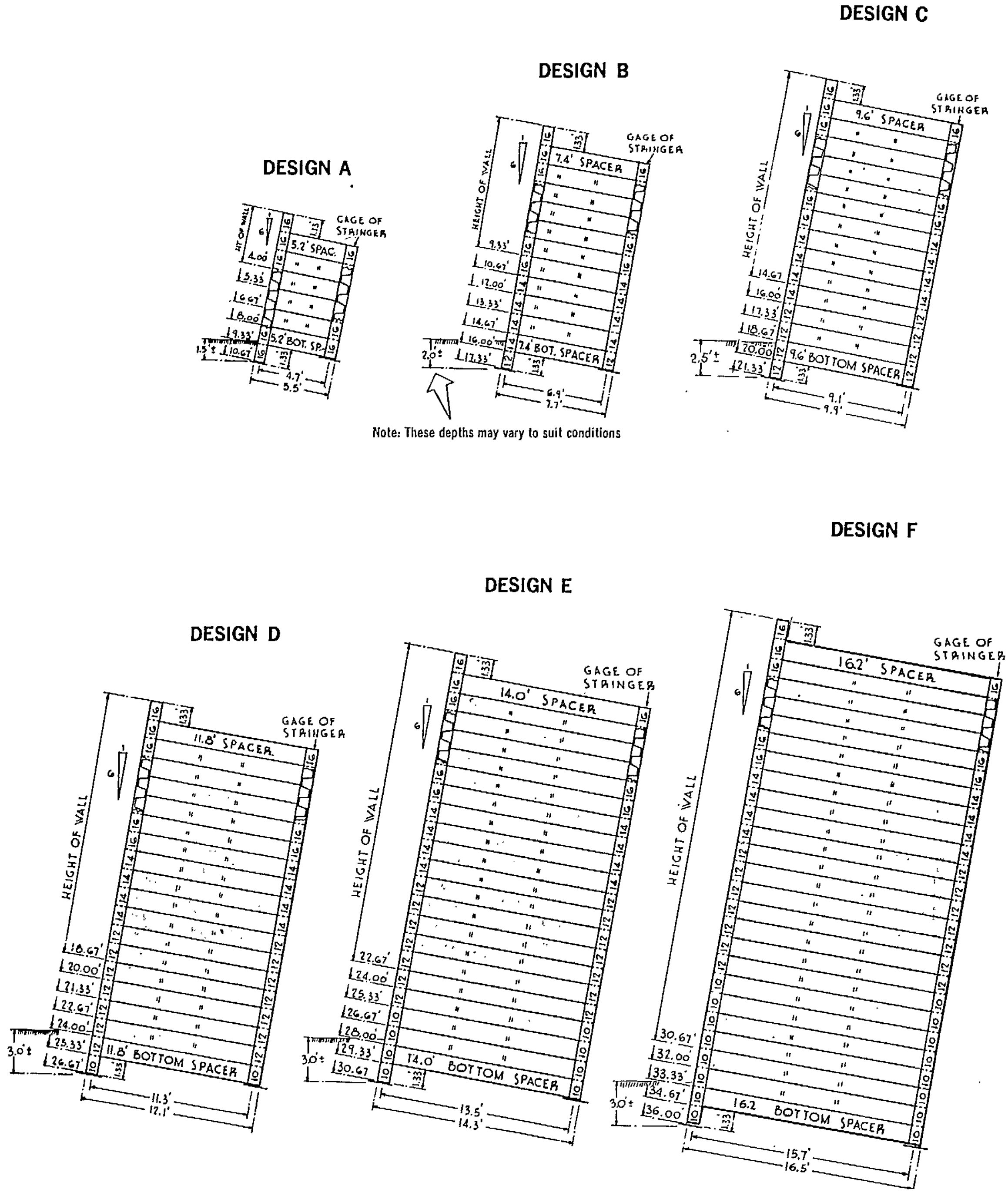


Fig 28 - Armco, Type 1 wall design details (18).

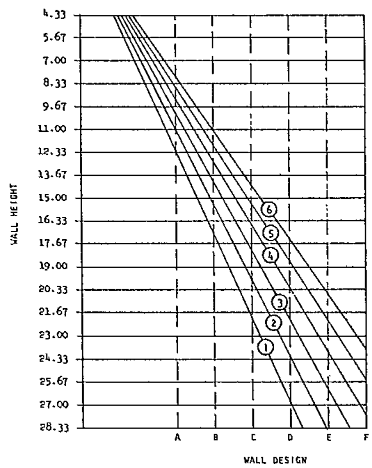


Fig 29 - Design chart for Armco, Type 2 crib walls (19).

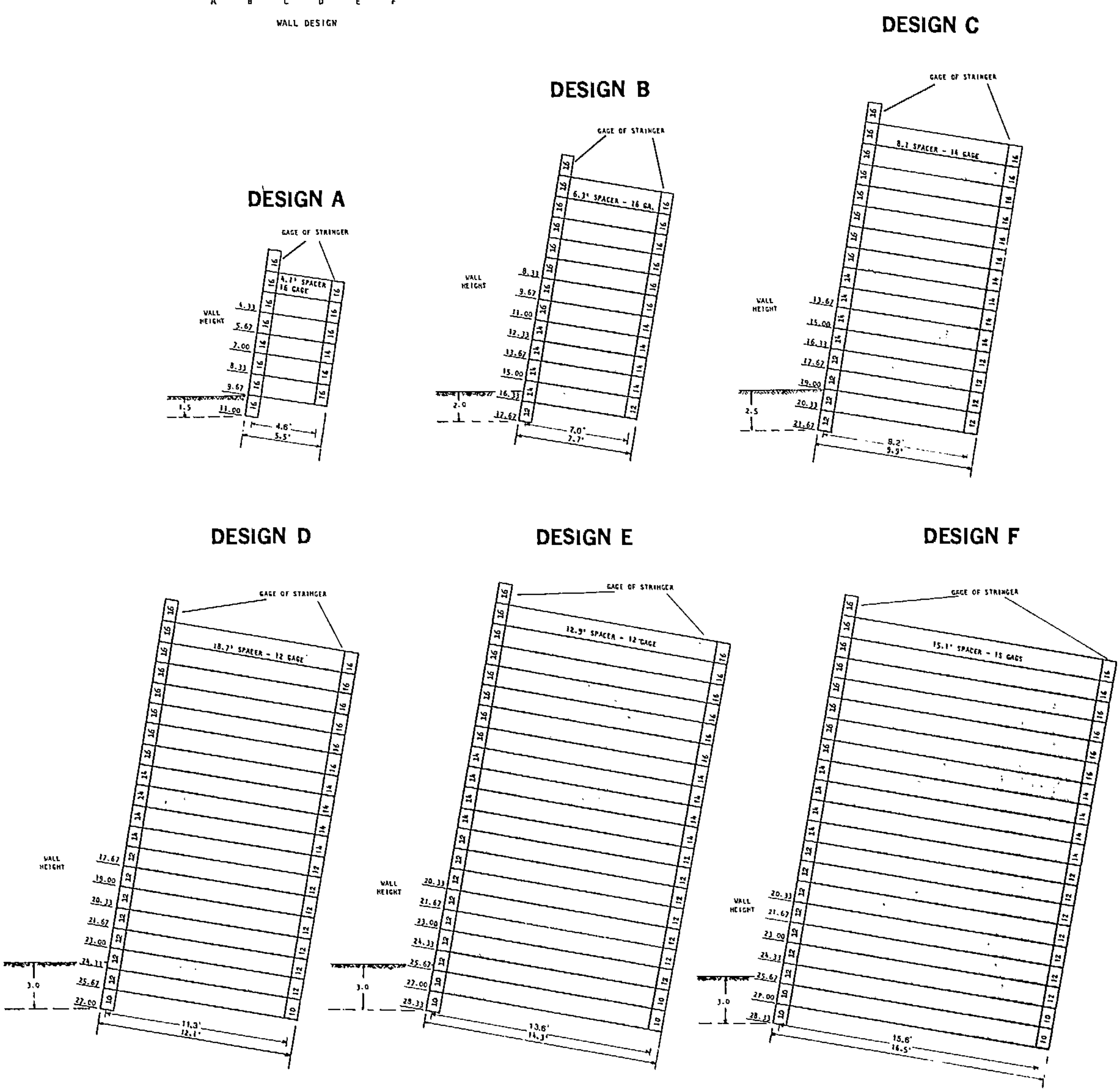


Fig 30 - Typical ratios of thickness to height for Armco, Type 2 walls (19).

Table 4: Selection of load condition for Kaiser cellular type aluminum walls

Load condition	Surcharge	Batter
1	Level	1:6
2	Slight and line load	1:6
3	Infinite	1:6
4	Level	Vertical
5	Slight and line load	Vertical
6	Infinite	Vertical

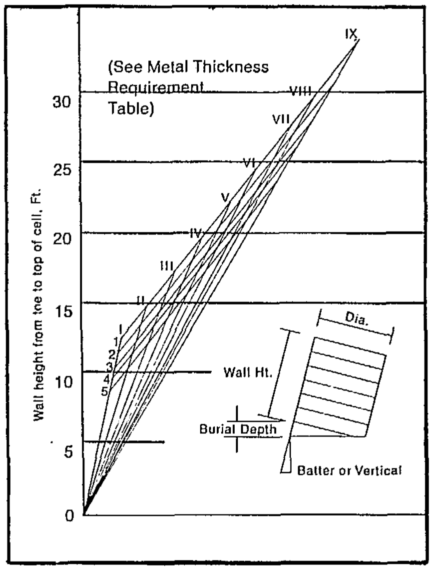


Fig 31 - Chart for selecting crib wall type (20).

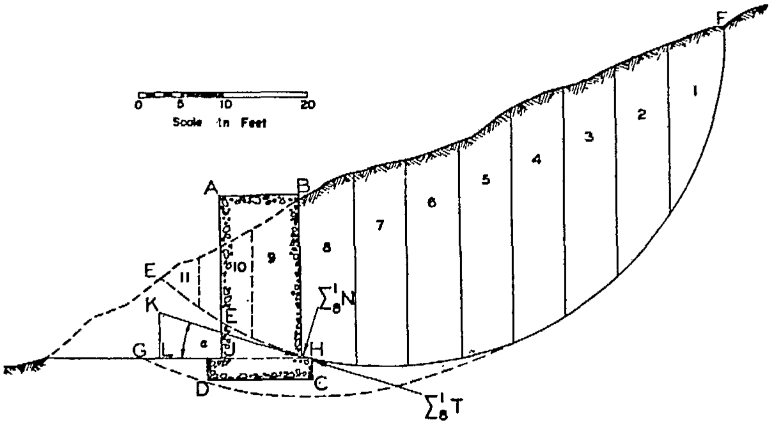


Fig 32 - Crib wall design criteria.

Table 5: Maximum cell height vs metal thickness

After Kaiser (20)

	Structure type	Cell diam. (ft)	Cell height (ft) for various metal thicknesses: 0.060" 16 ga	0.075" 14 ga	0.105" 12 ga	0.135" 10 ga	0.164" 8 ga
A	I	6.2	12***				
	II	7.7	15***				
	III	8.7	14	17***			
	IV	10.2	12	16	20***		
	V	11.3	10	14	20	22***	
	VI	12.8	10	12	16	22	24***
	VII	13.8	8	12	16	20	24
	VIII	15.3	8	10	14	18	22
	IX	16.5	8	10	14	16	20
B	I	6.2	12***				
	II	7.7	15***				
	III	8.7	17***				
	IV	10.2	18	20***			
	V	11.3	16	20	22***		
	VI	12.8	14	18	24***		
	VII	13.8	12	16	22	27***	
	VIII	15.3	12	14	20	26	29***
	IX	16.4	10	14	20	24	30

A For cells assembled with 1/2 in. diameter steel bolts* and back-filled with soil having an equivalent fluid pressure of 45 pcf.**

B For cells assembled with 1/2 in. diameter steel bolts* and back-filled with soil having an equivalent fluid pressure of 30 pcf.***

* If these cells are assembled with 5/8 in. diameter steel bolts instead of 1/2 in. the fill height may be increased by 12 per cent, provided the resulting height does not exceed the maximum height limitation noted in Structure Type Selection chart, Fig 35.

** The soil description for these equivalent pressures is given by Terzaghi and Peck (1967) as follows:

45 pcf = Residual soil with stones, fine silty sand, and granular materials with conspicuous clay content (non-saturated)

30 pcf = Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand and gravel).

*** Denotes that maximum height is determined by overturning for Type 1 loading condition rather than by hoop tension. Maximum height limitations for other load conditions can be obtained from the Structure Type Selection chart.

Table 6: Foundation pressures and stable wall height limits

After Kaiser (21)

Bearing pressure at toe is in tsf

Surcharge type	**H ft.	Bin type I	II	III	IV	V	VI	VII	VIII	LX
1	5	0.4	0.3	0.2	0.2	0.2	0.2	0.1	0.1	0.1
	10	0.9	0.7	0.6	0.6	0.5	0.5	0.4	0.4	0.4
	15	2.0	1.5	1.3	1.1	1.0	0.9	0.8	0.8	0.8
	20	---	2.8	2.3	1.9	1.7	0.5	1.3	1.3	1.2
	25	---	---	---	3.1	2.6	2.3	2.1	1.9	1.8
	30	---	---	---	---	---	3.4	3.0	2.7	2.5
	35	---	---	---	---	---	---	---	3.7	3.4
	h.max.	10	14	17	20	23	26	28 / 24*	31 / 22*	34 / 20*
2	5	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	10	1.1	1.0	0.8	0.8	0.7	0.6	0.6	0.6	0.6
	15	2.8	2.2	1.9	1.6	1.4	1.3	1.2	1.1	1.1
	20	---	---	3.5	2.9	2.6	2.3	2.1	1.9	1.8
	25	---	---	---	---	4.2	3.6	3.3	3.0	2.8
	30	---	---	---	---	---	---	4.9	4.4	4.1
	h.max.	11	13	15	18	20	22	24 / 23*	27 / 20*	29 / 18*
3	5	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.3	0.3
	10	1.7	1.4	1.2	1.1	1.0	0.9	0.8	0.8	0.8
	15	4.2	3.2	2.8	2.3	2.1	1.9	1.7	1.6	1.6
	20	---	---	5.2	4.3	3.8	3.3	3.1	2.8	2.6
	25	---	---	---	---	---	3.4	4.9	4.4	4.1
	30	---	---	---	---	---	5.4	---	6.5	6.0
	h.max.	10	12	14	16	18	20	22 / 21*	25 / 19*	26 / 17*
4	5	0.4	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.1
	10	1.1	0.8	0.7	0.6	0.5	0.5	0.4	0.4	0.4
	15	2.6	1.9	1.5	1.3	1.1	1.0	0.9	0.8	0.8
	20	---	3.6	2.9	2.3	2.0	1.7	1.5	1.4	1.3
	25	---	---	---	3.9	3.3	2.8	2.5	2.2	2.1
	30	---	---	---	---	---	4.3	3.8	3.3	3.0
	35	---	---	---	---	---	---	---	4.7	4.3
	h.max.	10	14	16	19	22	25	27 / 24*	30 / 22*	33 / 20I

Table 6: Foundation pressures and stable wall height limits - cont.

After Kaiser (21)

Bearing pressure at toe is in tsf

Surcharge type	**H ft.	I	II	III	IV	V	VI	VII	VIII	LX
		Bin type								
5	5	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.2	0.2
	10	1.5	1.2	1.0	0.9	0.8	0.7	0.7	0.6	0.6
	15	3.8	2.9	2.4	2.1	1.8	1.6	1.5	1.4	1.3
	20	---	---	4.8	3.9	3.4	2.9	2.7	2.4	2.3
	25	---	---	---	---	5.7	4.8	4.4	3.9	3.6
	30	---	---	---	---	---	---	---	5.8	5.4
	h.max.	10	13	15	17	19	22	23	26 / 20*	28 / 18*
6	5	0.5	0.5	0.4	0.4	0.3	0.3	0.3	0.3	0.3
	10	2.3	1.8	1.6	1.4	1.2	1.1	1.0	1.0	1.0
	15	---	---	3.8	3.2	2.8	2.5	2.3	2.1	2.0
	20	---	---	---	---	5.2	4.5	4.1	3.7	3.5
	h.max.	7	8	10	11	14	15	17	18	28 / 17*

NOTE: * Height limited by hoop tension of 8 ga. 3004H34 alloy sheet.

** Maximum bin height (H) limited by shear strength of shear plane at vertical soil-bin interface on the back face of bin. All heights are the actual height of bin measured in ft.

Table 7: Diameter and burial depth for Kaiser cellular-type aluminum walls

Type	Diameter (ft)	Burial depth at toe (ft)
I	6.2	1.5
II	7.7	1.5
III	8.7	2.0
IV	10.2	2.0
V	11.3	2.5
VI	12.8	2.5
VII	13.8	3.0
VIII	15.3	3.0
IX	16.4	3.0

Table 8: Maximum height (ft) of cell as limited by internal cell pressures

After Kaiser (21)

Surcharge type			1		4			2		5			3		6	
Sheet gauge		16	14	12	10	8	16	14	12	10	8	16	14	12	10	8
	I	19	--	--	--	--	20	--	--	--	--	19	--	--	--	--
	II	15	--	--	--	--	15	--	--	--	--	15	--	--	--	--
	III	13	17	--	--	--	13	17	--	--	--	12	16	--	--	--
BIN	IV	11	14	21	--	--	11	14	21	--	--	10	13	10	--	--
	V	10	13	19	25	--	9	12	18	24	--	8	11	17	23	--
TYPE	VI	8	11	16	21	26	7	10	15	20	25	6	9	14	19	24
	VII	8	10	15	20	24	7	9	14	19	23	5	7	12	17	21
	VIII	7	9	13	18	22	5	7	11	16	20	4	6	10	15	19
	IX	6	8	12	16	20	4	6	10	14	18	3	5	9	13	17

suppliers. Two popular suppliers in North America are Maccaferri Gabions of America and Bekaert Gabions. The structures supplied by these two firms are similar, although the former provides both standard and metric sizes, and includes more compartments in a basket than does the latter. Each type may be constructed with either a straight or a stepped front face. The basic sizes of Bekaert Gabions are shown in Table 9, and those of Maccaferri Gabions are shown in Tables 10 and 11.

48. A critical factor in the design and construction of a gabion wall as a gravity structure is to ensure that a realistic unit weight is used for both the basket fill material and the backfill material. If either is overestimated compared with actual field conditions, excessive lateral movement or even failure of the structure may occur. Figure 33 is a good guide for estimating the unit weight of gabion fill. Note that porosity is given as 0.30 and if it differs significantly, this chart should not be used.

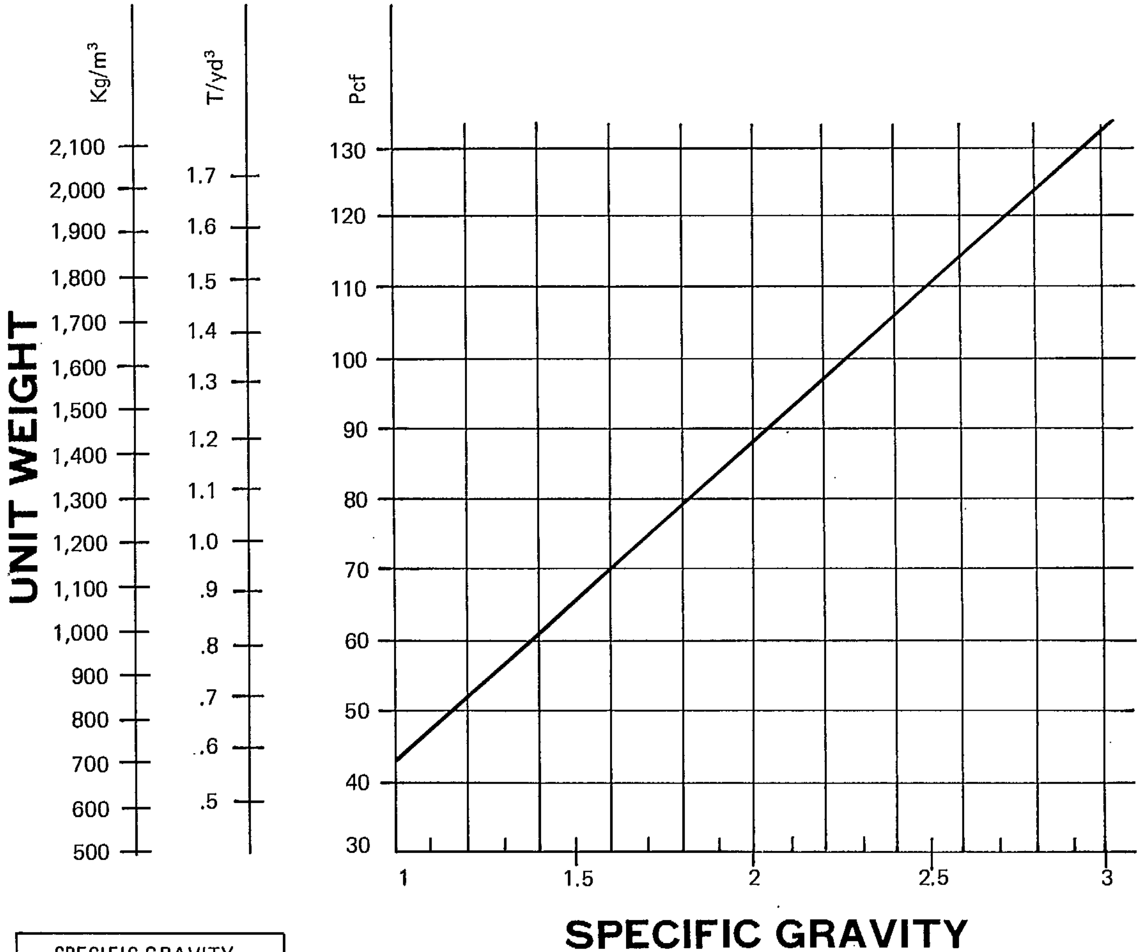
Table 9: Nominal size of Bekaert gabions

After Bekaert (24)

Size code		Size (ft)				Capacity
Letter	Colour	Length	Width	Depth	Partitions	(cu yd)
A	Blue	6	3	3	1	2
B	White	9	3	3	2	3
C	Black	12	3	3	3	4
D	Red	6	3	1 1/2	1	1
E	Green	9	3	1 1/2	2	1.5
F	Yellow	12	3	1 1/2	3	2
G	Blue/red	6	3	1	1	0.666
H	Blue/green	9	3	1	2	1
I	Blue/yellow	12	3	1	3	1.33

Table 10: Nominal size of Maccaferri standard gabions

Letter code	Length	Width	Height	Number of cells	Capacity cu yd	Color code
A	6'	3'	3'	2	2.0	Blue
B	9'	3'	3'	3	3.0	White
C	12'	3'	3'	4	4.0	Black
D	6'	3'	1'6"	2	1.0	Red
E	9'	3'	1'6"	3	1.5	Green
F	12'	3'	1'6"	4	2.0	Yellow
G	6'	3'	1'	2	0.66	Blue/red
H	9'	3'	1'	3	1.0	Blue/yellow
I	12'	3'	1'	4	1.33	Blue/green



SPECIFIC GRAVITY OF COMMON MATERIALS	
BASALT	3.0
BRICK	2.0
CONCRETE (Broken)	2.4
GRANITE	2.7
LIMESTONE	2.5
SANDSTONE	2.2
TRAP ROCK	2.7

Example:

GIVEN: SPECIFIC GRAVITY = 2.5
FIND: UNIT WEIGHT IN (a) Pcf, (b) T/yd^3, (c) Kg/m^3

SOLUTION: PROCEED VERTICALLY FROM S.G. = 2.5 TO INTERSECTION OF DIAGONAL LINE. THEN PROCEED HORIZONTALLY TO INTERSECTION OF VERTICAL LINE
AND FIND: (a) UNIT WEIGHT = 109 Pcf.
(b) " " = 1.48 T/yd^3
(c) " " = 1,760 Kg/m^3

Fig 33 - Chart for unit weight of gabion fill (22).

Table 11: Nominal sizes of Maccaferri metric gabions

Letter code	Length	Width	Weight	Number of cells	Capacity cu yd	Color code
A	6'6"	3'3"	3'3"	2	2.62	Blue
B	9'9"	3'3"	3'3"	3	3.93	White
C	13'1"	3'3"	3'3"	4	5.24	Black
D	6'6"	3'3"	1'8"	2	1.31	Red
E	9'9"	3'3"	1'8"	3	1.96	Green
F	13'1"	3'3"	1'8"	4	2.62	Yellow
G	6'6"	3'3"	1'	2	0.78	Blue/red
H	9'9"	3'3"	1'	3	1.18	Blue/yellow
I	13'1"	3'3"	1'	4	1.57	Blue/green

49. The basic designs of gabions are fairly simple and Fig 34-37 show common examples. It should be noted that although Fig 34 to 37 show wall heights of only 18 ft, gabion walls in Italy have been built up to 81 ft high and 36 ft thick (25) and have performed satisfactorily.

50. Gabion wire baskets are generally fabricated with galvanized wire, but for extreme corrosive conditions, they are available in PVC (polyvinyl chloride) coated wire.

51. If gabion structures are used to retain clay slopes, counterforts are recommended as illustrated in Fig 38. They should be built as headers and extend from the front of the wall to a point at least one gabion length beyond the slip circle of the bank. These counterforts serve both as drains and as structural members of the wall. The recommended spacing of counterforts is shown in Table 12.

BUTTRESSES

52. Rock buttresses are relatively inexpensive to construct on a unit basis but are usually considerably larger and have relatively wider bases than conventional retaining structures and therefore require more space. Royster (23) has found that buttresses work very well in stabilizing colluvial slides, but points out that they should be constructed on residual or in-place soils rather than on colluvium.

Table 12: Spacing of gabion counterforts*

Type of soil	Water content (%)	Cohesion (psf)	Counterfort spacing (ft)
Very soft clay	40	300	13
Soft clay	35	400	16.5
Medium clay	33-30	600-800	20-23
Stiff clay	27-25	1000-1500	26-30

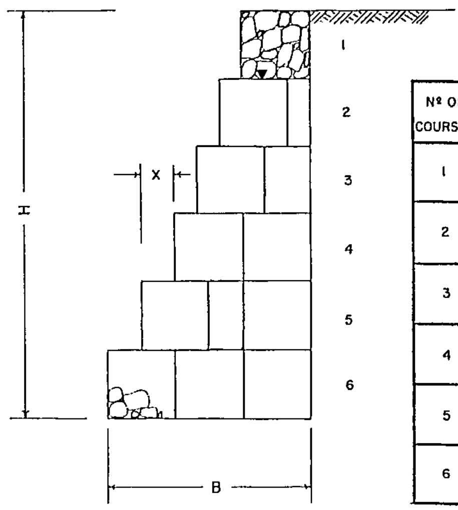
* After Reynolds and Protopapadakis (26)

53. The term "buttress" includes earth or rock dikes installed for either of two purposes (7):

a. to provide weight at the toe of a landslide, such as "toe support" or "strut" fills; and

b. to increase the shear strength of the soil by construction of a dike or buttress of material having substantially higher shear strength than the native soil.

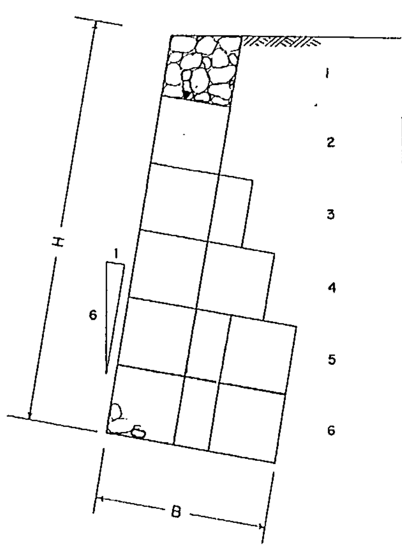
54. They are rarely used except at the toe of a landslide for the purpose of controlling the



"B" FRONT FACE STEPPED

Nº OF COURSES	H	B	X	REMARKS
1	3'-0"	3'-0"		
2	6'-0"	4'-0"	12"	
3	9'-0"	5'-0"	12"	
4	12'-0"	6'-0"	12"	
5	15'-0"	7'-6"	12"	USE COUNTERFORTS @ 9'-0" IN COURSE 4
6	18'-0"	9'-0"	12"	

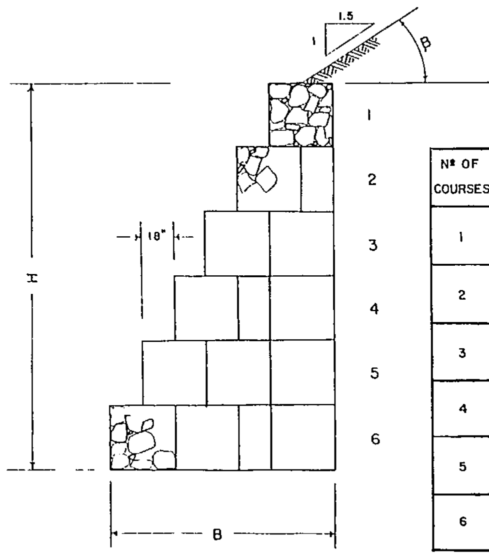
Fig 34 - Stepped face wall with horizontal backfill (22).



"A" FRONT FACE ON 1:6 BATTER

Nº OF COURSES	H	B	REMARKS
1	3'-0"	3'-0"	FRONT FACE MAY BE VERTICAL
2	6'-0"	3'-0"	
3	9'-0"	4'-6"	
4	12'-0"	6'-0"	
5	15'-0"	7'-6"	
6	18'-0"	7'-6"	

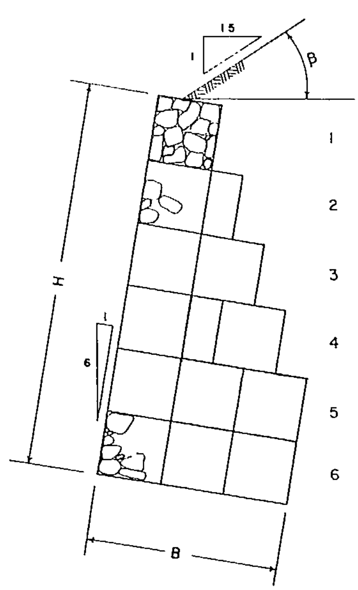
Fig 35 - Smooth front face wall with horizontal backfall (22).



FRONT FACE STEPPED

Nº OF COURSES	H	B	REMARKS
1	3'-0"	3'-0"	
2	6'-0"	4'-6"	
3	9'-0"	6'-0"	
4	12'-0"	7'-6"	
5	15'-0"	9'-0"	
6	18'-0"	10'-6"	

Fig 36 - Stepped front face wall with sloping backfill (22).



FRONT FACE ON 1:6 BATTER

Nº OF COURSES	H	B	REMARKS
1	3'-0"	3'-0"	FRONT FACE MAY BE VERTICAL
2	6'-0"	4'-6"	
3	9'-0"	6'-0"	
4	12'-0"	7'-6"	
5	15'-0"	9'-0"	
6	18'-0"	9'-0"	

Fig 37 - Smooth front face wall with sloping backfill (22).

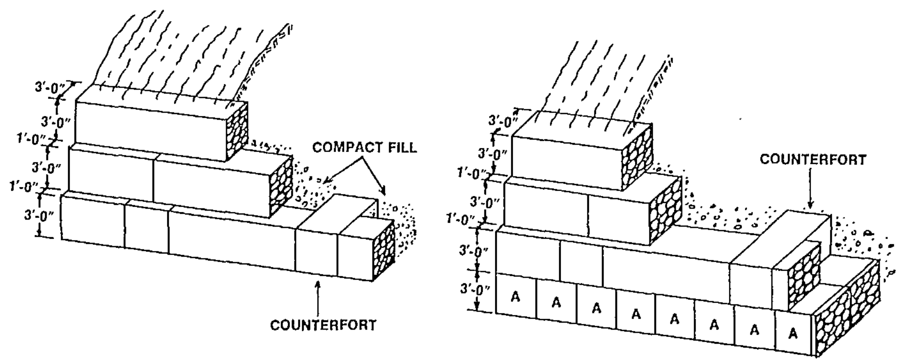


Fig 38 - Gabion retaining walls showing counterforts (24).

slide itself. Ladd (27) points out that where a wall must be built and the base rock is poor but variable in strength, a good solution is the angled underground buttress, extending forward and down as nearly parallel to the angle of thrust as possible.

55. The easiest way to design a buttress is to make an empirical estimate of the required size and then check the stability calculations to see if it is adequate. Royster (28) recommends as a rule of thumb, that the mass of the buttress should be about 1/4 to 1/5 the mass of the slide mass. Empirical relationships for sizing buttresses and other retaining structures have been summarized by Eckel (7) as shown in Table 13.

56. Once the first estimate of a buttress design is chosen, it must be checked for stability. Instability may develop in one of three ways (Fig 39):

a. shear through the buttress, along line HE or HJ, or any other critical slip surface,
b. foundation failure beneath the buttress, along arc FG, and
c. shear between the buttress and the foundation, along line CD.

57. In the stability analysis the resisting forces are compared with the driving forces to arrive at a factor of safety. Rock buttresses are more stable if placed on a rock foundation as foundation failures through rock are unlikely; this, however, is not always possible or practical.

58. Normally, it is easier to determine the size of a buttress on a preliminary basis by checking stability against a shear instability at the base of the buttress (line JH). A detailed analysis of buttress stability is given by Eckel (7).

MASONRY WALLS

59. Masonry walls may be practical in some locations. These are essentially gravity structures and may be designed according to basic soil mechanics practice as outlined in Fig 3. After a wall design is chosen, it should be checked for stability against internal shearing and foundation failure as described in the section on crib walls (Fig 32).

60. Masonry walls are structurally not very rigid and so accept small differential settlement with minimal structural damage. However, since they have no restraining members such as cribbing or wire baskets, excessive movements may destroy their structural integrity.

61. Basic designs for vertical and sloping front face masonry walls as used by the U.S. Dept of Agriculture, Soil Conservation Service (SCS) are shown in Fig 40 and 41.

Table 13: Empirical relations between various factors in the use of restraining devices to control active slides*

Type of treatment	Effect of quantity of moving mass	Effect of foundation conditions
1. Buttress at foot		
(a) Rockfill	Buttress should be 1/4 to 1/3 the volume of total moving mass to be retained.	Should extend at least 5 to 10 ft below slip-plane unless stable bedrock is encountered.
(b) Earthfill	Recompacted fill should be 1/3 to 1/2 that of total moving mass to be retained.	
2. Crib or retaining wall	Volume of crib should be 1/15 to 1/10 that of total moving mass to be retained.	Stable bedrock preferred. Otherwise, foundation should extend 4 to 7 ft below slip-plane.

* Subject to evaluation and experience in given locality.

Notes: 1. Relative stability: With the exception of rock buttresses, restraining structures are not recommended for controlling very unstable masses at the toe. Near the top of the landslide, piling, cribs, retaining walls, and tie-rodding of slopes can be used successfully.

2. In general, restraining structures are not recommended for falls or flows except as underpinning. If drainage is also provided, a restraining device may be helpful if area is permitted to drain before retainer is built.

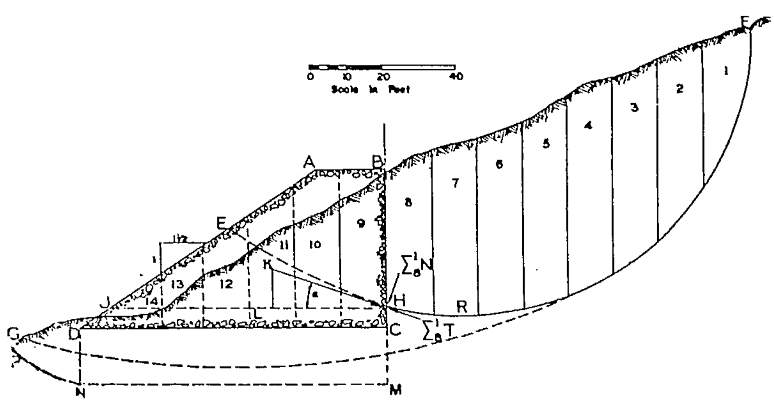
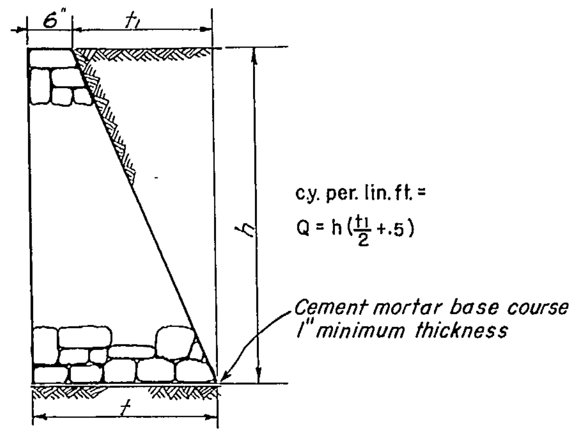


Fig 39 - Rock buttress design criteria (7).



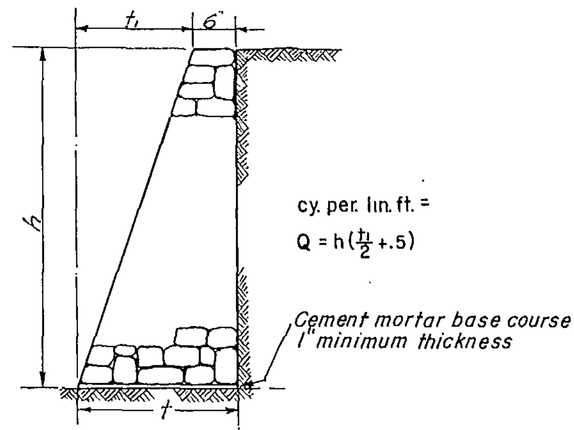
h in feet	P= 20		P= 33		P=50		P=62.5		P=80	
	t	cu. yds. lin. ft.	t	cu. yds. lin. ft.	t	cu. yds lin. ft.	t	cu. yds. lin. ft.	t	cu. yds. lin. ft.
2	0'-11"	.051	1'-2"	.061	1'-5"	.071	1'-7"	.077	1'-9"	.085
3	1'-4"	.102	1'-9"	.123	2'-1"	.145	2'-4"	.160	2'-8"	.176
4	1'-9"	.170	2'-4"	.207	2'-10"	.245	3'-2"	.271	3'-7"	.302
5	2'-3"	.255	2'-10"	.311	3'-6"	.374	3'-11"	.410	4'-6"	.459
6	2'-8"	.355	3'-5"	.437	4'-3"	.520	4'-9"	.582	5'-4"	.653
7	3'-2"	.474	4'-0"	.585	4'-11"	.707	5'-6"	.782	6'-3"	.876
8	3'-7"	.607	4'-7"	.755	5'-8"	.912	6'-4"	1.01	7'-2"	1.13
9	4'-1"	.760	5'-2"	.943	6'-4"	1.14	7'-1"	1.26	8'-1"	1.43
10	4'-6"	.925	5'-9"	1.15	7'-1"	1.40	7'-11"	1.55	8'-11"	1.75
11	4'-11"	1.11	6'-4"	1.39	7'-9"	1.68	8'-8"	1.87	9'-10"	2.10
12	5'-5"	1.31	6'-10"	1.64	8'-6"	2.00	9'-6"	2.22	10'-9"	2.50
13	5'-10"	1.53	7'-6"	1.91	9'-2"	2.33	10'-3"	2.59	11'-8"	2.92
14	6'-4"	1.76	8'-1"	2.21	9'-11"	2.69	11'-1"	3.00	12'-6"	3.37
15	6'-9"	2.01	8'-7"	2.53	10'-7"	3.08	11'-10"	3.42	13'-5"	3.86
16	7'-2"	2.28	9'-2"	2.87	11'-4"	3.50	12'-8"	3.89	14'-4"	4.39
17	7'-8"	2.56	9'-9"	3.23	12'-0"	3.93	13'-5"	4.38	15'-3"	4.94
18	8'-1"	2.86	10'-4"	3.61	12'-9"	4.40	14'-3"	4.90	16'-1"	5.53
19	8'-7"	3.18	10'-11"	4.01	13'-5"	4.90	15'-0"	5.45	17'-10"	6.15
20	9'-0"	3.52	11'-6"	4.44	14'-2"	5.42	15-9"	6.03	17'-11"	6.81

DESIGN ASSUMPTIONS

Wt. of masonry = 150 lbs./cu.ft.
Earth pressure: Horz = P lbs/cu.ft.
Vert = 100 lbs/cu.ft.
Resultant pressure passes thru base at 1/3 t from toe.
Unyielding foundation assumed

Where yielding foundation may exist, an extended reinforced concrete footing slab to be provided to allow resultant to fall in center of slab

Fig 40 - Design assumptions for vertical face masonry retaining walls.



h in feet	P=20 t	P=20 cu.yds. lin.ft.	P=33 t	P=33 cu.yds. lin.ft.	P=50 t	P=50 cu.yds. lin.ft.	P=62.5 t	P=62.5 cu.yds. lin.ft.	P=80 t	P=80 cu.yds. lin.ft.
2	0'-9"	.046	0'-11"	.053	1'-2"	.061	1'-4"	.066	1'-6"	.072
3	1'-1"	.088	1'-5"	.106	1'-9"	.124	1'-11"	.136	2'-2"	.150
4	1'-6"	.148	1'-10"	.176	2'-4"	.208	2'-7"	.214	2'-11"	.253
5	1'-10"	.216	2'-4"	.264	2'-11"	.314	3'-3"	.348	3'-8"	.380
6	2'-3"	.302	2'-10"	.368	3'-6"	.444	3'-11"	.480	4'-5"	.542
7	2'-7"	.401	3'-3"	.492	4'-1"	.591	4'-7"	.655	5'-1"	.725
8	3'-0"	.512	3'-9"	.631	4'-8"	.761	5'-2"	.859	5'-10"	.939
9	3'-4"	.640	4'-3"	.790	5'-3"	.953	5'-10"	1.06	6'-7"	1.18
10	3'-8"	.777	4'-8"	.962	5'-10"	1.16	6'-6"	1.29	7'-4"	1.44
11	4'-1"	.932	5'-2"	1.15	6'-5"	1.40	7'-2"	1.56	8'-0"	1.74
12	4'-5"	1.09	5'-8"	1.43	7'-0"	1.65	7'-10"	1.84	8'-9"	2.05
13	4'-10"	1.28	6'-1"	1.59	7'-6"	1.93	8'-5"	2.15	9'-6"	2.40
14	5'-2"	1.47	6'-7"	1.83	8'-1"	2.23	9'-1"	2.48	10'-3"	2.77
15	5'-7"	1.68	7'-1"	2.10	8'-8"	2.55	9'-9"	2.85	10'-11"	3.18
16	5'-11"	1.90	7'-6"	2.37	9'-3"	2.89	10'-5"	3.22	11'-8"	3.60
17	6'-3"	2.14	8'-0"	2.67	9'-10"	3.26	11'-1"	3.63	12'-5"	4.06
18	6'-8"	2.38	8'-6"	2.98	10'-5"	3.64	11'-8"	4.06	13'-2"	4.54
19	7'-0"	2.65	9'-0"	3.25	11'-0"	4.05	12'-4"	4.52	13'-10"	5.05
20	7'-5"	2.92	9'-5"	3.67	11'-7"	4.48	13'-10"	5.00	14'-7"	5.59

DESIGN ASSUMPTIONS

Wt. of masonry = 150 lbs./cu.ft.
Earth pressure: Horz = P lbs/cu.ft.
Vert = 100 lbs/cu.ft.
Resultant pressure passes thru base at 1/3 t from toe.
Unyielding foundation assumed.

Where yielding foundation may exist, an extended reinforced concrete footing slab to be provided to allow resultant to fall in center of slab.

Fig 41 - Design assumptions for sloping face masonry retaining walls (15).

REINFORCED EARTH

62. Reinforced earth is a patented process for a construction material, formed by the association of soil with linear metallic reinforcement. It is capable of withstanding significant tensile stresses parallel to the reinforcement.

63. Reinforced earth structures offer several advantages:

a. ease and speed of construction;
b. they are relatively inexpensive, often 20-50% below alternatives,
c. they are strong and stable,
d. they have no practical height or width limitation,
e. they are particularly suitable for construction on poor foundation soils.

64. Reinforced earth structures are usually designed for particular conditions and, as a result, there are no "standard designs". They depend on two basic requirements:

a. there must be sufficient friction between the soil and the reinforcing strips to prevent slipping.
b. there must be sufficient density of reinforcing strips in the mass to prevent internal structural failure.

65. The first condition - sufficient friction - is satisfied by ensuring that the length of the galvanized steel or aluminum strips is adequate. A length of 0.8 to 1.2 times the height of the structure is satisfactory in most applications. The necessary density of reinforcing strips is determined by calculating stresses within the mass.

66. The reinforced earth theory assumes that the soil within the structure is in a state of limit equilibrium and that the horizontal earth pressures corresponding to this state are transferred to the reinforcements by friction. The computations of the active horizontal earth pressures take into account all imposed external loading, geostatic and hydrostatic stresses, as well as dynamic forces from seismic or moving loads. The stress distribution is calculated from the total effect of all loadings at any point. Theoretically, stresses on the reinforcements are a maximum near, but not at, the face of the structure and decrease to zero at the opposite end.

67. Thus, the facing panels do not sustain the full lateral earth pressure normally associated with retaining structures. Rather, only minor stresses are imposed by tension at the face of the strips. The panels prevent the loss of soil and provide a stress continuity at the face end of the structure. Facing panels commonly are either precast concrete in a variety of architectural designs and textures (Fig 42) or metal elliptical facing elements (Fig 43).

68. The theoretical assumptions concerning reinforced earth have been verified by extensive instrumentation of full-scale structures in service. For details of reinforced earth design, the reader is referred to Vidal et al., (31), Schlosser et al., (32), and to Reinforced Earth Co. in Toronto and Montreal (29). A schematic of a reinforced earth structure is shown in Fig 1.

69. Construction requires only standard equipment, simple construction techniques and readily available materials, including:

a. galvanized metal or aluminum reinforcing strips 2 to 3 in. wide and approximately 1/8 in. thick with length varying according to the heigth of the structure and the magnitude of external loading;
b. precast concrete interlocking panels, each about 5 ft square and 7 in. thick, light enough to be placed by a small crane, or light metal facing elements;
c. earth backfill.

70. Using a crew of four or five men, construction can proceed at rates of between 750 and 1000 square feet of wall surface per day.

71. The construction process is basically repetitive:

a. The first row of precast concrete panels is placed;
b. the reinforcing strips are laid out horizontally and are bolted to the facing panels;
c. earth is backfilled and compacted as in ordinary earth embankment construction;
d. another row of facing panels is set into place, more strips are attached, and the backfilling-compacting process is repeated.

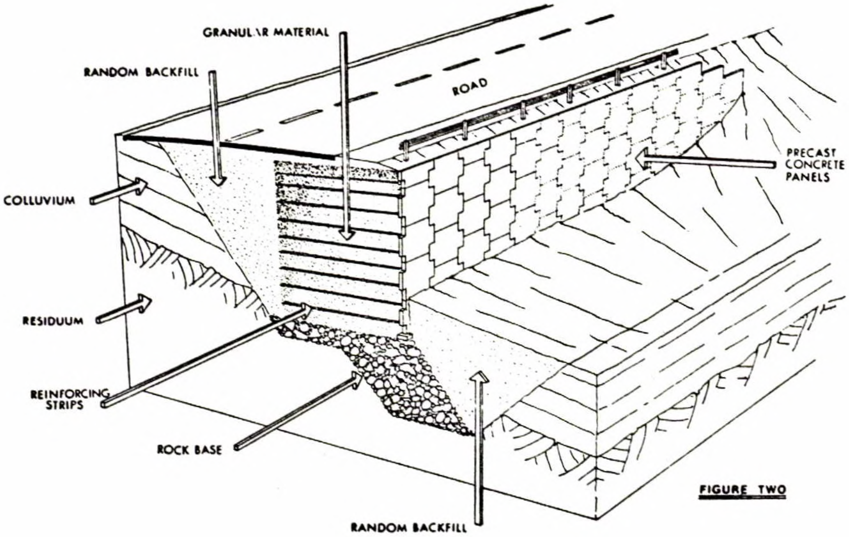


Fig 42 - Reinforced earth using pre-cast concrete panels (29).

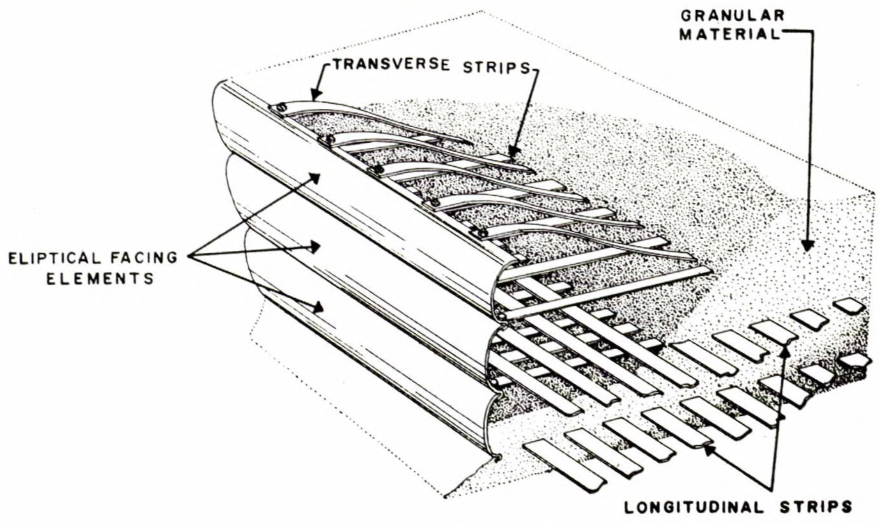


Fig 43 - Reinforced earth using elliptical metal facing elements (30).

72. The reinforced earth structure is complete and stable each time a layer of backfill is spread. Trucks, bulldozers and compactors, can drive on top of the wall while workmen fit additional facing panels and reinforcing strips into place.

DRAINS AND FILTERS

73. Any retaining structure which is not designed to withstand the full hydrostatic pressure of submerged soil should be provided with adequate drainage facilities to ensure that the backfill soil will not become saturated and submerged. Common types of retaining wall drainage are shown in Fig 44. Their application to cribs, gabions, and buttresses can be separated into two different cases.

74. One is that of open face crib structures with a fine-grained or impervious bin fill material, closed crib structures, or mansonry-rubble retaining walls that serve as a barrier to efficient water migration. These structures require an adequate drainage system as part of the wall and backfill design.

75. Drainage may be provided by various means. Weep holes 4 to 6 in. in diameter should be placed in the wall at intervals along its length and at the lowest elevation at which free outlet drainage can be maintained. The effectiveness of the weep holes is greatly enhanced by placing a vertical layer of crushed rock or coarse gravel about 9 to 12 in. thick directly behind the wall. If the backfill soil is fine-grained, a graded filter (filter criteria will be discussed later) should be placed between the soil and the rock to prevent clogging of the drainage system by migration of the soil fines into the coarse layer. Where it is not practicable to install weep holes through a wall, or if there is a question of whether they will be adequately maintained throughout the life of the structure, a longitudinal drain may be placed behind the wall at an elevation a little above the footing. Water collected by this drain may be carried to outlets at the ends of the wall or it may be discharged through headers extending through the wall at intervals of several hundred

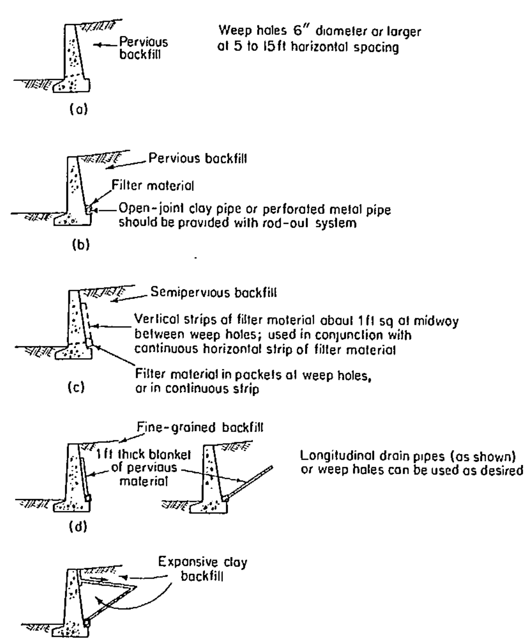


Fig 44 - Common types of retaining wall drainage: a) weep holes; b) longitudinal drain pipe; c) weep holes with filter strips; d) blanket drain; e) double blanket drain (33).

feet in the case of a very long wall.

76. The second drainage case is that of rockfill buttresses, gabions, dry rubble masonry walls, or open crib structures filled with gravel or rockfill with fine-grained backfill. These structures have adequate natural drainage but require a filter layer between them and the backfill to prevent clogging of the drainage system by migration of fines into the coarse layers. Royster (23) has included drain tiles in conjunction with free draining gabions and rock buttresses (Fig 1).

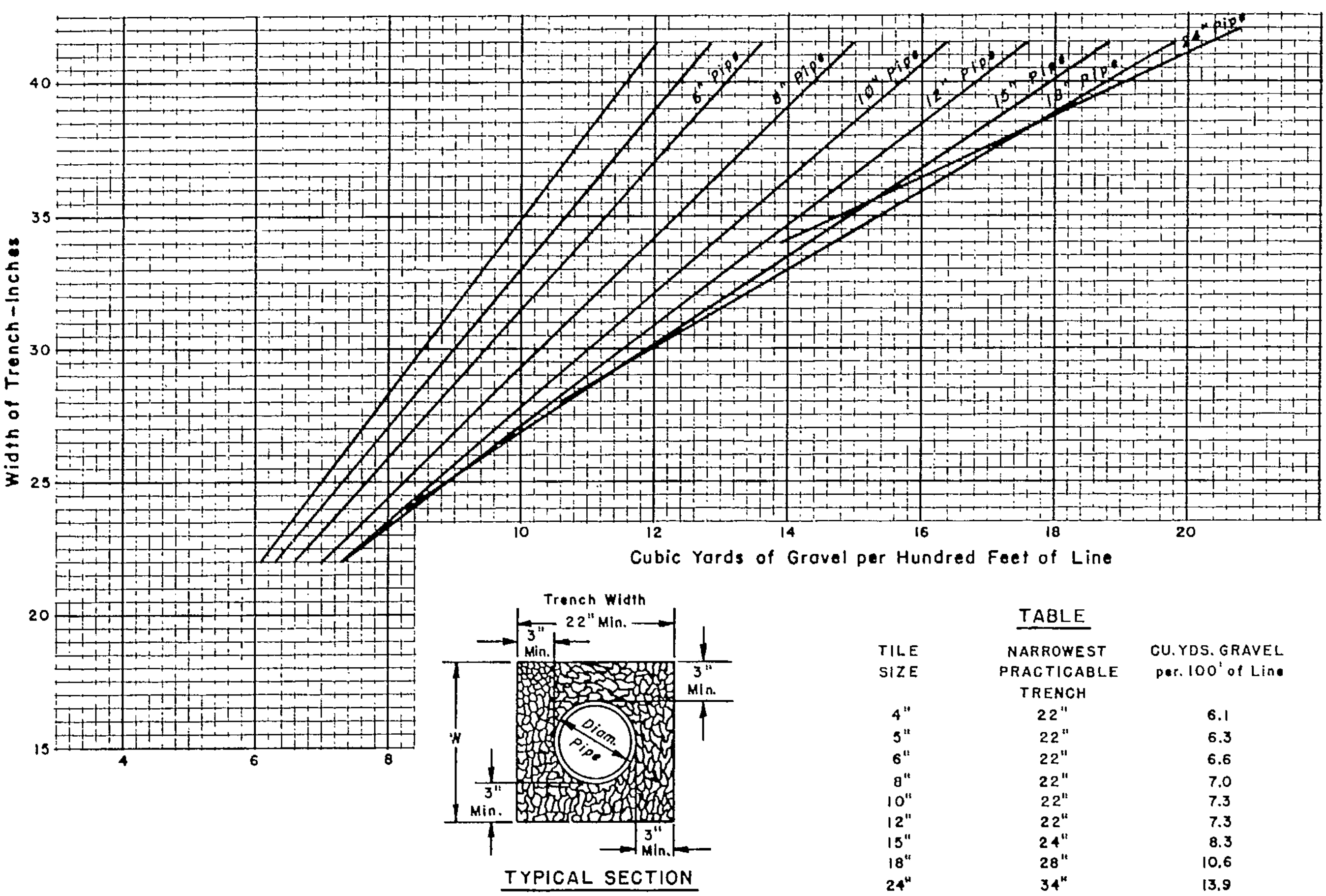
77. Filters must be sufficiently fine-grained to hold erodable material in place, but must at the same time be sufficiently coarse-grained to discharge all the water that reaches them. The accepted criteria for filter design suggested by Terzaghi is the following:

$$\frac{D_{15}\text{ (of filter)}}{D_{85}\text{ (of soil)}} < 4 \text{ to } 5 < \frac{D_{15}\text{ (of filter)}}{D_{15}\text{ (of soil)}} \qquad (2)$$

where D_{15} and D_{85} are particle sizes not exceeded by 15 and 85 wt % of the material respectively.

78. The left half of the equation satisfies the piping criterion and the right half the permeability criterion (34).

79. The most effective filter material is a well-graded gravel that meets the above criteria. This type may be used in drainage blankets (Fig 44(a) and (e)) or as backfill around drain tiles (Fig 44(b), (d), and (e)). Figure 45 is a chart for estimating gravel filter quantities for drain tiles.



TABLE

TILE SIZE	NARROWEST PRACTICABLE TRENCH	CU. YDS. GRAVEL per 100' of Line
4"	22"	6.1
5"	22"	6.3
6"	22"	6.6
8"	22"	7.0
10"	22"	7.3
12"	22"	7.3
15"	24"	8.3
18"	28"	10.6
24"	34"	13.9

Fig 45 - Chart for estimating gravel filter quantities (35).

CONSTRUCTION SPECIFICATIONS

80. Building specifications for the various retaining structures have been adapted from the following sources,

AASHO, (36)
AITC, (37)
AREA, (38)
ARMCO, (18,19)
AWPI, (13)
CHD, (39)
Kaiser, (20,21)
Maccaferri, (22)
NLMA, (40)
PCA Publication ST46, (17)
Royster, (23)
Schuster, (10)
USDOT, (41)
WSDH, (14)

These specifications may be used directly when specifying work to be done by outside contractors. All structural material and construction procedures should conform to the National Building Code of Canada or to appropriate provincial or municipal regulations.

CRIB WALLS

81. Crib structures may be made of cut timber, rough hewn logs, precast concrete blocks, or metal. Material specifications for each are outlined below. All crib type walls should be founded on stable ground (either natural or compacted) or on a bedrock foundation. Seelye (42) recommends that, because of the flexibility of crib walls, the foundations of "moderately high" walls need not extend below the frost line whereas "high" crib walls should be carried below the frost line. Where foundations are poor or inadequate, a concrete pad or footing should be constructed beneath the wall to reduce the contact pressure by spreading the load.

Cut Timber

82. Cut timber components of timber crib walls should be pressure treated with preservative. Suitable preservatives include creosote, penta chlorophenol in heavy oil, pentachlorophenol in mineral spirits or liquid petroleum gas, or water-borne salts. This type of treatment will protect against any type of rot and will ensure long service. Modifications in the field to any member that expose fresh wood surfaces, such as on-site cutting and drilling, should include the swabbing of the exposed surfaces with preservative as standard procedure.

83. All timber components should be of structural grade timber (Douglas fir or equivalent). All bolts should be galvanized. Bolt holes should be drilled within 1/16 in. of the required location and should be 1/16 in. larger in diameter than the bolt to prevent cracking due to stresses incurred by driven bolts.

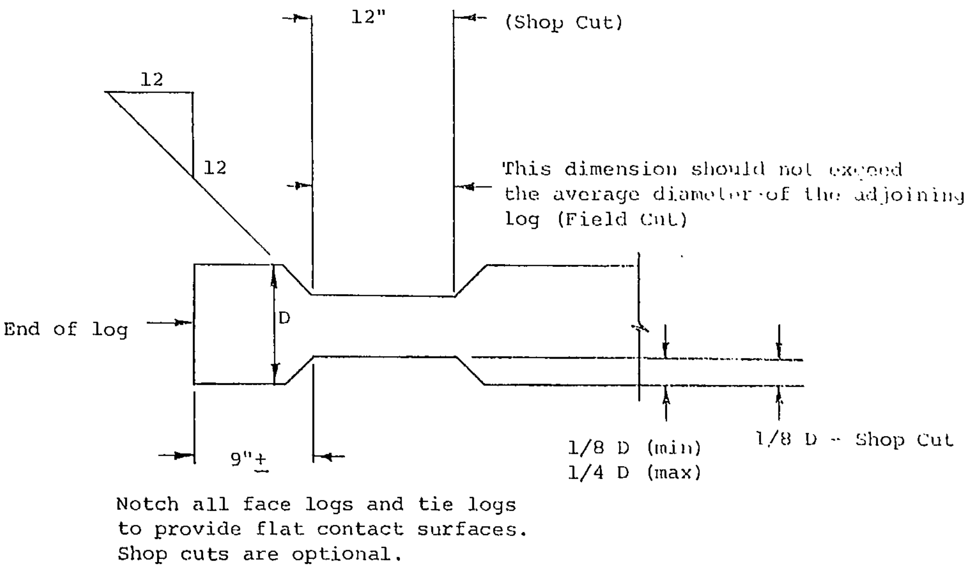
84. A washer should be used under all bolt heads and nuts which would otherwise come in contact with wood. Either cast or plate washers may be used and they should be designed to prevent excessive crushing of the wood when the bolts are tightened. For bolts or rods in tension, washers should be of sufficient size to develop the tension stress in the bolt or rod without exceeding the allowable unit stress in compression perpendicular to the grain for the species and grade of lumber used.

Logs

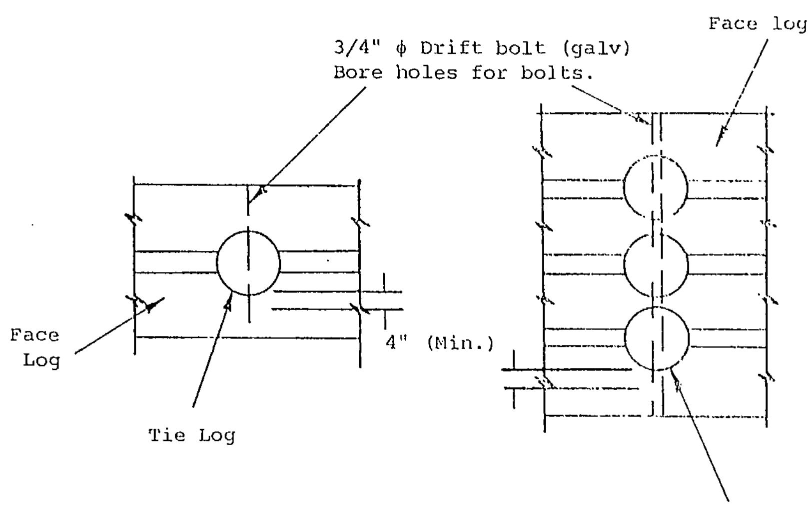
85. Logs used for structural members should be Douglas fir or equivalent and should be stripped of all bark and protruding branches at points of contact prior to use. This prevents deterioration of the bark and reduces natural internal adjustment.

86. Face logs should be not less than 16 ft long with as little taper as possible and should be no less than 10 in. in diameter at the small end. The base tier should be no less than 12 in. in diameter at the small end. Interior tie logs should have a diameter of no less than 8 in. at the small end.

87. Log connections may be either dapped and pinned with 3/4 in. diameter drift-bolts as shown in Fig 15 or they may be notched and pinned with 3/4 in. diameter drift-bolts as shown in Fig 46.



DETAIL FOR NOTCHING LOGS



DETAILS OF DRIFT BOLTING

Fig 46 - Joint details for log cribbing - U.S. Bureau of Public Roads (10).

The requirements for bolts, bolt holes and washers are the same as for cut timber in the previous section.

88. Each successive tier of logs or timbers should be drift-bolted to the one upon which it rests by drifts not less than 3/4 in. in diameter and of sufficient length to extend through 2 tiers and not less than 4 in. into the third tier.

89. Drift-bolts should be staggered and not more than 8 ft centre to centre in each tier. All end joints and splices should be half-lapped for 10 in. and drifted at the centre. Before assembling, all framed joints in contact should be well coated with preservative.

90. The length of ties should be sufficient to develop the required anchorage against overturning, and in no case should the length extending into the fill be less than two-thirds of the height of fill above the tie in question.

91. Ties should be anchored to the face walls by framing, either dovetailed or by sufficient projection behind the face of the crib to form proper anchorage. Ties should be anchored at the fill end to cross pieces fastened to them at right angles by drift-bolts or other suitable means.

92. Ties should be spaced not more than 8 ft centre to centre in any horizontal tier and should be staggered with the next adjacent tier of ties. Tiers of ties should be not more than 3 ft apart vertically.

Concrete

93. Concrete used in casting structural members of crib walls should have a minimum compressive strength of 3750 psi at 28 days. Each of the members in any arrangement, should bear at two points only, and the minimum percentage of reinforcement should be 0.9 per cent of the total cross sectional area.

94. Reinforcing steel should be protected from damage at all times. It should be free from dirt, detrimental scale, paint, oil, loose rust or other foreign substances.

95. All reinforcing bars should be bent cold unless otherwise permitted. Bars partially embedded in concrete should not be field bent except as shown on plans or permitted. Only competent men should be employed for cutting and bending, and proper equipment should be provided for such work. Should the engineer in charge approve the application of heat for field bending reinforcing bars, precautions should be taken to assure that the physical properties of the steel will not be materially altered.

96. Forms for concrete cribbing should be true to line and built of metal, plywood, or dressed lumber. A 3/4 in. chamfer strip should be used in all corners. Forms should be watertight and remain in place at least 24 hours after the concrete has been placed.

97. The concrete pour should be continuous and acceptable methods of vibration or compaction employed.

98. All members should be free from depressions, spalled, patched, or plastered surfaces or edges, or any other defect which may impair strength or durability. Cracked or otherwise defective members should be rejected.

99. Backfilling around cribbing should not be started until concrete test cylinders show a compressive strength of at least 80 per cent of the required 28 day compressive strength. In lieu of test cylinders to establish this, the concrete should be allowed to set for at least 14 days at a minimum temperature of 60° F or 21 days at a minimum temperature of 40° F.

100. Where used, connecting dowels should be of wrought iron or galvanized steel not less than 1 in. in diameter and of the required length. Casings for these dowels should be of galvanized steel or iron pipe not less than 1.25 in. inside diameter.

Steel

101. Structural members of steel bin-type crib walls should be not less than 16 gauge. All joints should be connected by flexible bolts or dowels of wrought iron or galvanized steel.

102. Metal sheets used to form the members of bin-type retaining walls (except grade plates and connecting channels) should be coated on both sides with a layer of asbestos fibres, applied in

a sheet form by pressing into a molten metallic bonding medium. Immediately after the metallic bond has solidified, the asbestos fibres should be thoroughly saturated with a bitmunious saturant. The finished sheets should be of first-class commercial quality, free from blisters and unsaturated spots.

103. The wall should consist of units that conform to the dimensions and thicknesses specified on the plans, and when assembled, should present a uniform workmanlike appearance. All units should be so fabricated that units of the same nominal size are fully interchangeable. No drilling, punching or drifting to correct defects in manufacture should be permitted. Any units having holes improperly punched should be replaced.

Aluminum

104. Aluminum alloy sheets should have a nominal cladding thickness on both sides of a 5% of the total composite thickness and should have the following mechanical properties:

	Thickness in.	
	0.051-0.113	0.114-0.249
Tensile strength		
Max (psi)	37,000	37,000
Min (psi)	31,000	31,000
Yield-Strength (0.2% Offset psi-Min)	24,000	24,000
Elongation % in 2 in. min.	4	5

Bolts and nuts for connecting the cells and facing sheets should be not less than 1/2 in. in diameter.

105. Aluminum bolt and nut material should have the following mechanical properties:

Diameter (in.)	0.125 to 8.000
Tensile strength (Min psi)	42,000
Yield-strength (0.2% Offset psi-Min)	35,000
Elongation % in 2 in.	10

106. Steel bolt and nut material should be hot, double-dipped galvanized, aluminized or cadmium-plated. Bolts should meet the following physical requirements:

	Material	
	High Strength Steel	Standard Carbon Steel
Tensile Strength (psi-Min.)	120,000	55,000
Shear (psi-Min.)	85,000	--
Brinell Hardness Number	241 to 302	Over 104

107. Aluminum sheet should have corrugations with a nominal pitch of 6 in. centre to centre of either crests or valleys. Depth of corrugations should nominally be 1 in.

108. Sheets should be curved to the bin diameter and should have a gross width of 25.5 in. and a variable gross length depending upon the geometry of the structure. Vertical joints in the sheets forming the walls of the cell should be drilled or punched with a double row of holes, 1/16 in. larger than the bolt diameter, on 3 in. centres to match the crests and valleys of the corrugations, the centre line of the outer row of holes should be not less than 1 in. from the ends of the sheet and the gauge between rows of holes should be not less than 1 3/4 in. The horizontal joints for all sheets should be drilled or punched with a single row of slotted holes spaced not less than 18 in. or greater than 21 in. Centre line of the row of holes should be centred on the crest or valley of a corrugation and not less than 3/4 in. from the edge of the sheet.

109. The facing sheets should be drilled or punched with a single row of holes, 1/16 in. larger than the bolt diameter, spaced on 3 in. centres to match the crests of the corrugations. The centre line of the row of holes should not be less than 7/8 in. from the end of the sheet.

110. Field drilling of holes to fasten facing sheets to cells is permissible. In this case, the punched facing sheet should be used as a pattern

118. Wire mesh should be non-raveling.

119. The selvedge on each sheet of mesh should be galvanized steel wire (as described above) two gauges heavier than used in the body of the mesh.

120. The wire mesh should have sufficient elasticity to permit elongation of the mesh equivalent to a minimum of 10 per cent of the length of the section of mesh under test without reducing the gauge or tensile strength of individual wire strands to values less than those for similar wire one gauge smaller.

121. Gabions should be supplied in the various sizes shown on the plans. Cages furnished by a manufacturer should be of uniform size.

122. All gabion dimensions should be subject to a tolerance limit of ± 3 per cent of manufacturer's stated sizes.

123. Rockfill for gabion baskets 12 in. or greater in thickness should have a size tolerance of 4-8 in. and should consist of a non-degradable rock such as limestone, quartzite, granite or broken concrete. Stone or broken concrete should have a minimum specific gravity of 2.25 and should be resistant to the action of air and water. Flaking or fragmental rock should not be permitted. Broken concrete may be used in the most convenient size for the intended purpose.

124. Gabions should be fabricated in such a manner that the sides, ends, lid, and diaphragms can be assembled at the construction site into rectangular units of the specified size. Gabions should be of single unit construction - the base, ends, and sides either to be woven into a single unit or one edge of these members connected to the base section of the unit in such a manner that strength and flexibility at the point of connection is at least equal to that of the mesh.

125. Gabions should be placed to conform with plan details. Riprap material should be placed in close contact so that maximum fill is obtained. The units may be filled by machine with only enough hand work to meet specification requirements.

126. Where the length of the gabion exceeds its width, the gabion should be equally divided, by diaphragms of the same mesh and gauge as the body and the holes should be drilled through the cell sheets after assembly.

111. The structures should be assembled following the manufacturer's shop drawings and assembly instructions. All bolts should be torqued to at least 25 ft-lbs. If a preset powered torque wrench is utilized, the contractor should be required to test only 1 per cent of the bolts at random for proper tightness.

112. Assembly procedures vary depending on the diameter of the cells. Generally it is expedient to: assemble one or more rings of a cell with facing sheets attached to one side only; rotate the cell until the facing sheets are in correct position on the adjacent previously placed cell; field drill the holes to attach the facing sheet to the previously placed cell; bolt the facing sheet fast; and proceed with the next cell in the same fashion. Field drilling of facing sheet attachment holes in one side of each cell permits alignment changes which may be necessitated by site conditions.

113. Filling of the cells or placing backfill behind the structure should not commence until the assembled portion of the structure has been duly inspected and approved.

114. All portions of the structure should be in true alignment before backfilling is started and care should be exercised to maintain reasonable alignment during backfilling operations.

GABIONS

115. Wire used in the body of the gabion mesh should be zinc coated, 11 gauge, and soft temper.

116. Tie and connecting wire should be supplied for securely fastening all edges of the gabions and diaphragms. Gabions should be provided with 4 cross connecting wires in each cell 1/2 unit high and 8 in each cell one unit high. Gabions should also have inner tie wires connecting the front face to the rear face at approximate spacing of 12 in. in both vertical and horizontal dimensions. Tie wire may be no more than 2 gauges lighter than gabion wire.

117. The longest dimension of the mesh openings should not exceed 4 in. for the gabions.

of the gabions, into cells whose length does not exceed the horizontal width. The gabion should be furnished with the necessary diaphragms secured in proper position on the base section in such a manner that no additional tying at this juncture will be necessary.

127. All perimeter edges of gabions should be securely selvedged or bound so that the joints formed by tying the selvedges have approximately the same strength as the body of the mesh.

128. Excavation for toe or cut-off walls should be made to the neat lines of the wall.

129. All gabion units should be tied together, each to its neighbor, along all contacting edges to form a continuous connecting structure.

BUTTRESSES

130. Buttresses should be constructed of large blocks of non-degradable rock such as limestone, quartzite, granite or broken concrete. If there is any doubt as to the suitability of the material, durability tests should be carried out to ensure that none of it flakes or fragments. All soil-like materials should be excluded from the fill.

131. The fill should have 50 per cent of the material greater than one cu ft in size and not more than 10 per cent passing the no. 2 mesh size. If these general specifications cannot be met, the size grading should ensure that buttress is free-draining.

MASONRY

132. Building masonry structures in the field as illustrated in Fig 40 and 41 may be done using available stone, either with a cement base binder (mortar rubble) or without a binder (dry rubble).

Mortar Rubble Masonry

133. Mortar rubble masonry includes the classes commonly known as coursed, random and random range work and should consist of roughly squared and dressed stone laid in cement mortar.

134. Stone for the masonry should be of approved quality, sound and durable, and free from segregations, seams, cracks, and other structural defects or imperfections tend to weaken its resistance to weather. It should be free from rounded, worn, or weathered surfaces. Any weathered stone should be rejected.

135. The stone should be free from dirt, oil, or any other substance which may prevent proper adhesion of the mortar.

136. Mortar for laying the stone should be composed of one part portland cement and three parts of mortar sand, unless otherwise specified.

137. Individual stones should have a thickness of not less than 8 in. and a width of not less than 1.5 times the thickness. No stones, except headers, should have a length less than 1.5 times their width. Stones should decrease in thickness from the bottom of the wall to the top.

138. Headers should hold in the heart of the wall the same size as in the face and should extend not less than 12 in. into the core or backing. They should occupy not less than 20% of the face area of the wall and should be evenly distributed. Headers in walls 2 ft or less thick should extend entirely through the wall.

139. The stones should be roughly squared on joints, beds, and faces. Selected stone, roughly squared and pitched to line, should be used at all angles and ends of walls. If specified, all corners or angles in exterior surfaces should be finished with a chisel draft.

140. All shaping or dressing of stone should be done before the stone is laid in the wall, and no dressing or hammering which will loosen the stone should be permitted after it is placed.

141. Stone masonry should not be constructed in freezing weather or when the stone is cold and shows signs of frost.

142. Masonry should be laid to line and in roughly level courses. The bottom or foundation courses should be composed of large, selected stones and all courses should be laid with bearing beds parallel to the natural bed of the material.

143. Each stone should be cleaned and thoroughly saturated with water before being set and the bed which is to receive it should be clean and well moistened. All stones should be well bedded in freshly made mortar. The mortar joints should

be full and the stones carefully settled in place before the mortar has set. No spalls should be permitted. Joints and beds should have an average thickness of not more than 1 in.

144. The vertical joints in each course should break with those in adjoining courses by at least 6 in. In no case should a vertical joint be so located as to occur directly above or below a header.

145. If any stone is moved or the joint broken, the stone should be taken up, the mortar thoroughly cleaned from bed and joints, and the stone reset in fresh mortar.

Dry Rubble Masonry

146. Dry rubble masonry includes the classes commonly known as coursed, random and random range work and should consist of roughly squared and dressed stone laid without mortar.

147. The stones should conform in quality and size to the requirements specified for masonry rubble.

148. Headers should conform to specifications for masonry rubble.

149. The stones should be roughly squared on joints, beds, and faces. Selected stone, roughly squared and pitched to line, should be used at all angles and ends of walls.

150. The masonry should be laid to line and in roughly level courses. The bottom or foundation courses should be composed of large, selected stones and all courses should be laid with bearing beds parallel to the natural bed of the material. Face joints shall not exceed 1 in. in width.

151. In laying dry rubble masonry, care should be taken that each stone takes a firm bearing on the underlying course at not less than three separate points. Open joints, both front and rear, should be "chinked" with spalls fitted to on their top and bottom surfaces, to secure firm bearing throughout the length of the stone.

152. When required, the open joints on the rear surfaces of abutments or retaining walls should be "slushed" thoroughly with mortar to prevent seepage of water through the joints.

REINFORCED EARTH STRUCTURES

Concrete Face Panels

153. Concrete should have a minimum compressive strength at 28 days of 4500 psi. Air entraining, retarding or accelerating agents or any additive containing chloride should not be used.

154. Tie strips, connecting pins, and PVC pin form and lifting and handling devices should be set in place prior to casting to the dimensions and tolerances shown on the plans.

155. Acceptability of the precast units should be on the basis of compression tests and visual inspection. The precast units should be considered acceptable regardless of curing age when compression test results indicate strength will conform to 28-day specifications. Panels may be considered acceptable for placing in the wall when 7-day strengths exceed 60 per cent of 28-day requirements.

156. The panels should be cast on a flat area, the front face of the form at the bottom, the back face on top. Tie strip guides should be set on the rear face. The concrete in each unit should be placed without interruption and should be consolidated by an approved vibrator, supplemented by any necessary hand-tamping to force concrete into the corners of the forms and prevent the formation of stone pockets or cleavage planes. Clear form oil of the same manufacture should be used throughout the casting operation.

157. The units should be cured for a sufficient length of time so that the concrete will develop the specified compressive strength. Any panel pour which does not reach specified strength within 28 days should be rejected.

158. The forms should remain in place until they can be removed without damage to the unit.

159. The rear face should have an unformed finish, and should be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of 1/4 in.

160. All units should be manufactured to the following tolerances:

all dimensions within 3/16 in.,

angular distortion with regard to the

height of the panel less than 0.2 in. in 5 ft, and

defects on formed surfaces, not more than 0.1 in., measured on a length of 5 in.

161. Compression tests to determine the minimum strength requirement should be made on cylinders. A minimum of three cylinders should be made from each day's production and cured in the same manner as precast units.

162. Units should be rejected if they do not meet all the above requirements. In addition they should be rejected if there are signs of imperfect mouldings, or honeycombed or open texture concrete.

163. The manufacturing date should be clearly scribed on the rear face of each panel.

164. All units should be handled, stored, and shipped in such a manner as to eliminate the danger of their being chipped, cracked or fractured and of excessive bending stresses. Panels being stored should be supported on firm blocking immediately adjacent to tie strips to avoid bending the tie strips.

165. Concrete for footings should have a minimum compressive strength of 3750 psi at 28 days.

Steel Face Panels

166. Steel face panels should be fabricated of cold rolled galvanized steel.

Reinforcing and Tie Strips

167. Reinforcing and tie strips should be shop fabricated of galvanized steel. They should be cut to the lengths and tolerances shown on the plans. The minimum bending radius of tie strips should be 1 in. All reinforcing and tie strips should be true to size and free from defects that may impair their strength or durability.

Fasteners

168. Bolts and nuts should be acceptable grade hexagonal cap screws and should be a nominal 1/2 in. by 1 in. size with a 3/4 in. thread length, hot dip galvanized.

Joint Filler

169. Filler for vertical joints between concrete face panels should be flexible open cell 2 in. x 2 in. polyethylene foam strips. Filler for horizontal joints between panels should be resin-bonded cork.

Joint Covers

170. Joint covers for steel face panels should be fabricated of cold rolled galvanized steel.

Select Granular Backfill Material

171. All backfill material in the structure should be free from organic or other deleterious material and should conform to the following gradation limits:

Sieve size	Per cent passing
10 in.	100
4 in.	100 - 75
No. 200	0 - 15

172. This material should have an angle of internal friction of not less than 25° as determined by standard triaxial or direct shear testing methods.

Construction Requirements

173. Excavations should be in reasonably close conformity with the limits and construction stages shown on the plans. The foundation for the structure should be level for a width equal to or exceeding the length of reinforcing strips or as shown on the plans. Prior to wall construction, the foundation should be compacted with a smooth wheel vibratory roller having a minimum weight of 6 tons.

174. An unreinforced concrete leveling footing should be provided at each panel foundation level when concrete face panels are specified. The footing should be cured a minimum of 12 hours before placing wall panels.

175. When erecting walls, precast concrete panels should be placed vertically with the aid of

a light crane and lifting beam. Panels are handled by eyes set into the upper edge of the panels. Panels should be placed in successive horizontal lifts in the sequence shown on the plans as backfill placement proceeds. When a panel is having fill placed behind it, it should be maintained in a vertical position by means of temporary wooden wedges at the junction of two adjacent panels on the external side of the wall. External bracing may also be required for the initial lift. Vertical tolerances and alignment should be 0.25 in. in 10 ft.

176. Skin elements are hand placed in successive horizontal lifts as indicated on the plans. Backfill should be maintained no more than two skin elements below the top of wall. Wooden wedges should be placed at no more than 5 ft centres between the skins on the exposed face to maintain verticality during backfilling. Batter boards should be used to maintain verticality for the first five lifts. All wedges should be removed as backfilling proceeds. A minimum of five lifts should at all times be supported by wedges.

177. Placing backfill should closely follow the erection of each panel lift. Backfill should be levelled roughly before placing and bolting strips. Reinforcing strips should be placed normal to the face of the wall. The maximum lift thickness should not exceed 10 in. (loose) and should follow panel erection closely. Lift thickness should be decreased if necessary to obtain specified density. Backfill compaction should not disturb or distort the reinforcing strips and panels.

BACKFILLING

Closed Face Structures

178. Backfill within the bins and behind the bins of closed face structures may be either granular and free-draining or cohesive and non-free-draining.

179. For backfilling with coarse broken rock of a specified size, backfilling may be done by machine with sufficient hand work to assure close contact with the structure and to minimize voids. Care should be taken, especially with concrete cribs, not to subject them to heavy impact and, with timber or log cribs, care should be taken to avoid distortion of members.

180. For soil-like backfill, the soil should be placed in 6-8 in. thick loose, even, horizontal lifts and tamped or compacted to approximately 95 per cent of the maximum density obtained by the standard Proctor test (36). Again, care should be taken not to damage structural components. If any holes or small openings are present in the walls, they should be blocked off by caulking, taping, or placing coarser material against the holes to prevent loss of fines.

181. At locations where headers are not uniformly supported, the fill should be tamped and compacted under and between them.

182. Filling of the cribs may progress simultaneously with erection. Backfilling behind a crib may progress with, but never ahead of, filling of the crib.

Open Face Structures

183. Backfilling for open face structures is the same as the above procedure for closed face structures. However, if the size of the backfill material is smaller than the openings, a layer of rock or stone spalls of sufficient size should be placed against the cribbing in advance of backfilling to prevent loss through the openings. As before, care should be taken not to damage the structure.

184. It must be remembered, especially in crib structures, that the backfill is part of the structure, and the need for stronger, less compressible, higher density backfills is extremely important to minimize effects of environmental changes.

DRAINS AND FILTERS

185. Fill material behind all retaining walls should be effectively drained by weep holes, horizontal drains, drainage blankets, or combinations of the above.

186. Weep holes should be placed at suitable

intervals. In counterfort type walls and bin or cell type structures, there should be at least one weep hole per compartment.

COSTS

187. As an introduction to costs, Baker (1) has summarized relative costs of many different landslide control measures (Table 14). These costs are out-of-date, but may serve to point out that some expedient other than a retaining structure might be more economical in some cases.

188. Because of the many variables involved - volume of excavation, quantity of backfill, haul distance, cost and availability of material, labour costs and associated maintenance costs for different types of structures - the following costs are only a rough guide.

Timber Cribs

189. Costs for constructing timber cribs vary considerably depending on the location of the work and on the availability of suitable timber.

190. Costs incurred in the Pacific Northwest portion of the U.S. for timber crib walls reported by Schuster (10) are shown in Table 15.

191. As can be seen in the table, the cost of shipping Douglas fir from the coast to Minnesota increases the cost considerably. In addition, the cost of construction labor depends greatly on whether cribs members are drilled in the field or not. Field drilling can increase the cost significantly.

LOG CRIBS

192. No cost data is available for log cribbing. However, if the logs have been cut from on site available timber owned by the mine, the material costs would probably be relatively low. Labor costs including cutting, drilling, and assembly of the logs might be compared with field drilling operations in Minnesota in 1971 at $5.00 per square foot of wall.

CONCRETE CRIBS

193. The cost of concrete cribbing (10) was about $4.50 to $4.75 per sq ft of wall at the Helca Mining installation in 1960. Details of this particular installation are not known, but availability of aggregates, haul distance from a mixing plant and haul distance to the job site have a direct influence on costs.

STEEL CRIBS

194. The Helca Mining installation (10) would have cost about $4.50 to $4.75 per sq ft in 1960, if built of Armco standard steel bins.

195. Armco has estimated average costs for their standard steel bin structures in the U.S. (43) as shown in Table 16. It should be emphasized that these are average costs for a particular design. Higher or lower wall heights for any particular design would result in higher or lower costs than the values given.

ALUMINUM CRIBS

196. Schuster (10) reported that Kaiser Aluminum cellular walls constructed in 1968-1970 cost between $15.00 to $17.00 per sq ft for walls up to 15 ft high in the Angeles National Forest, California. The cost of this type of wall was compared with concrete crib for use on a 17 ft high wall at Boise, Idaho, with the following results:

a. concrete crib, $16.00 per sq ft of wall face,
b. cellular aluminum, $10.50 per sq ft of wall face.

197. Other information indicates that these cellular aluminum walls cost from $4.00 to $10.00 per sq ft of face, depending on the amount of excavation involved, the availability of approved backfill material, and the actual diameter, gauge, and height of the cells.

198. Kaiser (44) reports costs for the material and wall assembly given in Tables 17 and 18.

GABIONS

198. Gabions structures have been reported by Schuster (10) to cost between $20 and $40 per cubic yard or $5.50 to $11.00 per sq ft of wall. The effect of haul distance for gabion stone can increase this cost considerably.

Table 14: 1950 unit costs for estimating expenditures of corrective measures

	Corrective measure	Item	Costs, in dollars	Unit
1.	Relocation	Excavation	0.75	cu yd
		Pavement	3.00	sq yd[a]
		Right of Way	Variable	
2.	Excavate, drain, and backfill	Excavation	3.50	cu yd[b]
		Drainage pipe	1.00	ft[c]
3.	Drainage	Drainage pipe	1.00	ft[c]
		Excavation	3.00	cu yd
		Porous backfill	2.50	cu yd[d]
		Jacked-in-place pipe	2.00	ft[e]
4.	Removal of material	Excavation	0.75	cu yd
		Right of way	Variable	
5.	Buttress at toe	Excavation	1.00	cu yd
		Backfill	3.00	cu yd
		Drainage pipe	1.00	ft
		Right of way	Variable	
6.	Bridging	Roadway surface	15.00	sq ft
7.	Cribbing	Face of cribbing	4.00	sq ft[f]
8.	Retaining wall	Face of wall	7.00	sq ft[f]
9.	Piling	Length of pile	5.00	ft[g]
10.	Sealing joint planes and open seams	Equipment rental	75.00	Day
		Drilling	3.00	ft
		Cement	4.00	bbl[d,h]
11.	Cementation of loose material	Equipment rental	75.00	Day
		Drilling	3.00	ft
		Cement	4.00	bbl[d,h]
12.	Chemical treatment-flocculation	Equipment rental	75.00	Day
		Drilling	3.00	ft[h]
		Admixture	Variable	
13.	Tie-rodding slopes	Length of pile	5.00	ft[g]
		Drilling	3.00	ft
		Steel	0.20	lb[d]
		Concrete	45.00	cu yd[d]
14.	Blasting	Drilling	3.00	ft
		Black powder	--	

Note

a - for flexible type pavement

b - earth moved twice

c - perforated pipe 6 in. in diameter

d - in place

e - pipe 6 in. in diameter, in place

f - in place, 8 ft high, gravity-type concrete

g - steel, in place

h - quantities difficult to estimate

Table 15: Timber crib wall costs

Type design	Year incurred	Location	Dollar costs per ft^2 of wall		
			Total	Material	Labor
AWPI	1972	Oregon	6.50	4.50	2.00
Perma Crib	1972	Oregon	5.00	3.50	1.50
U.S. Forest Service	1971	Oregon	7.50	4.50	3.00
Hecla Mining - Custom	1960	Idaho	2.25	--	--
AWPI	1971	Minnesota	13.00	8.00	5.00
U.S. Forest Service	1960's	Utah	7.00	--	--

Table 16: Costs of Armco bin structures

Wall type	Design type	Height (ft)	Canadian dollar cost*/ ft^2 of wall		
			Total	Materials	Labor
I	A	8'0"	9.60	6.50	1.80
I	B	13'4"	11.05	7.40	2.10
I	C	18'8"	12.90	8.60	2.50
I	D	22'8"	15.40	10.25	3.40
I	E	26'8"	17.25	11.30	3.50
II	A	8'4"	6.00	4.00	N/A
II	B	13'8"	6.90	4.60	N/A
II	C	17'8"	8.10	5.40	N/A
II	D	20'4"	9.75	6.50	N/A

* FOB site; does not include cost of backfilling.

Table 17: Material cost vs material thickness*

Gauge of metal	16	14	12	10	8
Metal thickness - in.	.060	.075	.105	.135	.164
Cost - U.S. \$/ft^2 of wall	4.50	5.40	7.20	9.00	10.70

Table 18: Assembly costs vs cell diameter*

Wall type	I	II	III	IV	V	VI	VII	VIII	IX
Cell diameter - ft	6.2	7.7	8.7	10.2	11.3	12.8	13.8	15.3	16.4
Assembly cost - U.S. $\$/ft^2$ of wall	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75

* Note that costs reported by Kaiser (44) do not include backfilling of the cells and are F.O.B., Spokane, Washington and hence do not include shipping and customs fee into Canada. The assembly costs reflect U.S. labor rates and in Canada may be different.

199. Maccaferri Gabions (25) estimate the 1975 cost of gabion structures in the U.S. to be about \$40 to \$45 per cu yd (\$11.00 to \$12.50 per sq ft of wall) in place; and about \$30 to \$35 per cu yd (\$8.25 to \$9.63 per sq ft of wall) in place in Canada (25, 45). For a \$40 per cu yd cost, about \$10 per cu yd is for the basket, about \$5 to \$10 per cu yd for the rock and about \$20 per cu yd for the labor and equipment to construct the wall. Maccaferri estimates that about 10 per cent can be saved in total labor costs if metric baskets are used rather than standard baskets since both require about the same amount of labor with the metric basket yielding more volume per unit of labor.

200. In mining applications, the cost of the rock could be reduced due to the immediate availability of broken or crushed rock. The only rock costs involved would be screening to ensure the proper sizes. In such a case the cost of a gabion structure would be reduced to approximately \$35 per cu yd or \$9.60 per sq ft of wall in the U.S. and \$25 per cu yd or \$6.85 per sq ft of wall in Canada.

201. If the haul or shipping distance of the gabion baskets is extremely long, the cost would increase accordingly. Therefore, availability ofthe gabion baskets could be a significant factor. Bekaert Gabions (46) have their only North American outlet in Reno, Nevada, and shipping costs to points in Canada east of Manitoba might be prohibitive. However, Maccaferri Gabions maintain outlets in Toronto, Montreal and Vancouver and could supply eastern as well as western Canada.

202. Costs of gabion structures reported in 1973 by Royster (23) were \$44 per cu yd (approximately \$12 per sq ft of wall).

BUTTRESSES

203. Limited cost data is available for constructing rock buttresses. Costs for such structures depend almost entirely on the excavation or placement of the rock since minimal appurtenant material is necessary.

204. Royster (23) reported in 1973 that buttress rock placement ranged from \$3 to \$6 per cu yd. Cost figures for 1976 from Royster (28) indicate buttress rock to vary from \$6 to \$12 per cu yd and costs to excavate slide material to be \$2 to \$6 per cu yd.

205. The above buttress rock costs do not differentiate between actual rock costs and transportation costs. In a mine, shot rock is generally already available so that the only actual cost involved would be for hauling to the desired location. If existing haul routes are close to the location of a planned buttress structure, the total rock cost could be quite minimal.

MASONRY WALLS

206. Limited cost data is available for mason-

ry-rubble or dry-rubble retaining walls. However, in mining, the rock would essentially be free with minimal haulage cost so that most of the cost involved would be for labor. Information from the Denver Stone Company (47) indicates that a good stone mason should be able to lay 100-150 sq ft of rock in one day. This footage is not square footage of wall face, but square footage of masonry, a single rock layer in thickness. As a result, the use of larger size stones would reduce the overall costs of a wall. If an average size of stone is used and the wall dimensions are known, a cost per square foot of wall could be calculated knowing the labor rates of stone masons at any particular location.

REINFORCED EARTH

207. Reinforced earth costs in Canada estimated by Gladstone (48) for a 10-20 ft high wall with sloping backfill would be about $13.00 per sq ft of wall for reinforced earth material and $1.50 to $2.00 per sq ft for erection (1976 figures). If the backfill is flat rather than sloped, the required reinforcement would be reduced and material costs would also be reduced accordingly. If the structure were erected by mine personnel rather than by an outside contractor, erection costs may be reduced 30 to 50 per cent. Also, larger walls afford lower unit costs for erection.

208. The reported cost of $10.00 per sq ft (1976) for material includes technical services from the Reinforced Earth Co., which covered the required design and engineering, construction drawings and specifications, and assistance prior to and during construction.

POST CONSTRUCTION MONITORING

209. It is good engineering practice to monitor the behavior and performance of retaining structures. If possible, it is advisable to install instrumentation during construction. In this way, if the wall shows instability during that period any inadequacies in the design will be evident before the entire expense of the completed wall has been committed. Ladd (27) points out cases in which walls have failed during construction; hence, there is a need to continuously monitor behaviour from the beginning.

TECHNIQUES

210. Schuster et al. (49) have monitored wall movement on timber cribs using inclinometers which measure the angular deviation from the original inclination of a tube rigidly attached to the structure at a particular orientation. This type of instrument is described in Chapter 8 - Monitoring of the manual.

211. The tube for the inclinometer is mounted on the structure at a convenient location. Readings of the inclinometer should initially be frequent; after construction is completed and the wall thus stabilized, they may be reduced to once a month or even once every second month. Readings should be more frequent after any drastic change in environmental conditions such as an increase in surcharge or after a heavy rainfall. Continual monitoring in such a fashion will give a continuous record of wall movement vs time and any sudden increase in movement may be an indication of possible instability.

212. Inclinometer readings may be used in different ways. They can be used to show variations with time of the position of points on the face of the wall relative to the base of the inclinometer tube, or to show the horizontal deflection of points on the face of the wall. Schuster (49) has pointed out that crib walls can change geometry drastically and still perform their overall design functions. Hence, any symptoms of impending instability must necessarily be carefully interpreted.

213. An alternative method of monitoring structural behavior would be to use conventional surveying techniques. However, the repeated surveys needed for such a monitoring program would probably be more expensive and less accurate than inclinometers.

214. Maccaferri Gabions (25) recommends for gabion walls that control pins be grouted into the gabion structure. The position of these pins should then be checked once a week for the first

month after construction, and once a month for the first year after construction. As before, it is necessary to distinguish between natural distortions and impending instability.

CASE HISTORIES IN MINING

215. Few case histories with any appreciable background information have been documented. There are several known cases of the use of retaining structures in open pit mining, but few details are available. One that is well documented is the Ruth Lake Mine of the Iron Ore Company of Canada where a crib structure was used, and another in South Africa where a post-tensioned buttress was used. Details of these follow.

RUTH LAKE MINE

216. The Ruth Lake Mine at Schefferville, Quebec was one of the first mines to go into production when the area was opened in 1954.

217. There was no prior knowledge regarding the operation of large, open pit mines in Ungava, nor was there much background information available with respect to pit design parameters such as optimum slope angles, climate, ground water conditions and detailed geological structure. As a result, mining progressed on a "design as you go" basis. Progress was maintained during the early 1960's in an attempt to analyze past slides in the area and to investigate strength parameters and to understand instability mechanisms.

218. By the summer of 1965, Ruth Lake Mine itself was beginning to show signs of significant instability. Essentially, the west wall of the mine, comprising Ruth Slate and underlain by Wishart Quartzite, began to move.

The Slide

219. On the west wall some 50,000 cubic yards of material slowly began to move. It should be noted that slides had occurred in the same general area over the previous several years, in some cases involving much larger quantities of material. Also of interest is that the slope angle prior to sliding was never greater than 50°, and was often as low as 32°. The present-day procedure of peripheral dewatering of open pits was only in the design stage in 1965 and the input dewatering system then in use at Ruth Lake was probably inadequate to reduce the adverse effects of the high natural water-table. Previous slides had been handled by digging them out, unloading the crest, or in some cases loading the toe where pit geometry and operating procedures permitted. The unique feature of the 1965 slide was that it threatened the main haul road into the pit, and no alternative access was available.

Remedy

220. It was decided that the most amenable technique available, commensurate with maintaining the haul road past the toe of the active slide area, was to load the toe. The required mode of loading did not allow for a wide base because of the haul road. Accordingly, a fabricated retaining structure was envisaged, allowing a heavy load to be applied over a narrow base area.

221. Design drawings for a crib wall were completed by October 1965 (Fig 47) and the main structure, almost 300 ft in length along the main haul-road, was constructed during the following winter. Figure 48 shows construction under way. Note that the toe of the slide had been excavated to make way for the crib.

222. By spring of 1966 the job was completed. Figure 49 shows the crib as it was in June 1966, with the haul road passing in front of it from left to right and then doubling back as a new sinking cut was made to the left of the photograph. Essentially, at this time the crib was isolated on a berm which was in fact the haul road.

223. The survey monitoring of the crib began early in 1966 at two-weekly intervals. By June, it had established that movement was taking place at the crib. Differential settlement of the crib resulted in failure of certain crib members. However, even though some members had failed individually, the structure as a whole continued to serve its intended purpose. Figure 50, taken on June 14, 1966, with backfilling still in

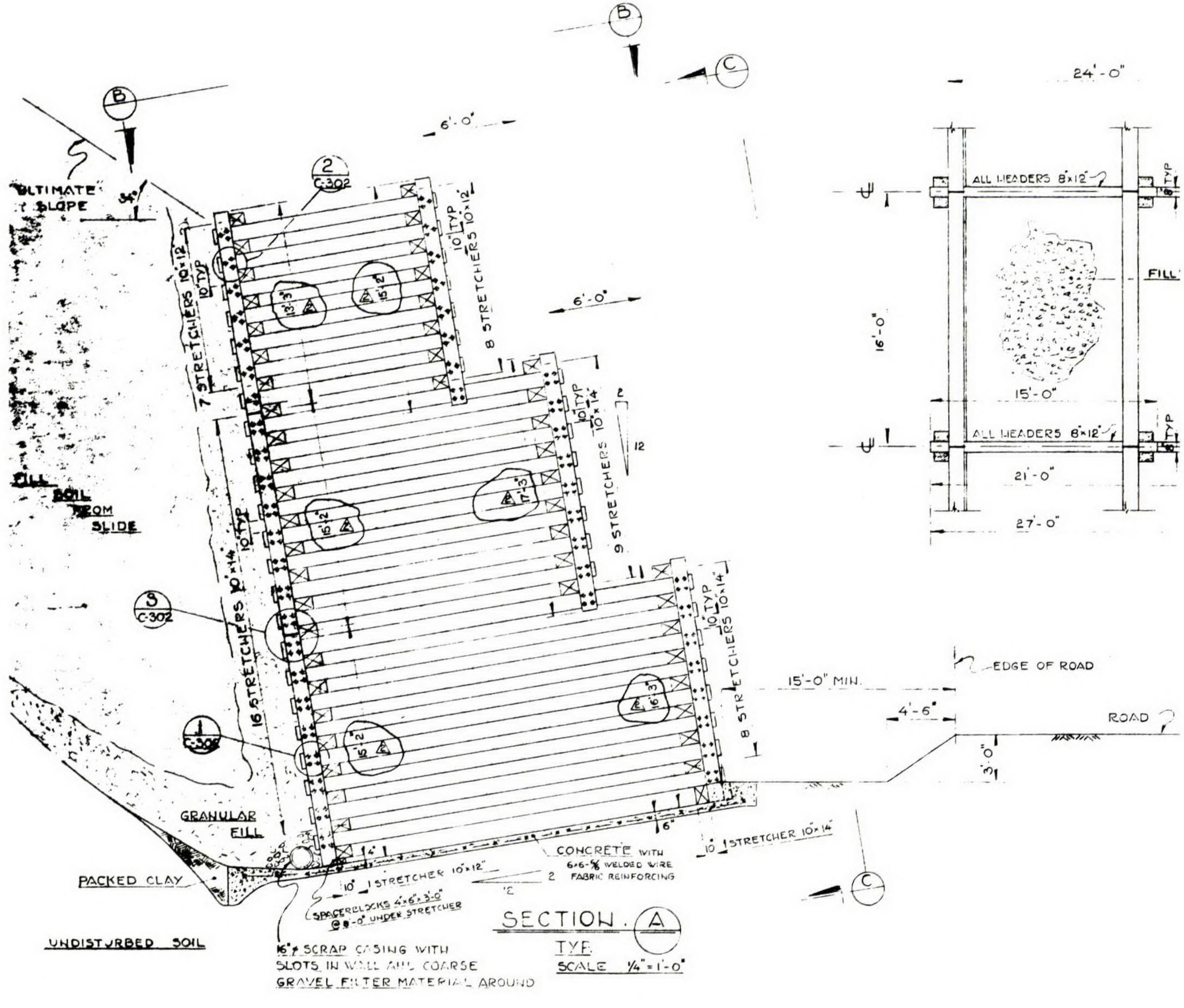


Fig 47 - Timber crib cross-section for IOC buttress.

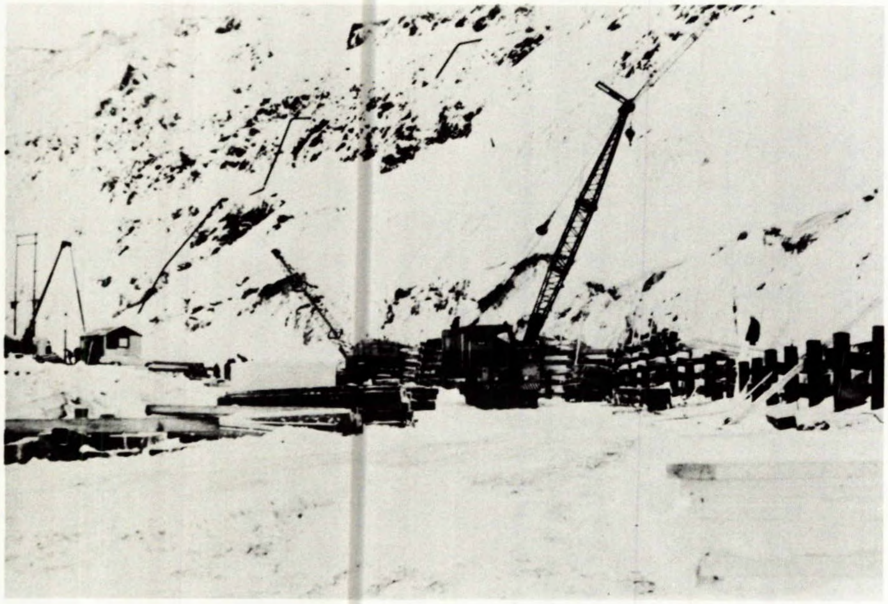


Fig 48 - Construction of buttress at Iron Ore Co. of Canada, IOC.

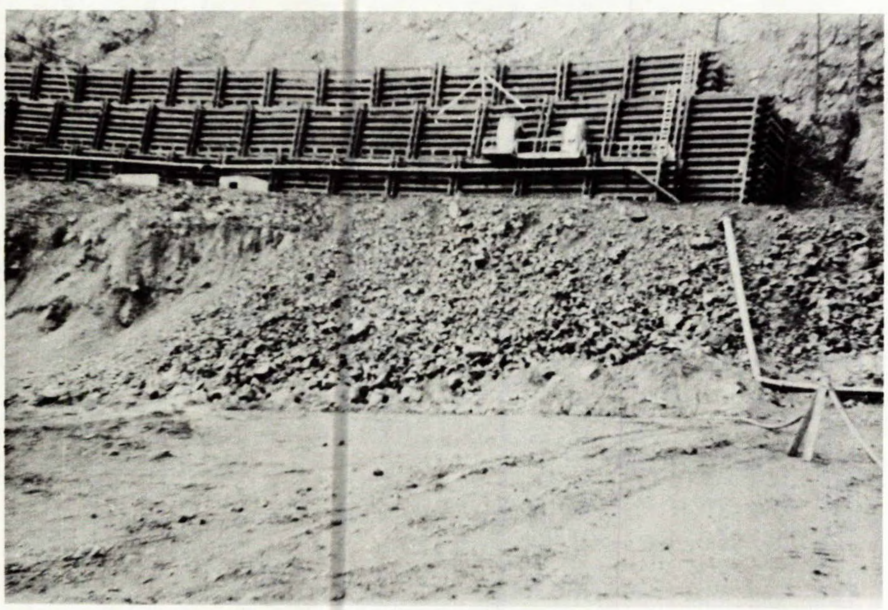


Fig 49 - Completed buttress at IOC.

Fig 50 - Structural damage to members of buttress at IOC due to settlement.

progress, shows structural damage to crib members. Figure 51 shows a longitudinal view of the crib. Note that a mud-flow partially covers the iron formation back fill.

224. Movement continued and by October 1966, near the end of the ore season, the crib had moved a total of 5 ft. In mid-October 1966, a small slide occurred on night shift. The edge of the haul road directly in front of the crib fell away and partially covered the portion of the main haul road immediately below. Although not a major instability, it was felt related to the major movement behind the crib. The pit was accordingly ordered evacuated and all equipment removed. During the next few shifts, the evacuation proceeded without mishap. However, as the last piece of equipment, a drill, was being brought out a larger slide occurred, again taking out a part of the haul road. This did not take place near the crib, but higher up the haul road towards the pit entrance. This final slide, on October 21, rendered the haul road unsafe and the pit was therefore abandoned.

225. As mentioned above, the pit was being dewatered. Abandoning the pit meant that no service vehicles could enter to maintain the pumps and pipelines. The mine flooded rapidly and as it did so, what can only be described as "peripheral instability" took place. All pit walls sloughed in as the water rose. Two months later, in mid-December, the water was over 100 ft deep.

Present

226. Abandoning the Ruth Lake Mine would normally be the end of the story, but in this particular case there is an interesting and instructive aftermath.

227. While the Ruth Lake crib was submerged after 1966, future mine planning for the Schefferville operations moved ahead. One of the priority orebodies to be mined was the Burnt Creek Mine. Burnt Creek was in fact an extension of the Ruth Lake orebody. Consequently, it could be anticipated that slope stability conditions might be similar. In addition, the planned bottom of the Burnt Creek pit was some 200 ft below the water level in the adjacent Ruth Lake Mine.

228. Plans were made to combat slope stability by proper design and adequate peripheral dewatering. Then, to prevent water from Ruth Lake Mine flowing in, a pumping program was started to lower the water level ahead of mining. By 1974, the crib in Ruth Lake Mine had begun to reappear above the water level and by early summer of 1975, emergence was complete. Figure 52and 53 show the crib as it was in early June 1975. The distortion of the crib is apparent but it is still in place, and still assisting in preventing major collapse, even after several years of complete submersion.

SOUTH AFRICAN SLIDE

229. Use of a post-tensioned buttress to stabilize a slide in an open pit mine in South Africa

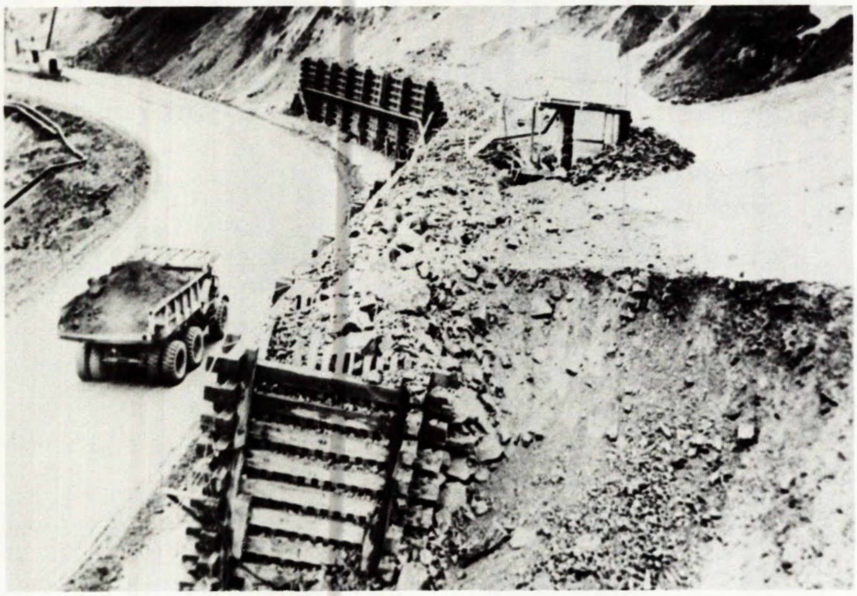


Fig 51 - Longitudinal view of IOC buttress.

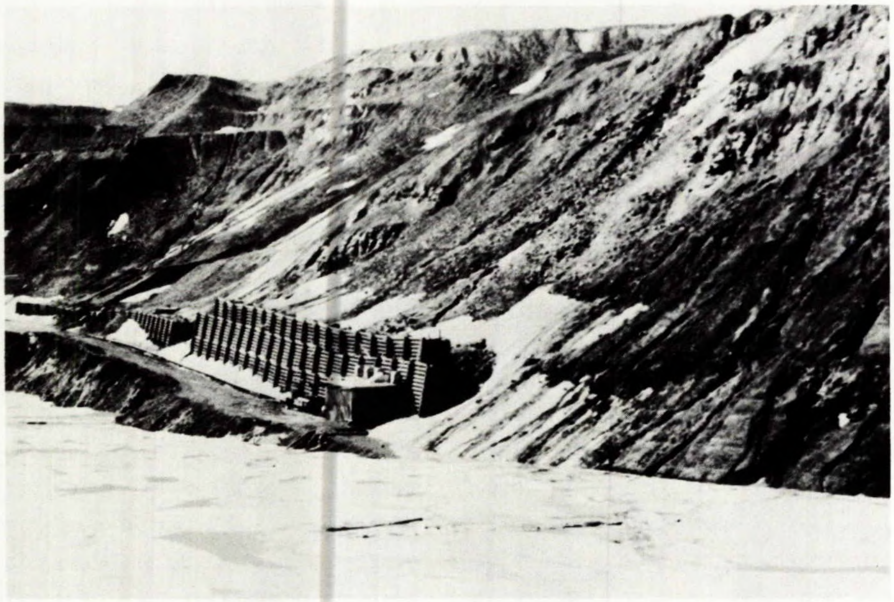


Fig 52 - IOC buttress after drainage of Ruth Lake.

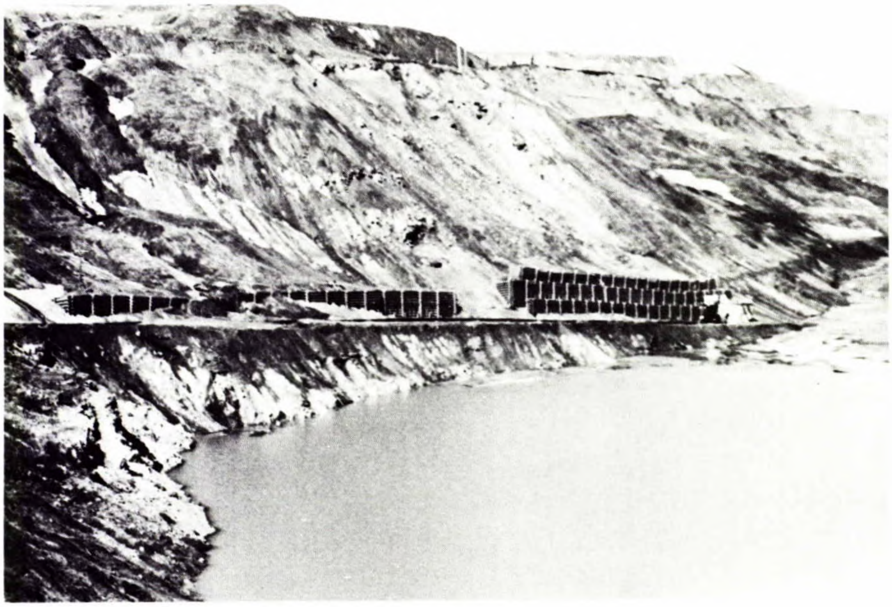


Fig 53 - IOC buttress after drainage of Ruth Lake.

has been reported by Yardley et al. (50). In December, 1963, the authors studied two slope failures both of which occurred in the north wall of an open pit iron mine. The study was concerned with two problems:

a. to determine if the two physically separate slides were a result of the same cause or whether they were independent;
b. to propose remedial action within certain economic and equipment limits.

The economic limitation derived from the fact that one slide had destroyed part of the main haulage road and threatened additional parts. An alternative haulage route some four miles greater in length was available. The arithmetic product of the remaining ore and the added ton-mile costs provided an economic limit for any proposed remedial action.

Structural Setting

230. The open pit is in the Biwabik iron formation striking N35°E and dipping 12° toward the south, while the north wall of the pit, where the slides occurred, was striking N35°W and sloping to the south.

231. A fault zone 50 to 100 ft wide, striking N50°E and dipping 25° to 30° to the southeast, passed through both slide areas as it cut across the strata. Up-dip the fault zone intersected the rock surface below about 100 ft of permeable glacial gravels which provided a path for water to move into the fault zone.

232. A prominent vertical set of joints was striking N45°W, roughly parallel to the benches and to the ore trough. Other joints occurred but did not appear to be particularly involved in the slope failures.

233. The upper slide area was in the Upper Cherty member of the iron formation, with its base in the top of the Lower Slaty member. The lower slide was in the Lower Cherty member which is stratigraphically below the Lower Slaty. A significant feature was the existence of a 2 to 4 ft thick shalylayer at the base of the upper slide. In the slide area the shaly layer was strongly altered to a greenish coloured semi-plastic material which could be deformed or squeezed by hand. Away from the slide zone, the layer was

unaltered and had the appearance of typical slaty iron formation.

234. Some idea of the physical nature of the squeezing layer was obtained from very limited test data. A uniaxial compression test of an 80.75 in. diameter specimen showed a compressive strength of about 60 psi. In a triaxial test with 40 psi confining pressure, failure occurred at about 100 psi. In a uniaxial test of a specimen from the same layer just outside the slide zone, an apparently unaltered specimen failed at about 4000 psi. The uniaxial failure strength of two test specimens of the iron formation collected 20 ft above the altered layer was 29,000 psi and 16,000 psi.

General Description and Chronology of the Slides

235. The two separate slides occurred on the north slope of the pit. One at an elevation of 500 to 600 ft above Lake Superior datum was referred to as the lower slide. The other at an elevation of 600 to 720 ft, centred about 400 ft further to the north and about 300 ft further west, was referred to as the upper slide. The upper slide involved destruction of part of the main haulage road. The first signs of failure were noted in February 1962, shortly after the removal of 7000 cu yd of material from the toe. No evidence of failure in the lower slide areas was reported at that time. Although cracking and some subsidence occurred, the haulage road was still usable. The lower area slid in August 1963, just after removal of a cut at the base of the bank adjacent to the old underground workings. The height of the instability, from toe to crown, was about 100 ft. There was no evidence of renewed movement in the upper slide area at that time. In November 1963, the upper slide renewed and extended its movement following removal of 10,000 cu yd of material along the toe bench. Again there was no evidence of new movement of the lower slide.

Causes of the Slides

236. A critical point of the study was whether the two slides were both results of the same root cause or whether they were unrelated. The conclusion reached was that the two slides were independent, and not related to a single deep seated cause.

237. It was also concluded that the lower slide resulted from a combination of:

a. the fault zone, which cut across its upper part and east side, with its related fracturing and decrease of cohesive support at the east edge of the slide area;
b. the caving method of mining used in the adjacent old underground work, which probably caused some additional fracturing of the rock to the north and some degree of opening up of the joints and bedding planes in that region;
c. water movement along the fault zone into the joints and fractures north of the caved area and consequent hydrostatic pressure effects which could have acted after deepening of the pit and lowering of the water table;
d. removal of support along the toe until instability resulted.

238. The upper slide resulted from a combination of:

a. the fault zone, which cut across it, but which occurred further west and stratigraphically higher than where it cut across the lower slide area;
b. existence of a particularly shaly layer at the base of the upper slide which was strongly altered by water percolating along and down through the fault zone (the resultant decrease in shear- cohesion made the layer subject to "squeezing out");
c. strong nearly-vertical joints, striking nearly parallel to the benches, and occasional build-up of water pressure in them because of the relatively impervious nature of the altered shaly layer at the base;
d. continued removal of material along the toe bench until the area reached a critical stage and instability resulted.

Remedial Action

239. Lower slide: Because the lower slide area was remedied by a fairly standard approach, only a

brief description is given. Basically the effective cohesion was insufficient to support a 1 to 1 slope, 100 ft high in the weakened rock in and near the fault zone, but was adequate to support a 50 ft slope. Hence, a change was made in the mining sequence so that no single bank exceeded 60 ft at any time. As of October, 1964, this had proved satisfactory and no further trouble had been experienced.

240. Upper slide: It was clear that regardless of the details, any remedial action would require removal of the slide material to unaltered rock below the squeezing layer and that rock backfill would be used. Economic considerations ruled out stripping back at road level to provide road room. This meant that 40 ton loads would be moving over rock underlain by the squeezing layer, then over rock plus backfill, then over backfill and to rock again.

241. It was felt that the part of the road and bank which had not yet slid was in a critical condition because all of the elements creating instability were still present. Dumping of rock fill against the bank would tend to inhibit further movement but might be inadequate if the altered layer continued to sqeeze out. The width of rock backfill which could be placed was limited by the size of the bench, and by future mining limits at a lower elevation.

242. A postensioned backfill system was designed to provide some additional resistance. The design consisted of the following steps:

a. removal of slide material, including the altered layer,
b. drilling anchor holes adjacent to the toe below the altered layer,
c. anchoring steel cables (old churn drill cable);
d. filling the toe area to above the altered layer with screened face rock (1/2 to 1 in.), to provide uniform bearing against the squeezing layer as well as good drainage,
e. placing some regular coarse backfill, stringing the cable, covering it for protection and placing bearing mats,
f. completing backfill to road elevation,
g. tensioning cables with jacks, and
h. adding further backfill to the top as settling occurred.

243. The principles are shown in Fig 54.

244. The remedy was decided upon in January. About 50,000 cu yd of slide was removed and 30,000 cu yd of backfill placed.

245. As of October, 1964 the remedy had worked. New instability did occur beyond the zone of post-tensioned cables but ceased where the post-tensioned zone began.

246. The remedial action provided the following stabilizing features. It provided additional direct resistance to squeezing of the altered layer. It increased the frictional resistance at the base of the fill. It provided some additional force against the whole bank. It increased the internal resistance of the fill. It also established a system whereby any slight outward movement of the rock increased the tension on the cables which in turn increased the resisting forces.

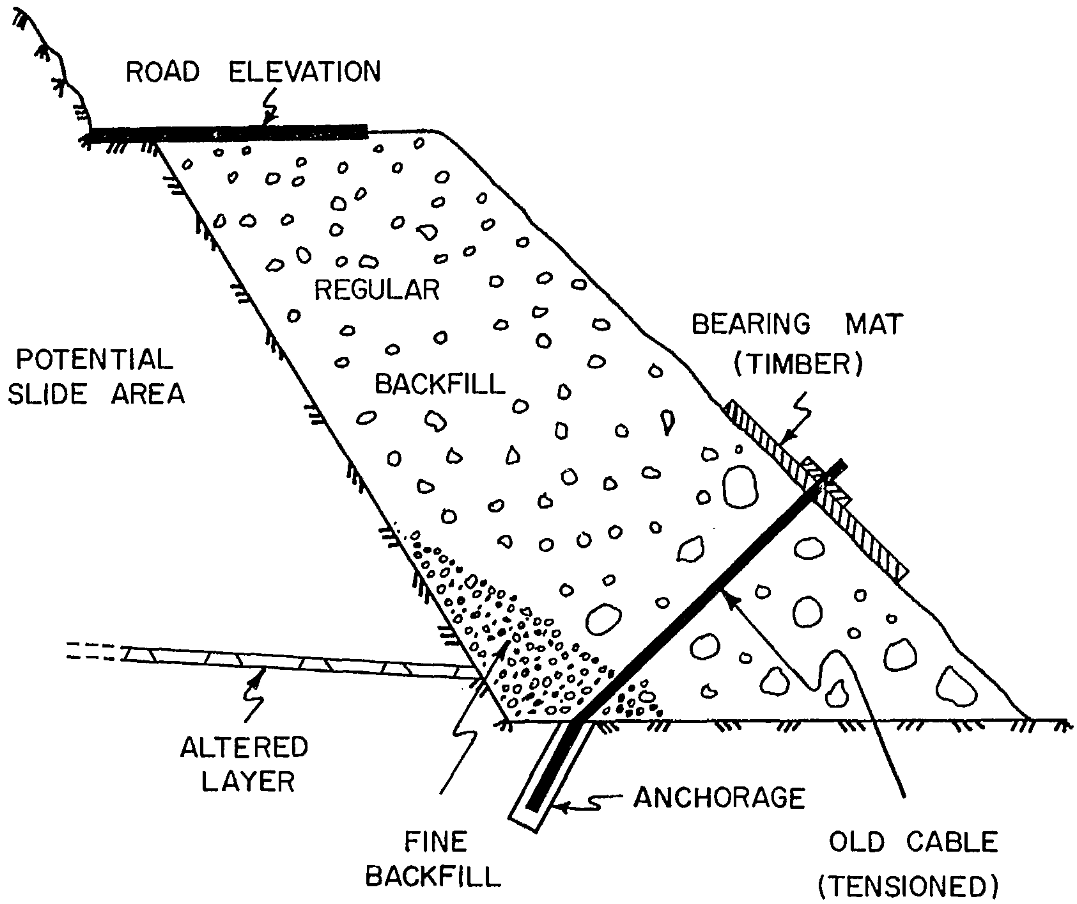


Fig 54 - Proposed post-tensioned rock buttress for stabilizing a slide in South Africa (50).

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